Structural Design Software

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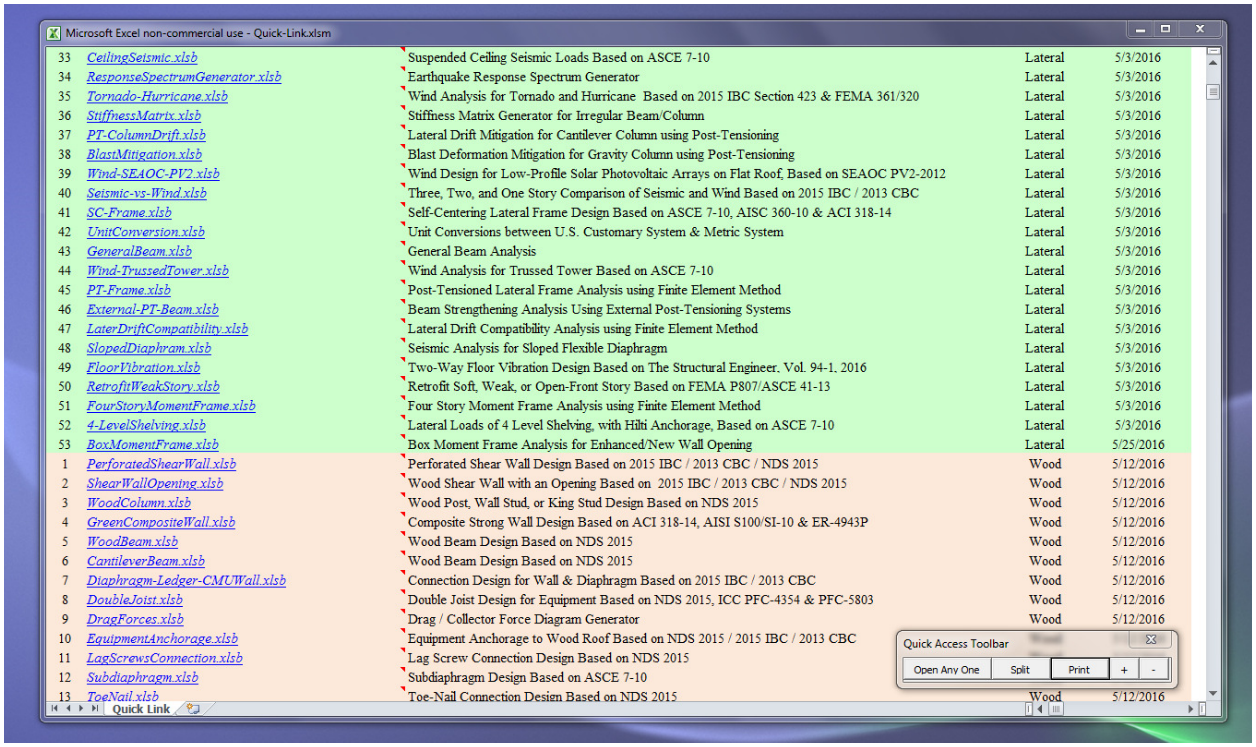
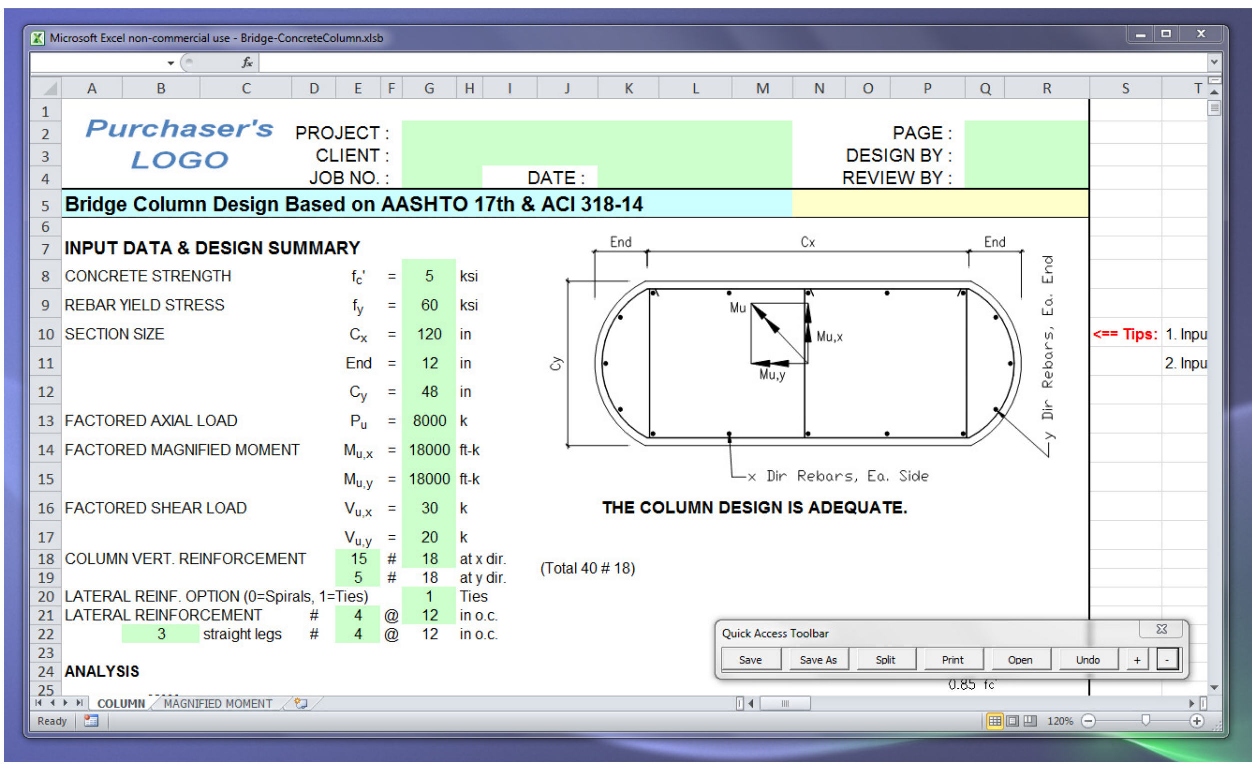

Microsoft Excel non-commercial use - Quick-Link.xlsm

33	CeilingSeismic.xlsb	Suspended Ceiling Seismic Loads Based on ASCE 7-10	Lateral	5/3/2016
34	ResponseSpectrumGenerator.xlsb	Earthquake Response Spectrum Generator	Lateral	5/3/2016
35	Tornado-Hurricane.xlsb	Wind Analysis for Tornado and Hurricane Based on 2015 IBC Section 423 & FEMA 361/320	Lateral	5/3/2016
36	StiffnessMatrix.xlsb	Stiffness Matrix Generator for Irregular Beam/Column	Lateral	5/3/2016
37	PT-ColumnDrift.xlsb	Lateral Drift Mitigation for Cantilever Column using Post-Tensioning	Lateral	5/3/2016
38	BlastMitigation.xlsb	Blast Deformation Mitigation for Gravity Column using Post-Tensioning	Lateral	5/3/2016
39	Wind-SEAOC-PV2.xlsb	Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roof, Based on SEAOC PV2-2012	Lateral	5/3/2016
40	Seismic-vs-Wind.xlsb	Three, Two, and One Story Comparison of Seismic and Wind Based on 2015 IBC / 2013 CBC	Lateral	5/3/2016
41	SC-Frame.xlsb	Self-Centering Lateral Frame Design Based on ASCE 7-10, AISC 360-10 & ACI 318-14	Lateral	5/3/2016
42	UnitConversion.xlsb	Unit Conversions between U.S. Customary System & Metric System	Lateral	5/3/2016
43	GeneralBeam.xlsb	General Beam Analysis	Lateral	5/3/2016
44	Wind-TrussedTower.xlsb	Wind Analysis for Trussed Tower Based on ASCE 7-10	Lateral	5/3/2016
45	PT-Frame.xlsb	Post-Tensioned Lateral Frame Analysis using Finite Element Method	Lateral	5/3/2016
46	External-PT-Beam.xlsb	Beam Strengthening Analysis Using External Post-Tensioning Systems	Lateral	5/3/2016
47	LaterDriftCompatibility.xlsb	Lateral Drift Compatibility Analysis using Finite Element Method	Lateral	5/3/2016
48	SlopedDiaphram.xlsb	Seismic Analysis for Sloped Flexible Diaphragm	Lateral	5/3/2016
49	FloorVibration.xlsb	Two-Way Floor Vibration Design Based on The Structural Engineer, Vol. 94-1, 2016	Lateral	5/3/2016
50	RetrofitWeakStory.xlsb	Retrofit Soft, Weak, or Open-Front Story Based on FEMA P807/ASCE 41-13	Lateral	5/3/2016
51	FourStoryMomentFrame.xlsb	Four Story Moment Frame Analysis using Finite Element Method	Lateral	5/3/2016
52	4-LevelShelving.xlsb	Lateral Loads of 4 Level Shelving, with Hilti Anchorage, Based on ASCE 7-10	Lateral	5/3/2016
53	BoxMomentFrame.xlsb	Box Moment Frame Analysis for Enhanced/New Wall Opening	Lateral	5/25/2016
1	PerforatedShearWall.xlsb	Perforated Shear Wall Design Based on 2015 IBC / 2013 CBC / NDS 2015	Wood	5/12/2016
2	ShearWallOpening.xlsb	Wood Shear Wall with an Opening Based on 2015 IBC / 2013 CBC / NDS 2015	Wood	5/12/2016
3	WoodColumn.xlsb	Wood Post, Wall Stud, or King Stud Design Based on NDS 2015	Wood	5/12/2016
4	GreenCompositeWall.xlsb	Composite Strong Wall Design Based on ACI 318-14, AISI S100/SI-10 & ER-4943P	Wood	5/12/2016
5	WoodBeam.xlsb	Wood Beam Design Based on NDS 2015	Wood	5/12/2016
6	CantileverBeam.xlsb	Wood Beam Design Based on NDS 2015	Wood	5/12/2016
7	Diaphragm-Ledger-CMUWall.xlsb	Connection Design for Wall & Diaphragm Based on 2015 IBC / 2013 CBC	Wood	5/12/2016
8	DoubleJoist.xlsb	Double Joist Design for Equipment Based on NDS 2015, ICC PFC-4354 & PFC-5803	Wood	5/12/2016
9	DragForces.xlsb	Drag / Collector Force Diagram Generator	Wood	5/12/2016
10	EquipmentAnchorage.xlsb	Equipment Anchorage to Wood Roof Based on NDS 2015 / 2015 IBC / 2013 CBC		
11	LagScrewsConnection.xlsb	Lag Screw Connection Design Based on NDS 2015		
12	Subdiaphragm.xlsb	Subdiaphragm Design Based on ASCE 7-10		
13	ToeNail.xlsb	Toe-Nail Connection Design Based on NDS 2015	Wood	5/12/2016



	Quick Open Link	www.Engineering-International.com	Group	Last Updated
1	FreeStandingWall.xlsb	Free Standing Masonry & Conctere Wall Design Based on TMS 402-16 & ACI 38-19	Foundation	5/17/2024
2	EccentricFooting.xlsb	Eccentric Footing Design Based on ACI 38-19	Foundation	5/17/2024
3	Flagpole.xlsb	Flagpole Footing Design Based on Chapter 18 of IBC & CBC	Foundation	5/17/2024
4	MasonryRetainingWall.xlsb	Masonry Retaining / Fence Wall Design Based on TMS 402-16 & ACI 38-19	Foundation	5/17/2024
5	ConcreteRetainingWall.xlsb	Concrete Retaining Wall Design Based on ACI 38-19	Foundation	5/17/2024
6	Masonry-Concrete-RetainingWall.xlsb	Retaining Wall Design, for Masonry Top & Concrete Bottom, Based on TMS 402-16 & ACI 38-19	Foundation	5/17/2024
7	ConcretePier.xlsb	Concrete Pier (Isolated Deep Foundation) Design Based on ACI 38-19	Foundation	5/17/2024
8	ConcretePile.xlsb	Drilled Cast-in-place Pile Design Based on ACI 38-19	Foundation	5/17/2024
9	PileCaps.xlsb	Pile Cap Design for 4, 3, 2-Piles Pattern Based on ACI 38-19	Foundation	5/17/2024
10	PileCapBalancedLoads.xlsb	Determination of Pile Cap Balanced Loads and Reactions	Foundation	5/17/2024
11	ConventionalSlabOnGrade.xlsb	Design of Conventional Slabs on Expansive & Compressible Soil Grade Based on ACI 360	Foundation	5/17/2024
12	PT-SlabOnGround.xlsb	Design of PT Slabs on Expansive Soil Ground Based on PTI DC10.5-12 & PTI 3rd Edition	Foundation	5/17/2024
13	BasementConcreteWall.xlsb	Basement Concrete Wall Design Based on ACI 38-19	Foundation	5/17/2024
14	BasementMasonryWall.xlsb	Basement Masonry Wall Design Based on TMS 402-16	Foundation	5/17/2024
15	BasementColumn.xlsb	Basement Column Supporting Lateral Resisting Frame Based on ACI 38-19	Foundation	5/17/2024
16	MRF-GradeBeam.xlsb	Grade Beam Design for Moment Resisting Frame Based on ACI 318-16	Foundation	5/17/2024
17	BraceGradeBeam.xlsb	Grade Beam Design for Brace Frame Based on ACI 38-19	Foundation	5/17/2024
18	GradeBeam.xlsb	Two Pads with Grade Beam Design Based on ACI 38-19 & AISC 360-22	Foundation	5/17/2024
19	CircularFooting.xlsb	Circular Footing Design Based on ACI 38-19	Foundation	5/17/2024
20	CombinedFooting.xlsb	Combined Footing Design Based on ACI 38-19	Foundation	5/17/2024
21	BoundarySpringGenerator.xlsb	Mat Boundary Spring Generator	Foundation	5/17/2024
22	DeepFooting.xlsb	Deep Footing Design Based on ACI 38-19	Foundation	5/17/2024
23	FootingAtPiping.xlsb	Design of Footing at Piping Based on ACI 38-19	Foundation	5/17/2024
24	IrregularFootingSoilPressure.xlsb	Soil Pressure Determination for Irregular Footing	Foundation	5/17/2024
25	PAD.xlsb	Pad Footing Design Based on ACI 38-19	Foundation	5/17/2024
26	PlainConcreteFooting.xlsb	Plain Concrete Footing Design Based on ACI 38-19	Foundation	5/17/2024
27	RestrainedRetainingWall.xlsb	Restrained Retaining Masonry & Concrete Wall Design Based on TMS 402 & ACI 318	Foundation	5/17/2024
28	RetainingWall-DSA-OSHPD.xlsb	Retaining Wall Design Based on 2022 CBC Chapter A	Foundation	5/17/2024
29	TankFooting.xlsb	Tank Footing Design Based on ACI 318-19, ASCE 7-22 & AWWA D103-19	Foundation	5/17/2024
30	TankAnchorage.xlsb	Tank Anchorage Design Based on ACI 318-19 & AWWA D103-19	Foundation	5/17/2024
31	TemporaryFootingforRectangularTank.xlsb	Temporary Tank Footing Design Based on ACI 38-19	Foundation	5/17/2024
32	UnderGroundWell.xlsb	Under Ground Well Design Based on ACI 350-06 & ACI 38-19	Foundation	5/17/2024
33	StudBearingWallFooting.xlsb	Footing Design for Stud Bearing Wall Based on 2021 IBC / ACI 38-19	Foundation	5/17/2024
34	WallFooting.xlsb	Footing Design of Shear Wall Based on ACI 38-19	Foundation	5/17/2024
35	FixedMomentCondition.xlsb	Fixed Moment Condition Design Based on ACI 38-19	Foundation	5/17/2024
36	FloodWay.xlsb	Concrete Floodway Design Based on ACI 350-06 & ACI 38-19	Foundation	5/17/2024
37	LateralEarthPressure.xlsb	Lateral Earth Pressure of Rigid Wall Based on AASHTO 17th & 2021 IBC	Foundation	5/17/2024
38	Shoring.xlsb	Sheet Pile Wall Design Based on 2021 IBC / 2022 CBC / ACI 38-19	Foundation	5/17/2024
39	CompositeElementDurability.xlsb	Composite Element Design Based on AISC 360-22 & ACI 38-19	Foundation	5/17/2024
40	SeismicEarthPressure.xlsb	Seismic Earth Pressure of Deep Stiff Wall Based on FEMA P-750 & AASHTO/IBC	Foundation	5/17/2024
41	Caisson.xlsb	Caisson Design Based on 2021 IBC & 2022 CBC	Foundation	5/17/2024
42	RectangularMachineFooting.xlsb	Rectangular Machine or Tank Footing Design Based on ACI 38-19	Foundation	5/17/2024
43	Tieback.xlsb	Sheet Pile Wall, with Tieback Anchors, Design Based on AASHTO (HB-17), 2021 IBC & ACI 38-19	Foundation	5/17/2024
44	ScrewPiles.xlsb	Screw Pile Design Based on 2021 IBC & AISC 360-22	Foundation	5/17/2024
45	SlopeStability.xlsb	Slope Stability Analysis Based on AASHTO 17th & 2021 IBC	Foundation	5/17/2024
46	UnderFootingSewer.xlsb	Underground Utilities Way Design Based on AASHTO-17th & 2021 IBC	Foundation	5/17/2024
47	LandslideRepair.xlsb	Landslide Repair Design Based on 2021 IBC & AASHTO 17th	Foundation	5/17/2024
48	RingFoundation.xlsb	Ring Foundation Design Based on 2021 IBC & ACI 38-19	Foundation	5/17/2024
49	DrivenPile.xlsb	Driven Precast Concrete Pile Design Based on 2021 IBC & ACI 38-19	Foundation	5/17/2024
50	EquipmentFooting.xlsb	Foundation Design for Dynamic Equipment Based on ACI 351.3 & ACI 38-19	Foundation	5/17/2024
51	Counterfort-Retaining-Wall.xlsb	Counterfort Retaining Wall Design Based on 2021 IBC & ACI 38-19	Foundation	5/17/2024
52	Retaining-Wall-Repair.xlsb	Retaining Wall Repair Design Based on AASHTO/2021 IBC & TMS 402-16	Foundation	5/17/2024
53	MatFoundation.xlsb	RC Mat Slab Design Based on 2021 IBC, ACI 38-19, AASHTO 17th Edition & ACI 360	Foundation	5/17/2024
54	BinWall.xlsb	Trapezoidal Loads Retaining Wall Design Based on ACI 318-19	Foundation	5/17/2024
55	TreeRootFoundation.xlsb	Tree Root Foundation Design Based on AASHTO (HB-17), 2021 IBC & 2022 CBC	Foundation	5/17/2024
56	ElasticStripFoundation.xlsb	Elastic Strip Foundation Analysis using Finite Element Method Based on 2021 IBC	Foundation	5/17/2024
57	PT-Rebar-GroundSlab.xlsb	Design of PT Slabs with Rebar Stiffening Beam Based on ACI 318-19, PTI DC10.5-12 & PTI 3rd Edition	Foundation	5/17/2024
58	SonotubeFooting.xlsb	Sonotube Footing Design Based on 2021 IBC, ASCE 7-22, & ACI 318-19	Foundation	5/17/2024
59	RigidFootingMomentCapacity.xlsb	Rigid Footing Moment Capacity Design Based on ASCE 41-17 & ACI 318-19	Foundation	5/17/2024
1	PerforatedShearWall.xlsb	Perforated Shear Wall Design Based on 2021 IBC / 2022 CBC / NDS 2018	Wood	5/17/2024
2	ShearWallOpening.xlsb	Wood Shear Wall with an Opening Based on 2021 IBC / 2022 CBC / NDS 2018	Wood	5/17/2024
3	WoodColumn.xlsb	Wood Post, Wall Stud, or King Stud Design Based on NDS 2018	Wood	5/17/2024
4	GreenCompositeWall.xlsb	Composite Strong Wall Design Based on ACI 318-19, AISI S100/SI-10 & ESR-3064P	Wood	5/17/2024
5	WoodBeam.xlsb	Wood Beam Design Based on NDS 2018	Wood	5/17/2024
6	CantileverBeam.xlsb	Wood Beam Design Based on NDS 2018	Wood	5/17/2024
7	Diaphragm-Ledger-CMUWall.xlsb	Connection Design for Wall & Diaphragm Based on 2021 IBC / 2022 CBC	Wood	5/17/2024
8	DoubleJoist.xlsb	Double Joist Design for Equipment Based on NDS 2018, ICC PFC-4354 & PFC-5803	Wood	5/17/2024
9	DragForces.xlsb	Drag / Collector Force Diagram Generator	Wood	5/17/2024
10	EquipmentAnchorage.xlsb	Equipment Anchorage to Wood Roof Based on NDS 2018 / 2021 IBC / 2022 CBC	Wood	5/17/2024
11	LagScrewsConnection.xlsb	Lag Screw Connection Design Based on NDS 2018	Wood	5/17/2024
12	Subdiaphragm.xlsb	Subdiaphragm Design Based on ASCE 7-22	Wood	5/17/2024
13	ToeNail.xlsb	Toe-Nail Connection Design Based on NDS 2018	Wood	5/17/2024
14	TopPlateConnection.xlsb	Top Plate Connection Design Based on NDS 2018	Wood	5/17/2024
15	Truss-Wood.xlsb	Wood Truss Design Based on NDS 2018	Wood	5/17/2024
16	WoodBoltConnection.xlsb	Bolt Connection Design Based on NDS 2018	Wood	5/17/2024
17	WoodDiaphragm.xlsb	Wood Diaphragm Design Based on SDPWS-21	Wood	5/17/2024
18	WoodJoist.xlsb	Wood Joist Design Based on NDS 2018 / NDS 01, ICC PFC-4354 & PFC-5803	Wood	5/17/2024
19	WoodShearWall.xlsb	Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21	Wood	5/17/2024
20	WoodTables.xlsb	Tables for Wood Post Design Based on NDS 2018	Wood	5/17/2024
21	TransferDiaphragm-Wood.xlsb	Wood Diaphragm Design for a Discontinuity of Type 4 out-of-plane offset irregularity	Wood	5/17/2024
22	WoodPolePile.xlsb	Wood Pole or Pile Design Based on NDS 2018	Wood	5/17/2024
23	WoodMember.xlsb	Wood Member (Beam, Column, Brace, Truss Web & Chord) Design Based on NDS 2018	Wood	5/17/2024
24	BendingPostAtColumn.xlsb	Connection Design for Bending Post at Concrete Column Based on NDS 2018 & ACI 318-19	Wood	5/17/2024
25	CurvedMember.xlsb	Curved Wood Member (Wood Torsion) Design Based on NDS 2018	Wood	5/17/2024
26	StrongCustomFrame.xlsb	4E-SMF with Wood Nailer Design Based on AISC 358-22 & NDS 2018	Wood	5/17/2024
27	CLT-TwoWayFloor.xlsb	Two-Way Floor Design Based on NDS 2018, using Cross-Laminated Timber (CLT), by FEM	Wood	5/17/2024

28	CLT-ShearWall.xlsb	Shear Wall Design, using Cross-Laminated Timber (CLT), Based on NDS 2018	Wood	5/17/2024
29	HybridMember.xlsb	Hybrid Member (Wood & Metal) Design Based on NDS 2018, AISI S100 & ESR-3064P	Wood	5/17/2024
30	BeamReinforcement.xlsb	Beam Reinforcement Design by Finite Element Method	Wood	5/17/2024
31	BambooShearWall.xlsb	Shear Wall Design, using Laminated Bamboo, Based on NDS 2018	Wood	5/17/2024
32	WoodRepairProtection.xlsb	Wood Repair & Protection Design Based on 2016 CEBC, ASCE 41-17, ACI 318-19 & NDS 2018	Wood	5/17/2024
33	ToFixSaggingBeam.xlsb	To Fix Sagging Beam, Using External Post-Tensioning Systems, Based on NDS 2018	Wood	5/17/2024
34	ToFixSaggingGirder.xlsb	To Fix Sagging Girder, by Bent HSS Tube Arch, Based on NDS 2018 & AISC 360-22	Wood	5/17/2024
35	MechanicallyLaminatedDecking.xlsb	Mechanically Laminated Decking Design Based on 2022 CBC/2021 IBC 2304.9	Wood	5/17/2024
36	WoodAnchorage.xlsb	Sill Plate/Nailer Connection Design Based on NDS 2018	Wood	5/17/2024
37	FlitchPlateBeam.xlsb	Flitch Plate Beam Design Based on AISC 360-22 & NDS 2018	Wood	5/17/2024
38	CantileverWoodDiaphragm.xlsb	Cantilever Wood Diaphragm Design Based on SDPWS-21	Wood	5/17/2024
39	NotchingDesign.xlsb	Notching Design for Wood and Steel Beam Based on 2021 IBC, NDS 2018, & AISC 360-22	Wood	5/17/2024
40	TudorArches.xlsb	Tudor Arches Design Using Finite Element Method in Structural Mechanics	Wood	5/17/2024
41	CLT-Wall-Wind.xlsb	Wall of Cross-Laminated Timber (CLT) Design, for Perpendicular to Plane Loads, Based on NDS 2018	Wood	5/17/2024
1	MasonryShearWall-CBC.xlsb	Masonry Shear Wall Design Based on 2022 CBC Chapter A (both ASD and SD)	Masonry	5/17/2024
2	MasonryShearWall-IBC.xlsb	Masonry Shear Wall Design Based on TMS 402-16 & 2021 IBC (both ASD and SD)	Masonry	5/17/2024
3	AnchorageToMasonry.xlsb	Fastener Anchorage Design in Masonry Based on TMS 402-16	Masonry	5/17/2024
4	FlushWallPilaster-CBC.xlsb	Masonry Flush Wall Pilaster Design Based on 2022 CBC Chapter A	Masonry	5/17/2024
5	FlushWallPilaster-IBC.xlsb	Masonry Flush Wall Pilaster Design Based on TMS 402-16	Masonry	5/17/2024
6	BearingWallOpening.xlsb	Design of Masonry Bearing Wall with Opening Based on TMS 402-16	Masonry	5/17/2024
7	BendingPostAtTopWall.xlsb	Design for Bending Post at Top of Wall, Based on TMS 402-16	Masonry	5/17/2024
8	DevelopmentSpliceMasonry.xlsb	Development & Splice of Reinforcement in Masonry Based on TMS 402-16 & 2021 IBC & 2022 CBC	Masonry	5/17/2024
9	Elevator-DSA-OSHPD.xlsb	Elevator Masonry Wall Design Based on 2022 CBC Chapter A & 2021 IBC	Masonry	5/17/2024
10	GirderAtWall.xlsb	Design for Girder at Masonry Wall Based on TMS 402-16	Masonry	5/17/2024
11	HorizontalBendingWall.xlsb	Masonry Wall Design at Horizontal Bending Based on TMS 402-16	Masonry	5/17/2024
12	MasonryBeam.xlsb	Masonry Beam Design Based on TMS 402-16	Masonry	5/17/2024
13	MasonryBearingWall-CBC.xlsb	Allowable & Strength Design of Masonry Bearing Wall Based on 2022 CBC Chapter A	Masonry	5/17/2024
14	MasonryBearingWall-IBC.xlsb	Allowable & Strength Design of Masonry Bearing Wall Based on TMS 402-16 & 2021 IBC	Masonry	5/17/2024
15	MasonryColumn-CBC.xlsb	Masonry Column Design Based on 2022 CBC Chapter A	Masonry	5/17/2024
16	MasonryColumn-IBC.xlsb	Masonry Column Design Based on TMS 402-16 & 2021 IBC	Masonry	5/17/2024
17	BeamToWall.xlsb	Beam to Wall Anchorage Design Based on TMS 402-16	Masonry	5/17/2024
18	CollectorToWall.xlsb	Collector to Wall Connection Design Based on TMS 402-16	Masonry	5/17/2024
19	HybridMasonry.xlsb	Hybrid Masonry Wall Design Based on TMS 402-16	Masonry	5/17/2024
20	PT-MasonryShearWall.xlsb	Post-Tensioned Masonry Shear Wall Design Based on TMS 402-16 (SD Method)	Masonry	5/17/2024
21	MasonryWallOpening.xlsb	Masonry Shear Wall with Opening Design Using Finite Element Method	Masonry	5/17/2024
22	MasonryCracking.xlsb	Anticipated Cracking Design of Masonry Wall Based on TMS 402-16	Masonry	5/17/2024
23	EnhanceExistingColumn.xlsb	Existing Column Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19/TMS 402-16	Masonry	5/17/2024
24	EnhanceExistingWall.xlsb	Existing Wall Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19/TMS 402-16	Masonry	5/17/2024
25	GrillageBeamWall.xlsb	Design for Grillage Beam Masonry Wall Based on TMS 402-16	Masonry	5/17/2024
26	MasonryElement.xlsb	Masonry Plate/Shell Element Design (ASD) Based on 2021 IBC & TMS 402-16	Masonry	5/17/2024
27	LightlyLoadedColumn.xlsb	Lightly Loaded Column Design Based on TMS 402-16 (UNCRACKED and CRACKED)	Masonry	5/17/2024
1	Wind-ASCE7-22.xlsb	Wind Analysis Based on ASCE 7-22	Lateral	5/17/2024
2	Seismic-2021IBC.xlsb	Seismic Analysis Based on 2021 IBC	Lateral	5/17/2024
3	Wind-ASCE7-16.xlsb	Wind Analysis Based on ASCE 7-16	Lateral	5/17/2024
4	PipeRiser.xlsb	MCE Level Seismic Design for Metal Pipe/Riser Based on ASCE 7-22 & AISI S100	Lateral	5/17/2024
5	RigidDiaphragm.xlsb	Rotation Analysis of Rigid Diaphragm Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
6	FlexibleDiaphragm.xlsb	Flexible Diaphragm Analysis	Lateral	5/17/2024
7	TwoStoryMomentFrame.xlsb	Two Story Moment Frame Analysis using Finite Element Method	Lateral	5/17/2024
8	X-BracedFrame.xlsb	X-Braced Frame Analysis using Finite Element Method	Lateral	5/17/2024
9	OpenStructureWind.xlsb	Wind Analysis for Open Structure (Solar Panels) Based on ASCE 7-22, 10 & 05	Lateral	5/17/2024
10	RoofScreenWind.xlsb	Wind Load, on Roof Screen / Roof Equipment, Based on ASCE 7-22, 10 & 05	Lateral	5/17/2024
11	AxialRoofDeck.xlsb	Axial Capacity of 1 1/2" Type "B" Roof Deck Based on ICBO ER-2078P	Lateral	5/17/2024
12	DeformationCompatibility.xlsb	Column Deformation Compatibility Design using Finite Element Method	Lateral	5/17/2024
13	DiscontinuousShearWall.xlsb	Discontinuous Shear Wall Analysis Using Finite Element Method	Lateral	5/17/2024
14	FlexibleDiaphragmOpening.xlsb	Flexible Diaphragm with an Opening Analysis	Lateral	5/17/2024
15	Handrail.xlsb	Handrail Design Based on AISC 360-22 & ACI 318-19	Lateral	5/17/2024
16	InteriorWallLateralForce.xlsb	Interior Wall Lateral Forces Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
17	LateralFrameFormulas.xlsb	Lateral Frame Formulas	Lateral	5/17/2024
18	LiveLoad.xlsb	Live Load Reduction Based on ASCE 7-22, 2021 IBC / 2022 CBC	Lateral	5/17/2024
19	Seismic-SingleFamilyDwellings.xlsb	Seismic Analysis for Family Dwellings Based on 2021 IBC / 2022 CBC & ASCE 7-22	Lateral	5/17/2024
20	ShadeStructureWind.xlsb	Wind Analysis for Shade Open Structure Based on ASCE 7-22, 10 & 05	Lateral	5/17/2024
21	ShearWallForces.xlsb	Shear Wall Analysis for Shear Wall with Opening Using Finite Element Method	Lateral	5/17/2024
22	ShearWall-NewOpening.xlsb	Relative Rigidity Determination for Shear Wall with New Opening	Lateral	5/17/2024
23	ShearWallRigidity.xlsb	Rigidity for Shear Wall & Shear Wall with Opening Using Finite Element Method	Lateral	5/17/2024
24	Sign.xlsb	Sign Design Based on AISC 360-22, ACI 318-19, and IBC 1807.3	Lateral	5/17/2024
25	SignWind.xlsb	Wind Analysis for Freestanding Wall & Sign Based on ASCE 7-22, 10 & 05	Lateral	5/17/2024
26	Snow.xlsb	Snow Load Analysis Based on ASCE 7-22, 10, 05, & UBC	Lateral	5/17/2024
27	WallLateralForce-CBC.xlsb	Lateral Force for One-Story Wall Based on 2022 CBC	Lateral	5/17/2024
28	WallLateralForce-IBC.xlsb	Lateral Force for One-Story Wall Based on 2021 IBC	Lateral	5/17/2024
29	Seismic-ASCE7-22.xlsb	Seismic Analysis Based on ASCE 7-22	Lateral	5/17/2024
30	WindGirtDeflection.xlsb	Wind Girt Deflection Analysis of Wood, Metal Stud, and/or Steel Tube	Lateral	5/17/2024
31	StorageRacks.xlsb	Lateral Loads of Storage Racks, with Hilti & Red Head Anchorage, Based on ASCE 7-22	Lateral	5/17/2024
32	Wind-Alternate.xlsb	Wind Analysis for Building with h < 60 ft, Based on 2021 IBC/ASCE 7-22	Lateral	5/17/2024
33	CeilingSeismic.xlsb	Suspended Ceiling Seismic Loads Based on ASCE 7-22	Lateral	5/17/2024
34	ResponseSpectrumGenerator.xlsb	Earthquake Response Spectrum Generator	Lateral	5/17/2024
35	Tornado-Hurricane.xlsb	Wind Analysis for Tornado and Hurricane Based on 2021 IBC Section 423 & FEMA 361/320	Lateral	5/17/2024
36	StiffnessMatrix.xlsb	Stiffness Matrix Generator for Irregular Beam/Column	Lateral	5/17/2024
37	PT-ColumnDrift.xlsb	Lateral Drift Mitigation for Cantilever Column using Post-Tensioning	Lateral	5/17/2024
38	BlastMitigation.xlsb	Blast Deformation Mitigation for Gravity Column using Post-Tensioning	Lateral	5/17/2024
39	Wind-SolarPanels.xlsb	Wind Design for Rooftop Solar Panels Based on ASCE 7-22	Lateral	5/17/2024
40	Seismic-vs-Wind.xlsb	Three, Two, and One Story Comparison of Seismic and Wind Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
41	SC-Frame.xlsb	Self-Centering Lateral Frame Design Based on ASCE 7-22, AISC 360-22 & ACI 318-19	Lateral	5/17/2024
42	UnitConversion.xlsb	Unit Conversions between U.S. Customary System & Metric System	Lateral	5/17/2024
43	GeneralBeam.xlsb	General Beam Analysis	Lateral	5/17/2024
44	Wind-TrussedTower.xlsb	Wind Analysis for Trussed Tower Based on ASCE 7-22	Lateral	5/17/2024
45	PT-Frame.xlsb	Post-Tensioned Lateral Frame Analysis using Finite Element Method	Lateral	5/17/2024
46	External-PT-Beam.xlsb	Beam Strengthening Analysis Using External Post-Tensioning Systems	Lateral	5/17/2024

47	LaterDriftCompatibility.xlsb	Lateral Drift Compatibility Analysis using Finite Element Method	Lateral	5/17/2024
48	SlopedDiaphram.xlsb	Seismic Analysis for Sloped Flexible Diaphragm	Lateral	5/17/2024
49	FloorVibration.xlsb	Two-Way Floor Vibration Design Based on The Structural Engineer, Vol. 94-1, 2016	Lateral	5/17/2024
50	RetrofitWeakStory.xlsb	Retrofit Soft, Weak, or Open-Front Story Based on FEMA P807/ASCE 41-17	Lateral	5/17/2024
51	FourStoryMomentFrame.xlsb	Four Story Moment Frame Analysis using Finite Element Method	Lateral	5/17/2024
52	4-LevelShelving.xlsb	Lateral Loads of 4 Level Shelving, with Hilti Anchorage, Based on ASCE 7-22	Lateral	5/17/2024
53	BoxMomentFrame.xlsb	Box Moment Frame Analysis for Enhanced/New Wall Opening	Lateral	5/17/2024
54	High-RiseBuilding.xlsb	High-Rise Structural Embedded Design Based on 2022 CBC/2021 IBC	Lateral	5/17/2024
55	Bracing-FlexibleDiaphragm.xlsb	Flexible Diaphragm Design with Tension Rod Cross Bracing	Lateral	5/17/2024
56	BaseIsolatedBuilding.xlsb	Base Isolated Building Design Based on ASCE 7-22	Lateral	5/17/2024
57	CanopyWind.xlsb	Wind Load on Canopy Based on ASCE 7-22 Section 30.9	Lateral	5/17/2024
58	BinSiloWind.xlsb	Wind Analysis for Bin or Silo, Supported by Columns, Based on ASCE 7-22	Lateral	5/17/2024
59	CircularWind.xlsb	Wind Analysis for Circular Structure Based on ASCE 7-22	Lateral	5/17/2024
60	CircularDiaphragm.xlsb	Circular Flexible Diaphragm Analysis	Lateral	5/17/2024
61	NewRoofLoads.xlsb	Support Design, for New Loads on Existing Roof, Based on ASCE 41-17, AISC 360-22 & ACI 318-19	Lateral	5/17/2024
62	ReversedLateralFrame.xlsb	Reversed Lateral Frame Design Based on ASCE 41-17 & 7-22, AISC 360-22 & ACI 318-19	Lateral	5/17/2024
63	ArchRoofWind.xlsb	Wind Analysis for Open Arch Roof Based on ASCE 7-22	Lateral	5/17/2024
64	KBRF.xlsb	Knee Braced Moment Resisting Frame (KBRF) Analysis using Finite Element Method	Lateral	5/17/2024
65	Green-Roof.xlsb	Green Roof Seismic Analysis Based on 2021 IBC, ASCE 41-17 & ASCE 7-22	Lateral	5/17/2024
66	PoleMountClamp.xlsb	Pole Mount Clamp Design Based on ACI 318-19 & AISC 360-22	Lateral	5/17/2024
67	PondingDesign.xlsb	Ponding Design for Roof Beam Based on 2021 IBC, 2022 CBC, & AISC 360-22	Lateral	5/17/2024
68	Typ-Truss.xlsb	Typical Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
69	Fink-Truss.xlsb	Fink Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
70	Howe-Truss.xlsb	Howe Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
71	Attic-Truss.xlsb	Attic Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
72	Floor-Truss.xlsb	Flat Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
73	Scissor-Truss.xlsb	Scissor Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC	Lateral	5/17/2024
74	SolarCarport.xlsb	Solar Carport Pole & Footing Design Based on AISC 360-22, ACI 318-19, and 2021 IBC 1807.3	Lateral	5/17/2024
75	TensionOnly-BracedFrame.xlsb	Tension-Only Braced Frame Analysis using Finite Element Method	Lateral	5/17/2024
76	NonbuildingSeismic.xlsb	Nonbuilding Seismic Analysis Based on ASCE 7-22 Chapter 15	Lateral	5/17/2024
77	TwoSpanFrame.xlsb	Two Span Moment Frame Analysis using Finite Element Method	Lateral	5/17/2024
78	SetBackFrame.xlsb	Set Back Moment Frame Analysis using Finite Element Method	Lateral	5/17/2024
79	ContainerBuilding.xlsb	Container Building Lateral Design Based on 2021 IBC / 2022 CBC & ASCE 7-22	Lateral	5/17/2024
80	BlastLoads.xlsb	Determination of Blast Loads on Buildings Based on BIPS 06/FEMA 426, & UFC 3-340-02	Lateral	5/17/2024
1	Aluminum-I-WF-Capacity.xlsb	Aluminum I or WF Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)	Aluminum	5/17/2024
2	Aluminum-C-CS-Capacity.xlsb	Aluminum C or CS Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)	Aluminum	5/17/2024
3	Aluminum-RT-Capacity.xlsb	Aluminum RT Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)	Aluminum	5/17/2024
4	Aluminum-PIPE-Capacity.xlsb	Aluminum PIPE Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)	Aluminum	5/17/2024
5	StructuralGlass.xlsb	Glass Wall/Window/Stair Design, Based on ASTM E1300, using Finite Element Method	Aluminum	5/17/2024
6	P-Delta-Effect.xlsb	P-Delta Effect Analysis by Finite Element Method	Aluminum	5/17/2024
7	CopperPipe.xlsb	Copper Pipe Design using Finite Element Method	Aluminum	5/17/2024
1	TwoWaySlab.xlsb	Two-Way Slab Design Based on ACI 318-19 using Finite Element Method	Concrete	5/17/2024
2	VoidedBiaxialSlabs.xlsb	Voided Two-Way Slab Design Based on ACI 318-19	Concrete	5/17/2024
3	AnchorageToConcrete.xlsb	Base Plate and Group Anchors Design Based on ACI 318-19 & AISC 360-22	Concrete	5/17/2024
4	AnchorageToPedestal.xlsb	Anchorage to Pedestal Design Based on ACI 318-19 & AISC 360-22	Concrete	5/17/2024
5	CircularColumn.xlsb	Circular Column Design Based on ACI 318-19	Concrete	5/17/2024
6	ConcreteColumn.xlsb	Concrete Column Design Based on ACI 318-19	Concrete	5/17/2024
7	SuperCompositeColumn.xlsb	Super Composite Column Design Based on AISC 360-22 & ACI 318-19	Concrete	5/17/2024
8	SpecialShearWall-CBC.xlsb	Special Concrete Shear Wall Design Based on ACI 318-19 & 2022 CBC Chapter A	Concrete	5/17/2024
9	OrdinaryShearWall.xlsb	Ordinary Concrete Shear Wall Design Based on ACI 318-19	Concrete	5/17/2024
10	ConcretePool.xlsb	Concrete Pool Design Based on ACI 318-19	Concrete	5/17/2024
11	Corbel.xlsb	Corbel Design Based on IBC 09 / ACI 318-19	Concrete	5/17/2024
12	CouplingBeam.xlsb	Coupling Beam Design Based on ACI 318-19	Concrete	5/17/2024
13	DeepBeam.xlsb	Deep Beam Design Based on ACI 318-19	Concrete	5/17/2024
14	Non-DeepBeam.xlsb	Typical Member Section (Non Deep Beam) Design Based on ACI 318-19	Concrete	5/17/2024
15	DevelopmentSpliceConcrete.xlsb	Development & Splice of Reinforcement Based on ACI 318-19	Concrete	5/17/2024
16	EquipmentMounting.xlsb	Design for Equipment Anchorage Based on 2021 IBC & 2022 CBC Chapter A	Concrete	5/17/2024
17	ExistingShearWall.xlsb	Verify Existing Concrete Shear Wall Based on ASCE 41-17 / 2022 CBC & 2021 IBC	Concrete	5/17/2024
18	Friction.xlsb	Shear Friction Reinforcing Design Based on ACI 318-19	Concrete	5/17/2024
19	PipeConcreteColumn.xlsb	Pipe Concrete Column Design Based on ACI 318-19	Concrete	5/17/2024
20	PT-ConcreteFloor.xlsb	Design of Post-Tensioned Concrete Floor Based on ACI 318-19	Concrete	5/17/2024
21	Punching.xlsb	Slab Punching Design Based on ACI 318-19	Concrete	5/17/2024
22	Slab.xlsb	Concrete Slab Perpendicular Flexure & Shear Capacity Based on ACI 318-19	Concrete	5/17/2024
23	VoidedSectionCapacity.xlsb	Voided Section Design Based on ACI 318-19	Concrete	5/17/2024
24	DiaphragmShear.xlsb	Concrete Diaphragm in-plane Shear Design Based on ACI 318-19	Concrete	5/17/2024
25	SMRF-ACI.xlsb	Seismic Design for Special Moment Resisting Frame Based on ACI 318-19	Concrete	5/17/2024
26	SpecialShearWall-IBC.xlsb	Special Reinforced Concrete Shear Wall Design Based on ACI 318-19 & 2021 IBC	Concrete	5/17/2024
27	SuspendedAnchorage.xlsb	Suspended Anchorage to Concrete Based on 2021 IBC & 2022 CBC	Concrete	5/17/2024
28	TiltupPanel.xlsb	Tilt-up Panel Design based on ACI 318-19	Concrete	5/17/2024
29	Multi-StoryTilt-Up.xlsb	Multi-Story Tilt-Up Wall Design Based on ACI 318-19	Concrete	5/17/2024
30	WallPier.xlsb	Wall Pier Design Based on 2022 CBC & 2021 IBC	Concrete	5/17/2024
31	BeamPenetration.xlsb	Design for Concrete Beam with Penetration Based on ACI 318-19	Concrete	5/17/2024
32	ColumnSupportingDiscontinuous.xlsb	Column Supporting Discontinuous System Based on ACI 318-19	Concrete	5/17/2024
33	PlateShellElement.xlsb	Plate/Shell Element Design Based on ACI 318-19	Concrete	5/17/2024
34	TransferDiaphragm-Concrete.xlsb	Concrete Diaphragm Design for a Discontinuity of Type 4 out-of-plane offset irregularity	Concrete	5/17/2024
35	Silo-Chimney-Tower.xlsb	Concrete Silo / Chimney / Tower Design Based on ASCE 7-22, ACI 318-19 & ACI 313-16	Concrete	5/17/2024
36	ConcreteBeam.xlsb	Concrete Beam Design, for New or Existing, Based on ACI 318-19	Concrete	5/17/2024
37	AnchorageWithCircularBasePlate.xlsb	Anchorage Design, with Circular Base Plate, Based on ACI 318-19 & AISC 360-22	Concrete	5/17/2024
38	DirectCompositeBeam.xlsb	Composite Beam/Collector Design, without Metal Deck, Based on AISC 360-22 & ACI 318-19	Concrete	5/17/2024
39	CompositeMomentConnection.xlsb	Composite Moment Connection Design Based on ACI 318-19	Concrete	5/17/2024
40	MetricBars.xlsb	Flexural & Axial Design for Custom Metric Bars Based on Linear Distribution of Strain (ACI 318-19)	Concrete	5/17/2024
41	EnhanceExistingBeam.xlsb	Existing Concrete Beam Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19	Concrete	5/17/2024
42	EnhanceExistingFloor.xlsb	Existing Concrete Floor Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19	Concrete	5/17/2024
43	BearingWall-ICF.xlsb	Bearing Wall Design of Insulated Concrete Form (ICF) Based on ACI 318-19 & 2021 IBC	Concrete	5/17/2024
44	Lintel-ICF.xlsb	Lintel Design of Insulated Concrete Form (ICF) Based on ACI 318-19 & 2021 IBC	Concrete	5/17/2024
45	ConnectionToWall.xlsb	Design for Connection to Wall Based on AISC 360-22 & ACI 318-19	Concrete	5/17/2024
46	IrregularSection.xlsb	Irregular Section Design of Concrete Beam/Column Based on & ACI 318-19	Concrete	5/17/2024

47	CoupledShearWalls.xlsb	Coupled Shear Walls Design Based on ASCE 7-22 & ACI 318-19	Concrete	5/17/2024
48	ConcreteStair.xlsb	Concrete Stair Design Based on 2021 IBC & ACI 318-19	Concrete	5/17/2024
49	SlabOnWall.xlsb	Design for Two-Way Concrete Slab on Wall Based on ACI 318-19 using Finite Element Method	Concrete	5/17/2024
1	BeamConnection.xlsb	Beam Connection Design Based on AISC 360-22	Steel	5/17/2024
2	AngleCapacity.xlsb	Angle Steel Member Capacity Based on AISC 360-22	Steel	5/17/2024
3	HSS-WF-Capacity.xlsb	Tube, Pipe, or WF Member Capacity Based on AISC 360-22	Steel	5/17/2024
4	MetalStuds.xlsb	Metal Member Design Based on AISI S100-07/SI-10 (2021 IBC) & ESR-3064P	Steel	5/17/2024
5	SMRF-CBC.xlsb	Seismic Design for Special Moment Resisting Frames Based on 2022 CBC	Steel	5/17/2024
6	SCBF-Parallel.xlsb	Seismic Design for Special Concentrically Braced Frames Based on CBC/IBC & AISC 341-22	Steel	5/17/2024
7	SCBF-Perpendicular.xlsb	Bracing Connection Design, with Perpendicular Gusset, Based on CBC/IBC & AISC 341-22	Steel	5/17/2024
8	ColumnAboveBeam.xlsb	Connection Design for Column above Beam, Based on AISC Manual & AISC 360-22	Steel	5/17/2024
9	BeamGravity.xlsb	Steel Gravity Beam Design Based on AISC 360-22	Steel	5/17/2024
10	BeamWithTorsion.xlsb	WF Simply Supported Beam Design with Torsional Loading Based on AISC 360-22	Steel	5/17/2024
11	HSS-Torsion.xlsb	HSS (Tube, Pipe) Member Design with Torsional Loading Based on AISC 360-22	Steel	5/17/2024
12	FixedBoltedJoint.xlsb	Fixed Bolted Joint, with Beam Sitting on Top of Column, Based on AISC 358-22 8ES/4ES & FEMA-350	Steel	5/17/2024
13	BraceConnection.xlsb	Typical Bracing Connection Capacity Based on AISC 360-22	Steel	5/17/2024
14	BRBF.xlsb	Buckling-Restrained Braced Frames Based on AISC 360-22 & AISC 341-22	Steel	5/17/2024
15	BSEP-SMF.xlsb	Bolted Seismic Moment Connection Based on AISC 341-22, 358-22, 360-22 & FEMA-350	Steel	5/17/2024
16	BoltedMomentConnection.xlsb	Bolted Non-Seismic Moment Connection Based on AISC 341-22, 358-22, 360-22 & FEMA-350	Steel	5/17/2024
17	ChannelCapacity.xlsb	Channel Steel Member Capacity Based on AISC 360-22	Steel	5/17/2024
18	CompositeCollectorBeam.xlsb	Composite Collector Beam with Seismic Loads Based on 2022 CBC / 2021 IBC	Steel	5/17/2024
19	CompositeFloorBeam.xlsb	Composite Beam Design Based on AISC Manual 9th	Steel	5/17/2024
20	CompositeFloorBeamWithCantilever.xlsb	Composite Beam Design Based on AISC 360-22 / 2021 IBC / 2022 CBC	Steel	5/17/2024
21	CompositeFloorGirder.xlsb	Composite Girder Design Based on AISC 360-22 / 2021 IBC / 2022 CBC	Steel	5/17/2024
22	DragConnection.xlsb	Drag Connection Based on AISC 360-22 & AISC 341-22	Steel	5/17/2024
23	DragForcesforBraceFrame.xlsb	Drag / Collector Forces for Brace Frame	Steel	5/17/2024
24	EBF-CBC.xlsb	Seismic Design for Eccentrically Braced Frames Based on 2022 CBC & AISC 341-22	Steel	5/17/2024
25	EBF-IBC.xlsb	Seismic Design for Eccentrically Braced Frames Based on 2021 IBC & AISC 341-22	Steel	5/17/2024
26	EnhancedCompositeBeam.xlsb	Enhanced Composite Beam Design Based on AISC 360-22 / 2021 IBC / 2022 CBC	Steel	5/17/2024
27	EnhancedSteelBeam.xlsb	Enhanced Steel Beam Design Based on AISC 360-22	Steel	5/17/2024
28	ExteriorMetalStudWall.xlsb	Exterior Metal Stud Wall Design Based on AISI S100-07/SI-10 & ESR-3064P	Steel	5/17/2024
29	FloorDeck.xlsb	Depressed Floor Deck Capacity (Non-Composite)	Steel	5/17/2024
30	GussetGeometry.xlsb	Gusset Plate Dimensions Generator	Steel	5/17/2024
31	MetalShearWall.xlsb	Metal Shear Wall Design Based on AISI S100-07/SI-10, ER-5762 & ESR-3064P	Steel	5/17/2024
32	MetalShearWallOpening.xlsb	Metal Shear Wall with an Opening Based on AISI S100-07/SI-10, ER-5762 & ESR-3064P	Steel	5/17/2024
33	Metal-Z-Purlins.xlsb	Metal Z-Purlins Design Based on AISI S100-07/SI-10	Steel	5/17/2024
34	OCBF-CBC.xlsb	Ordinary Concentrically Braced Frames Based on 2022 CBC & AISC 341-22	Steel	5/17/2024
35	OCBF-IBC.xlsb	Ordinary Concentrically Braced Frames Based on 2021 IBC & AISC 341-22	Steel	5/17/2024
36	CantileverFrame.xlsb	Web-Tapered Cantilever Frame Design Based on AISC-ASD 9th, Appendix F	Steel	5/17/2024
37	OMRF-CBC.xlsb	Intermediate/Ordinary Moment Resisting Frames Based on 2022 CBC	Steel	5/17/2024
38	OMRF-IBC.xlsb	Intermediate/Ordinary Moment Resisting Frames Based on 2021 IBC	Steel	5/17/2024
39	PlateGirder.xlsb	Plate Girder Design Based on AISC 360-22	Steel	5/17/2024
40	RectangularSection.xlsb	Rectangular Section Member Design Based on AISC 360-22	Steel	5/17/2024
41	RoofDeck.xlsb	Design of 1 1/2" Type "B" Roof Deck Based on ICBO ER-2078P	Steel	5/17/2024
42	BasePlate.xlsb	Base Plate Design Based on AISC 360-22	Steel	5/17/2024
43	SMRF-IBC.xlsb	Special Moment Resisting Frames Based on 2021 IBC, AISC 341-22 & AISC 358-22	Steel	5/17/2024
44	SPSW.xlsb	Seismic Design for Special Plate Shear Wall Based on AISC 341-22 & AISC 360-22	Steel	5/17/2024
45	SteelColumn.xlsb	Steel Column Design Based on AISC 360-22	Steel	5/17/2024
46	SteelStair.xlsb	Steel Stair Design Based on AISC 360-22	Steel	5/17/2024
47	TripleW-Shapes.xlsb	Simply Supported Member of Triple W-Shapes Design Based on AISC 360-22	Steel	5/17/2024
48	PortalFrame.xlsb	Portal Frame Analysis using Finite Element Method	Steel	5/17/2024
49	WebTaperedPortal.xlsb	Web Tapered Portal Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25	Steel	5/17/2024
50	WebTaperedFrame.xlsb	Web Tapered Frame Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25	Steel	5/17/2024
51	WebTaperedGirder.xlsb	Web Tapered Girder Design Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25	Steel	5/17/2024
52	WeldConnection.xlsb	Weld Connection Design Based on AISC 360-22	Steel	5/17/2024
53	WF-Opening.xlsb	Check Capacity of WF Beam at Opening Based on AISC 360-22	Steel	5/17/2024
54	HSS-Opening.xlsb	Check Capacity of HSS Tube Beam at Opening Based on AISC 360-22	Steel	5/17/2024
55	MomentAcrossGirder.xlsb	Design for Fully Restrained Moment Connection across Girder Based on AISC 360-22	Steel	5/17/2024
56	BeamSplice.xlsb	Beam Bolted Splice Design Based on AISC 360-22	Steel	5/17/2024
57	FilledCompositeColumn.xlsb	Filled Composite Column Design Based on AISC 360-22 & ACI 318-19	Steel	5/17/2024
58	CellularBeam.xlsb	Cellular Beam Design Based on AISC 360-22	Steel	5/17/2024
59	DoubleAngleCapacity.xlsb	Double Angle Capacity Based on AISC 360-22	Steel	5/17/2024
60	T-ShapeCapacity.xlsb	T-Shape Member Capacity Based on AISC 360-22	Steel	5/17/2024
61	CantileverColumn.xlsb	Cantilever Column & Footing Design Based on AISC 360-22, ACI 318-19, and IBC 1807.3	Steel	5/17/2024
62	Truss-Metal.xlsb	Light Gage Truss Design Based on AISI S100-07/SI-10 & ESR-3064P	Steel	5/17/2024
63	SleeveJointConnection.xlsb	Sleeve Joint Connection Design, for Steel Cell Tower / Sign, Based on AISC 360-22	Steel	5/17/2024
64	MomentToColumnWeb.xlsb	Moment Connection Design for Beam to Weak Axis Column Based on AISC 360-22	Steel	5/17/2024
65	ConXL.xlsb	Seismic Bi-axial Moment Frame Design Based on AISC 358-22 & ACI 318-19	Steel	5/17/2024
66	ThinCompositeBeam.xlsb	Thin Composite Beam/Collector Design Based on AISC 360-22 & ACI 318-19	Steel	5/17/2024
67	BoltConnection.xlsb	Bolt Connection Design Based on AISC 360-22	Steel	5/17/2024
68	SCCS-OCCS.xlsb	Cantilever Column System (SCCS/OCCS) Design Based on AISC 341-22, AISC 360-22 & ACI 318-19	Steel	5/17/2024
69	Non-PrismaticCompositeGirder.xlsb	Non-Prismatic Composite Girder Design Based on AISC 360-22 / 2022 CBC / 2021 IBC	Steel	5/17/2024
70	EndPlateConnection.xlsb	Endplate Splice Moment Connection Based on AISC 341-22, 358-22, 360-22 & FEMA-350	Steel	5/17/2024
71	Z-ProfileTreadRiser.xlsb	Flexure Capacity for Z-Profile Tread and Riser Based on AISC 360-22	Steel	5/17/2024
72	SC-WB.xlsb	Strong-Column Weak-Beam Design Based on AISC 341-22 and AISC 360-22	Steel	5/17/2024
73	C-PSW-CF.xlsb	Composite Plate Shear Wall Design Based on AISC 341-22 & ACI 318-19 - Concrete Filled (C-PSW/CF)	Steel	5/17/2024
74	MTBF.xlsb	Multi-Tiered Braced Frame Design Based on AISC 341-22	Steel	5/17/2024
75	SeismicColumn.xlsb	Filled Composite Column (FCC) Design for C-SMF/C-IMF/C-OCF Based on ASCE 7-22, AISC 341-22 & ACI 318-19	Steel	5/17/2024
76	SCBF-4-Story.xlsb	Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-22	Steel	5/17/2024
77	SCBF-3-Story.xlsb	Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-22	Steel	5/17/2024
78	SCBF-2-Story.xlsb	Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-22	Steel	5/17/2024
79	T-SMF.xlsb	Double-Tee Connection Design for SMF Based on AISC 341-22, 358-22, 360-22	Steel	5/17/2024
80	SteelCorbel.xlsb	Steel Corbel Design Based on AISC-ASD 9th, Appendix F	Steel	5/17/2024
81	Stiffeners.xlsb	Proportions and Stiffeners Design for I-Shaped Member Based on AISC 360-22	Steel	5/17/2024
82	MomentToTubeColumn.xlsb	Moment Connection Design for Beam to Tube Column Based on AISC 360-22	Steel	5/17/2024
83	Prestressed-Steel-Arch.xlsb	Prestressed Steel Arch Design Based on 2021 IBC/2022 CBC & AISC 360-22	Steel	5/17/2024
84	RoofBentGirder.xlsb	Web-Tapered Roof Girder Design Based on AISC-ASD 9th Appendix F and 2021 IBC/2022 CBC 1605	Steel	5/17/2024

1	*Arch-Bridge.xlsb*	Arch Bridge Analysis using Finite Element Method	Bridge	5/17/2024
2	*Bridge-ConcreteGirder.xlsb*	Prestressed Concrete Girder Design for Bridge Structure Based on AASHTO 17th Edition & ACI 318-19	Bridge	5/17/2024
3	*Bridge-ConcreteColumn.xlsb*	Bridge Column Design Based on AASHTO 17th & ACI 318-19	Bridge	5/17/2024
4	*Bridge-BoxSection.xlsb*	Bridge Design for Prestressed Concrete Box Section Based on AASHTO 17th Edition & ACI 318-19	Bridge	5/17/2024
5	*ConcreteTunnel.xlsb*	Concrete Tunnel Design Based on AASHTO-17th & ACI 318-19	Bridge	5/17/2024
6	*DoubleTee.xlsb*	Prestressed Double Tee Design Based on AASHTO 17th Edition & ACI 318-19	Bridge	5/17/2024
7	*BoxCulvert.xlsb*	Concrete Box Culvert Design Based on AASHTO 17th Edition & ACI 318-19	Bridge	5/17/2024
8	*SteelRoadPlate.xlsb*	Steel Road Plate Design Based on AASHTO 17th Edition & AISC 360-22 using Finite Element Method	Bridge	5/17/2024
9	*FlangeTaperedGirder.xlsb*	Flange Tapered Plate Girder Design Based on AISC 360-22	Bridge	5/17/2024
10	*PrestressedConcreteCircularHollowSection.xlsb*	Prestressed Concrete Circular Hollow Pole/Pile Design Based on ACI 318-19 & AASHTO 17th	Bridge	5/17/2024
11	*Falsework.xlsb*	Falsework Design for Steel Girder Bridge Based on NDS 2018 & AASHTO 17th	Bridge	5/17/2024
12	*PolygonCapacity.xlsb*	Polygon Section Member (Tubular Steel Pole) Design Based on ASCE 48-14	Bridge	5/17/2024
13	*Truss-Bridge.xlsb*	Truss Analysis using Finite Element Method	Bridge	5/17/2024
14	*ConcreteWall-Mount.xlsb*	Mounting Design on Concrete Wall/Tunnel Based on FEMA E-74, 2021 IBC, and 2022 CBC Chapter A	Bridge	5/17/2024
15	*VehicularBarrierWall.xlsb*	Vehicular Barrier Wall Design Based on ASCE 7-22 & ACI 318-19	Bridge	5/17/2024
16	*FootbridgeVibration.xlsb*	Footbridge Vibration Design Based on The Structural Engineer, Vol. 94-1, 2016	Bridge	5/17/2024
17	*MSE-Wall.xlsb*	Design of Mechanically Stabilized Earth Wall Based on AASHTO/2021 IBC & TMS 402-16	Bridge	5/17/2024
18	*ElastomericBearingBridge.xlsb*	Elastomeric Bearing Bridge Analysis using Finite Element Method	Bridge	5/17/2024
19	*CableStructure.xlsb*	Cable Structure Design Based on ASCE 19-10 & AASHTO 17th	Bridge	5/17/2024
20	*WildlifeCrossing.xlsb*	Wildlife Crossing Design Based on AASHTO-17th & ACI 318-19	Bridge	5/17/2024
21	*ArchBridgeLimits.xlsb*	Arch Bridge Limits Analysis Based on ACI 318-19, AISC 360-22 & AASHTO-17th	Bridge	5/17/2024
22	*BollardAnchorage.xlsb*	Bollard/Flagpole Anchorage Design Based on ACI 318-19	Bridge	5/17/2024
23	*Repairing-Bridge.xlsb*	Bridge Design and Repair, by Added New Arch, using Finite Element Method	Bridge	5/17/2024
24	*Curved-Pipe-Tube.xlsb*	Curved Steel HSS (Tube, Pipe) Member Design Based on AISC 360-22	Bridge	5/17/2024
25	*ArticulatingConcreteBlock.xlsb*	Articulating Concrete Block (ACB) Design Based on NCMA ACB Manual 2nd Edition	Bridge	5/17/2024
26	*HybridRetaining.xlsb*	Hybrid Retaining Structural Design Based on 2021 IBC/AASHTO, TMS 402-16 & AISC 360-22	Bridge	5/17/2024
27	*SuperCompositeGirder.xlsb*	Super Composite Girder Design Based on 2022 CBC/2021 IBC, AISC 360-22 & ACI 318-19	Bridge	5/17/2024
28	*TowerDrift.xlsb*	Tower Drift Analysis for Cable Stayed Bridge by Finite Element Method	Bridge	5/17/2024
29	*UndergroundVault.xlsb*	Masonry Vault Design Based on 2021 IBC & TMS 402-16	Bridge	5/17/2024
30	*HeavyHaulRailway.xlsb*	Subgrade Design for Heavy Haul Railway on Soft Soil Based on AASHTO & ACI 318-19	Bridge	5/17/2024
31	*UndergroundRectangularBox.xlsb*	Underground Rectangular Section Design using Finite Element Method	Bridge	5/17/2024
32	*GabionRetainingWall.xlsb*	Design of Gabion Retaining Wall Based on AASHTO 17th & 2021 IBC	Bridge	5/17/2024
33	*SteelSheetPiling.xlsb*	Steel Retaining Wall Design Based on 2021 IBC / 2022 CBC & AISC 360-22	Bridge	5/17/2024
34	*CantileverDiaphragm.xlsb*	Cantilever Diaphragm Analysis using Tension-Only Braced Frame	Bridge	5/17/2024
35	*VehicleSecurityBarrier.xlsb*	Vehicle Security Barrier Design Based on AASHTO-17th, 2021 IBC, AISC 341-22, & ACI 318-19	Bridge	5/17/2024
36	*FiberWrapColumn.xlsb*	Column Repair Design of Carbon Fiber Wrap Based on 2021 IBC, ACI 318-19, & AASHTO-17th	Bridge	5/17/2024
37	*PostCompressionStructure.xlsb*	Post-Compression Structure Analysis using Finite Element Method	Bridge	5/17/2024
38	*HybridSuspensionBridge.xlsb*	Hybrid Suspension Bridge Design Based on ASCE 19-10 & AASHTO 17th	Bridge	5/17/2024
39	*HeavyLoadsSlab.xlsb*	Analysis of Concrete Floor Slabs on Grade Subjected to Heavy Loads Based on AASHTO/ACI 318-19	Bridge	5/17/2024
40	*SeismicSlopeStability.xlsb*	Seismic Slope Stability Analysis Based on Mononobe-Okabe Method, AASHTO 17th & 2021 IBC	Bridge	5/17/2024
41	*FlexiblePipeCover.xlsb*	Flexible Pipe Cover Design Based on AASHTO / NCSPA Design Manual	Bridge	5/17/2024
42	*CurvedRigidFooting.xlsb*	Curved Rigid Footing Design Based on ASCE 41-17 & ACI 318-19	Bridge	5/17/2024
43	*CSP-DeepFoundation.xlsb*	CSP Deep Foundation Design Based on 2021 IBC, ACI 318-19 & AASHTO 17th	Bridge	5/17/2024

STRUCTURAL DESIGN SOFTWARE

This web site provides structural design software which created using Microsoft Windows Excel 2010/2013 or 2016 Office 365. Each spreadsheet contains formulas, reference code sections, and graphic drawings. The software are nice and easy on all Win Tablet/Phone. The analysis results can be copied and pasted to AutoCAD. The Example is intended for re-use and is loaded with floating comments as well as ActiveX pull-down menus for variable choices.

The top benefit of the software is VAB Events, which is the same as blockchain. (The blockchain doesn't disrupt databases, but it disrupts how databases get SYNCHRONIZED between each other.) When purchaser input a single cell value, all section forces/analysis CON-CURRENTLY balanced/updated per structural mechanics.

It is free to download, by click software name, for limited version (demo only). For professional version (xlsb/xlsm filename extension), a Package of all ***392*** listed software, the normal price is $1850. (We sell entire Package Licenses, not individual.)

(What's New?) **(User's Book)** **(Unit Conversions)**

Aluminum & Glass Design

1	Aluminum I or WF Member	Aluminum I or WF Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)
2	Aluminum C or CS Member	Aluminum C or CS Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)
3	Aluminum RT Member	Aluminum RT Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)
4	Aluminum PIPE Member	Aluminum PIPE Member Capacity Based on Aluminum Design Manual 2015 (ADM-I)
5	Structural Glass	Glass Wall/Window/Stair Design, Based on ASTM E1300, using Finite Element Method
6	P-Delta Effect	P-Delta Effect Analysis by Finite Element Method
7	Copper Pipe	Copper Pipe Design using Finite Element Method

Concrete Design Group

1	Custom Metric Bars	Flexural & Axial Design for Custom Metric Bars Based on Linear Distribution of Strain (ACI 318-19)
2	Voided Biaxial Slab	Voided Two-Way Slab Design Based on ACI 318-19
3	Anchorage to Concrete	Base Plate and Group Anchors Design Based on ACI 318-19 & AISC 360-22
4	Anchorage to Pedestal	Anchorage to Pedestal Design Based on ACI 318-19 & AISC 360-22
5	Circular Column	Circular Column Design Based on ACI 318-19
6	Concrete Column	Concrete Column Design Based on ACI 318-19
7	Super Composite Column	Super Composite Column Design Based on AISC 360-22 & ACI 318-19
8	Special Shear Wall - CBC	Special Concrete Shear Wall Design Based on ACI 318-19 & 2022 CBC Chapter A
9	Ordinary Shear Wall	Ordinary Concrete Shear Wall Design Based on ACI 318-19
10	Concrete Pool	Concrete Pool Design Based on ACI 318-19
11	Corbel	Corbel Design Based on 2021 IBC / ACI 318-19
12	Coupling Beam	Coupling Beam Design Based on ACI 318-19
13	Deep Beam	Deep Beam Design Based on ACI 318-19
14	Non Deep Beam	Typical Member Section (Non Deep Beam) Design Based on ACI 318-19
15	Equipment Mounting	Design for Equipment Anchorage Based on ASCE 7-22 Supplement 1 & 2022 CBC Chapter A
16	Existing Shear Wall	Verify Existing Concrete Shear Wall Based on ASCE 41-17 / 2022 CBC / 2021 IBC
17	Friction	Shear Friction Reinforcing Design Based on ACI 318-19
18	Pipe Concrete Column	Pipe Concrete Column Design Based on ACI 318-19
19	PT-Concrete Floor	Design of Post-Tensioned Concrete Floor Based on ACI 318-19

20	Punching	Slab Punching Design Based on ACI 318-19
21	Concrete Slab	Concrete Slab Perpendicular Flexure & Shear Capacity Based on ACI 318-19
22	Voided Section Capacity	Voided Section Design Based on ACI 318-19
23	Concrete Diaphragm	Concrete Diaphragm in-plane Shear Design Based on ACI 318-19
24	SMRF - ACI	Seismic Design for Special Moment Resisting Frame Based on ACI 318-19
25	Special Shear Wall - IBC	Special Reinforced Concrete Shear Wall Design Based on ACI 318-19 & 2021 IBC
26	Suspended Anchorage	Suspended Anchorage to Concrete Based on 2021 IBC & 2022 CBC
27	Tiltup Panel	Tilt-up Panel Design based on ACI 318-19
28	Wall Pier	Wall Pier Design Based on 2022 CBC & 2021 IBC
29	Beam Penetration	Design for Concrete Beam with Penetration Based on ACI 318-19
30	Column Supporting Discontinuous	Column Supporting Discontinuous System Based on ACI 318-19
31	Plate Shell Element	Plate/Shell Element Design Based on ACI 318-19
32	Transfer Diaphragm - Concrete	Concrete Diaphragm Design for a Discontinuity of Type 4 out-of-plane offset irregularity
33	Silo/Chimney/Tower Design	Concrete Silo / Chimney / Tower Design Based on ASCE 7-22, ACI 318-19 & ACI 313-16
34	Concrete Beam	Concrete Beam Design, for New or Existing, Based on ACI 318-19
35	Anchorage with Circular Base Plate	Anchorage Design, with Circular Base Plate, Based on ACI 318-19 & AISC 360-22
36	Direct Composite Beam	Composite Beam/Collector Design, without Metal Deck, Based on AISC 360-22 & ACI 318-19
37	Multi-Story Tilt-Up	Multi-Story Tilt-Up Wall Design Based on ACI 318-19
38	Composite Moment Connection	Composite Moment Connection Design Based on ACI 318-19
39	Concrete Development & Splice	Development & Splice of Reinforcement Based on ACI 318-19
40	Two Way Slab	Two-Way Slab Design Based on ACI 318-19 using Finite Element Method
41	Existing Beam Enhancement	Existing Concrete Beam Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19
42	Existing Floor Enhancement	Existing Concrete Floor Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19
43	Bearing Wall for ICF	Bearing Wall Design of Insulated Concrete Form (ICF) Based on ACI 318-19 & 2021 IBC
44	Lintel for ICF	Lintel Design of Insulated Concrete Form (ICF) Based on ACI 318-19 & 2021 IBC
45	Gusset To Wall	Design for Connection to Wall Based on AISC 360-22 & ACI 318-19
46	Irregular Section	Irregular Section Design of Concrete Beam/Column Based on & ACI 318-19
47	Coupled Shear Walls	Coupled Shear Walls Design Based on ASCE 7-22 & ACI 318-19
48	Concrete Stair	Concrete Stair Design Based on 2021 IBC & ACI 318-19
49	Slab on Wall	Design for Two-Way Concrete Slab on Wall Based on ACI 318-19 using Finite Element Method

Foundation Design Group

1	Slope Stability Analysis	Slope (Wild Fired Mountain) Stability Analysis Based on AASHTO 17th & 2021 IBC
2	Basement Concrete Wall	Basement Concrete Wall Design Based on ACI 318-19
3	Flagpole	Flagpole Footing Design Based on 2021 IBC Chapter 18
4	Masonry Retaining Wall	Masonry Retaining / Fence Wall Design Based on TMS 402-16 & ACI 318-19
5	Concrete Retaining Wall	Concrete Retaining Wall Design Based on ACI 318-19
6	Masonry-Concrete Retaining Wall	Retaining Wall Design, for Masonry Top & Concrete Bottom, Based on TMS 402-16 & ACI 318-19
7	Concrete Pier	Concrete Pier (Isolated Deep Foundation) Design Based on ACI 318-19
8	Concrete Pile	Drilled Cast-in-place Pile Design Based on ACI 318-19
9	Pile Caps	Pile Cap Design for 4, 3, 2-Piles Pattern Based on ACI 318-19
10	Pile Cap Balanced Loads	Determination of Pile Cap Balanced Loads and Reactions
11	Conventional Slab on Grade	Design of Conventional Slabs on Expansive & Compressible Soil Grade Based on ACI 360
12	Caisson	Caisson Design Based on 2021 IBC & 2022 CBC
13	Eccentric Footing	Eccentric Footing Design Based on ACI 318-19
14	Basement Masonry Wall	Basement Masonry Wall Design Based on TMS 402-16
15	Basement Column	Basement Column Supporting Lateral Resisting Frame Based on ACI 318-19

16	MRF-Grade Beam	Grade Beam Design for Moment Resisting Frame Based on ACI 318-19
17	Brace Grade Beam	Grade Beam Design for Brace Frame Based on ACI 318-19
18	Grade Beam	Two Pads with Grade Beam Design Based on ACI 318-19 & AISC 360-22
19	Circular Footing	Circular Footing Design Based on ACI 318-19
20	Combined Footing	Combined Footing Design Based on ACI 318-19
21	Boundary Spring Generator	Mat Boundary Spring Generator
22	Deep Footing	Deep Footing Design Based on ACI 318-19
23	Footing at Piping	Design of Footing at Piping Based on ACI 318-19
24	Irregular Footing Soil Pressure	Soil Pressure Determination for Irregular Footing
25	PAD Footing	Pad Footing Design Based on ACI 318-19
26	Plain Concrete Footing	Plain Concrete Footing Design Based on ACI 318-19
27	Restrained Retaining Wall	Restrained Retaining Masonry & Concrete Wall Design Based on TMS 402 & ACI 318
28	Retaining Wall for DSA /OSHPD	Retaining Wall Design Based on 2022 CBC Chapter A
29	Tank Footing	Tank Footing Design Based on ACI 318-19, ASCE 7-22 & AWWA D103-19
30	Tank Anchorage	Tank Anchorage Design Based on ACI 318-19 & AWWA D103-19
31	Under Ground Well	Under Ground Well Design Based on ACI 350-06 & ACI 318-19
32	Stud Bearing Wall Footing	Footing Design for Stud Bearing Wall Based on 2021 IBC / ACI 318-19
33	Wall Footing	Footing Design of Shear Wall Based on ACI 318-19
34	Fixed Moment Condition	Fixed Moment Condition Design Based on ACI 318-19
35	Flood Way	Concrete Floodway Design Based on ACI 350-06 & ACI 318-19
36	Lateral Earth Pressure	Lateral Earth Pressure of Rigid Wall Based on AASHTO 17th & 2021 IBC
37	Shoring	Sheet Pile Wall Design Based on 2021 IBC / 2022 CBC / ACI 318-19
38	Composite Element Durability	Composite Element (Tension Pile) Design Based on AISC 360-22 & ACI 318-19
39	Seismic Earth Pressure	Seismic Earth Pressure of Deep Stiff Wall Based on FEMA P-750 & AASHTO/IBC
40	Free Standing Wall	Free Standing Masonry & Conctere Wall Design Based on TMS 402-16 & ACI 318-19
41	Rectangular Machine Footing	Rectangular Machine or Tank Footing Design Based on ACI 318-19
42	Tieback Wall	Sheet Pile Wall, with Tieback Anchors, Design Based on AASHTO (HB-17), 2021 IBC & ACI 318-19
43	Screw Piles	Screw Pile Design Based on 2021 IBC & AISC 360-22
44	PT-Slab on Ground	Design of PT Slabs on Expansive Soil Ground Based on PTI DC10.5-12 & PTI 3rd Edition
45	Under Footing Sewer	Underground Utilities Way Design Based on AASHTO-17th & 2021 IBC
46	Landslide Repair	Landslide Repair Design Based on 2021 IBC, ACI 318-19 & AASHTO 17th
47	Ring Foundation	Ring Foundation Design Based on 2021 IBC & ACI 318-19
48	Driven Pile	Driven Precast Concrete Pile Design Based on 2021 IBC & ACI 318-19
49	Equipment Footing	Foundation Design for Dynamic Equipment Based on ACI 351.3 & ACI 318-19
50	Counterfort Retaining Wall	Counterfort Retaining Wall Design Based on 2021 IBC & ACI 318-19
51	Retaining Wall Repair	Retaining Wall Repair Design Based on AASHTO/2021 IBC & TMS 402-16
52	Mat Foundation	RC Mat Slab Design Based on 2021 IBC, ACI 318-19, AASHTO 17th Edition & ACI 360
53	Bin Wall	Trapezoidal Loads Retaining Wall Design Based on ACI 318-19
54	Tree Root Foundation	Tree Root Foundation Design Based on AASHTO (HB-17), 2021 IBC & 2022 CBC
55	Elastic Strip Foundation	Elastic Strip Foundation Analysis using Finite Element Method Based on 2021 IBC
56	PT-Rebar Ground Slab	Design of PT Slabs with Rebar Stiffening Beam Based on ACI 318-19, PTI DC10.5-12 & PTI 3rd Edition
57	Temporary Footing for Rectangular Tank	Temporary Tank Footing Design Based on ACI 318-19
58	Sonotube Footing	Sonotube Footing Design Based on 2021 IBC, ASCE 7-22, & ACI 318-19
59	Rigid Footing Moment Capacity	Rigid Footing Moment Capacity Design Based on ASCE 41-17 & ACI 318-19

Lateral Analysis Group

1	Seismic - ASCE 7-22	Seismic Analysis Based on ASCE 7-22 (Equivalent Lateral Force Procedure & Modal Response Spectrum Analysis)
2	Shade Structure Wind	Wind Analysis for Shade Open Structure Based on ASCE 7-22, 10 & 05
3	Circular Structure Wind	Wind Analysis for Circular Structure Based on ASCE 7-22
4	Metal Pipe/Riser	MCE Level Seismic Design for Metal Pipe/Riser Based on ASCE 7-22 & AISI S100
5	Rigid Diaphragm	Rotation Analysis of Rigid Diaphragm Based on 2021 IBC / 2022 CBC (Why not Semi-Rigid?)
6	Flexible Diaphragm	Flexible Diaphragm Analysis
7	Two Story Moment Frame	Two Story Moment Frame Analysis using Finite Element Method
8	X - Braced Frame	X-Braced Frame Analysis using Finite Element Method
9	Open Structure Wind	Wind Analysis for Open Structure (Solar Panels) Based on ASCE 7-22, 10 & 05
10	Roof Screen/Equipment Wind	Wind Load, on Roof Screen / Roof Equipment, Based on ASCE 7-22, 10 & 05
11	Axial Roof Deck	Axial Capacity of 1 1/2" Type "B" Roof Deck Based on ICBO ER-2078P
12	Deformation Compatibility	Column Deformation Compatibility Design using Finite Element Method
13	Discontinuous Shear Wall	Discontinuous Shear Wall Analysis Using Finite Element Method
14	Flexible Diaphragm Opening	Flexible Diaphragm with an Opening Analysis
15	Hand Rail	Handrail Design Based on AISC 360-22 & ACI 318-19
16	Interior Wall Lateral Force	Interior Wall Lateral Forces Based on 2021 IBC / 2022 CBC
17	Lateral Frame Formulas	Lateral Frame Formulas
18	Live Load	Live Load Reduction Based on ASCE 7-22, 2021 IBC / 2022 CBC
19	New Roof Loads	Support Design, for New Loads on Existing Roof, Based on ASCE 41-17, AISC 360-22 & ACI 318-19
20	Wind - ASCE7-22	Wind Analysis Based on ASCE 7-22
21	Shear Wall Forces	Shear Wall Analysis for Shear Wall with Opening Using Finite Element Method
22	Shear Wall - New Opening	Relative Rigidity Determination for Shear Wall with New Opening
23	Shear Wall Rigidity	Rigidity for Shear Wall & Shear Wall with Opening Using Finite Element Method
24	Sign	Sign Design Based on AISC 360-22, ACI 318-19, and IBC 1807.3
25	Sign Wind	Wind Analysis for Freestanding Wall & Sign Based on ASCE 7-22, 10 & 05
26	Snow	Snow Load Analysis Based on ASCE 7-22, 10, 05, & UBC
27	Wall Lateral Force - CBC	Lateral Force for One-Story Wall Based on 2022 CBC
28	Wall Lateral Force - IBC	Lateral Force for One-Story Wall Based on 2022 CBC/2021 IBC
29	High-Rise Building	High-Rise Structural Embedded Design Based on 2022 CBC/2021 IBC
30	Wind Girt Deflection	Wind Girt Deflection Analysis of Wood, Metal Stud, and/or Steel Tube
31	Storage Racks	Lateral Loads of Storage Racks, with Hilti & Red Head Anchorage, Based on ASCE 7-22
32	Wind Alternate Method	Wind Analysis for Building with h < 60 ft, Based on 2021 IBC/ASCE 7-22
33	Ceiling Seismic Loads	Suspended Ceiling Seismic Loads Based on ASCE 7-22
34	Response Spectrum Generator	Earthquake Response Spectrum Generator
35	Tornado and Hurricane	Wind Analysis for Tornado and Hurricane Based on 2022 CBC/2021 IBC 423 & FEMA 361/320
36	Stiffness Matrix Generator	Stiffness Matrix Generator for Irregular Beam/Column
37	PT-Column Drift	Lateral Drift Mitigation for Cantilever Column (Monorail Column) using Post-Tensioning
38	Blast Mitigation	Blast/Explosion Deformation Mitigation for Gravity Column using Post-Tensioning
39	Wind - Solar Panels	Wind Design for Rooftop Solar Panels Based on ASCE 7-22
40	Wind - ASCE7-16	Wind Analysis Based on ASCE 7-16
41	Self-Centering Frame	Self-Centering Lateral Frame Design Based on ASCE 7-22, AISC 360-22 & ACI 318-19
42	General Beam	General Beam Analysis, including Lateral-Torsional Buckling Length
43	Trussed Tower Wind	Wind Analysis for Trussed Tower Based on ASCE 7-22
44	PT Lateral Frame	Post-Tensioned Lateral Frame Analysis using Finite Element Method
45	External PT Beam	Beam Strengthening Analysis Using External Post-Tensioning Systems
46	Later Drift Compatibility	Lateral Drift Compatibility Analysis using Finite Element Method
47	Sloped Diaphram Analysis	Seismic Analysis for Sloped Flexible Diaphragm
48	Floor Vibration	Two-Way Floor Vibration Design Based on The Structural Engineer, Vol. 94-1, 2016

49	Retrofit Weak Story	Retrofit Soft, Weak, or Open-Front Story Based on FEMA P807/ASCE 41-17 (LA Ordinance 183893 & 184081)
50	Four Story Moment Frame	Four Story Moment Frame Analysis using Finite Element Method
51	4 Level Shelving	Lateral Loads of 4 Level Shelving, with Hilti Anchorage, Based on ASCE 7-22
52	Box Moment Frame	Box Moment Frame Analysis for Enhanced/New Wall Opening
53	Seismic vs Wind	Three, Two, and One Story Comparison of Seismic and Wind Based on 2021 IBC / 2022 CBC
54	Bracing Flexible Diaphragm	Flexible Diaphragm Retrofit Design with Tension Rod Cross Bracing
55	Base Isolated Building	Base Isolated Building Design Based on ASCE 7-22
56	Canopy Wind	Wind Load on Canopy Based on ASCE 7-22 Section 30.9
57	Seismic - 2021 IBC	Seismic Analysis Based on 2021 IBC (Equivalent Lateral Force Procedure, ASCE 7-22)
58	Bin Silo Wind	Wind Analysis for Bin or Silo, Supported by Columns, Based on ASCE 7-22
59	Circular Diaphragm	Circular Flexible Diaphragm Analysis
60	Seismic - Single Family Dwellings	Seismic Analysis for Family Dwellings Based on 2021 IBC / 2022 CBC & ASCE 7-22
61	Reversed Lateral Frame	Reversed Lateral Frame Design Based on ASCE 41-17 & 7-22, AISC 360-22 & ACI 318-19 (LA Ordinance 183893 & 184081)
62	Arch Roof Wind	Wind Analysis for Open Arch Roof Based on ASCE 7-22
63	Knee Braced	Knee Braced Moment Resisting Frame (KBRF) Analysis using Finite Element Method
64	Green Roof	Green Roof Seismic Analysis Based on 2021 IBC, ASCE 41-17 & ASCE 7-22
65	Pole Mount Clamp	Pole Mount Clamp Design Based on ACI 318-19 & AISC 360-22
66	Ponding Design	Ponding Design for Roof Beam Based on 2021 IBC, 2022 CBC, & AISC 360-22
67	Typical Truss	Typical Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC
68	Fink Truss	Fink Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC
69	Howe Truss	Howe Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC
70	Attic Truss	Attic Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC
71	Floor Truss	Flat Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC
72	Scissor Truss	Scissor Truss Analysis by Finite Element Method Based on 2021 IBC / 2022 CBC
73	Solar Carport	Solar Carport Pole & Footing Design Based on AISC 360-22, ACI 318-19, and 2021 IBC 1807.3
74	Tension-Only Braced Frame	Tension-Only Braced Frame Analysis using Finite Element Method
75	Nonbuilding Seismic	Nonbuilding Seismic Analysis Based on ASCE 7-22 Chapter 15
76	Two Span Frame	Two Span Moment Frame Analysis using Finite Element Method
77	Set Back Frame	Set Back Moment Frame Analysis using Finite Element Method
78	Container Building	Container Building Lateral Design Based on 2021 IBC / 2022 CBC & ASCE 7-22
79	Blast Loads	Determination of Blast Loads on Buildings Based on BIPS 06/FEMA 426, & UFC 3-340-02

Infrastructure (Bridge) Design Group

1	Repairing Bridge	Bridge Design and Repair, by Added New Arch, using Finite Element Method
2	Bridge Concrete Column	Bridge Column Design Based on AASHTO 17th & ACI 318-19
3	Bridge Box Section	Bridge Design for Prestressed Concrete Box Section Based on AASHTO 17th Edition & ACI 318-19
4	Concrete Tunnel	Concrete Tunnel Design Based on AASHTO-17th & ACI 318-19
5	Double Tee	Prestressed Double Tee Design Based on AASHTO 17th Edition & ACI 318-19

6	Concrete Box Culvert	Concrete Box Culvert Design Based on AASHTO 17th Edition & ACI 318-19
7	Steel Road Plate	Steel Road Plate Design Based on AASHTO 17th Edition & AISC 360-22 using Finite Element Method
8	Flange Tapered Girder	Flange Tapered Plate Girder Design Based on AISC 360-22
9	Prestressed Concrete Pole/Pile	Prestressed Concrete Circular Hollow Pole/Pile Design Based on ACI 318-19 & AASHTO 17th
10	Falsework	Falsework Design for Steel Girder Bridge Based on NDS 2018 & AASHTO 17th
11	Polygon Capacity	Polygon Section Member (Tubular Steel Pole) Design Based on ASCE 48-14
12	Concrete Wall-Mount	Mounting Design on Concrete Wall/Tunnel Based on FEMA E-74, 2021 IBC, and 2022 CBC Chapter A
13	Truss Bridge	Truss Analysis using Finite Element Method
14	Bridge Concrete Girder	Prestressed Concrete Girder Design for Bridge Structure Based on AASHTO 17th Edition & ACI 318-19
15	Vehicular Barrier Wall	Vehicular Barrier Wall Design Based on ASCE 7-22 & ACI 318-19
16	Footbridge Vibration	Footbridge Vibration Design Based on The Structural Engineer, Vol. 94-1, 2016
17	MSE Wall	Design of Mechanically Stabilized Earth Wall Based on AASHTO/2021 IBC & TMS 402-16
18	Elastomeric Bearing Bridge	Elastomeric Bearing Bridge Analysis using Finite Element Method
19	Cable Structure	Cable Structure Design Based on ASCE 19-10 & AASHTO 17th
20	Arch Bridge	Arch Bridge Analysis using Finite Element Method
21	Arch Bridge Limits	Arch Bridge Limits Analysis Based on ACI 318-19, AISC 360-22 & AASHTO-17th
22	Bollard Anchorage	Bollard/Flagpole Anchorage Design Based on ACI 318-19
23	Wildlife Crossing	Wildlife Crossing Design Based on AASHTO-17th & ACI 318-19
24	Curved Pipe Tube	Curved Steel HSS (Tube, Pipe) Member Design Based on AISC 360-22
25	Articulating Concrete Block	Articulating Concrete Block (ACB) Design Based on NCMA ACB Manual 2nd Edition
26	Hybrid Retaining	Hybrid Retaining Structural Design Based on 2021 IBC/AASHTO, TMS 402-16 & AISC 360-22
27	Super Composite Girder	Super Composite Girder Design Based on 2022 CBC / 2021 IBC, AISC 360-22 & ACI 318-19
28	Tower Drift	Tower Drift Analysis for Cable Stayed Bridge by Finite Element Method
29	Underground Vault	Masonry Vault Design Based on 2021 IBC & TMS 402-16
30	Railway Subgrade	Subgrade Design for Heavy Haul Railway on Soft Soil Based on AASHTO & ACI 318-19
31	Underground Rectangular Box	Underground Rectangular Section Design using Finite Element Method
32	Gabion Retaining Wall	Design of Gabion Retaining Wall Based on AASHTO 17th & 2021 IBC
33	Steel Sheet Piling	Steel Retaining Wall Design Based on 2021 IBC / 2022 CBC & AISC 360-22
34	Cantilever Diaphragm	Cantilever Diaphragm Analysis using Tension-Only Braced Frame
35	Vehicle Security Barriers	Vehicle Security Barrier Design Based on AASHTO-17th, 2021 IBC, AISC 341-22, & ACI 318-19
36	Fiber Wrap Column	Column Repair Design of Carbon Fiber Wrap Based on 2021 IBC, ACI 318-19, & AASHTO-17th
37	Post-Compression Structure	Post-Compression Structure Analysis using Finite Element Method
38	Hybrid Suspension Bridge	Hybrid Suspension Bridge Design Based on ASCE 19-10 & AASHTO 17th
39	Heavy Loads Concrete Slab	Analysis of Concrete Floor Slabs on Grade Subjected to Heavy Loads Based on AASHTO/ACI 318-19
40	Seismic Slope Stability	Seismic Slope Stability Analysis Based on Mononobe-Okabe Method, AASHTO 17th & 2021 IBC
41	Flexible Pipe Cover	Flexible Pipe Cover Design Based on AASHTO / NCSPA Design Manual
42	Curved Rigid Footing	Curved Rigid Footing Design Based on ASCE 41-17 & ACI 318-19
43	CSP Deep Foundation	CSP Deep Foundation Design Based on 2021 IBC, ACI 318-19 & AASHTO 17th

Wood Design Group

1	To Fix Sagging Girder	To Fix Sagging Girder, by Bent HSS Tube Arch, Based on NDS 2018 & AISC 360-22
2	To Fix Sagging Beam	To Fix Sagging Beam, Using External Post-Tensioning Systems, Based on NDS 2018

3	Perforated Shear Wall	Perforated Shear Wall Design Based on 2021 IBC / 2022 CBC / NDS 2018
4	Shear Wall Opening	Wood Shear Wall with an Opening Based on 2021 IBC / 2022 CBC / NDS 2018
5	Wood Beam	Wood Beam Design Based on NDS 2018
6	Cantilever Beam	Gravity Wood Beam Design Based on NDS 2018
7	Diaphragm-Ledger-CMU Wall	Connection Design for Wall & Diaphragm Based on 2021 IBC / 2022 CBC
8	Double Joist	Double Joist Design for Equipment Based on NDS 2018, ICC PFC-4354 & PFC-5803
9	Drag Forces	Drag / Collector Force Diagram Generator
10	Equipment Anchorage	Equipment Anchorage to Wood Roof Based on NDS 2018 / 2021 IBC / 2022 CBC
11	Lag Screws Connection	Lag Screw Connection Design Based on NDS 2018
12	Subdiaphragm	Subdiaphragm Design Based on ASCE 7-22
13	Toe Nail	Toe-Nail Connection Design Based on NDS 2018
14	Top Plate Connection	Top Plate Connection Design Based on NDS 2018
15	Wood Truss	Wood Truss Design Based on NDS 2018
16	Wood Bolt Connection	Bolt Connection Design Based on NDS 2018
17	Wood Diaphragm	Wood Diaphragm Design Based on SDPWS-21
18	Wood Joist	Wood Joist Design Based on NDS 2018 / NDS 01, ICC PFC-4354 & PFC-5803
19	Wood Shear Wall	Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21 (Why Ev NOT Applied?)
20	Wood Design Tables	Tables for Wood Post Design Based on NDS 2018
21	Transfer Diaphragm - Wood	Wood Diaphragm Design for a Discontinuity of Type 4 out-of-plane offset irregularity
22	Wood Column	Wood Post, Wall Stud, or King Stud Design Based on NDS 2018
23	Green Composite Wall	Composite Strong Wall Design Based on ACI 318-19, AISI S100/SI-10 & ESR-3064P
24	Bending Post at Column	Connection Design for Bending Post at Concrete Column Based on NDS 2018 & ACI 318-19
25	Curved Member	Curved Wood Member (Wood Torsion) Design Based on NDS 2018
26	Wood Member	Wood Member (Beam, Column, Brace, Truss Web & Chord) Design Based on NDS 2018
27	Strong Custom Frame	4E-SMF with Wood Nailer Design Based on AISC 358-22 & NDS 2018
28	Hybrid Member	Hybrid Member (Wood & Metal) Design Based on NDS 2018, AISI S100 & ESR-3064P
29	Beam Reinforcement	Beam Reinforcement Design by Finite Element Method
30	Wood Pole Pile	Wood Pole or Pile Design Based on NDS 2018
31	Bamboo Shear Wall	Shear Wall Design, using Laminated Bamboo, Based on NDS 2018
32	Wood Repair and Protection	Wood Repair & Protection Design Based on 2016 CEBC, ASCE 41-17, ACI 318-19 & NDS 2018
33	CLT Shear Wall	Shear Wall Design, using Cross-Laminated Timber (CLT), Based on NDS 2018
34	Mechanically Laminated Decking	Mechanically Laminated Decking Design Based on 2022 CBC/2021 IBC 2304.9
35	CLT Two Way Floor	Two-Way Floor Design Based on NDS 2018, using Cross-Laminated Timber (CLT), by FEM
36	Flitch Plate Beam	Flitch Plate Beam Design Based on AISC 360-22 & NDS 2018
37	Wood Anchorage	Sill Plate/Nailer Connection Design Based on NDS 2018
38	Cantilever Wood Diaphragm	Cantilever Wood Diaphragm Design Based on SDPWS-21
39	Notching Design	Notching Design for Wood and Steel Beam Based on 2021 IBC, NDS 2018, & AISC 360-22
40	Tudor Arches Design	Tudor Arches Design Using Finite Element Method in Structural Mechanics
41	CLT Wall Wind	Wall of Cross-Laminated Timber (CLT) Design, for Perpendicular to Plane Loads, Based on NDS 2018

Steel Design Group

1	Filled Composite Column	Filled Composite Column Design Based on AISC 360-22 & ACI 318-19
2	Cellular Beam	Cellular Beam Design Based on AISC 360-22
3	Double Angle Capacity	Double Angle Capacity Based on AISC 360-22
4	Metal Studs	Metal Member Design Based on AISI S100/SI-10 (2021 IBC) & ESR-3064P

5	SMRF - CBC	Seismic Design for Special Moment Resisting Frames Based on 2022 CBC
6	SCBF-Parallel	Seismic Design for Special Concentrically Braced Frames Based on CBC/IBC & AISC 341-22
7	SCBF-Perpendicular	Bracing Connection Design, with Perpendicular Gusset, Based on CBC/IBC & AISC 341-22
8	Column Above Beam	Connection Design for Column above Beam, Based on AISC Manual & AISC 360-22
9	Beam Gravity	Steel Gravity Beam Design Based on AISC 360-22
10	WF Beam with Torsion	WF Simply Supported Beam Design with Torsional Loading Based on AISC 360-22
11	HSS (Tube, Pipe) Torsion	HSS (Tube, Pipe) Member Design with Torsional Loading Based on AISC 360-22
12	Fixed Bolted Joint	Fixed Bolted Joint, with Beam Sitting on Top of Column, Based on AISC 358-22 8ES/4ES & FEMA-350
13	Brace Connection	Typical Bracing Connection Capacity Based on AISC 360-22
14	BRBF	Buckling-Restrained Braced Frames Based on AISC 360-22 & AISC 341-22
15	BSEP - SMF	Bolted Seismic Moment Connection Based on AISC 341-22, 358-22, 360-22 & FEMA-350
16	Bolted Moment Connection	Bolted Non-Seismic Moment Connection Based on AISC 341-22, 358-22, 360-22 & FEMA-350
17	Channel Capacity	Channel Steel Member Capacity Based on AISC 360-22
18	Composite Collector Beam	Composite Collector Beam with Seismic Loads Based on 2022 CBC / 2021 IBC
19	Composite Floor Beam	Composite Beam Design Based on AISC Manual 9th
20	Composite Floor Beam with Cantilever	Composite Beam Design Based on AISC 360-22 / 2021 IBC / 2022 CBC
21	Composite Floor Girder	Composite Girder Design Based on AISC 360-22 / 2021 IBC / 2022 CBC
22	Drag Connection	Drag Connection Based on AISC 360-22 & AISC 341-22
23	Drag Forces for Brace Frame	Drag / Collector Forces for Brace Frame
24	EBF - CBC	Seismic Design for Eccentrically Braced Frames Based on 2022 CBC & AISC 341-22
25	EBF - IBC	Seismic Design for Eccentrically Braced Frames Based on 2021 IBC & AISC 341-22
26	Enhanced Composite Beam	Enhanced Composite Beam Design Based on AISC 360-22 / 2021 IBC / 2022 CBC
27	Enhanced Steel Beam	Enhanced Steel Beam Design Based on AISC 360-22
28	Exterior Metal Stud Wall	Exterior Metal Stud Wall Design Based on AISI S100/SI-10 & ESR-3064P
29	Floor Deck	Depressed Floor Deck Capacity (Non-Composite)
30	Gusset Geometry	Gusset Plate Dimensions Generator
31	Metal Shear Wall	Metal Shear Wall Design Based on AISI S100/SI-10, ER-5762 & ESR-3064P
32	Metal Shear Wall Opening	Metal Shear Wall with an Opening Based on AISI S100/SI-10, ER-5762 & ESR-3064P
33	Metal Z Purlins	Metal Z-Purlins Design Based on AISI S100/SI-10
34	OCBF - CBC	Ordinary Concentrically Braced Frames Based on 2022 CBC & AISC 341-22
35	OCBF - IBC	Ordinary Concentrically Braced Frames Based on 2021 IBC & AISC 341-22
36	Web-Tapered Cantilever Frame	Web-Tapered Cantilever Frame Design Based on AISC-ASD 9th, Appendix F
37	OMRF - CBC	Intermediate/Ordinary Moment Resisting Frames Based on 2022 CBC
38	OMRF - IBC	Intermediate/Ordinary Moment Resisting Frames Based on 2021 IBC
39	Plate Girder	Plate Girder Design Based on AISC 360-22
40	Rectangular Section	Rectangular Section Member Design Based on AISC 360-22
41	Roof Deck	Design of 1 1/2" Type "B" Roof Deck Based on ICBO ER-2078P
42	Base Plate	Base Plate Design Based on AISC 360-22
43	SMRF - IBC	Special Moment Resisting Frames Based on 2021 IBC, AISC 341-22 & 358-22
44	SPSW	Seismic Design for Special Plate Shear Wall Based on AISC 341-22 & AISC 360-22
45	Steel Column	Steel Column Design Based on AISC 360-22
46	Steel Stair	Steel Stair Design Based on AISC 360-22
47	Triple W Shapes	Simply Supported Member of Triple W-Shapes Design Based on AISC 360-22
48	Portal Frame	Portal Frame Analysis using Finite Element Method
49	Web Tapered Portal	Web Tapered Portal Design Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25

50	Web Tapered Frame	Web Tapered Frame Design Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25
51	Web Tapered Girder	Web Tapered Girder Design Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25
52	Weld Connection	Weld Connection Design Based on AISC 360-22
53	WF Opening	Check Capacity of WF Beam at Opening Based on AISC 360-22
54	HSS Opening	Check Capacity of HSS Tube Beam at Opening Based on AISC 360-22
55	Beam Bolted Splice	Beam Bolted Splice Design Based on AISC 360-22
56	C-PSW/CF	Composite Plate Shear Wall Design Based on AISC 341-22 & ACI 318-19 - Concrete Filled (C-PSW/CF)
57	MT-OCBF/SCBF	Multi-Tiered Braced Frame Design Based on AISC 341-22
58	HSS-WF Capacity	Tube, Pipe, or WF Member Capacity Based on AISC 360-22
59	T-Shape Capacity	T-Shape Member Capacity Based on AISC 360-22
60	Fence Column & Footing	Cantilever Column & Footing Design Based on AISC 360-22, ACI 318-19, and IBC 1807.3
61	Metal Truss	Light Gage Truss Design Based on AISI S100/SI-10 & ESR-3064P
62	Sleeve Joint Connection	Sleeve Joint Connection Design, for Steel Cell Tower / Sign, Based on AISC 360-22
63	Moment to Column Web	Moment Connection Design for Beam to Weak Axis Column Based on AISC 360-22
64	Beam Connection	Beam Connection Design Based on AISC 360-22
65	ConXL	Seismic Bi-axial Moment Frame Design Based on AISC 358-22 & ACI 318-19
66	Bolt Connection	Bolt Connection Design Based on AISC 360-22
67	SCCS and/or OCCS	Cantilever Column System (SCCS/OCCS) Design Based on AISC 341-22/360-22 & ACI 318-19
68	Non-Prismatic Composite Girder	Non-Prismatic Composite Girder Design Based on AISC 360-22 / 2022 CBC / 2021 IBC
69	Endplate Connection	Endplate Splice Moment Connection Based on AISC 341-22, 358-22, 360-22 & FEMA-350
70	Z-Profile Tread and Riser	Flexure Capacity for Z-Profile Tread and Riser Based on AISC 360-22
71	Strong-Column Weak-Beam	Strong-Column Weak-Beam Design Based on AISC 341-22 and AISC 360-22
72	Thin Composite Beam	Thin Composite Beam/Collector Design Based on AISC 360-22 & ACI 318-19
73	Angle Capacity	Angle Steel Member Capacity Based on AISC 360-22
74	Seismic Column	Filled Composite Column (FCC) Design for C-SMF/C-IMF/C-OCF Based on ASCE 7-22, AISC 341-22 & ACI 318-19
75	SCBF for 2-Story	Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-22
76	SCBF for 3-Story	Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-22
77	SCBF for 4-Story	Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-22
78	T-SMF Connection	Double-Tee Connection Design for SMF Based on AISC 341-22, 358-22, 360-22
79	Steel Corbel	Steel Corbel Design Based on AISC-ASD 9th, Appendix F
80	Stiffeners	Proportions and Stiffeners Design for I-Shaped Member Based on AISC 360-22
81	Moment to Tube Column	Moment Connection Design for Beam to Tube Column Based on AISC 360-22
82	Prestressed Steel Arch	Prestressed Steel Arch Design Based on 2021 IBC/2022 CBC & AISC 360-22
83	Roof Bent Girder	Web-Tapered Roof Girder Design Based on AISC-ASD 9th Appendix F and 2021 IBC/2022 CBC 1605
84	Moment across Girder	Design for Fully Restrained Moment Connection across Girder Based on AISC 360-22
85	Forbidden City-TYP	Typical Frame Design of Web Curved Portal Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25
86	Forbidden City-HIP	Mono Hip Frame Design of Web Curved Portal Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25

Masonry Design Group

1	Masonry Shear Wall - CBC	Masonry Shear Wall Design Based on 2022 CBC Chapter A (both ASD and SD)
2	Masonry Shear Wall - IBC	Masonry Shear Wall Design Based on TMS 402-16 & 2021 IBC (both ASD and SD)
3	Anchorage to Masonry	Fastener Anchorage Design in Masonry Based on TMS 402-16
4	Flush Wall Pilaster - CBC	Masonry Flush Wall Pilaster Design Based on 2022 CBC Chapter A
5	Flush Wall Pilaster - IBC	Masonry Flush Wall Pilaster Design Based on TMS 402-16 & 2021 IBC
6	Bearing Wall Opening	Design of Masonry Bearing Wall with Opening Based on TMS 402-16
7	Bending Post at Top Wall	Design for Bending Post at Top of Wall, Based on TMS 402-16
8	Development Splice Masonry	Development & Splice of Reinforcement in Masonry Based on TMS 402-16 & 2021 IBC & 2022 CBC
9	Elevator for DSA / OSHPD	Elevator Masonry Wall Design Based on 2022 CBC Chapter A & 2021 IBC
10	Girder at Wall	Design for Girder at Masonry Wall Based on TMS 402-16
11	Horizontal Bending Wall	Masonry Wall Design at Horizontal Bending Based on TMS 402-16
12	Masonry Beam	Masonry Beam Design Based on TMS 402-16
13	Masonry Bearing Wall - CBC	Allowable & Strength Design of Masonry Bearing Wall Based on 2022 CBC Chapter A
14	Masonry Bearing Wall - IBC	Allowable & Strength Design of Masonry Bearing Wall Based on TMS 402-16
15	Masonry Column - CBC	Masonry Column Design Based on 2022 CBC Chapter A
16	Masonry Column - IBC	Masonry Column Design Based on TMS 402-16 & 2021 IBC
17	Beam to Wall	Beam to Wall Anchorage Design Based on TMS 402-16
18	Collector to Wall	Collector to Wall Connection Design Based on TMS 402-16
19	Hybrid Masonry Wall	Hybrid Masonry Wall Design Based on TMS 402-16
20	PT-Masonry Shear Wall	Post-Tensioned Masonry Shear Wall Design Based on TMS 402-16 (LEED Gold)
21	Masonry Shear Wall Opening	Masonry Shear Wall with Opening Design Using Finite Element Method
22	Masonry Cracking	Anticipated Cracking Design of Masonry Wall Based on TMS 402-16
23	Existing Column Enhancement	Existing Column Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19/TMS 402-16
24	Existing Wall Enhancement	Existing Wall Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-19/TMS 402-16
25	Grillage Beam Wall	Design for Grillage Beam Masonry Wall Based on TMS 402-16
26	Masonry Element	Masonry Plate/Shell Element Design (ASD) Based on 2021 IBC & TMS 402-16
27	Lightly Loaded Column	Lightly Loaded Column Design Based on TMS 402-16 (UNCRACKED and CRACKED)

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Daniel T. Li
Engineering International

PROJECT :
CLIENT :
JOB NO. :
DATE :

PAGE :
DESIGN BY :
REVIEW BY :

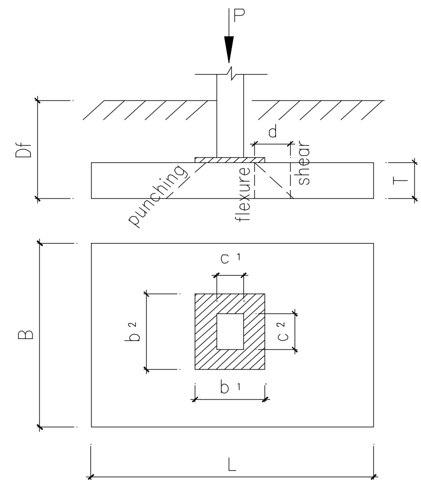
Pad Footing Design Based on ACI 318-08

INPUT DATA

COLUMN WIDTH	c_1	=	5	in
COLUMN DEPTH	c_2	=	5	in
BASE PLATE WIDTH	b_1	=	16	in
BASE PLATE DEPTH	b_2	=	16	in
FOOTING CONCRETE STRENGTH	f_c'	=	2.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	25	k
AXIAL LIVE LOAD	P_{LL}	=	4.5	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	1	Seismic,SD
SEISMIC AXIAL LOAD	P_{LAT}	=	-6	k, SD
SURCHARGE	q_s	=	0	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	2	ft
FOOTING THICKNESS	T	=	12	in
ALLOW SOIL PRESSURE	Q_a	=	2.5	ksf
FOOTING WIDTH	B	=	3	ft
FOOTING LENGTH	L	=	4	ft
BOTTOM REINFORCING		#	5	

DESIGN SUMMARY

FOOTING WIDTH		B	=	3.00	ft	
FOOTING LENGTH		L	=	4.00	ft	
FOOTING THICKNESS		T	=	12	in	
LONGITUDINAL REINF.	3	#	5	@	15	in o.c.
TRANSVERSE REINF.	4	#	5	@	14	in o.c.



THE PAD DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS (IBC SEC.1605.3.2 & ACI 318-08 SEC.9.2.1)

CASE 1:	DL + LL	P	=	30	kips	1.2 DL + 1.6 LL	Pu	=	37	kips
CASE 2:	DL + LL + E / 1.4	P	=	25	kips	1.2 DL + 1.0 LL + 1.0 E	Pu	=	29	kips
CASE 3:	0.9 DL + E / 1.4	P	=	18	kips	0.9 DL + 1.0 E	Pu	=	17	kips

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

$$q_{MAX} = \frac{P}{BL} + q_s + (0.15 - w_S)T =$$

CASE 1	CASE 2	CASE 3
2.50 ksf,	2.14 ksf,	1.56 ksf

q_{MAX} < $k\,Q_a$, **[Satisfactory]**

where k = 1 for gravity loads, 4/3 for lateral loads.

DESIGN FOR FLEXURE (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y}$$

$$\rho_{MAX} = \frac{0.85 \beta_1 f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$$

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d} \ , \ \frac{4}{3}\rho\right)$$

	LONGITUDINAL	TRANSVERSE
d	8.69	8.38
b	36	48
$q_{u,max}$	3.10	3.10
M_u	11.35	7.00
ρ	0.001	0.000
ρ_{min}	0.001	0.001
A_s	0.39	0.25
ReqD	2 # 5	1 # 5
Max. Spacing	18 in o.c.	18 in o.c.
USE	3 # 5 @ 15 in o.c.	4 # 5 @ 14 in o.c.
ρ_{max}	0.013	0.013
Check $\rho_{prod} < \rho_{max}$	**[Satisfactory]**	**[Satisfactory]**

(cont'd)

CHECK FLEXURE SHEAR (ACI 318-08 SEC.9.3.2.3, 15.5.2, 11.1.3.1, & 11.2)

$$\phi V_n = 2\phi b d\sqrt{f'_c}$$

	LONGITUDINAL	TRANSVERSE
V_u	7.80	4.52
ϕ	0.75	0.75
ϕV_n	23.5	30.2
Check $V_u < \phi V_n$	**[Satisfactory]**	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318-08 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$\phi V_n = (2 + y)\phi\sqrt{f'_c}A_p \quad = \quad 97.42 \text{ kips}$$

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

β_c = ratio of long side to short side of concentrated load = 1.00

b_0 = $c_1 + c_2 + b_1 + b_2 + 4d$ = 76.1 in

A_p = b_0 d = 649.4 in^2

y = MIN(2 , 4 / β_c , 40 d / b_0) = 2.0

$$V_u = P_u, \max\left[1 - \frac{1}{BL}\left(\frac{b_1 + c_1}{2} + d\right)\left(\frac{b_2 + c_2}{2} + d\right)\right] = \quad 29.40 \text{ kips} \quad < \quad \phi V_n$$

[Satisfactory]

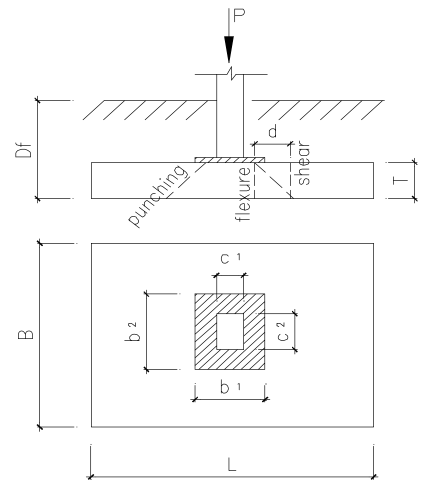
Pad Footing Design Based on ACI 318-95 / CBC 2001

INPUT DATA

COLUMN WIDTH	c_1	=	5	in
COLUMN DEPTH	c_2	=	5	in
BASE PLATE WIDTH	b_1	=	16	in
BASE PLATE DEPTH	b_2	=	16	in
FOOTING CONCRETE STRENGTH	f_c'	=	2.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	25	k
AXIAL LIVE LOAD	P_{LL}	=	4.5	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	1	Seismic,SD
SEISMIC AXIAL LOAD	P_{LAT}	=	-6	k, SD
SURCHARGE	q_s	=	0	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	2	ft
FOOTING THICKNESS	T	=	12	in
ALLOW SOIL PRESSURE	Q_a	=	2.5	ksf
FOOTING WIDTH	B	=	3	ft
FOOTING LENGTH	L	=	4	ft
BOTTOM REINFORCING		#	5	

DESIGN SUMMARY

FOOTING WIDTH		B	=	3.00	ft	
FOOTING LENGTH		L	=	4.00	ft	
FOOTING THICKNESS		T	=	12	in	
LONGITUDINAL REINF.	3	#	5	@	15	in o.c.
TRANSVERSE REINF.	4	#	5	@	14	in o.c.



THE PAD DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS (CBC SEC.1612.3.2, 1612.2.1 & ACI 318-95 SEC.9.2.1)

CASE 1:	DL + LL	P	=	30	kips	1.4 DL + 1.7 LL	Pu	=	43	kips
CASE 2:	DL + LL + E / 1.4	P	=	25	kips	1.2 DL + 1.0 LL + 1.0 E	Pu	=	29	kips
CASE 3:	0.9 DL + E / 1.4	P	=	18	kips	0.9 DL + 1.0 E	Pu	=	17	kips

CHECK SOIL BEARING CAPACITY (ACI 318-95 SEC.15.2.2)

$$q_{MAX} = \frac{P}{BL} + q_s + (0.15 - w_S)T =$$

CASE 1	CASE 2	CASE 3
2.50 ksf,	2.14 ksf,	1.56 ksf

q_{MAX} < $k Q_a$, **[Satisfactory]**

where k = 1 for gravity loads, 4/3 for lateral loads.

DESIGN FOR FLEXURE (ACI 318-95 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho = \frac{0.85 f_c'\left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y} \qquad \rho_{MAX} = 0.75\left(\frac{0.85 \beta_1 f_c'}{f_y}\,\frac{87}{87 + f_y}\right) \qquad \rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

	LONGITUDINAL	TRANSVERSE
d	8.69	8.38
b	36	48
$q_{u,max}$	3.55	3.55
M_u	13.02	8.02
ρ	0.001	0.001
ρ_{min}	0.001	0.001
A_s	0.45	0.29
ReqD	2 # 5	1 # 5
Max. Spacing	18 in o.c.	18 in o.c.
USE	3 # 5 @ 15 in o.c.	4 # 5 @ 14 in o.c.
ρ_{max}	0.013	0.013
Check $\rho_{prod} < \rho_{max}$	**[Satisfactory]**	**[Satisfactory]**

(cont'd)

CHECK FLEXURE SHEAR (ACI 318-95 SEC.9.3.2.3, 15.5.2, 11.1.3.1, & 11.3)

$$\phi V_n = 2\phi b d \sqrt{f'_c}$$

	LONGITUDINAL	TRANSVERSE
V_u	8.94	5.18
ϕ	0.85	0.85
ϕV_n	26.6	34.2
Check $V_u < \phi V_n$	**[Satisfactory]**	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318-95 SEC.15.5.2, 11.12.1.2, 11.12.6, & 13.5.3.2)

$$\phi V_n = (2 + y)\phi\sqrt{f'_c}A_p \quad = \quad 110.41 \text{ kips}$$

where ϕ = 0.85 (ACI 318-95, Section 9.3.2.3)

β_c = ratio of long side to short side of concentrated load = 1.00

b_0 = $c_1 + c_2 + b_1 + b_2 + 4d$ = 76.1 in

A_p = b_0 d = 649.4 in^2

y = MIN(2 , 4 / β_c , 40 d / b_0) = 2.0

$$V_u = P_{u,\max}\left[1 - \frac{1}{BL}\left(\frac{b_1 + c_1}{2} + d\right)\left(\frac{b_2 + c_2}{2} + d\right)\right] = \quad 33.71 \text{ kips} \quad < \quad \phi V_n$$ **[Satisfactory]**

Eccentric Footing Design Based on ACI 318-08

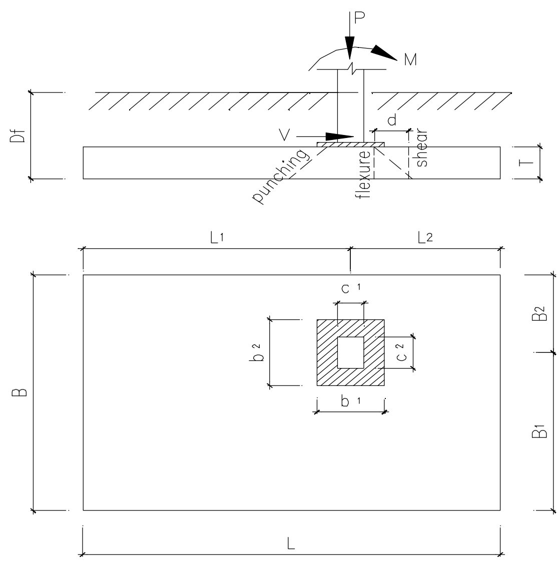
INPUT DATA

COLUMN WIDTH	c_1	=	5	in
COLUMN DEPTH	c_2	=	5	in
BASE PLATE WIDTH	b_1	=	16	in
BASE PLATE DEPTH	b_2	=	16	in
FOOTING CONCRETE STRENGTH	f_c'	=	2.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	50	k
AXIAL LIVE LOAD	P_{LL}	=	4.5	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	0	Wind,ASD
WIND AXIAL LOAD	P_{LAT}	=	1	k, ASD
WIND MOMENT LOAD	M_{LAT}	=	15	ft-k, ASD
WIND SHEAR LOAD	V_{LAT}	=	2.5	k, ASD
SURCHARGE	q_s	=	0.1	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	2	ft
FOOTING THICKNESS	T	=	12	in
ALLOW SOIL PRESSURE	Q_a	=	3	ksf
FOOTING WIDTH	B_1	=	10	ft
	B_2	=	6	ft
FOOTING LENGTH	L_1	=	6	ft
	L_2	=	1	ft
REINFORCING SIZE		#	5	

THE FOOTING DESIGN IS ADEQUATE.

DESIGN SUMMARY

FOOTING WIDTH	B	=	16.00	ft
FOOTING LENGTH	L	=	7.00	ft
FOOTING THICKNESS	T	=	12	in
LONGITUDINAL REINF., TOP	1 # 5			
LONGITUDINAL REINF., BOT.	23 # 5 @ 8 in o.c.			
TRANSVERSE REINF., BOT.	6 # 5 @ 15 in o.c.			



ANALYSIS

DESIGN LOADS AT TOP OF FOOTING (IBC SEC.1605.3.2 & ACI 318-08 SEC.9.2.1)

CASE 1:	DL + LL	P	=	55	kips	1.2 DL + 1.6 LL	Pu	=	67	kips
		M	=	136	ft-kips		Mu	=	168	ft-kips
		e	=	2.5	ft, fr cl ftg		eu	=	2.5	ft, fr cl ftg
CASE 2:	DL + LL + 1.3 W	P	=	56	kips	1.2 DL + LL + 1.6 W	Pu	=	66	kips
		M	=	162	ft-kips		Mu	=	189	ft-kips
		V	=	3	kips		Vu	=	4	kips
		e	=	2.9	ft, fr cl ftg		eu	=	2.9	ft, fr cl ftg
CASE 3:	DL + LL + 0.65 W	P	=	55	kips	0.9 DL+ 1.6 W	Pu	=	47	kips
		M	=	147	ft-kips		Mu	=	141	ft-kips
		V	=	1	kips		Vu	=	4	kips
		e	=	2.7	ft, fr cl ftg		eu	=	3.0	ft, fr cl ftg

CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 9.2 **>** F = 1.6 / 0.9 = 1.78 **[Satisfactory]**

Where M_O = $M_{LAT} + V_{LAT}\,T - P_{LAT}L_2$ = 17 k-ft

P_{ftg} = (0.15 kcf) T B L = 16.80 k, footing weight

P_{soil} = $w_s\,(D_f - T)\,B\,L$ = 12.32 k, soil weight

M_R = $P_{DL}L_2 + 0.5\,(P_{ftg} + P_{soil})\,L$ = 152 k-ft

FOR REVERSED LATERAL LOADS,

M_R / M_O = 28.7 **>** F = 1.6 / 0.9 **[Satisfactory]**

Where M_O = $M_{LAT} + V_{LAT}\,D_f - P_{LAT}L_1$ = 14 k-ft

M_R = $P_{DL}L_1 + 0.5\,(P_{ftg} + P_{soil})\,L$ = 402 k-ft

CHECK SLIDING (IBC 09 1807.2.3)

$1.5\,(V_{Lat,\,ASD})$ = 3.75 kips **<** $\mu\,\Sigma W$ = 26.72 kips **[Satisfactory]**

Where ` μ = 0.4

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

Service Loads	CASE 1		CASE 2		CASE 3		
P	54.5		55.8		55.2		k
e	2.5		3.0		2.7		ft (from center of footing)
q_S B L	11.2		11.2		11.2		k, (surcharge load)
$(0.15\text{-}w_S)$T B L	4.5		4.5		4.5		k, (footing increased)
Σ P	70.2		71.5		70.8		k
e_L	1.9	> L/6	2.3	> L/6	2.1	> L/6	ft
e_B	1.6	< B/6	1.6	< B/6	1.6	< B/6	ft
q_L	30.0		40.2		33.4		k / ft
q_{max}	3.0		4.0		3.3		ksf
q_{allow}	3.0		4.0		4.0		ksf

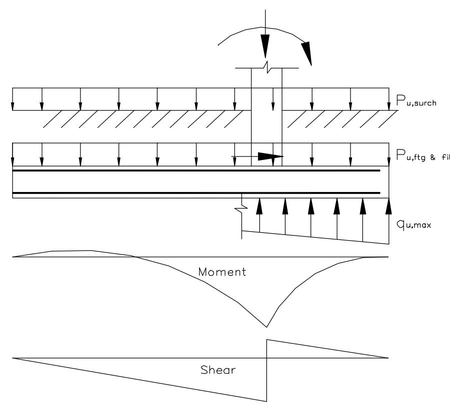
Where

$$q_L = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e_L}{L}\right)}{L}, & for \quad e_L \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3(0.5L-e_L)}, & for \quad e_L > \dfrac{L}{6} \end{cases} \qquad q_{MAX} = \begin{cases} \dfrac{q_L\left(1+\dfrac{6e_B}{B}\right)}{B}, & for \quad e_B \le \dfrac{B}{6} \\ \dfrac{2q_L}{3(0.5B-e_B)}, & for \quad e_B > \dfrac{B}{6} \end{cases}$$

[Satisfactory]

DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)



$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} \qquad \rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} \qquad \rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1		CASE 2		CASE 3		
P_u	67.2		66.1		46.6		k
e_u	2.5		2.9		3.1		ft
γq_S B L	17.9		11.2		0.0		k, (factored surcharge load)
$\gamma[0.15T + w_s(D_f - T)]BL$	34.9		34.9		26.2		k, (factored footing & backfill loads)
ΣP_u	120.1		112.2		72.8		k
e_u	1.4	> L/6	1.7	> L/6	2.0	> L/6	ft
$q_{u,\,max}$	2.381		2.630		2.002		ksf

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **1**

Section	0	0.25 L_1	0.50 L_1	0.75 L_1	Col_L	Col_R	0.25 L_2	0.50 L_2	0.75 L_2	L
X_u (ft, dist. from left of footing)	0	1.50	3.00	4.50	5.56	6.44	6.25	6.50	6.75	7.00
$M_{u,col}$ (ft-k)	0	0	0	0	0	-29.4	-16.8	-33.6	-50.4	-67.2
$V_{u,col}$ (k)	0	0.0	0.0	0.0	0.0	67.2	67.2	67.2	67.2	67.2
$P_{u,surch}$ (klf)	2.56	2.56	2.56	2.56	2.56	2.56	2.56	2.56	2.56	2.56
$M_{u,surch}$ (ft-k)	0	-2.9	-11.5	-25.9	-39.6	-53.0	-50.0	-54.1	-58.3	-62.7
$V_{u,surch}$ (k)	0	3.8	7.7	11.5	14.2	16.5	16.0	16.6	17.3	17.9
$P_{u,ftg\ \&\ fill}$ (klf)	4.99	4.99	4.99	4.99	4.99	4.99	4.99	4.99	4.99	4.99
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-5.6	-22.5	-50.5	-77.2	-103.4	-97.5	-105.5	-113.7	-122.3
$V_{u,ftg\ \&\ fill}$ (k)	0	7.5	15.0	22.5	27.8	32.1	31.2	32.4	33.7	34.9
$q_{u,soil}$ (ksf)	0.00	0.51	1.02	1.53	1.89	2.19	2.13	2.21	2.30	2.38
$M_{u,soil}$ (ft-k)	0	189.5	288.9	316.3	302.3	275.2	282.0	272.8	262.9	252.2
$V_{u,soil}$ (k)	0	-48.2	-84.1	-107.8	-117.2	-120.3	-120.0	-120.3	-120.4	-120.1
Σ M_u (ft-k)	**0**	**181.1**	**254.9**	**239.8**	**185.5**	**89.3**	**117.7**	**79.7**	**40.5**	**0**
Σ V_u (kips)	**0**	**-36.9**	**-61.5**	**-73.8**	**-75.2**	**-4.5**	**-5.6**	**-4.0**	**-2.2**	**0**

(cont'd)

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **2**

Section	0	0.25 L_1	0.50 L_1	0.75 L_1	Col_L	Col_R	0.25 L_2	0.50 L_2	0.75 L_2	L
X_u (ft, dist. from left of footing)	0	1.50	3.00	4.50	5.56	6.44	6.25	6.50	6.75	7.00
$M_{u,col}$ (ft-k)	0	0	0	0	0	-0.9	11.5	-5.1	-21.6	-38.1
$V_{u,col}$ (k)	0	0.0	0.0	0.0	0.0	66.1	66.1	66.1	66.1	66.1
$P_{u,surch}$ (klf)	1.60	1.60	1.60	1.60	1.60	1.60	1.60	1.60	1.60	1.60
$M_{u,surch}$ (ft-k)	0	-1.8	-7.2	-16.2	-24.8	-33.2	-31.3	-33.8	-36.5	-39.2
$V_{u,surch}$ (k)	0	2.4	4.8	7.2	8.9	10.3	10.0	10.4	10.8	11.2
$P_{u,ftg\ \&\ fill}$ (klf)	4.99	4.99	4.99	4.99	4.99	4.99	4.99	4.99	4.99	4.99
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-5.6	-22.5	-50.5	-77.2	-103.4	-97.5	-105.5	-113.7	-122.3
$V_{u,ftg\ \&\ fill}$ (k)	0	7.5	15.0	22.5	27.8	32.1	31.2	32.4	33.7	34.9
$q_{u,soil}$ (ksf)	0.00	0.00	1.13	1.69	2.09	2.42	2.35	2.44	2.54	2.63
$M_{u,soil}$ (ft-k)	0	0.0	261.9	276.8	255.6	224.3	231.8	221.7	210.9	199.6
$V_{u,soil}$ (k)	0	0.0	-84.2	-106.0	-113.2	-114.1	-114.3	-114.0	-113.3	-112.2
Σ M_u (ft-k)	**0**	**-7.4**	**232.2**	**210.0**	**153.6**	**86.8**	**114.6**	**77.4**	**39.2**	**0**
Σ V_u (kips)	**0**	**9.9**	**-64.4**	**-76.3**	**-76.6**	**-5.6**	**-7.0**	**-5.0**	**-2.7**	**0**

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **3**

Section	0	0.25 L_1	0.50 L_1	0.75 L_1	Col_L	Col_R	0.25 L_2	0.50 L_2	0.75 L_2	L
X_u (ft, dist. from left of footing)	0	1.50	3.00	4.50	5.56	6.44	6.25	6.50	6.75	7.00
$M_{u,col}$ (ft-k)	0	0	0	0	0	7.6	16.4	4.7	-7.0	-18.6
$V_{u,col}$ (k)	0	0.0	0.0	0.0	0.0	46.6	46.6	46.6	46.6	46.6
$P_{u,surch}$ (klf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{u,surch}$ (ft-k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$V_{u,surch}$ (k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{u,ftg\ \&\ fill}$ (klf)	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-4.2	-16.8	-37.9	-57.9	-77.6	-73.1	-79.1	-85.3	-91.7
$V_{u,ftg\ \&\ fill}$ (k)	0	5.6	11.2	16.8	20.8	24.1	23.4	24.3	25.3	26.2
$q_{u,soil}$ (ksf)	0.00	0.00	0.86	1.29	1.59	1.84	1.79	1.86	1.93	2.00
$M_{u,soil}$ (ft-k)	0	0.0	167.1	170.4	151.9	128.1	133.7	126.2	118.4	110.3
$V_{u,soil}$ (k)	0	0.0	-58.7	-72.5	-76.2	-75.2	-75.7	-75.0	-74.1	-72.8
Σ M_u (ft-k)	**0**	**-4.2**	**150.3**	**132.5**	**94.0**	**58.2**	**76.9**	**51.8**	**26.2**	**0**
Σ V_u (kips)	**0**	**5.6**	**-47.4**	**-55.7**	**-55.3**	**-4.5**	**-5.7**	**-4.1**	**-2.2**	**0**

DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	use	ρ_{provD}
Top Longitudinal	-7.4	ft-k	9.69	0.0001	0.0001	0.0129	no limit	1 # 5	0.0002
Bottom Longitudinal	254.9	ft-k	8.69	0.0025	0.0041	0.0129	18	23 # 5 @ 8 in o.c.	0.0043
Bottom Transverse	1	ft-k / ft	8.38	0.0004	0.0003	0.0129	18	6 # 5 @ 15 in o.c.	0.0026

[Satisfactory]

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$		$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$		check $V_u < \phi\ V_c$
Longitudinal	76.6	k	125	k	**[Satisfactory]**
Transverse	4.3	k / ft	8	k / ft	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318-08 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right]$$

$$R = \frac{P_u b_1 b_2}{A_f}$$

$$A_P = 2(b_1 + b_2)d$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$A_f = BL$$

$$\phi v_c(psi) = \phi(2 + y)\sqrt{f_c'}$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

$$b_0 = \frac{A_P}{d}, b_1 = (0.5c_1 + 0.5b_1 + d), b_2 = (0.5c_2 + 0.5b_2 + d)$$

Case	P_u	M_u	b_1	b_2	b_0	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\ (psi)}$	$\phi\ v_c$
1	67.2	168.0	18.9	18.9	0.5	0.4	1.0	2.0	112.0	4.4	1.5	1.9	105.3	150.0
2	66.1	189.3	18.9	18.9	0.5	0.4	1.0	2.0	112.0	4.4	1.5	1.9	103.7	150.0
3	46.6	140.5	18.9	18.9	0.5	0.4	1.0	2.0	112.0	4.4	1.0	1.9	73.2	150.0

[Satisfactory]

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

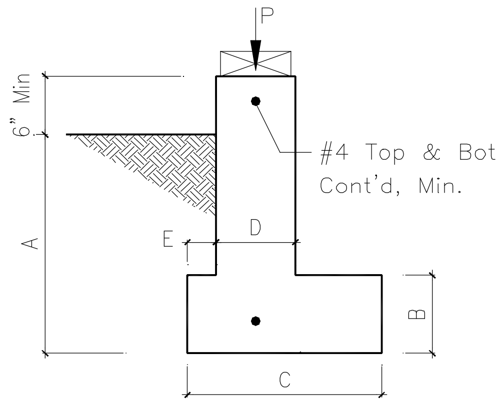
Footing Design for Stud Bearing Wall Based on IBC 09 / ACI 318-08

INPUT DATA & DESIGN SUMMARY

FOOTING SIZE	A	=	18	in
	B	=	6	in
	C	=	16	in
	D	=	8	in
	E	=	3	in
FOOTING CONCRETE STRENGTH	fc'	=	2.5	ksi
AXIAL DEAD LOAD (per linear foot)	P_{DL}	=	1.5	k / ft
AXIAL LIVE LOAD (per linear foot)	P_{LL}	=	0.6	k / ft
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	1	Seismic,SD
LATERAL LOAD (per linear foot)	P_{LAT}	=	0.8	k / ft, SD

(holdown force converted to load per linear foot)

SURCHARGE	q_s	=	0.1	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
ALLOWABLE SOIL PRESSURE	Q_a	=	3	ksf



THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS (IBC SEC.1605.3.2 & ACI 318-08 SEC.9.2.1)

CASE 1:	DL + LL	P	=	2.10	k / ft	1.2 DL + 1.6 LL	Pu	=	2.76	k / ft
CASE 2:	DL + LL + E / 1.4	P	=	2.67	k / ft	1.2 DL + 1.0 LL + 1.0 E	Pu	=	3.20	k / ft
CASE 3:	0.9 DL + E / 1.4	P	=	1.92	k / ft	0.9 DL + 1.0 E	Pu	=	2.15	k / ft

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

Service Loads	CASE 1	CASE 2	CASE 3	
P	2.10	2.67	1.92	k / ft
e	1.0	1.0	1.0	in (from center of footing)
q_s C	0.13	0.13	0.13	k / ft, (surcharge load)
$(0.15-w_s)$ Area	0.07	0.07	0.07	k / ft, (footing increased)
Σ P	2.3	2.9	2.1	k / ft
e	1.0	1.0	1.0	in
q_{max}	2.36	2.95	2.17	ksf
Q_a	3.00	3.00	3.00	ksf

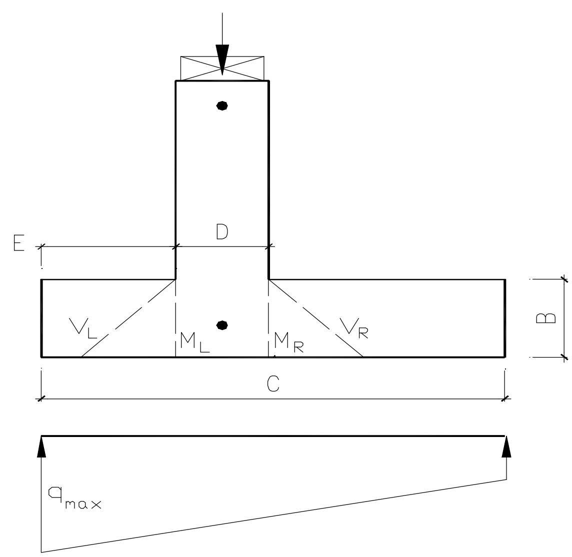
Where [Satisfactory]

$$q_{max} = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e}{C}\right)}{C}, & for \quad e \le \dfrac{C}{6} \\[2ex] \dfrac{2(\Sigma P)}{3(0.5C-e)}, & for \quad e > \dfrac{C}{6} \end{cases}$$

DESIGN FOR FLEXURE (ACI 318-08 SEC.22.5.1)

$$\phi M_n = MIN\left(5\lambda\phi\sqrt{f'_c}S,\ 0.85\phi f'_c S\right) = 0.90 \text{ ft-kips / ft}$$

where	λ	=	1.0	(ACI 318-08, Section 8.6.1)
	ϕ	=	0.6	(ACI 318-08, Section 9.3.5)
	S	=	elastic section modulus of section	
		=	72	in^3/ ft



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FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	2.8	3.2	2.2	k / ft
e_u	1.0	1.0	1.0	in (from center of footing)
$\gamma\, q_s\, C$	0.21	0.21	0.21	k / ft, (factored surcharge load)
γ[0.15AC - (0.15-Ws) (C-D) (A-B)]	0.33	0.33	0.25	k / ft, (factored footing & backfill loads)
$\Sigma\, P_u$	3.30	3.74	2.61	k / ft
e_u	0.9	0.9	0.9	in
E	3.0	3.0	3.0	in
$q_{u,\, max}$	3.27	3.73	2.58	ksf
$q_{u,\, VL}$	3.27	3.73	2.58	ksf
$q_{u,\, ML}$	2.98	3.38	2.35	ksf
$q_{u,\, MR}$	2.18	2.46	1.72	ksf
$q_{u,\, VR}$	1.68	1.88	1.33	ksf
$q_{u,\, min}$	1.68	1.88	1.33	ksf
$M_{u,\, L}$	0.09	0.10	0.07	ft-k / ft
$M_{u,\, R}$	0.13	0.15	0.10	ft-k / ft
$V_{u,\, L}$	0.00	0.00	0.00	k / ft
$V_{u,\, R}$	0.00	0.00	0.00	k / ft

$M_{u,\, max}$ = 0.15 ft-k / ft < $\phi\, M_n$ [Satisfactory]

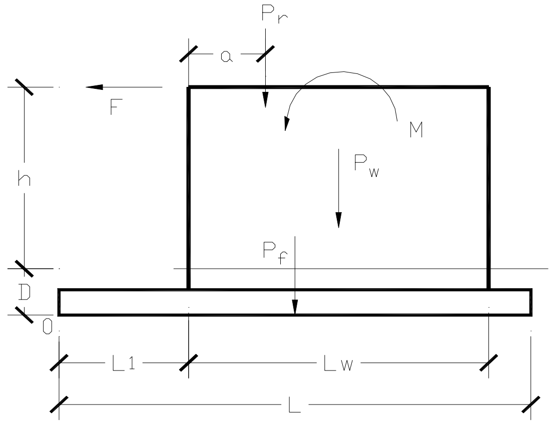
CHECK FLEXURE SHEAR (ACI 318-08 SEC.22.5.4)

$\phi V_n = \frac{4}{3} \lambda \phi \sqrt{f'_c} B$ = 2.88 k / ft

where ϕ = 0.6 (ACI 318-08, Section 9.3.5)

$V_{u,\, max}$ = 0.00 k / ft < $\phi\, V_n$ [Satisfactory]

Footing Design of Shear Wall Based on ACI 318-08



INPUT DATA

WALL LENGTH	L_W =	16.5	ft		
WALL HEIGHT	h =	11	ft		
WALL THICKNESS	t =	8	in		
FOOTING LENGTH	L =	25	ft		
	L_1 =	4.25	ft		
FOOTING WIDTH	B =	5	ft		
FOOTING THICKNESS	T =	24	in		
FOOTING EMBEDMENT DEPTH	D =	2	ft		
ALLOWABLE SOIL PRESSURE	q_a =	4	ksf		
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	110	kips		
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	110	kips		
TOP LOAD LOCATION	a =	8.25	ft		
WALL SELF WEIGHT	P_W =	15.6	kips		
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic		
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	62.5	kips		
	M =	812.4	ft-kips		
CONCRETE STRENGTH	f_c' =	4	ksi		
REBAR YIELD STRESS	f_y =	60	ksi		
TOP BARS, LONGITUDINAL		4	#	8	
BOTTOM BARS, LONGITUDINAL		9	#	9	
BOTTOM BARS, TRANSVERSE	#	6	@	12	in o.c.

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

$F = M_R / M_O =$ 1.25 **>** 1.4 x 0.75 / 0.9 for seismic [Satisfactory]

Where P_f = 36.25 kips (footing self weight)

$M_O = F (h + D) + M =$ 1625 ft-kips (overturning moment)

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_W (L_1 + 0.5L_W) =$ 2023 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_S = 25 kips (soil weight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_W + (P_f - P_S) =$ 246.85 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_W (L_1 + 0.5L_W) =$ 3398 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P =$ 5.32 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 4.58 \text{ ksf} < 4/3\, q_a$$

[Satisfactory]

Where e = 5.32 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

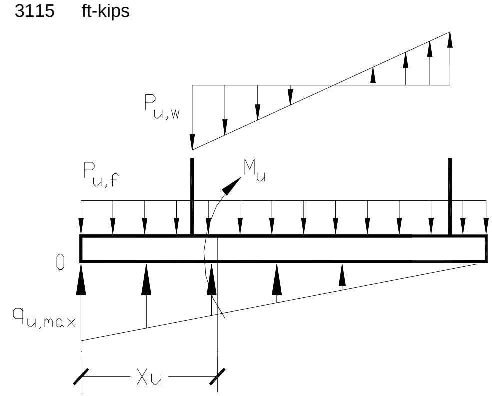
$M_{u,R} =$ $1.2 [P_{r,DL} (L_1 + a) + P_f (0.5 L) + P_W (L_1 + 0.5L_W)] + 0.5 P_{r,LL}(L_1 + a) =$ 3115 ft-kips

$M_{u,o} =$ $1.4 [F(h + D) + M] =$ 2275 ft-kips

$P_u =$ $1.2 (P_{r,DL} + P_f + P_W) + 0.5 P_{r,LL} =$ 249 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 9.13 ft

$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} = 9.85 \text{ ksf}$$



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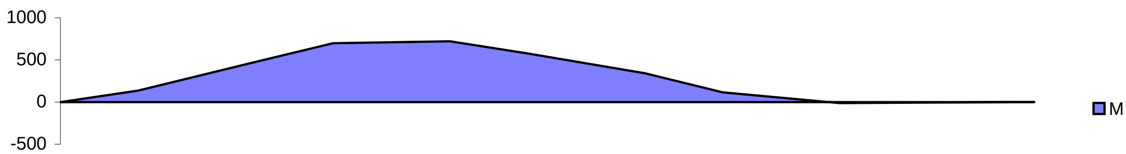
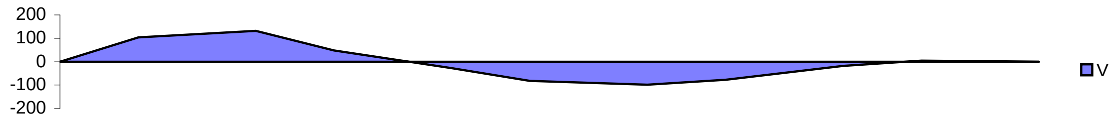
BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	2.50	5.00	7.50	10.00	12.50	15.00	17.50	20.00	22.50	25.00
$P_{u,w}$ (klf)	0.0	0.0	58.0	42.9	27.7	12.5	-2.7	-17.9	-33.1	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	-17	-296	-842	-1562	-2359	-3139	-3808	-4332	-4846
$V_{u,w}$ (kips)	0	0	-45	-171	-260	-310	-322	-296	-232	-206	-206
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-5	-22	-49	-87	-136	-196	-266	-348	-440	-544
$V_{u,f}$ (kips)	0	-4	-9	-13	-17	-22	-26	-30	-35	-39	-44
q_u (ksf)	-9.9	-7.4	-5.0	-2.5	-0.1	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	141	514	1043	1652	2275	2898	3521	4144	4767	5390
$V_{u,q}$ (kips)	0	108	185	233	249	249	249	249	249	249	249
ΣM_u (ft-k)	**0**	**136**	**475**	**699**	**722**	**577**	**343**	**115**	**-12**	**-5**	**0**
ΣV_u (kips)	**0**	**104**	**132**	**48**	**-28**	**-82**	**-99**	**-77**	**-18**	**4**	**0**





Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2 \phi b d (f_c')^{0.5}$	
Top Longitudinal	-12	ft-k	20.50	0.0001	0.0026	132	kips	132	kips
Bottom Longitudinal	722	ft-k	20.44	0.0068	0.0073	132	kips	132	kips
Bottom Transverse	5	ft-k / ft	19.50	0.0018	0.0019	4	kips / ft	25	kips / ft

Where $$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0206 [Satisfactory]

Daniel T. Li
Engineering International

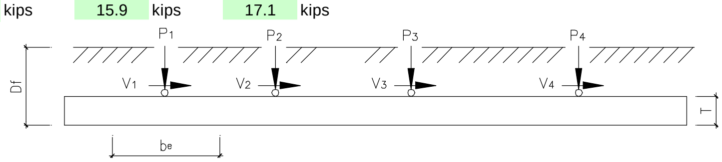
PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

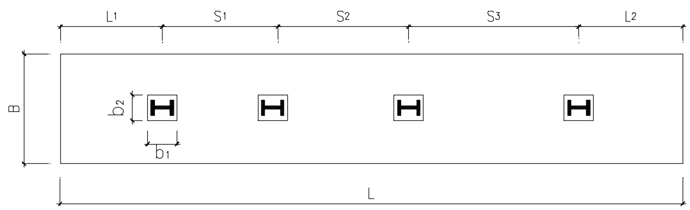
Grade Beam Design for Brace Frame Based on ACI 318-08

INPUT DATA

			COL#1		COL#2		COL#3		COL#4		
AXIAL DEAD LOAD	P_{DL}	=	15	kips	35	kips	39	kips	18	kips	<== Input 0 , if no column. Typical
AXIAL LIVE LOAD	P_{LL}	=	5.1	kips	4.3	kips	2.3	kips	2.3	kips	<== Non concurrent roof live load & lateral
LATERAL LOAD (0=WIND, 1=SEISMIC)				1	Seismic, SD						
SEISMIC AXIAL LOAD, SD	P_{LAT}	=	-80	kips	80	kips	-75	kips	75	kips	<== Negative value for uplift
SEISMIC SHEAR LOAD, SD	V_{LAT}	=	14.9	kips	18.8	kips	15.9	kips	17.1	kips	

STEEL COLUMN WIDTH	c_1	=	10.1	in
STEEL COLUMN DEPTH	c_2	=	8.02	in
BASE PLATE WIDTH	b_1	=	18	in
BASE PLATE DEPTH	b_2	=	18	in
CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE SOIL PRESSURE	Q_a	=	2	ksf
DISTANCE TO LEFT EDGE	L_1	=	5	ft
DISTANCE BETWEEN COLUMNS	S_1	=	20	ft
	S_2		16	ft
	S_3		20	ft
DISTANCE TO RIGHT EDGE	L_2	=	5	ft
FOOTING WIDTH	B	=	7.5	ft
FTG EMBEDMENT DEPTH	D_f	=	3	ft
FOOTING THICKNESS	T	=	24	in
SURCHARGE	q_s	=	0.1	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
LONGITUDINAL REINFORCING BAR SIZE		#	8	
TRANSVERSE REINFORCING BAR SIZE		#	5	





BAND WIDTH	b_e = 7.5	ft, for each col.
LONG. REINF AT TOP	5 # 8 @ 20 in o.c., cont.	
LONG. REINF AT BOTTOM	13 # 8 @ 7 in o.c., cont.	
TRANS. REINF. AT BAND WIDTH	7 # 5 @ 14 in o.c., bottom	

DESIGN SUMMARY

FOOTING LENGTH	L	=	66.00	ft
FOOTING WIDTH	B	=	7.50	ft
FOOTING THICKNESS	T	=	24	in

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS AT TOP OF GRADE BEAM (IBC SEC.1605.3.2 & ACI 318 SEC.9.2.1)

SERVICE LOADS		COL # 1		COL # 2		COL # 3		COL #4			TOTAL	
CASE 1 : DL + LL	P =	20	k	39	k	41	k	20	k		121.0	k
										(e =	0.18	ft, fr CL of GB)
CASE 2 : DL + LL + E / 1.4	P =	-37	k	96	k	-12	k	74	k		121.0	k
										(e =	18.48	ft, fr CL of GB)
	V =	11	k	13	k	11	k	12	k		47.6	k
CASE 3 : 0.9 DL + E / 1.4	P =	-44	k	89	k	-18	k	70	k		96.3	k
										(e =	24.08	ft, fr CL of GB)
	V =	11	k	13	k	11	k	12	k		47.6	k
FACTORED LOADS												
CASE 1 : 1.2 DL + 1.6 LL	P_u =	26	k	49	k	50	k	25	k		150.8	k
										(e_u =	-0.08	ft, fr CL of GB)
CASE 2 : 1.2 DL + 1.0 LL + 1.0 E	P_u =	-57	k	126	k	-26	k	99	k		142.4	k
										(e_u =	22.08	ft, fr CL of GB)
	V_u =	15	k	19	k	16	k	17	k		66.7	k
CASE 3 : 0.9 DL + 1.0 E	P_u =	-67	k	112	k	-40	k	91	k		96.3	k
										(e_u =	33.28	ft, fr CL of GB)
	V_u =	15	k	19	k	16	k	17	k		66.7	k

CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 3.13 **>** F = 0.75 / 0.9 = 0.83 **[Satisfactory]**

Where M_O = $(V_{LAT\,1} + V_{LAT\,2} + V_{LAT\,3} + V_{LAT\,4})\,T - P_{LAT\,1}(L - L_1) - P_{LAT\,2}(S_2 + S_3 + L_2) - P_{LAT\,3}(S_3 + L_2) - P_{LAT\,4}\,L_2$ = 3233 k-ft

P_{ftg} = (0.15 kcf) T B L = 148.50 k, footing weight

P_{soil} = $w_s\,(D_f - T)\,B\,L$ = 54.45 k, soil weight

M_R = $P_{DL\,1}(L - L_1) + P_{DL\,2}(S_2 + S_3 + L_2) + P_{DL\,3}(S_3 + L_2) + P_{DL\,4}\,L_2 + 0.5\,(P_{ftg} + P_{soil})\,L$ = 10112 k-ft

(cont'd)

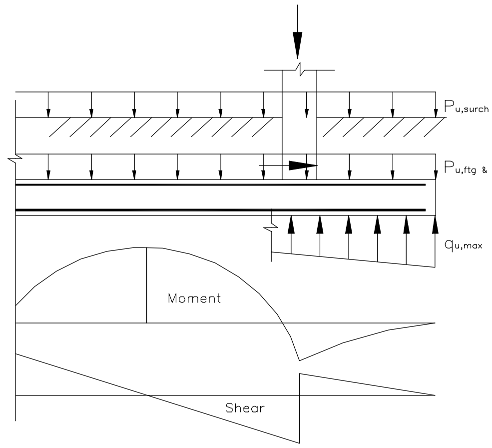
CHECK SOIL BEARING CAPACITY (ACI 318 SEC.15.2.2)

Service Loads	CASE 1		CASE 2		CASE 3		
P	121.0		121.0		96.3		k
e	0.2		19.3		25.1		ft, (at base, including V T / P)
q_S B L	49.5		49.5		0.0		k, (surcharge load)
$(0.15-w_S)$T B L	39.6		39.6		35.6		k, (footing increased)
Σ P	210.1		210.1		131.9		k
e	0.1	< L/6	11.1	> L/6	18.3	> L/6	ft
q_{max}	0.4		0.9		0.8		ksf
q_{allow}	2.0		2.7		2.7		ksf

Where

$$q_{MAX} = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

[Satisfactory]



DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)

$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d},\ \frac{4}{3}\rho\right)$$

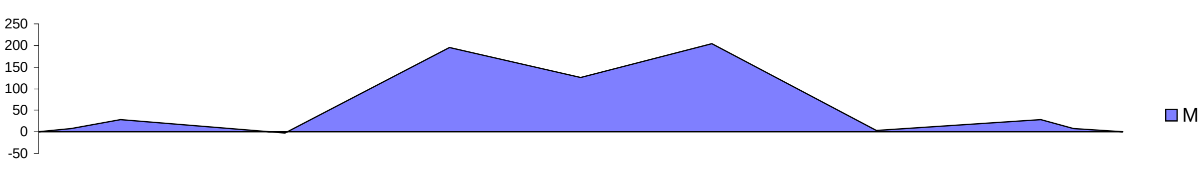
$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f'_c}}\right)}{f_y}$$

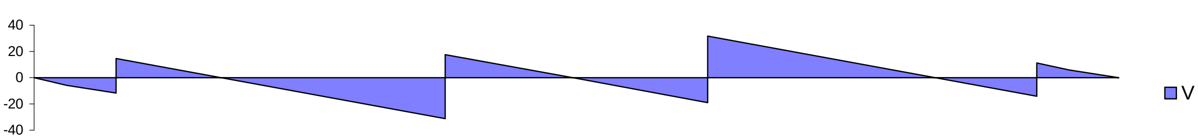
FACTORED SOIL PRESSURE

Factored Loads	CASE 1		CASE 2		CASE 3		
P_u	150.8		142.4		96.3		k
e_u	-0.1		23.0		34.7		ft, (at base, including V_u T / P_u)
γ q_S B L	79.2		49.5		0.0		k, (factored surcharge load)
γ [0.15 T + w_S (D_f - T)] B L	243.5		243.5		182.7		k, (factored footing & backfill loads)
Σ P_u	473.5		435.4		279.0		k
e_u	0.0	< L/6	7.5	< L/6	12.0	> L/6	ft
$q_{u,\ max}$	0.954		1.482		1.179		ksf

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **1**

Section	0	mid L_1	C 1 left	C 1 right	mid S_1	C 2 left	C 2 right	mid S_2	C 3 left	C 3 right	mid S_3	C 4 left	C 4 right	mid L_2	L
X_u (ft)	0	2.50	5.00	5.00	15.00	25.00	25.00	33.00	41.00	41.00	51.00	61.00	61.00	63.50	66.00
$M_{u,col}$ (ft-k)	0	0	0	0	-262	-523	-523	-1,124	-1,724	-1,724	-2,979	-4,234	-4,234	-4,611	-4,988
$V_{u,col}$ (k)	0	0.0	0.0	26.2	26.2	26.2	75.0	75.0	75.0	125.5	125.5	125.5	150.8	150.8	150.8
$P_{u,surch}$ (klf)	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
$M_{u,surch}$ (ft-k)	0	-4	-15	-15	-135	-375	-375	-653	-1009	-1009	-1561	-2233	-2233	-2419	-2614
$V_{u,surch}$ (k)	0	3.0	6.0	6.0	18.0	30.0	30.0	39.6	49.2	49.2	61.2	73.2	73.2	76.2	79.2
$P_{u,ftg\ \&\ fill}$ (klf)	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-12	-46	-46	-415	-1153	-1153	-2009	-3101	-3101	-4799	-6865	-6865	-7440	-8037
$V_{u,ftg\ \&\ fill}$ (k)	0	9.2	18.5	18.5	55.4	92.3	92.3	121.8	151.3	151.3	188.2	225.1	225.1	234.3	243.5
$q_{u,soil}$ (ksf)	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.95	0.95	0.95	0.95
$M_{u,soil}$ (ft-k)	0	22	90	90	809	2246	2246	3913	6038	6038	9341	13360	13360	14477	15639
$V_{u,soil}$ (k)	0	-18.0	-35.9	-35.9	-107.8	-179.6	-179.6	-237.0	-294.4	-294.4	-366.1	-437.7	-437.7	-455.6	-473.5
Σ M_u (ft-k)	**0**	**7**	**29**	**29**	**-3**	**195**	**195**	**126**	**205**	**205**	**3**	**28**	**28**	**7**	**0**
Σ V_u (kips)	**0**	**-5.8**	**-11.5**	**14.7**	**-8.3**	**-31.2**	**17.7**	**-0.6**	**-18.9**	**31.6**	**8.8**	**-13.9**	**11**	**5.7**	**0.0**

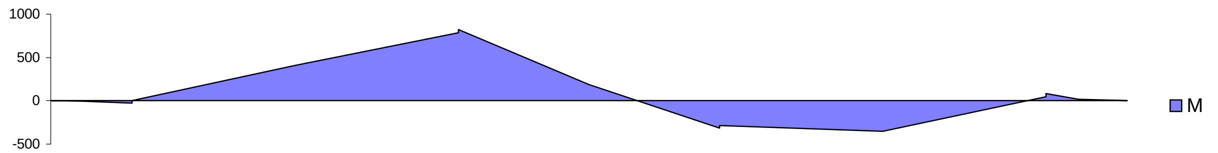


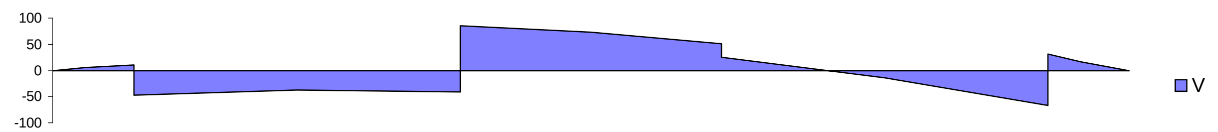


(cont'd)

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **2**

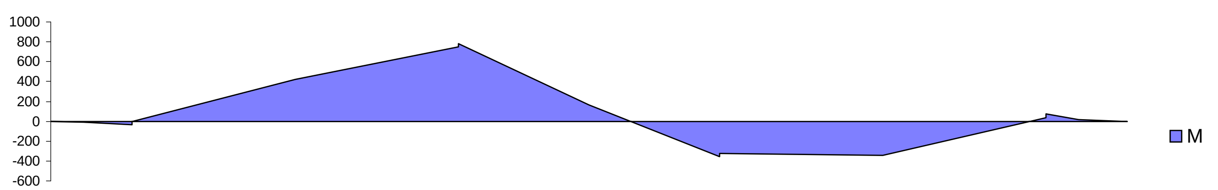
Section	0	mid L_1	C 1 left	C 1 right	mid S_1	C 2 left	C 2 right	mid S_2	C 3 left	C 3 right	mid S_3	C 4 left	C 4 right	mid L_2	L
X_u (ft)	0	2.50	5.00	5.00	15.00	25.00	25.00	33.00	41.00	41.00	51.00	61.00	61.00	63.50	66.00
$M_{u,col}$ (ft-k)	0	0	0	30	599	1,168	1,205	650	95	127	-308	-743	-709	-1,065	-1,421
$V_{u,col}$ (k)	0	0.0	0.0	-56.9	-56.9	-56.9	69.4	69.4	69.4	43.5	43.5	43.5	142.4	142.4	142.4
$P_{u,surch}$ (klf)	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
$M_{u,surch}$ (ft-k)	0	-2	-9	-9	-84	-234	-234	-408	-630	-630	-975	-1395	-1395	-1512	-1634
$V_{u,surch}$ (k)	0	1.9	3.8	3.8	11.3	18.8	18.8	24.8	30.8	30.8	38.3	45.8	45.8	47.6	49.5
$P_{u,ftg\ \&\ fill}$ (klf)	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-12	-46	-46	-415	-1153	-1153	-2009	-3101	-3101	-4799	-6865	-6865	-7440	-8037
$V_{u,ftg\ \&\ fill}$ (k)	0	9.2	18.5	18.5	55.4	92.3	92.3	121.8	151.3	151.3	188.2	225.1	225.1	234.3	243.5
$q_{u,soil}$ (ksf)	0.28	0.32	0.37	0.37	0.55	0.73	0.73	0.88	1.03	1.03	1.21	1.39	1.39	1.44	1.48
$M_{u,soil}$ (ft-k)	0	7	29	29	311	1007	1007	1953	3322	3322	5733	9050	9050	10037	11091
$V_{u,soil}$ (k)	0	-5.6	-12.1	-12.1	-46.6	-94.8	-94.8	-143.2	-200.4	-200.4	-284.1	-381.6	-381.6	-408.1	-435.4
Σ M_u (ft-k)	**0**	**-7**	**-27**	**3**	**411**	**787**	**825**	**186**	**-315**	**-283**	**-349**	**46**	**81**	**20**	**0**
Σ V_u (kips)	**0**	**5.5**	**10.1**	**-46.8**	**-36.9**	**-40.7**	**85.6**	**72.7**	**51.1**	**25.2**	**-14.2**	**-67.2**	**31.7**	**16.3**	**0.0**

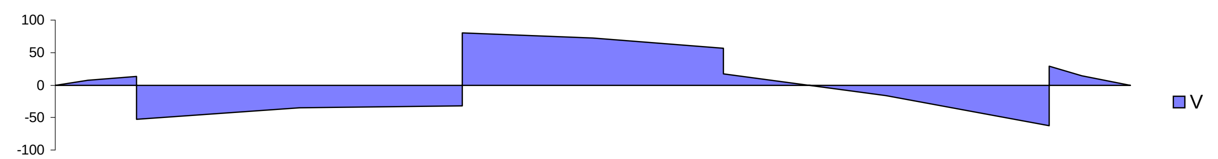




FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **3**

Section	0	mid L_1	C 1 left	C 1 right	mid S_1	C 2 left	C 2 right	mid S_2	C 3 left	C 3 right	mid S_3	C 4 left	C 4 right	mid L_2	L
X_u (ft)	0	2.50	5.00	5.00	15.00	25.00	25.00	33.00	41.00	41.00	51.00	61.00	61.00	63.50	66.00
$M_{u,col}$ (ft-k)	0	0	0	30	695	1,360	1,397	1,037	677	709	658	607	641	401	160
$V_{u,col}$ (k)	0	0.0	0.0	-66.5	-66.5	-66.5	45.0	45.0	45.0	5.1	5.1	5.1	96.3	96.3	96.3
$P_{u,surch}$ (klf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{u,surch}$ (ft-k)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
$V_{u,surch}$ (k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{u,ftg\ \&\ fill}$ (klf)	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-9	-35	-35	-311	-865	-865	-1507	-2326	-2326	-3599	-5149	-5149	-5580	-6028
$V_{u,ftg\ \&\ fill}$ (k)	0	6.9	13.8	13.8	41.5	69.2	69.2	91.3	113.5	113.5	141.1	168.8	168.8	175.7	182.7
$q_{u,soil}$ (ksf)	0.00	0.00	0.04	0.04	0.23	0.41	0.41	0.56	0.71	0.71	0.90	1.09	1.09	1.13	1.18
$M_{u,soil}$ (ft-k)	0	0	0	0	41	252	252	637	1292	1292	2599	4581	4581	5198	5868
$V_{u,soil}$ (k)	0	0.0	-0.3	-0.3	-10.3	-34.2	-34.2	-63.5	-101.7	-101.7	-162.1	-236.5	-236.5	-257.3	-279.0
Σ M_u (ft-k)	**0**	**-9**	**-34**	**-5**	**425**	**747**	**785**	**168**	**-357**	**-325**	**-342**	**39**	**73**	**19**	**0**
Σ V_u (kips)	**0**	**6.9**	**13.5**	**-53.0**	**-35.3**	**-31.5**	**80.0**	**72.8**	**56.8**	**16.9**	**-15.9**	**-62.6**	**28.6**	**14.7**	**0.0**





DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}(in)	use	ρ_{provD}
Top Longitudinal	-357	ft-k	21.50	0.0020	0.0019	0.0155	no limit	5 # 8 @ 20 in o.c., cont.	0.0020
Bottom Longitudinal	825	ft-k	20.50	0.0021	0.0052	0.0155	18	13 # 8 @ 7 in o.c., cont.	0.0056
Bottom Transverse, b_e	2	ft-k / ft	19.69	0.0011	0.0001	0.0155	18	7 # 5 @ 14 in o.c.	0.0012

[Satisfactory]

(cont'd)

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$		$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$		check $V_u < \phi\ V_c$
Longitudinal	86	k	152	k	**[Satisfactory]**
Transverse	1	k / ft	19	k / ft	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J}$$

$$A_P = 2(b_1 + b_2)d$$

$$\phi v_c(psi) = \phi(2 + y)\sqrt{f'_c}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right]$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

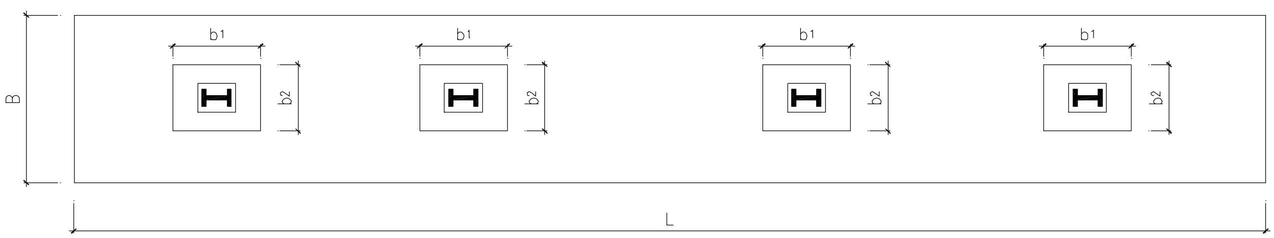
$$R = \frac{P_u b_1 b_2}{A_f}$$

$$A_f = B b_e$$

$$b_0 = \frac{A_P}{d},\ b_1 = (c_1 + d),\ b_2 = (c_2 + d)$$

Column	Case	P_u	M_u	b_1	b_2	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\ (psi)}$	$\phi\ v_c$
Col. 1	1	26.2	0.0	33.7	32.7	0.4	1.3	2.0	56.3	18.2	3.6	25.8	8.6	164.3
	2	0.0	14.9	33.7	32.7	0.4	1.3	2.0	56.3	18.2	0.0	25.8	2.3	164.3
	3	0.0	14.9	33.7	32.7	0.4	1.3	2.0	56.3	18.2	0.0	25.8	2.3	164.3
Col. 2	1	48.9	0.0	33.7	32.7	0.4	1.3	2.0	56.3	18.2	6.7	25.8	16.1	164.3
	2	126.3	18.8	33.7	32.7	0.4	1.3	2.0	56.3	18.2	17.2	25.8	44.6	164.3
	3	111.5	18.8	33.7	32.7	0.4	1.3	2.0	56.3	18.2	15.2	25.8	39.7	164.3
Col. 3	1	50.5	0.0	33.7	32.7	0.4	1.3	2.0	56.3	18.2	6.9	25.8	16.7	164.3
	2	0.0	15.9	33.7	32.7	0.4	1.3	2.0	56.3	18.2	0.0	25.8	2.4	164.3
	3	0.0	15.9	33.7	32.7	0.4	1.3	2.0	56.3	18.2	0.0	25.8	2.4	164.3
Col. 4	1	25.3	0.0	33.7	32.7	0.4	1.3	2.0	56.3	18.2	3.4	25.8	8.3	164.3
	2	98.9	17.1	33.7	32.7	0.4	1.3	2.0	56.3	18.2	13.5	25.8	35.3	164.3
	3	91.2	17.1	33.7	32.7	0.4	1.3	2.0	56.3	18.2	12.4	25.8	32.7	164.3

[Satisfactory]



Daniel T. Li
Engineering International

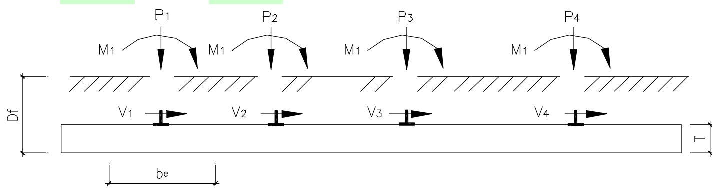
PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

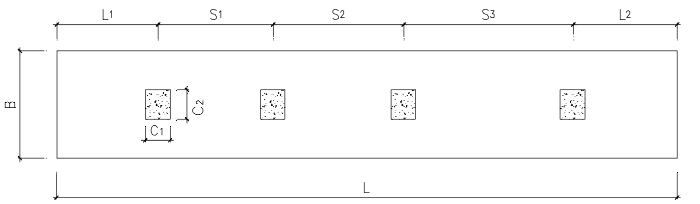
Grade Beam Design for Moment Resisting Frame Based on ACI 318-08

INPUT DATA

		COL#1		COL#2		COL#3		COL#4		
AXIAL DEAD LOAD	P_{DL} =	15	kips	35	kips	39	kips	18	kips	<== Input 0 , if no column. Typical
AXIAL LIVE LOAD	P_{LL} =	5.1	kips	4.3	kips	2.3	kips	2.3	kips	<== Non concurrent roof live load & lateral
LATERAL LOAD (0=WIND, 1=SEISMIC)		1	Seismic, SD							
SEISMIC AXIAL LOAD, SD	P_{LAT} =	10	kips	15	kips	12	kips	25	kips	<== Negative value for uplift
SEISMIC BENDING LOAD, SD	M_{LAT} =	295	ft-kips	501.5	ft-kips	531	ft-kips	324.5	ft-kips	
SEISMIC SHEAR LOAD, SD	V_{LAT} =	14.9	kips	18.8	kips	15.9	kips	17.1	kips	
COLUMN WIDTH	c_1 =	36	in	36	in	36	in	36	in	
COLUMN DEPTH	c_2 =	24	in	24	in	24	in	24	in	

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE SOIL PRESSURE	Q_a	=	2	ksf
DISTANCE TO LEFT EDGE	L_1	=	2	ft
DISTANCE BETWEEN COLUMNS	S_1	=	20	ft
	S_2		16	ft
	S_3		20	ft
DISTANCE TO RIGHT EDGE	L_2	=	2	ft
FOOTING WIDTH	B	=	4	ft
FTG EMBEDMENT DEPTH	D_f	=	4	ft
FOOTING THICKNESS	T	=	24	in
SURCHARGE	q_s	=	0.1	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
LONGITUDINAL REINFORCING BAR SIZE		#	8	
TRANSVERSE REINFORCING BAR SIZE		#	5	





BAND WIDTH b_e = 4.0 ft, for each col.
LONG. REINF AT TOP 6 # 8 @ 8 in o.c., cont.
LONG. REINF AT BOTTOM 6 # 8 @ 8 in o.c., cont.
TRANS. REINF. AT BAND WIDTH 4 # 5 @ 14 in o.c., bottom

DESIGN SUMMARY

FOOTING LENGTH	L	=	60.00	ft
FOOTING WIDTH	B	=	4.00	ft
FOOTING THICKNESS	T	=	24	in

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS AT TOP OF GRADE BEAM (IBC SEC.1605.3.2 & ACI 318 SEC.9.2.1)

SERVICE LOADS		COL # 1		COL # 2		COL # 3		COL #4		TOTAL	
CASE 1 : DL + LL	P =	20	k	39	k	41	k	20	k	121.0	k
									(e =	0.18	ft, fr CL of GB)
CASE 2 : DL + LL + E / 1.4	P =	27	k	50	k	50	k	38	k	165.3	k
	M =	211	ft-k	358	ft-k	379	k	232	ft-k	1180.0	ft-k
									(e =	8.98	ft, fr CL of GB)
	V =	11	k	13	k	11	k	12	k	47.6	k
CASE 3 : 0.9 DL + E / 1.4	P =	21	k	42	k	44	k	34	k	140.6	k
	M =	211	ft-k	358	ft-k	379	k	232	ft-k	1180.0	ft-k
									(e =	11.15	ft, fr CL of GB)
	V =	11	k	13	k	11	k	12	k	47.6	k
FACTORED LOADS											
CASE 1 : 1.2 DL + 1.6 LL	P_u =	26	k	49	k	50	k	25	k	150.8	k
									(e_u =	-0.08	ft, fr CL of GB)
CASE 2 : 1.2 DL + 1.0 LL + 1.0 E	P_u =	33	k	61	k	61	k	49	k	204.4	k
	M_u =	295	ft-k	502	ft-k	531	k	325	ft-k	1652.0	ft-k
									(e_u =	10.24	ft, fr CL of GB)
	V_u =	15	k	19	k	16	k	17	k	66.7	k
CASE 3 : 0.9 DL + 1.0 E	P_u =	24	k	47	k	47	k	41	k	158.3	k
	M_u =	295	ft-k	502	ft-k	531	k	325	ft-k	1652.0	ft-k
									(e_u =	13.60	ft, fr CL of GB)
	V_u =	15	k	19	k	16	k	17	k	66.7	k

CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 15.04 **>** F = 0.75 / 0.9 = 0.83 **[Satisfactory]**

Where M_O = $\Sigma M_{LAT} + (\Sigma V_{LAT}) D_f - P_{LAT\,1}(L - L_1) - P_{LAT\,2}(S_2 + S_3 + L_2) - P_{LAT\,3}(S_3 + L_2) - P_{LAT\,4} L_2$ = 455 k-ft

P_{ftg} = (0.15 kcf) T B L = 72.00 k, footing weight

P_{soil} = $w_s (D_f - T)$ B L = 52.80 k, soil weight

M_R = $P_{DL\,1}(L - L_1) + P_{DL\,2}(S_2 + S_3 + L_2) + P_{DL\,3}(S_3 + L_2) + P_{DL\,4} L_2 + 0.5 (P_{ftg} + P_{soil}) L$ = 6838 k-ft

(cont'd)

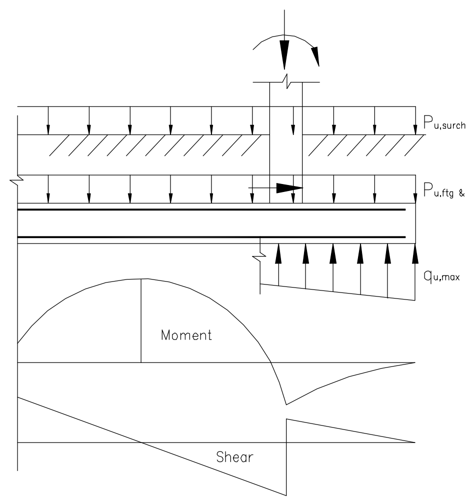
CHECK SOIL BEARING CAPACITY (ACI 318 SEC.15.2.2)

Service Loads	CASE 1	CASE 2	CASE 3	
P	121.0	165.3	140.6	k
e	0.2	9.6	11.8	ft, (at base, including V T / P)
q_s B L	24.0	24.0	0.0	k, (surcharge load)
$(0.15-w_s)$T B L	19.2	19.2	17.3	k, (footing increased)
Σ P	164.2	208.5	157.9	k
e	0.1 < L/6	7.6 < L/6	10.5 > L/6	ft
q_{max}	0.7	1.5	1.4	ksf
q_{allow}	2.0	2.7	2.7	ksf

Where

$$q_{MAX} = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

[Satisfactory]



DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)

$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} \qquad \rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

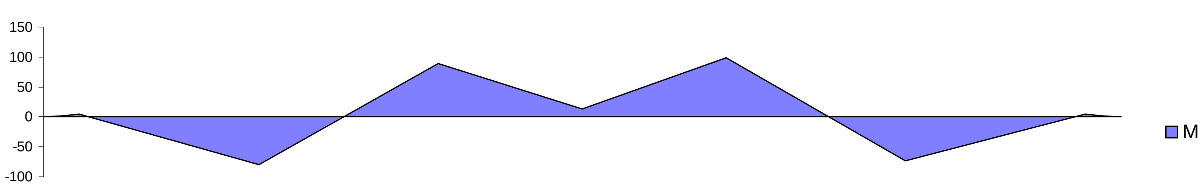
$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d},\ \frac{4}{3}\rho\right) \qquad \rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f'_c}}\right)}{f_y}$$

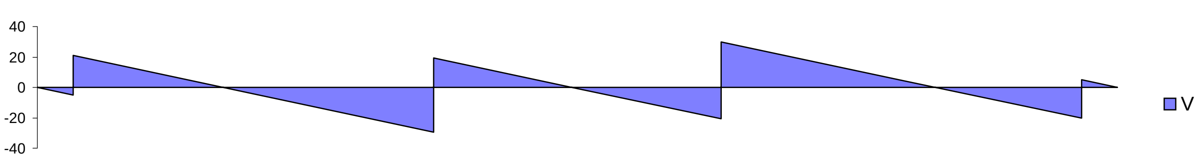
FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	150.8	204.4	158.3	k
e_u	-0.1	10.9	14.4	ft, (at base, including V_u T / P_u)
γ q_s B L	38.4	24.0	0.0	k, (factored surcharge load)
γ [0.15 T + w_s (D_f - T)] B L	149.8	149.8	112.3	k, (factored footing & backfill loads)
Σ P_u	339.0	378.2	270.6	k
e_u	0.0 < L/6	5.9 < L/6	8.4 < L/6	ft
$q_{u, max}$	1.407	2.503	2.080	ksf

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **1**

Section	0	mid L_1	C 1 left	C 1 right	mid S_1	C 2 left	C 2 right	mid S_2	C 3 left	C 3 right	mid S_3	C 4 left	C 4 right	mid L_2	L
X_u (ft)	0	1.00	2.00	2.00	12.00	22.00	22.00	30.00	38.00	38.00	48.00	58.00	58.00	59.00	60.00
$M_{u,col}$ (ft-k)	0	0	0	0	-262	-523	-523	-1,124	-1,724	-1,724	-2,979	-4,234	-4,234	-4,385	-4,536
$V_{u,col}$ (k)	0	0.0	0.0	26.2	26.2	26.2	75.0	75.0	75.0	125.5	125.5	125.5	150.8	150.8	150.8
$P_{u,surch}$ (klf)	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64
$M_{u,surch}$ (ft-k)	0	0	-1	-1	-46	-155	-155	-288	-462	-462	-737	-1076	-1076	-1114	-1152
$V_{u,surch}$ (k)	0	0.6	1.3	1.3	7.7	14.1	14.1	19.2	24.3	24.3	30.7	37.1	37.1	37.8	38.4
$P_{u,ftg \& fill}$ (klf)	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496	2.496
$M_{u,ftg \& fill}$ (ft-k)	0	-1	-5	-5	-180	-604	-604	-1123	-1802	-1802	-2875	-4198	-4198	-4344	-4493
$V_{u,ftg \& fill}$ (k)	0	2.5	5.0	5.0	30.0	54.9	54.9	74.9	94.8	94.8	119.8	144.8	144.8	147.3	149.8
$q_{u,soil}$ (ksf)	1.42	1.42	1.42	1.42	1.42	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41
$M_{u,soil}$ (ft-k)	0	3	11	11	408	1371	1371	2548	4087	4087	6519	9514	9514	9844	10181
$V_{u,soil}$ (k)	0	-5.7	-11.3	-11.3	-68.0	-124.6	-124.6	-169.8	-214.9	-214.9	-271.4	-327.7	-327.7	-333.3	-339.0
Σ M_u (ft-k)	**0**	**1**	**5**	**5**	**-79**	**89**	**89**	**13**	**99**	**99**	**-73**	**5**	**5**	**1**	**0**
Σ V_u (kips)	**0**	**-2.5**	**-5.1**	**21.1**	**-4.2**	**-29.4**	**19.5**	**-0.7**	**-20.7**	**29.7**	**4.7**	**-20.3**	**5**	**2.5**	**0.0**

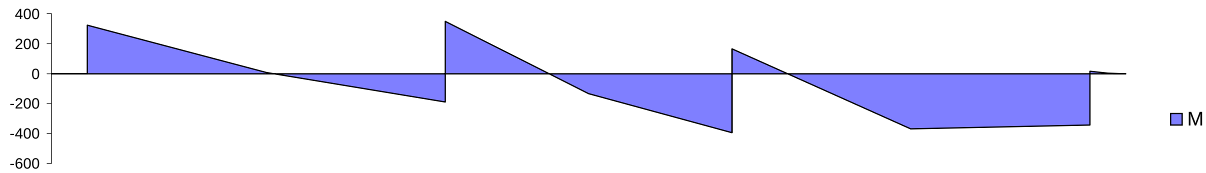


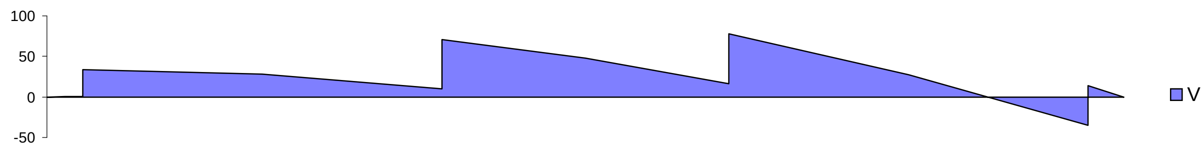


(cont'd)

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **2**

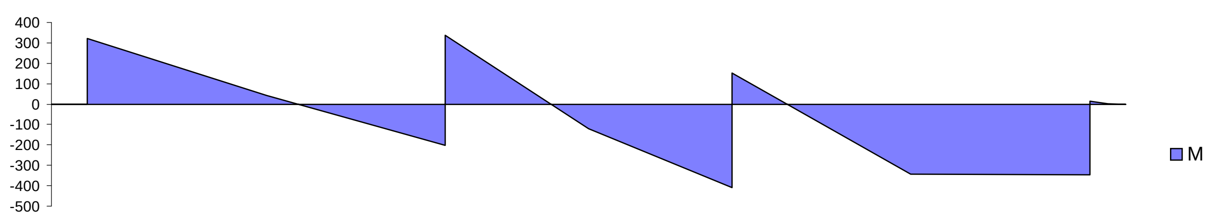
Section	0	mid L_1	C 1 left	C 1 right	mid S_1	C 2 left	C 2 right	mid S_2	C 3 left	C 3 right	mid S_3	C 4 left	C 4 right	mid L_2	L
X_u (ft)	0	1.00	2.00	2.00	12.00	22.00	22.00	30.00	38.00	38.00	48.00	58.00	58.00	59.00	60.00
$M_{u,col}$ (ft-k)	0	0	0	325	-6	-337	202	-553	-1,309	-746	-2,301	-3,856	-3,497	-3,701	-3,906
$V_{u,col}$ (k)	0	0.0	0.0	33.1	33.1	33.1	94.4	94.4	94.4	155.5	155.5	155.5	204.4	204.4	204.4
$P_{u,surch}$ (klf)	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
$M_{u,surch}$ (ft-k)	0	0	-1	-1	-29	-97	-97	-180	-289	-289	-461	-673	-673	-696	-720
$V_{u,surch}$ (k)	0	0.4	0.8	0.8	4.8	8.8	8.8	12.0	15.2	15.2	19.2	23.2	23.2	23.6	24.0
$P_{u,ftg\ \&\ fill}$ (klf)	2.496	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-1	-5	-5	-180	-604	-604	-1123	-1802	-1802	-2875	-4198	-4198	-4344	-4493
$V_{u,ftg\ \&\ fill}$ (k)	0	2.5	5.0	5.0	30.0	54.9	54.9	74.9	94.8	94.8	119.8	144.8	144.8	147.3	149.8
$q_{u,soil}$ (ksf)	0.65	0.68	0.71	0.71	1.02	1.33	1.33	1.58	1.82	1.82	2.13	2.44	2.44	2.47	2.50
$M_{u,soil}$ (ft-k)	0	1	5	5	222	847	847	1723	3003	3003	5266	8382	8382	8745	9119
$V_{u,soil}$ (k)	0	-2.7	-5.4	-5.4	-40.0	-87.0	-87.0	-133.4	-187.8	-187.8	-266.9	-358.4	-358.4	-368.2	-378.2
Σ M_u (ft-k)	**0**	**0**	**0**	**324**	**8**	**-191**	**348**	**-133**	**-397**	**166**	**-371**	**-345**	**14**	**4**	**0**
Σ V_u (kips)	**0**	**0.2**	**0.4**	**33.5**	**27.8**	**9.9**	**71.2**	**47.9**	**16.6**	**77.7**	**27.6**	**-34.9**	**14.0**	**7.1**	**0.0**

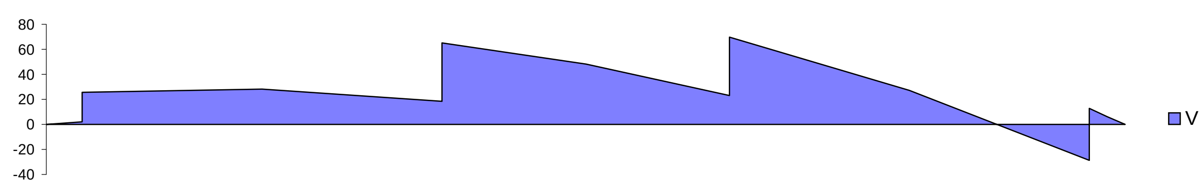




FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **3**

Section	0	mid L_1	C 1 left	C 1 right	mid S_1	C 2 left	C 2 right	mid S_2	C 3 left	C 3 right	mid S_3	C 4 left	C 4 right	mid L_2	L
X_u (ft)	0	1.00	2.00	2.00	12.00	22.00	22.00	30.00	38.00	38.00	48.00	58.00	58.00	59.00	60.00
$M_{u,col}$ (ft-k)	0	0	0	325	90	-145	394	-166	-726	-163	-1,334	-2,505	-2,147	-2,305	-2,463
$V_{u,col}$ (k)	0	0.0	0.0	23.5	23.5	23.5	70.0	70.0	70.0	117.1	117.1	117.1	158.3	158.3	158.3
$P_{u,surch}$ (klf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{u,surch}$ (ft-k)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
$V_{u,surch}$ (k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{u,ftg\ \&\ fill}$ (klf)	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-1	-4	-4	-135	-453	-453	-842	-1352	-1352	-2157	-3149	-3149	-3258	-3370
$V_{u,ftg\ \&\ fill}$ (k)	0	1.9	3.7	3.7	22.5	41.2	41.2	56.2	71.1	71.1	89.9	108.6	108.6	110.4	112.3
$q_{u,soil}$ (ksf)	0.18	0.21	0.24	0.24	0.56	0.87	0.87	1.13	1.38	1.38	1.70	2.02	2.02	2.05	2.08
$M_{u,soil}$ (ft-k)	0	0	2	2	87	395	395	887	1667	1667	3148	5308	5308	5566	5833
$V_{u,soil}$ (k)	0	-0.8	-1.7	-1.7	-17.6	-46.1	-46.1	-78.2	-118.3	-118.3	-179.9	-254.2	-254.2	-262.4	-270.6
Σ M_u (ft-k)	**0**	**-1**	**-2**	**323**	**42**	**-203**	**336**	**-122**	**-410**	**152**	**-343**	**-346**	**13**	**3**	**0**
Σ V_u (kips)	**0**	**1.1**	**2.1**	**25.6**	**28.4**	**18.5**	**65.0**	**48.0**	**22.8**	**69.9**	**27.0**	**-28.6**	**12.6**	**6.4**	**0.0**





DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}(in)	use	ρ_{provD}
Top Longitudinal	-410	ft-k	21.50	0.0020	0.0043	0.0155	no limit	6 # 8 @ 8 in o.c., cont.	0.0046
Bottom Longitudinal	348	ft-k	20.50	0.0021	0.0040	0.0155	18	6 # 8 @ 8 in o.c., cont.	0.0048
Bottom Transverse, b_e	2	ft-k / ft	19.69	0.0005	8.1E-05	0.0155	18	4 # 5 @ 14 in o.c.	0.0013

[Satisfactory]

(cont'd)

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$	$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$	check $V_u < \phi\ V_c$
Longitudinal	78 k	81 k	**[Satisfactory]**
Transverse	2 k / ft	19 k / ft	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J}$$

$$A_P = 2(b_1 + b_2)d$$

$$\phi v_c(psi) = \phi(2+y)\sqrt{f'_c}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right]$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

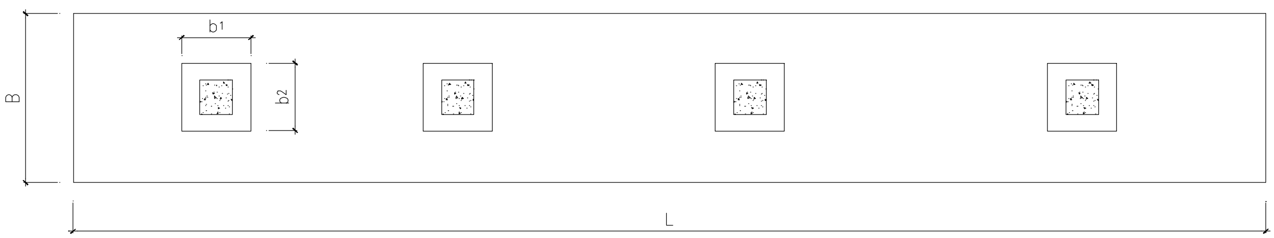
$$R = \frac{P_u b_1 b_2}{A_f}$$

$$A_f = B b_e$$

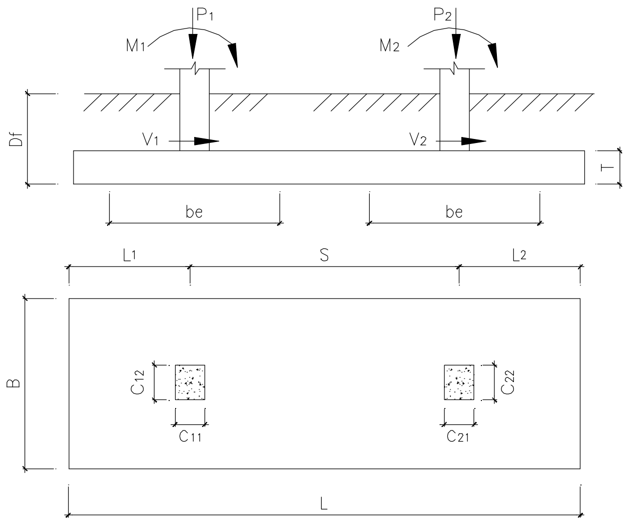
$$b_0 = \frac{A_P}{d},\ b_1 = (c_1 + d),\ b_2 = (c_2 + d)$$

Column	Case	P_u	M_u	b_1	b_2	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\ (psi)}$	$\phi\ v_c$
Col. 1	1	26.2	0.0	55.7	43.7	0.4	1.5	2.0	16.0	27.2	27.6	95.1	-0.4	164.3
	2	33.1	309.9	55.7	43.7	0.4	1.5	2.0	16.0	27.2	35.0	95.1	22.1	164.3
	3	23.5	309.9	55.7	43.7	0.4	1.5	2.0	16.0	27.2	24.8	95.1	22.2	164.3
Col. 2	1	48.9	0.0	55.7	43.7	0.4	1.5	2.0	16.0	27.2	51.6	95.1	-0.7	164.3
	2	61.3	520.3	55.7	43.7	0.4	1.5	2.0	16.0	27.2	64.7	95.1	37.0	164.3
	3	46.5	520.3	55.7	43.7	0.4	1.5	2.0	16.0	27.2	49.1	95.1	37.2	164.3
Col. 3	1	50.5	0.0	55.7	43.7	0.4	1.5	2.0	16.0	27.2	53.3	95.1	-0.7	164.3
	2	61.1	546.9	55.7	43.7	0.4	1.5	2.0	16.0	27.2	64.5	95.1	38.9	164.3
	3	47.1	546.9	55.7	43.7	0.4	1.5	2.0	16.0	27.2	49.7	95.1	39.1	164.3
Col. 4	1	25.3	0.0	55.7	43.7	0.4	1.5	2.0	16.0	27.2	26.7	95.1	-0.4	164.3
	2	48.9	341.6	55.7	43.7	0.4	1.5	2.0	16.0	27.2	51.6	95.1	24.2	164.3
	3	41.2	341.6	55.7	43.7	0.4	1.5	2.0	16.0	27.2	43.5	95.1	24.3	164.3

[Satisfactory]



Combined Footing Design Based on ACI 318-08



INPUT DATA

			COL#1	COL#2	
COLUMN WIDTH	c_1	=	18	18	in
COLUMN DEPTH	c_2	=	18	18	in
AXIAL DEAD LOAD	P_{DL}	=	13	26	k
AXIAL LIVE LOAD	P_{LL}	=	6.25	12.5	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	1	Seismic SD	
SEISMIC AXIAL LOAD, SD	P_{LAT}	=	-300	300	k
SEISMIC SHEAR LOAD, SD	V_{LAT}	=	12.5	13.75	k
SEISMIC MOMENT, SD	M_{LAT}	=	4.578	4.578	k-ft
CONCRETE STRENGTH	f_c'	=	3	ksi	
REBAR YIELD STRESS	f_y	=	60	ksi	
ALLOWABLE SOIL PRESSURE	Q_a	=	2	ksf	
DISTANCE TO LEFT EDGE	L_1	=	36	ft	
DISTANCE BETWEEN COLUMNS	S	=	30	ft	
DISTANCE TO RIGHT EDGE	L_2	=	36	ft	
FOOTING WIDTH	B	=	7.5	ft	
FTG EMBEDMENT DEPTH	D_f	=	5	ft	
FOOTING THICKNESS	T	=	48	in	
SURCHARGE	q_s	=	0.1	ksf	
SOIL WEIGHT	w_s	=	0.11	kcf	
LONGITUDINAL REINFORCING BAR SIZE		#	10		
TRANSVERSE REINFORCING BAR SIZE		#	5		

BAND WIDTH	b_e =	7.5 ft
LONG. REINF AT TOP		10 # 10 @ 9 in o.c., cont.
LONG. REINF AT BOTTOM		13 # 10 @ 7 in o.c., cont.
TRANS. REINF. AT BAND WIDTH		8 # 5 @ 12 in o.c., bottom

DESIGN SUMMARY

FOOTING LENGTH	L	=	102.00	ft
FOOTING WIDTH	B	=	7.50	ft
FOOTING THICKNESS	T	=	48	in

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS AT TOP OF FOOTING (IBC SEC.1605.3.2 & ACI 318 SEC.9.2.1)

SERVICE LOADS			COL # 1		COL # 2				TOTAL	
CASE 1 : DL + LL	P	=	19	k	39	k			58	k
							(e	=	5.00	ft, fr CL ftg)
CASE 2 : DL + LL + E / 1.4	P	=	-195	k	253	k			58	k
	M	=	12.2	ft-k	13.1	ft-k			25.3	ft-k
							(e	=	116.76	ft, fr CL ftg)
	V	=	9	k	10	k			19	k
CASE 3 : 0.9 DL + E / 1.4	P	=	-203	k	238	k			35	k
	M	=	12.2	ft-k	13.1	ft-k			25.3	ft-k
							(e	=	188.87	ft, fr CL ftg)
	V	=	9	k	10	k			19	k
FACTORED LOADS										
CASE 1 : 1.2 DL + 1.6 LL	Pu	=	26	k	51	k			77	k
							(eu	=	5.00	ft, fr CL ftg)
CASE 2 : 1.2 DL + 1.0 LL + 1.0 E	Pu	=	-278	k	344	k			66	k
	Mu	=	17.1	ft-k	18.3	ft-k			35.4	ft-k
							(eu	=	142.84	ft, fr CL ftg)
	Vu	=	13	k	14	k			26	k
CASE 3 : 0.9 DL + 1.0 E	Pu	=	-288	k	323	k			35	k
	Mu	=	17.1	ft-k	18.3	ft-k			35.4	ft-k
							(eu	=	262.42	ft, fr CL ftg)
	Vu	=	13	k	14	k			26	k

CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 3.24 **>** F = 0.75 / 0.9 = 0.83 **[Satisfactory]**

Where M_O = $M_{LAT\,1} + M_{LAT\,2} + (V_{LAT\,1} + V_{LAT\,2})\,T - P_{LAT\,1}(L - L_1) - P_{LAT\,2}L_2$ = 9114 k-ft

P_{ftg} = (0.15 kcf) T B L = 459.00 k, footing weight

P_{soil} = $w_s (D_f - T)\,B\,L$ = 84.15 k, soil weight

M_R = $P_{DL\,1}(L - L_1) + P_{DL\,2}L_2 + 0.5\,(P_{ftg} + P_{soil})\,L$ = 29495 k-ft

(cont'd)

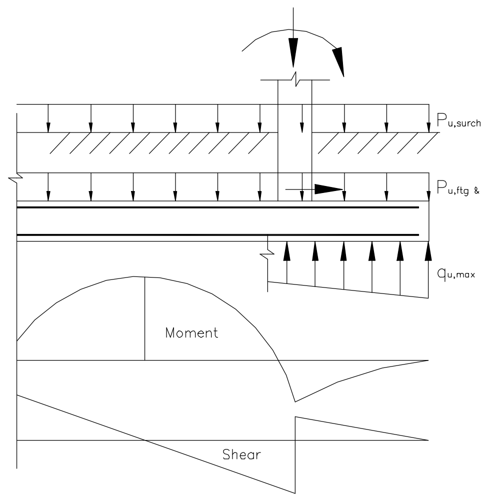
CHECK SOIL BEARING CAPACITY (ACI 318 SEC.15.2.2)

Service Loads	CASE 1		CASE 2		CASE 3		
P	57.8		57.8		35.1		k
e	5.0		118.1		191.0		ft, (at base, including V T / P)
q_S B L	76.5		76.5		0.0		k, (surcharge load)
$(0.15\text{-}w_S)$T B L	122.4		122.4		110.2		k, (footing increased)
Σ P	256.7		256.7		145.3		k
e	1.1	< L/6	26.6	> L/6	46.2	> L/6	ft
q_{max}	0.4		0.9		2.7		ksf
q_{allow}	2.0		2.7		2.7		ksf

Where

$$q_{MAX} = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

[Satisfactory]



DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)

$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d},\ \frac{4}{3}\rho\right)$$

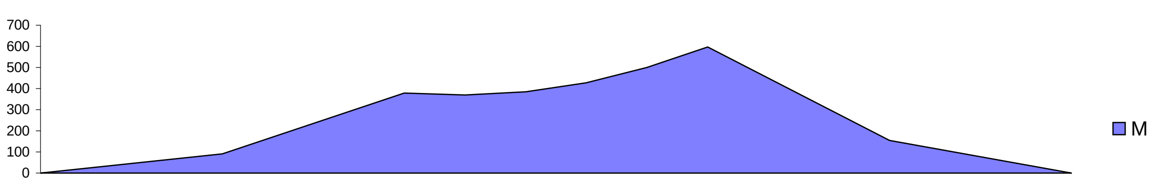
$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f'_c}}\right)}{f_y}$$

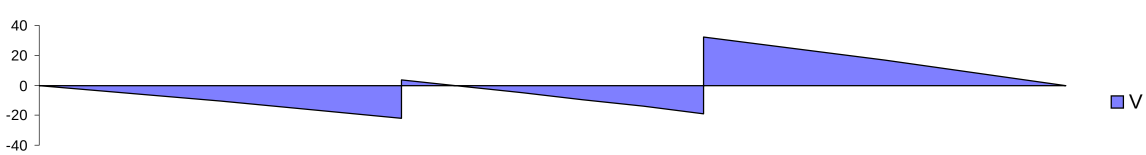
FACTORED SOIL PRESSURE

Factored Loads	CASE 1		CASE 2		CASE 3		
P_u	76.8		65.6		35.1		k
e_u	5.0		144.4		265.4		ft, (at base, including V_u T / P_u)
γ q_S B L	122.4		76.5		0.0		k, (factored surcharge load)
γ [0.15 T + w_S (D_f - T)] B L	651.8		651.8		488.8		k, (factored footing & backfill loads)
Σ P_u	851.0		793.8		523.9		k
e_u	0.5	< L/6	11.9	< L/6	17.8	> L/6	ft
$q_{u,\ max}$	1.142		1.766		1.402		ksf

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **1**

Section	0	0.5 L_1	L_1 left	L_1 right	0.2 S	0.4 S	0.6 S	0.8 S	L_2 left	L_2 right	0.5 L_2	L
X_u (ft)	0	18.00	36.00	36.00	42.00	48.00	54.00	60.00	66.00	66.00	84.00	102.00
$M_{u,col}$ (ft-k)	0	0	0	0	-154	-307	-461	-614	-768	-768	-2,150	-3,533
$V_{u,col}$ (k)	0	0.0	0.0	25.6	25.6	25.6	25.6	25.6	25.6	76.8	76.8	76.8
$P_{u,surch}$ (klf)	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
$M_{u,surch}$ (ft-k)	0	-194	-778	-778	-1058	-1382	-1750	-2160	-2614	-2614	-4234	-6242
$V_{u,surch}$ (k)	0	21.6	43.2	43.2	50.4	57.6	64.8	72.0	79.2	79.2	100.8	122.4
$P_{u,ftg\ \&\ fill}$ (klf)	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-1035	-4141	-4141	-5636	-7361	-9317	-11502	-13917	-13917	-22544	-33241
$V_{u,ftg\ \&\ fill}$ (k)	0	115.0	230.0	230.0	268.4	306.7	345.1	383.4	421.7	421.7	536.8	651.8
$q_{u,soil}$ (ksf)	1.08	1.09	1.10	1.10	1.11	1.11	1.11	1.12	1.12	1.12	1.13	1.14
$M_{u,soil}$ (ft-k)	0	1320	5296	5296	7217	9436	11955	14775	17897	17897	29082	43016
$V_{u,soil}$ (k)	0	-146.9	-295.2	-295.2	-344.9	-394.8	-444.9	-495.1	-545.5	-545.5	-697.5	-851.0
Σ M_u (ft-k)	**0**	**90**	**378**	**378**	**369**	**385**	**428**	**499**	**598**	**598**	**154**	**0**
Σ V_u (kips)	**0**	**-10.3**	**-21.9**	**3.7**	**-0.6**	**-4.9**	**-9.4**	**-14.1**	**-18.9**	**32.3**	**16.8**	**0**





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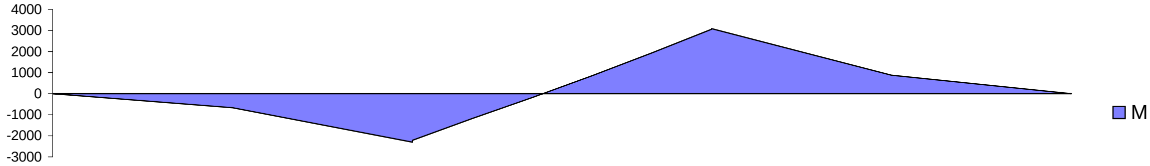
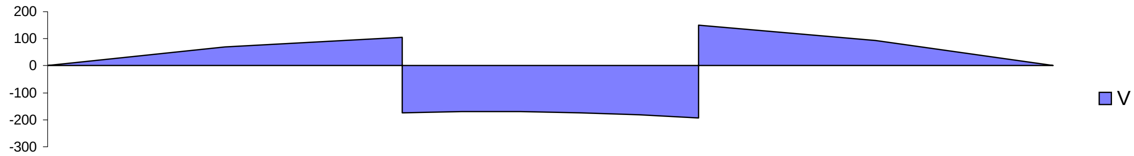
FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **2**

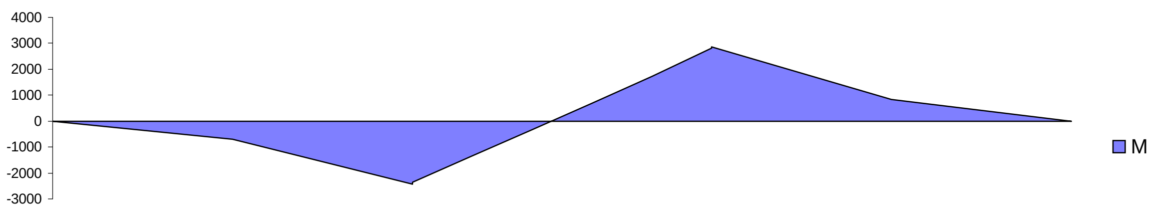
Section	0	0.5 L_1	L_1 left	L_1 right	0.2 S	0.4 S	0.6 S	0.8 S	L_2 left	L_2 right	0.5 L_2	L
X_u (ft)	0	18.00	36.00	36.00	42.00	48.00	54.00	60.00	66.00	66.00	84.00	102.00
$M_{u,col}$ (ft-k)	0	0	0	67	1,736	3,405	5,074	6,743	8,412	8,451	7,305	6,125
$V_{u,col}$ (k)	0	0.0	0.0	-278.2	-278.2	-278.2	-278.2	-278.2	-278.2	65.6	65.6	65.6
$P_{u,surch}$ (klf)	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
$M_{u,surch}$ (ft-k)	0	-122	-486	-486	-662	-864	-1094	-1350	-1634	-1634	-2646	-3902
$V_{u,surch}$ (k)	0	13.5	27.0	27.0	31.5	36.0	40.5	45.0	49.5	49.5	63.0	76.5
$P_{u,ftg\ \&\ fill}$ (klf)	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39	6.39
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-1035	-4141	-4141	-5636	-7361	-9317	-11502	-13917	-13917	-22544	-33241
$V_{u,ftg\ \&\ fill}$ (k)	0	115.0	230.0	230.0	268.4	306.7	345.1	383.4	421.7	421.7	536.8	651.8
$q_{u,soil}$ (ksf)	0.31	0.57	0.82	0.82	0.91	0.99	1.08	1.17	1.25	1.25	1.51	1.77
$M_{u,soil}$ (ft-k)	0	480	2337	2337	3370	4649	6196	8035	10188	10188	18770	31017
$V_{u,soil}$ (k)	0	-59.1	-153.0	-153.0	-192.0	-234.8	-281.5	-332.1	-386.5	-386.5	-572.8	-793.8
Σ M_u (ft-k)	**0**	**-676**	**-2289**	**-2222**	**-1191**	**-172**	**859**	**1925**	**3049**	**3088**	**885**	**0**
Σ V_u (kips)	**0**	**69.4**	**104.1**	**-174.1**	**-170.2**	**-170.2**	**-174.1**	**-181.8**	**-193.4**	**150.3**	**92.5**	**0**

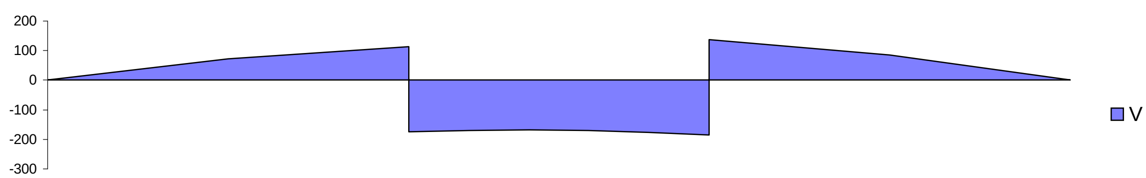




FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **3**

Section	0	0.5 L_1	L_1 left	L_1 right	0.2 S	0.4 S	0.6 S	0.8 S	L_2 left	L_2 right	0.5 L_2	L
X_u (ft)	0	18.00	36.00	36.00	42.00	48.00	54.00	60.00	66.00	66.00	84.00	102.00
$M_{u,col}$ (ft-k)	0	0	0	67	1797	3527	5256	6986	8716	8,755	8,158	7,526
$V_{u,col}$ (k)	0	0.0	0.0	-288.3	-288.3	-288.3	-288.3	-288.3	-288.3	35.1	35.1	35.1
$P_{u,surch}$ (klf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{u,surch}$ (ft-k)	0	0	0	0	0	0	0	0	0	0	0	0
$V_{u,surch}$ (k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{u,ftg\ \&\ fill}$ (klf)	4.79	4.79	4.79	4.79	4.79	4.79	4.79	4.79	4.79	4.79	4.79	4.79
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-776	-3106	-3106	-4227	-5521	-6987	-8627	-10438	-10438	-16908	-24931
$V_{u,ftg\ \&\ fill}$ (k)	0	86.3	172.5	172.5	201.3	230.0	258.8	287.6	316.3	316.3	402.6	488.8
$q_{u,soil}$ (ksf)	0.00	0.22	0.47	0.47	0.56	0.64	0.73	0.81	0.90	0.90	1.15	1.40
$M_{u,soil}$ (ft-k)	0	68	670	670	1097	1674	2424	3371	4536	4536	9575	17405
$V_{u,soil}$ (k)	0	-12.9	-59.8	-59.8	-83.0	-110.0	-140.8	-175.4	-213.8	-213.8	-351.8	-523.9
Σ M_u (ft-k)	**0**	**-709**	**-2435**	**-2368**	**-1333**	**-321**	**693**	**1730**	**2814**	**2853**	**824**	**0**
Σ V_u (kips)	**0**	**73.3**	**112.8**	**-175.5**	**-170.0**	**-168.2**	**-170.3**	**-176.1**	**-185.8**	**137.6**	**85.9**	**0**





DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}(in)	use	ρ_{provD}
Top Longitudinal	-2435	ft-k	45.37	0.0019	0.0030	0.0155	no limit	10 # 10 @ 9 in o.c., cont.	0.0031
Bottom Longitudinal	3088	ft-k	44.37	0.0019	0.0041	0.0155	18	13 # 10 @ 7 in o.c., cont.	0.0041
Bottom Transverse, b_e	1	ft-k / ft	43.42	0.0006	6.9E-06	0.0155	18	8 # 5 @ 12 in o.c.	0.0006

[Satisfactory]

(cont'd)

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$	$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$	check $V_u < \phi\ V_c$
Longitudinal	193 k	328 k	**[Satisfactory]**
Transverse	0 k / ft	43 k / ft	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J}$$

$$A_P = 2\left(b_1 + b_2\right)d$$

$$\phi v_c(psi) = \phi\left(2 + y\right)\sqrt{f'_c}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right]$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

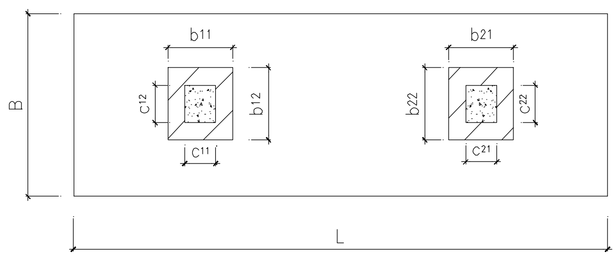
$$R = \frac{P_u b_1 b_2}{A_f}$$

$$A_f = B b_e$$

$$b_0 = \frac{A_P}{d},\ b_1 = \left(c_1 + d\right),\ b_2 = \left(c_2 + d\right)$$

Column	Case	P_u	M_u	b_1	b_2	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\ (psi)}$	$\phi\ v_c$
Col. 1	1	25.6	0.0	61.4	61.4	0.4	1.0	2.0	56.3	74.1	11.9	363.8	1.3	164.3
	2	0.0	0.0	61.4	61.4	0.4	1.0	2.0	56.3	74.1	0.0	363.8	0.0	164.3
	3	0.0	0.0	61.4	61.4	0.4	1.0	2.0	56.3	74.1	0.0	363.8	0.0	164.3
Col. 2	1	51.2	0.0	61.4	61.4	0.4	1.0	2.0	56.3	74.1	23.8	363.8	2.6	164.3
	2	343.7	18.3	61.4	61.4	0.4	1.0	2.0	56.3	74.1	160.1	363.8	17.6	164.3
	3	323.4	18.3	61.4	61.4	0.4	1.0	2.0	56.3	74.1	150.6	363.8	16.6	164.3

[Satisfactory]



Daniel T. Li
Engineering International

PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

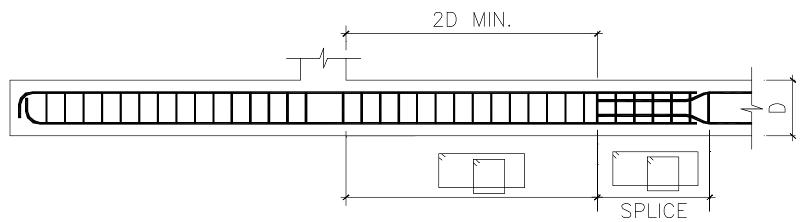
Seismic Design for Combined Footing, Based on ACI 318-08

DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	3	ksi
REBAR YIELD STRESS	f_y =	60	ksi
FOOTING WIDTH	W =	90	in
FOOTING THICKNESS	D =	48	in
DISTANCE BETWEEN COLUMN	L =	30	ft

COMBINED FOOTING LONGITUDINAL REINFORCING

TOP	12	#	10
	(d =	43.74	in)
	(1 Layer)		
BOTTOM	13	#	10
	(d =	43.74	in)
	(1 Layer)		



COMBINED FOOTING HOOPS (ACI 21.5.3)

LOCATION	AT END	AT SPLICE
LENGTH	96 in	70 in
	(2h)	$MAX\{0.075 f_y \alpha\beta\gamma d_b / [(f_c')^{0.5}(c+K_{tr})/d_b], 12\}$
BAR	7 Legs # 5	7 Legs # 5
SPACING	@ 10 in o.c.	@ 4 in o.c.
	$MIN(d/4, 8d_b, 24d_t, 12)$	MIN(d/4, 4)

THE SEISMIC DESIGN IS ADEQUATE.

ANALYSIS

CHECK GB SECTION REQUIREMENTS (ACI 21.5.1)

$L_n = L - c_1 =$ 28.50 ft **>** 4 d = 14.58 ft **[Satisfactory]**

W / D = 1.88 **>** 0.3 **[Satisfactory]**

W = 90 in **>** 10 in **[Satisfactory]**

< $c_1 + 1.5D =$ 90 in **[Satisfactory]**

CHECK SEISMIC FLEXURAL REQUIREMENTS

(ACI 21.5.2.1) ρ_{top} = 0.004 **>** $\rho_{min} = MIN[3(f_c')^{0.5}/f_y, 200/f_y] =$ 0.003 **[Satisfactory]**

< ρ_{max} = 0.025 **[Satisfactory]**

ρ_{bot} = 0.004 **>** ρ_{min} = 0.003 **[Satisfactory]**

< ρ_{max} = 0.025 **[Satisfactory]**

(ACI 21.5.2.2) $M_{n,top}$ **>** $(1/2)M_{n,bot}$ **[Satisfactory]**

where $M_{n,bot} = \rho_{bot} bd^2 f_y (1 - 0.588\rho_{bot} f_y/f_c') =$ 3433 ft-kips

$M_{n,top} = \rho_{top} bd^2 f_y (1 - 0.588\rho_{top} f_y/f_c') =$ 3181 ft-kips

CHECK GB SHEAR STRENGTH (ACI 21.5.4)

$V_e = (M_{pr,top} + M_{pr,bot}) / L_n =$ 286.5 kips **<** $8\phi(f_c')^{0.5}bd =$ 1293.7 kips **[Satisfactory]**

< $\phi[2(f_c')^{0.5}bd + A_v f_y d/s] =$ 750.5 kips **[Satisfactory]**

where $M_{pr,top} = \rho_{top} bd^2 f_y (1.25 - 0.919\rho_{top} f_y/f_c') =$ 3929 ft-kips

$M_{pr,bot} = \rho_{bot} bd^2 f_y (1.25 - 0.919\rho_{bot} f_y/f_c') =$ 4235 ft-kips

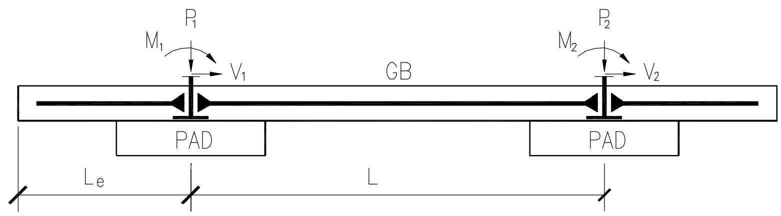
ϕ = 0.75 (ACI 9.3.2.3)

A_v = 2.17 in^2

Two Pads with Grade Beam Design Based on ACI 318-08 & AISC 360-05

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH		f_c' =	4	ksi
REBAR YIELD STRESS		f_y =	60	ksi
SQUARE PAD SIZE		B =	8	ft
		T =	16	in
GRADE BEAM SIZE	Steel		W18X65	
	Outside Concrete Width	W =	36	in
	Outside Concrete Depth	D =	36	in
COLUMN DISTANCE		L =	22	ft
GRADE BEAM EXTENSION		L_e =	5	ft



FRAME AXIAL LOADS, ASD	$P_{D,1}$ =	25	kips	$P_{D,2}$ =	25	kips	(Dead Load)
	$P_{L,1}$ =	15	kips	$P_{L,2}$ =	15	kips	(Live Load)
GRAVITY AXIAL LOADS, ASD	$P_{E,1}$ =	-30	kips	$P_{E,2}$ =	30	kips	(Seismic Load)
	$V_{E,1}$ =	50	kips	$V_{E,2}$ =	30	kips	(Seismic Load)
SEISMIC LOADS, ASD	$M_{E,1}$ =	50	ft-kips	$M_{E,2}$ =	50	ft-kips	(Seismic Load)

ALLOW SOIL PRESSURE Q_a = 2.5 ksf

PAD REINFORCING 8 # 6 @ 12 o.c., Each Way, Bottom.

THE GRADE BEAM DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING AT CENTER BOTTOM OF PAD 2 (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

$M_O = M_{E,1} + M_{E,2} + (V_{E,1}+V_{E,2})(D+T) - P_{E,1}L$ = 1106.7 ft-kips

$M_R = (P_{D,1} + \gamma_{conc}\, B^2T)\, L + 0.5\gamma_{conc\ \&\ steel}(L + 2L_e)\, L\, D\, W$ = 1322.7 > $0.75 \times 1.4\, M_O / (0.9)$ = 1291 ft-kips

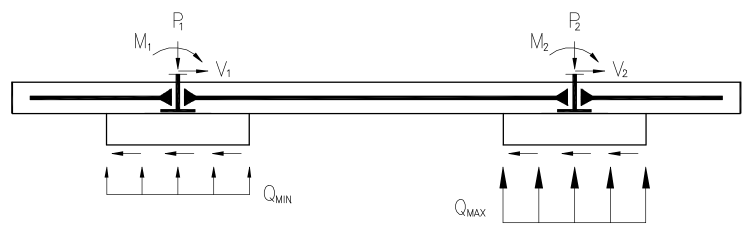
[Satisfactory]

CHECK SOIL BEARING CAPACITY

$$Q_{MAX} = \frac{M_O}{B^2L} + \frac{P_{D,2}+P_{L,2}+\left(\gamma_{Conc\&Steel}-\gamma_{SOIL}\right)\left[B^2T+WD\left(0.5L+L_e\right)\right]}{B^2} =$$ 1.57 ksf, (net pressure) < 4/3 Q_a [Satisfactory]

where γ_{conc} = 0.15 kcf

γ_{soil} = 0.11 kcf



CHECK PAD FLEXURAL REINFORCING

$$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\frac{M_u}{0.383Bd^2f_c'}}\right)}{f_y} =$$ 0.0020 < ρ_{provd} = 0.0030

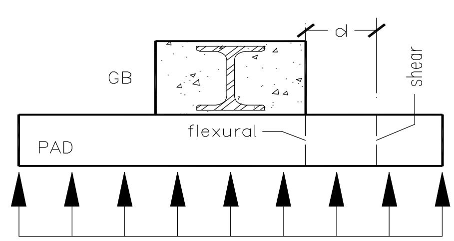
where d = 12.25 in

$Q_{u,max} = 1.5\, Q_{max}$ = 2.36 ksf, factor 1.5 for SD

$M_u = 0.125\, (B-W)^2\, B\, Q_{u,max}$ = 129 ft-kips

ρ_{max} = 0.0206 (ACI 10.2.7.3 & 10.3.5)

ρ_{min} = 0.0018 (ACI 7.12.2.1) [Satisfactory]



TRANS SECTION

(cont'd)

CHECK PAD ONE WAY SHEAR CAPACITY

V_u < ϕV_n [Satisfactory]

where $V_u = 0.5\, (B - W)\, B\, Q_{u,max} - d\, B\, Q_{u,max}$ = 27.9 kips

$\phi V_n = \phi\, 2\, d\, B\, (f_c')^{0.5}$ = 111.6 kips

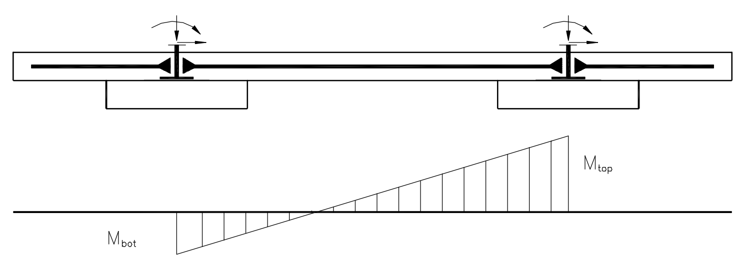
ϕ = 0.75

CHECK STEEL GB FLEXURAL CAPACITY

A	***Z_x***	***F_y***	***Ω_b***
19.1	133	50	1.67

(AISC 365-05 F1)

$M_{allowable} = Z_x\, F_y\, /\, \Omega_b$ = 331.8 ft-kips



GB MOMENT DIAGRAM

(cont'd)

> **$Max(M_{top}, M_{bot})$** [Satisfactory] (AISC 360-05 F1)

$$M_{top} = \left[M_{GB,wt} + \left(P_{D,1} + P_{L,1} + P_{E,1} + Wt_{PAD,1} - Q_{MIN}B^2\right)L - 0.5V_{E,1}D - M_{E,1} - Q_{MIN}\left(V_{E,1} + V_{E,2}\right)/\left(Q_{MAX} + Q_{MIN}\right)\left(0.5D + T\right)\right] =$$ 329 ft-kips

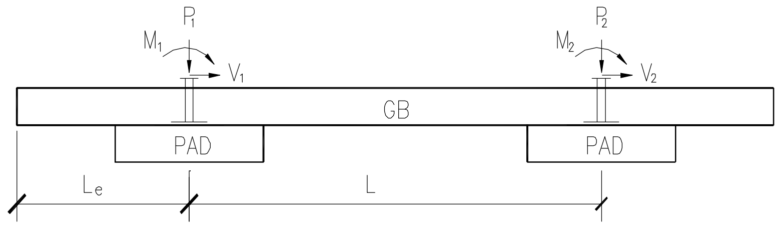
$$M_{bot} = \left[-M_{GB,wt} - \left(P_{D,2} + P_{L,2} + P_{E,2} + Wt_{PAD,1} - Q_{MAX}B^2\right)L - 0.5V_{E,2}D - M_{E,2} - Q_{MAX}\left(V_{E,1} + V_{E,2}\right)/\left(Q_{MAX} + Q_{MIN}\right)\left(0.5D + T\right)\right] =$$ 168 ft-kips

where $Q_{MAX} = \dfrac{M_O}{B^2L} + \dfrac{P_{D,2} + P_{L,2} + \gamma_{Conc\ \&\ Steel}\left[B^2T + WD\left(0.5L + L_e\right)\right]}{B^2} =$ 1.97 ksf, (full ASD pressure)

Q_{MIN} = 0.36 ksf, (full ASD pressure)

Two Pads with Grade Beam Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY



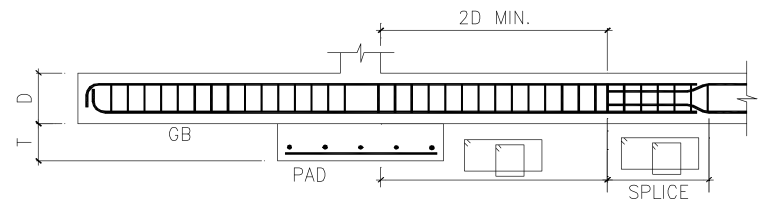
CONCRETE STRENGTH	f_c' =	3	ksi				
REBAR YIELD STRESS	f_y =	60	ksi				
SQUARE PAD SIZE	B =	8	ft				
	T =	16	in				
GRADE BEAM SIZE	W =	36	in				
	D =	36	in				
COLUMN DISTANCE	L =	22	ft				
GRADE BEAM EXTENSION	L_e =	5	ft				
GRAVITY AXIAL LOADS, ASD	$P_{D,1}$ =	25	kips	$P_{D,2}$ =	25	kips	(Dead Load)
	$P_{L,1}$ =	15	kips	$P_{L,2}$ =	15	kips	(Live Load)
SEISMIC LOADS, ASD	$P_{E,1}$ =	-30	kips	$P_{E,2}$ =	30	kips	(Seismic Load)
	$V_{E,1}$ =	50	kips	$V_{E,2}$ =	30	kips	(Seismic Load)
	$M_{E,1}$ =	50	ft-kips	$M_{E,2}$ =	50	ft-kips	(Seismic Load)

ALLOW SOIL PRESSURE Q_a = 2.5 ksf

PAD REINFORCING 8 # 6 @ 12 o.c., Each Way, Bottom.

GRADE BEAM LONGITUDINAL REINFORCING

TOP	7	#	7
	(d =	31.94	in)
	(1 Layer)		
BOTTOM	7	#	7
	(d =	31.94	in)
	(1 Layer)		



GRADE BEAM HOOPS (ACI 21.5.3)

LOCATION	AT END	AT SPLICE
LENGTH	72 in	48 in
	(2h)	MAX{$0.075 f_y \alpha\beta\gamma d_b / [(f_c')^{0.5}(c+K_{tr})/d_b]$, 12}
BAR	4 Legs # 5	4 Legs # 5
	(Legs to alternate long bars supported, ACI 7.10.5.3)	
SPACING	@ 7 in o.c.	@ 4 in o.c.
	MIN(d/4, $8d_b$, $24d_t$, 12)	MIN(d/4, 4)

THE GRADE BEAM DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING AT CENTER BOTTOM OF PAD 2 (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_O = $M_{E,1} + M_{E,2} + (V_{E,1}+V_{E,2})(D+T) - P_{E,1}L$ = 1106.7 ft-kips

M_R = $(P_{D,1} + \gamma_{conc} B^2 T)\, L + 0.5\gamma_{conc}(L + 2L_e)\, L\, D\, W$ = 1306.8 > $0.75 \times 1.4\, M_O / (0.9)$ = 1291 ft-kips

[Satisfactory]

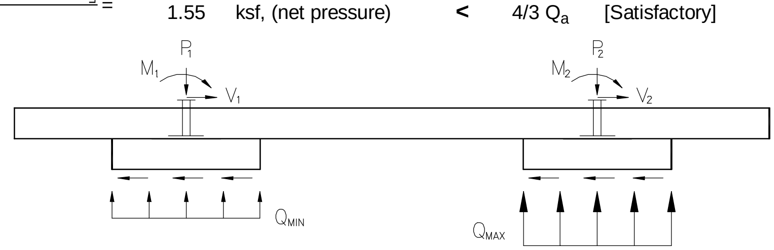
CHECK SOIL BEARING CAPACITY

$$Q_{MAX} = \frac{M_O}{B^2 L} + \frac{P_{D,2} + P_{L,2} + (\gamma_{CONC} - \gamma_{SOIL})\left[B^2 T + WD\,(0.5L + L_e)\right]}{B^2} =$$

1.55 ksf, (net pressure) < 4/3 Q_a [Satisfactory]

where γ_{conc} = 0.15 kcf

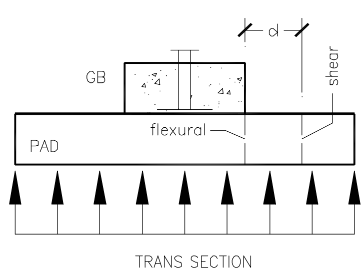
γ_{soil} = 0.11 kcf



CHECK PAD FLEXURAL REINFORCING

$$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \frac{M_u}{0.383 B d^2 f_c'}}\right)}{f_y} =$$

0.0020 < ρ_{provd} = 0.0030



where d = 12.25 in

$Q_{u,max} = 1.5\, Q_{max}$ = 2.33 ksf, factor 1.5 for SD

$M_u = 0.125\,(B-W)^2\, B\, Q_{u,max}$ = 128 ft-kips

ρ_{max} = 0.0155 (ACI 10.2.7.3 & 10.3.5)

ρ_{min} = 0.0018 (ACI 7.12.2.1) [Satisfactory]

(cont'd)

CHECK PAD ONE WAY SHEAR CAPACITY

V_u < ϕV_n [Satisfactory]

where $V_u = 0.5\ (B - W)\ B\ Q_{u,max} - d\ B\ Q_{u,max} =$ 27.6 kips

$\phi V_n = \phi\ 2\ d\ B\ (f_c')^{0.5} =$ 96.6 kips

$\phi =$ 0.75

CHECK GB SECTION REQUIREMENTS (ACI 21.5.1)

$P_u = 1.5(V_{E,1} - V_{E,2}) =$ 30 kips < $0.1A_g f_c' =$ 388.8 kips [Satisfactory]

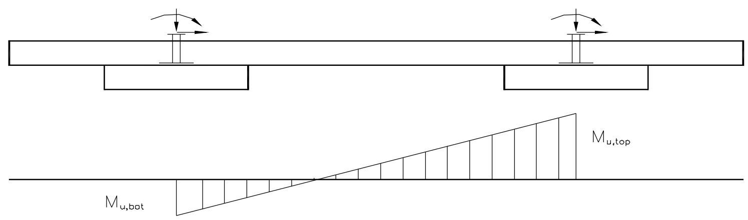
$L_n = L - B =$ 14.00 ft > $4\ d =$ 10.65 ft [Satisfactory]

W / D = 1.00 > 0.3 [Satisfactory]

W = 36 in > 10 in [Satisfactory]

< B+1.5D = 150 in [Satisfactory]

CHECK GB FLEXURAL REQUIREMENTS



GB MOMENT DIAGRAM

(ACI 21.3.5.1) $\rho_{top} =$ 0.004 > $\rho_{min} = MIN[3(f_c')^{0.5}/f_y,\ 200/f_y] =$ 0.003 [Satisfactory]

< $\rho_{max} =$ 0.025 [Satisfactory]

$\rho_{bot} =$ 0.004 > $\rho_{min} =$ 0.003 [Satisfactory]

< $\rho_{max} =$ 0.025 [Satisfactory]

(ACI 21.5.2.2) $M_{n,top}$ > $(1/2)M_{n,bot}$ [Satisfactory]

where $M_{n,bot} = \rho_{bot}\ bd^2 f_y\ (1 - 0.588\rho_{bot}\ f_y/f_c') =$ 642 ft-kips > $M_{u,bot} / \phi$ [Satisfactory]

$M_{n,top} = \rho_{top}\ bd^2 f_y\ (1 - 0.588\rho_{top}\ f_y/f_c') =$ 642 ft-kips > $M_{u,top} / \phi$ [Satisfactory]

$\phi =$ 0.9

$$M_{u,top} = 1.5\left[M_{GB,wt} + \left(P_{D,1} + P_{L,1} + P_{E,1} + Wt_{PAD,1} - Q_{MIN}B^2\right)L - 0.5V_{E,1}D - M_{E,1} - Q_{MIN}\left(V_{E,1} + V_{E,2}\right)/\left(Q_{MAX} + Q_{MIN}\right)\left(0.5D + T\right)\right] =$$ 453 ft-kips

$$M_{u,bot} = 1.5\left[-M_{GB,wt} - \left(P_{D,2} + P_{L,2} + P_{E,2} + Wt_{PAD,1} - Q_{MAX}B^2\right)L - 0.5V_{E,2}D - M_{E,2} - Q_{MAX}\left(V_{E,1} + V_{E,2}\right)/\left(Q_{MAX} + Q_{MIN}\right)\left(0.5D + T\right)\right] =$$ 217 ft-kips

where $Q_{MAX} = \dfrac{M_O}{B^2 L} + \dfrac{P_{D,2} + P_{L,2} + \gamma_{CONC}\left[B^2 T + WD\left(0.5L + L_e\right)\right]}{B^2} =$ 1.95 ksf, (full ASD pressure)

$Q_{MIN} =$ 0.38 ksf, (full ASD pressure)

Factor 1.5 is for SD

CHECK GB SHEAR STRENGTH (ACI 21.5.4)

$V_e = (M_{pr,\ top} + M_{pr,bot}) / L_n =$ 113.3 kips < $8\phi(f_c')^{0.5}bd =$ 377.8 kips [Satisfactory]

< $\phi[2(f_c')^{0.5}bd + A_v f_y d/s] =$ 349.0 kips [Satisfactory]

where $M_{pr,top} = \rho_{top}\ bd^2 f_y\ (1.25 - 0.919\rho_{top}\ f_y/f_c') =$ 793 ft-kips

$M_{pr,bot} = \rho_{bot}\ bd^2 f_y\ (1.25 - 0.919\rho_{bot}\ f_y/f_c') =$ 793 ft-kips

$\phi =$ 0.75 (ACI 9.3.2.3)

$A_v =$ 1.24 in^2

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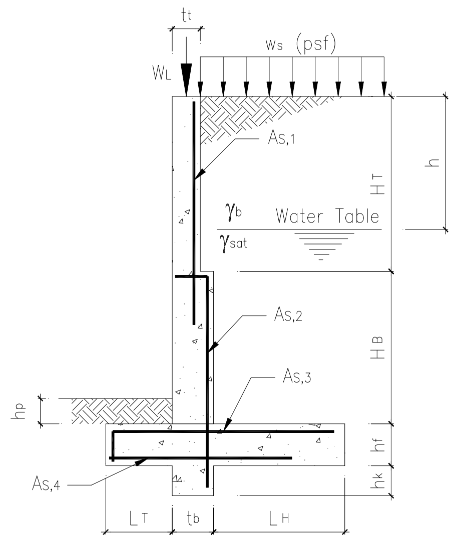
Retaining Wall Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	4	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	$P_a = k_a \gamma_b$	=	30	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	118	pcf		
WATER TABLE DEPTH	h	=	24.5	ft		
PASSIVE PRESSURE	P_p	=	300	psf / ft		
SURCHARGE WEIGHT	w_s	=	220	psf		
WALL TOP LIVE LOAD	W_L	=	4000	lbs / ft		
FRICTION COEFFICIENT	μ	=	0.4			
ALLOW SOIL PRESSURE	Q_a	=	4	ksf		
THICKNESS OF TOP STEM	t_t	=	18	in		
THICKNESS OF KEY & STEM	t_b	=	18	in		
TOE WIDTH	L_T	=	3	ft		
HEEL WIDTH	L_H	=	8	ft		
HEIGHT OF TOP STEM	H_T	=	10	ft		
HEIGHT OF BOT. STEM	H_B	=	10	ft		
FOOTING THICKNESS	h_f	=	18	in		
KEY DEPTH	h_k	=	36	in		
SOIL OVER TOE	h_p	=	0	in		
TOP STEM VERT. REINF. ($A_{s,1}$)		#	9	@	6	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
TOP STEM HORIZ. REINF. (ACI 14.1.2)			#	6	@ 9 in o.c., at soil face	
BOT. STEM VERT. REINF. ($A_{s,2}$)		#	9	@	6	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
BOT. STEM HORIZ. REINF. (ACI 14.1.2)			#	6	@ 9 in o.c., at soil face	
TOP REINF.OF FOOTING ($A_{s,3}$)		#	7	@	7	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	7	@	12	in

[THE WALL DESIGN IS ADEQUATE.]



ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a\,h^2 + h\,P_a\,H + 0.5\,[P_a\,(\gamma_{sat} - \gamma_w)\,/\,\gamma_b + \gamma_w]\,H^2$ = 6.93 kips

Where h = 21.5 ft , H = 0 ft

$H_s = w_s\,P_a\,(H_T + H_B + h_f)\,/\,\gamma_b$ = 1.29 kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$ = 3.04 kips

$W_s = w_s\,(L_H + t_b - t_t)$ = 1.76 kips

$W_b = W_{b1} + W_{b2}$ = 17.60 kips

Where W_{b1} = 17.6 kips , W_{b2} = 0.00 kips

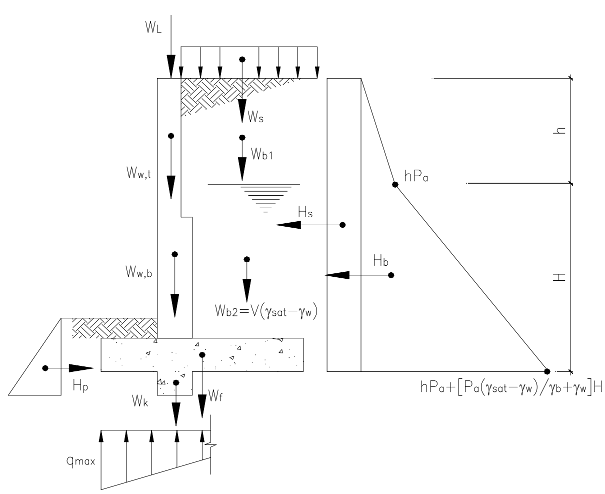
$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$ = 2.81 kips

$W_k = h_k\,t_b\,\gamma_c$ = 0.68 kips

$W_{w,t} = t_t\,H_T\,\gamma_c$ = 2.25 kips

$W_{w,b} = t_b\,H_B\,\gamma_c$ = 2.25 kips

W_L = 4.00 kips



FACTORED LOADS

$\gamma H_b = 1.6\,H_b$ = 11.09 kips

$\gamma H_s = 1.6\,H_s$ = 2.06 kips

$\gamma W_s = 1.6\,W_s$ = 2.82 kips

$\gamma W_b = 1.2\,W_b$ = 21.12 kips

$\gamma W_f = 1.2\,W_f$ = 3.38 kips

$\gamma W_k = 1.2\,W_k$ = 0.81 kips

$\gamma W_{w,t} = 1.2\,W_{w,t}$ = 2.70 kips

$\gamma W_{w,b} = 1.2\,W_{w,b}$ = 2.70 kips

$\gamma W_L = 1.6\,W_L$ = 6.40 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	6.93	11.09	7.17	49.69	79.51
H_s	1.29	2.06	10.75	13.87	22.19
Σ	8.22	13.16		**63.56**	101.70

(cont'd)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	1.76	2.82	8.50	14.96	23.94
W_b	17.60	21.12	8.50	149.60	179.52
W_f	2.81	3.38	6.25	17.58	21.09
W_k	0.68	0.81	3.75	2.53	3.04
$W_{w,t}$	2.25	2.70	3.75	8.44	10.13
$W_{w,b}$	2.25	2.70	3.75	8.44	10.13
Σ	27.35	33.52		**201.54**	247.84

M_{HP} = 0.17 ft-kips/ft

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx + M_{HP}}{\Sigma Hy} =$$ 3.174 **>** 1.5

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

$$L = L_T + t_b + L_H$$ = 12.50 ft

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy - M_{HP}}{\Sigma W}$$ = 1.21 ft

$$q_{MAX} = \begin{cases} \dfrac{(\Sigma W + W_L)\left(1 + \dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2(\Sigma W + W_L)}{3B(0.5L - e)}, & for \quad e > \dfrac{L}{6} \end{cases} =$$ 3.96 ksf **<** Q_a **[Satisfactory]**

CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-08 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 20 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 60 plf

B = h P_a = 600 plf

C = [P_a (γ_{sat} - γ_w) / γ_b + γ_w] H' : 0 plf

At base of top stem

M_u = 12.80 ft-kips

V_u = 3.36 kips

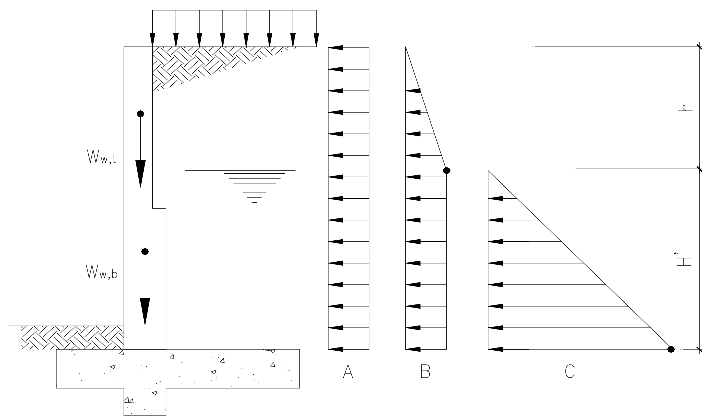
P_u = 2.70 kips

At base of bottom stem

M_u = 83.20 ft-kips

V_u = 11.52 kips

P_u = 5.40 kips



			At top stem		At base of bottom stem	
$\phi M_n = \phi\left[A_s f_y\left(d - \dfrac{A_s f_y - P_u}{1.7bf'_c}\right)\right]$		=	125.99	ft-kips ,	126.28	ft-kips
			>	M_u	>	M_u
				[Satisfactory]		**[Satisfactory]**
where	d	=	15.44	in ,	15.44	in
	b	=	12	in ,	12	in
	ϕ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
	A_s	=	2	in^2 ,	2	in^2
	ρ	=	0.011		0.011	
$\rho_{MAX} = \dfrac{0.85\beta_1 f'_c}{f_y}\dfrac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$		=	0.021		0.021	
			>	ρ	>	ρ
				[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\dfrac{t}{d}$		=	0.002		0.002	
			<	ρ	<	ρ
				[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-08 SEC.15.5.2, 11.1.3.1, & 11.2)

		At top stem		At base of bottom stem	
$\phi V_n = 2\phi bd\sqrt{f'_c}$	=	17.57	kips ,	17.57	kips
		>	V_u	>	V_u
			[Satisfactory]		**[Satisfactory]**

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

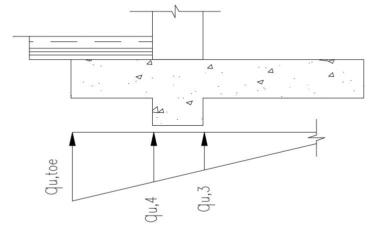
(cont'd)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} \quad = \quad 0.021 \qquad\qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} \quad = \quad 0.001$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{\left(q_{u,3} + 2q_{u,heel}\right) b L_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3} b S^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} \quad = \quad 58.12 \text{ ft-kips}$$

$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.005$$



where	d	=	15.56 in	$q_{u, toe}$	=	6.24	ksf
	e_u	=	1.99 ft	$q_{u, heel}$	=	0.15	ksf
	S	=	n/a	$q_{u, 3}$	=	4.04	ksf

$(A_{S, 3})_{required}$ = 0.86 in^2 / ft **<** $A_{S, 3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} \quad = \quad 0.021 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.001$$

$$M_{u,4} = \frac{\left(q_{u,4} + 2q_{u,toe}\right) b L_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f \quad = \quad 24.67 \text{ ft-kips}$$

where	d	=	14.56	in
	$q_{u, 4}$	=	4.78	ksf

$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.002$$

$(A_{S, 4})_{required}$ = 0.38 in^2 / ft **<** $A_{S, 4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 09 1807.2.3)

1.5 (H_b + H_s) = 12.3 kips **<** $H_p + \mu\,\Sigma W$ = 13.98 kips

[Satisfactory]

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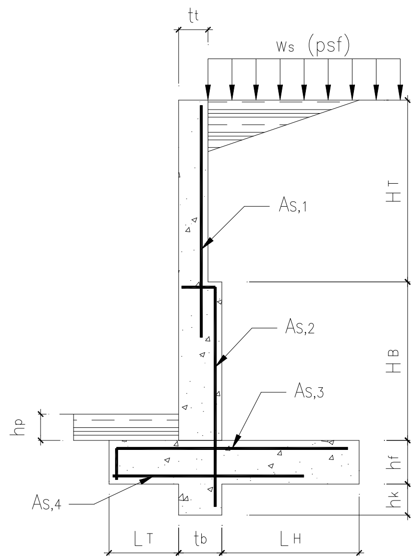
Retaining Wall Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	4	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a	=	30	pcf (equivalent fluid pressure)		
PASSIVE PRESSURE	P_p	=	300	psf / ft		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SURCHARGE WEIGHT	w_s	=	220	psf		
FRICTION COEFFICIENT	μ	=	0.4			
ALLOW SOIL PRESSURE	Q_a	=	4	ksf		
THICKNESS OF TOP STEM	t_t	=	18	in		
THICKNESS OF KEY & STEM	t_b	=	18	in		
TOE WIDTH	L_T	=	3	ft		
HEEL WIDTH	L_H	=	8	ft		
HEIGHT OF TOP STEM	H_T	=	10	ft		
HEIGHT OF BOT. STEM	H_B	=	10	ft		
FOOTING THICKNESS	h_f	=	18	in		
KEY DEPTH	h_k	=	36	in		
SOIL OVER TOE	h_p	=	0	in		
TOP STEM VERT. REINF. ($A_{s,1}$)		#	9	@	6	in o.c., at soil face
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
TOP STEM HORIZ. REINF. (ACI 14.1.2)			#	6	@ 9 in o.c., at soil face	
BOT. STEM VERT. REINF. ($A_{s,2}$)		#	9	@	6	in o.c., at soil face
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
BOT. STEM HORIZ. REINF. (ACI 14.1.2)			#	6	@ 9 in o.c., at soil face	
TOP REINF.OF FOOTING ($A_{s,3}$)		#	7	@	6	in
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	7	@	12	in

[THE WALL DESIGN IS ADEQUATE.]



ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a\,(H_T + H_B + h_f)^2 = 6.93$ kips

$H_s = w_s\,P_a\,(H_T + H_B + h_f) / \gamma_b = 1.29$ kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2 = 3.04$ kips

$W_s = w_s\,(L_H + t_b - t_t) = 1.76$ kips

$W_b = [H_T\,(L_H + t_b - t_t) + H_B\,L_H]\,\gamma_b = 17.60$ kips

$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c = 2.81$ kips

$W_k = h_k\,t_b\,\gamma_c = 0.68$ kips

$W_{w,t} = t_t\,H_T\,\gamma_c = 2.25$ kips

$W_{w,b} = t_b\,H_B\,\gamma_c = 2.25$ kips

FACTORED LOADS

$\gamma H_b = 1.6\,H_b = 11.09$ kips

$\gamma H_s = 1.6\,H_s = 2.06$ kips

$\gamma W_s = 1.6\,W_s = 2.82$ kips

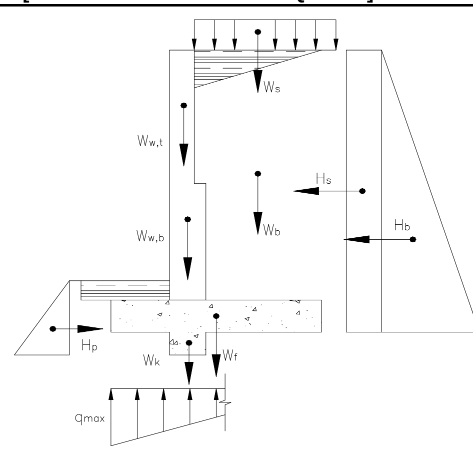
$\gamma W_b = 1.2\,W_b = 21.12$ kips

$\gamma W_f = 1.2\,W_f = 3.38$ kips

$\gamma W_k = 1.2\,W_k = 0.81$ kips

$\gamma W_{w,t} = 1.2\,W_{w,t} = 2.70$ kips

$\gamma W_{w,b} = 1.2\,W_{w,b} = 2.70$ kips



OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	6.93	11.09	7.17	49.69	79.51
H_s	1.29	2.06	10.75	13.87	22.19
Σ	8.22	13.16		**63.56**	101.70

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	1.76	2.82	8.50	14.96	23.94
W_b	17.60	21.12	8.50	149.60	179.52
W_f	2.81	3.38	6.25	17.58	21.09
W_k	0.68	0.81	3.75	2.53	3.04
$W_{w,t}$	2.25	2.70	3.75	8.44	10.13
$W_{w,b}$	2.25	2.70	3.75	8.44	10.13
Σ	27.35	33.52		**201.54**	247.84

$M_{HP} = 0.17$ ft-kips/ft

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx + M_{HP}}{\Sigma Hy} = 3.174 > 1.5$$

[Satisfactory]

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

$$L = L_T + t_b + L_H$$ = 12.50 ft

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy - M_{HP}}{\Sigma W}$$ = 1.21 ft

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$ = 3.46 ksf < Q_a **[Satisfactory]**

CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

		At top stem		At base of bottom stem	
$M_u = \gamma\left(\dfrac{P_a y^3}{6} + \dfrac{P_a y^2 w_s}{2\gamma_b}\right)$	=	12.80	ft-kips ,	83.20	ft-kips
$P_u = \gamma W_w$	=	2.70	kips ,	5.40	kips
$\phi M_n = \phi\left[A_S f_y\left(d - \dfrac{A_S f_y - P_u}{1.7bf'_c}\right)\right]$	=	125.99	ft-kips ,	126.28	ft-kips
		>	***M*u**	**>**	***M*u**
			[Satisfactory]		**[Satisfactory]**
where d	=	15.44	in ,	15.44	in
b	=	12	in ,	12	in
ϕ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
A_s	=	2	in^2 ,	2	in^2
ρ	=	0.011		0.011	
$\rho_{MAX} = \dfrac{0.85\beta_1 f'_c}{f_y}\dfrac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$	=	0.021		0.021	
		>	ρ	**>**	ρ
			[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\dfrac{t}{d}$	=	0.002		0.002	
		<	ρ	**<**	ρ
			[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-08 SEC.15.5.2, 11.1.3.1, & 11.2)

		At top stem		At base of bottom stem	
$V = \gamma\left(\dfrac{P_a y^2}{2} + \dfrac{w_s P_a y}{\gamma_b}\right)$	=	3.36	kips ,	11.52	kips
$\phi V_n = 2\phi bd\sqrt{f'_c}$	=	17.57	kips ,	17.57	kips
		>	***V*u**	**>**	***V*u**
			[Satisfactory]		**[Satisfactory]**

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

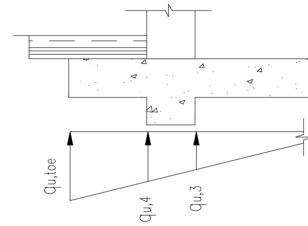
$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$$ = 0.021

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d}$$ = 0.001

$$M_{u,3} = \begin{cases} \dfrac{L_H}{2}\left(\gamma w_s + \gamma w_b + \dfrac{L_H}{L}\gamma w_f\right) - \dfrac{\left(q_{u,3} + 2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{L_H}{2}\left(\gamma w_s + \gamma w_b + \dfrac{L_H}{L}\gamma w_f\right) - \dfrac{q_{u,3}bS^2}{6}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$ = 63.30 ft-kips

(cont'd)

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.005$$



where							
d	=	15.56	in	$q_{u,\ toe}$	=	5.11	ksf
e_u	=	1.89	ft	$q_{u,\ heel}$	=	0.25	ksf
S	=	n/a		$q_{u,\ 3}$	=	3.36	ksf

$(A_{S,3})_{required}$ = 0.94 in^2 / ft < $A_{S,3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} \quad = \quad 0.021 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.001$$

$$M_{u,4} = \frac{(q_{u,4} + 2 q_{u,toe}) b L_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f \quad = \quad 20.04 \text{ ft-kips}$$

where			
d	=	14.56	in
$q_{u,\ 4}$	=	3.94	ksf

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.002$$

$(A_{S,4})_{required}$ = 0.31 in^2 / ft < $A_{S,4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 09 1807.2.3)

1.5 (H_b + H_s) = 12.3 kips < $H_p + \mu\, \Sigma W$ = 13.98 kips
[Satisfactory]

Daniel T. Li Engineering International	PROJECT :		PAGE :
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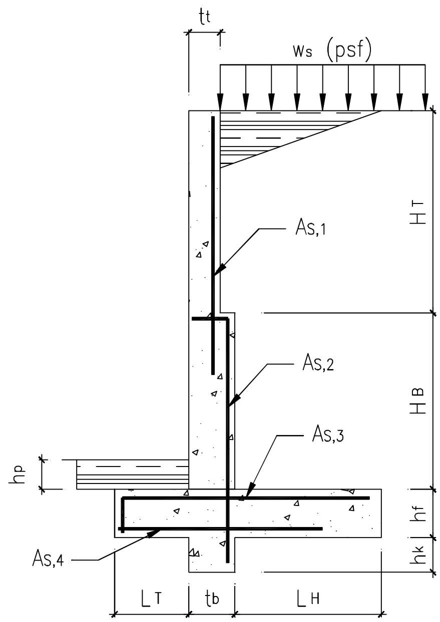
	JOB NO. :	DATE :	REVIEW BY :

Retaining Wall Design Based on 2010 CBC Chapter A

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a	=	45	pcf (equivalent fluid pressure)		
PASSIVE PRESSURE	P_p	=	450	psf / ft		
SEISMIC GROUND SHAKING	P_E	=	48	psf / ft (1807A.2.2)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SURCHARGE WEIGHT	w_s	=	50	psf		
FRICTION COEFFICIENT	μ	=	0.35			
ALLOW SOIL PRESSURE	Q_a	=	3	ksf		
THICKNESS OF TOP STEM	t_t	=	14	in		
THICKNESS OF KEY & STEM	t_b	=	22	in		
TOE WIDTH	L_T	=	5	ft		
HEEL WIDTH	L_H	=	13	ft		
HEIGHT OF TOP STEM	H_T	=	6	ft		
HEIGHT OF BOT. STEM	H_B	=	6	ft		
FOOTING THICKNESS	h_f	=	21	in		
KEY DEPTH	h_k	=	26	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	7	@	12	in o.c.
$A_{s,1}$ LOCATION (1 = at middle, 2 = at soil face)				2	at soil face	
TOP STEM HORIZ. REINF. (ACI 14.1.2)			#	6	@ 12 in o.c., at soil face	
BOT. STEM REINF. ($A_{s,2}$)		#	8	@	8	in o.c.
$A_{s,2}$ LOCATION (1 = at middle, 2 = at soil face)				0	at soil face	
BOT. STEM HORIZ. REINF. (ACI 14.1.2)			#	6	@ 8 in o.c., at soil face	
TOP REINF.OF FOOTING ($A_{s,3}$)		#	9	@	4	in
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	8	@	24	in



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS

$H_b = 0.5\, P_a\, (H_T + H_B + h_f)^2$ = 4.25 kips

$H_s = w_s\, P_a\, (H_T + H_B + h_f) / \gamma_b$ = 0.28 kips

$H_p = 0.5\, P_p\, (h_p + h_f + h_k)^2$ = 4.39 kips

$H_E = 0.5\, P_E\, (H_T + H_B + h_f)^2$ = 4.54 kips

$W_s = w_s\, (L_H + t_b - t_t)$ = 0.68 kips

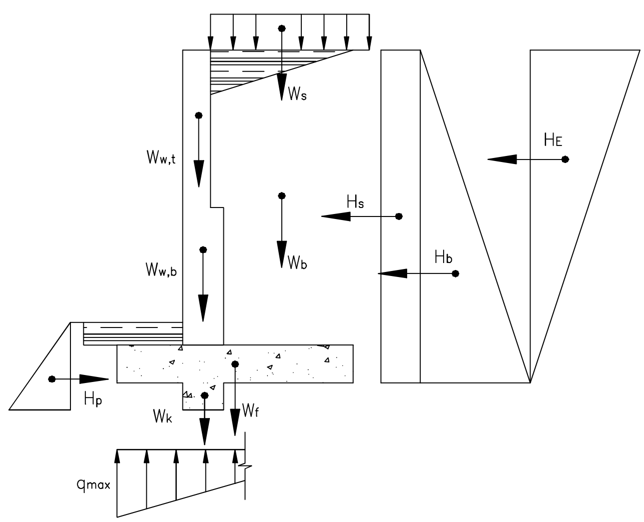
$W_b = [H_T\, (L_H + t_b - t_t) + H_B\, L_H]\, \gamma_b$ = 17.60 kips

$W_f = h_f\, (L_H + t_b + L_T)\, \gamma_c$ = 5.21 kips

$W_k = h_k\, t_b\, \gamma_c$ = 0.60 kips

$W_{w,t} = t_t\, H_T\, \gamma_c$ = 1.05 kips

$W_{w,b} = t_b\, H_B\, \gamma_c$ = 1.65 kips



FACTORED LOADS

$\gamma H_b = 1.6\, H_b$ = 6.81 kips

$\gamma H_s = 1.6\, H_s$ = 0.45 kips

$\gamma H_E = 1.6\, H_E$ = 7.26 kips

$\gamma W_s = 1.6\, W_s$ = 1.09 kips

$\gamma W_b = 1.2\, W_b$ = 21.12 kips

$\gamma W_f = 1.2\, W_f$ = 6.25 kips

$\gamma W_k = 1.2\, W_k$ = 0.72 kips

$\gamma W_{w,t} = 1.2\, W_{w,t}$ = 1.26 kips

$\gamma W_{w,b} = 1.2\, W_{w,b}$ = 1.98 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	4.25	6.81	4.58	19.50	31.20
H_E	4.54	7.26	9.17	41.59	66.55
H_s	0.28	0.45	6.88	1.93	3.09
Σ	9.07	14.52		**63.02**	100.84

(cont'd)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.68	1.09	13.00	8.88	14.21
W_b	17.60	21.12	13.16	231.66	277.99
W_f	5.21	6.25	9.92	51.63	61.95
W_k	0.60	0.72	5.92	3.53	4.23
$W_{w,t}$	1.05	1.26	5.58	5.86	7.04
$W_{w,b}$	1.65	1.98	5.92	9.76	11.72
Σ	26.79	32.42		**311.32**	377.14

M_{HP} = 0.85 ft-kips/ft

OVERTURNING FACTOR OF SAFETY (1806A.1)

$$SF = \frac{\Sigma Wx + M_{HP}}{\Sigma Hy} = 4.953 > 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318-05 SEC.15.2.2)

$$L = L_T + t_b + L_H = 19.83 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy + M_{HP}}{\Sigma W} = 0.61 \text{ ft} < L/3$$

[Satisfactory]

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 1.60 \text{ ksf} < Q_a$$

[Satisfactory]

CHECK FLEXURE CAPACITY, $A_{S,1}$ & $A_{S,2}$, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

		At top stem		At base of bottom stem	
$M_u = \gamma\left(\dfrac{P_a y^3}{6} + \dfrac{P_a y^2 w_s}{2\gamma_b} + M_E\right)$	=	19.42	ft-kips ,	89.64	ft-kips
$P_u = \gamma W_w$	=	1.26	kips ,	3.24	kips
$\phi M_n = \phi\left[A_S f_y\left(d - \dfrac{A_S f_y - P_u}{1.7bf'_c}\right)\right]$	=	29.52	ft-kips ,	98.07	ft-kips
		>	M_u	>	M_u
			[Satisfactory]		**[Satisfactory]**
where d	=	11.50	in ,	19.50	in
b	=	12	in ,	12	in
φ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
A_s	=	0.6	in^2 ,	1.185	in^2
ρ	=	0.004		0.005	
$\rho_{MAX} = \dfrac{0.85\beta_1 f'_c}{f_y}\dfrac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$	=	0.015		0.015	
		>	ρ	>	ρ
			[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\dfrac{t}{d}$	=	0.002		0.001	
		<	ρ	<	ρ
			[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-05 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem	
$V = \gamma\left(\dfrac{P_a y^2}{2} + \dfrac{w_s P_a y}{\gamma_b} + V_E\right)$	=	6.45	kips ,	12.84	kips
$V_{allowable} = 2\phi bd\sqrt{f'_c}$	=	11.34	kips ,	19.23	kips
		>	V	>	V
where φ = 0.75 (ACI 318-05, Section 9.3.2.3)			**[Satisfactory]**		**[Satisfactory]**

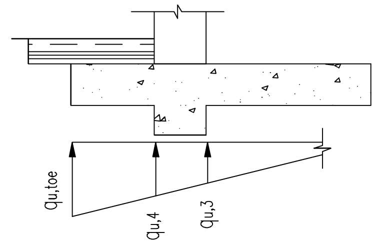
(cont'd)

CHECK HEEL FLEXURE CAPACITY, $A_{S,3}$, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon u}{\varepsilon u + \varepsilon t} \quad = \quad 0.015$$

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} \quad = \quad 0.001$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma W_s + \gamma W_b + \frac{L_H}{L}\gamma W_f\right) - \frac{\left(q_{u,3} + 2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma W_s + \gamma W_b + \frac{L_H}{L}\gamma W_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} \quad = \quad 69.78 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} \quad = \quad 0.004$$

where	d	=	18.44 in	$q_{u,\,toe}$	=	2.32	ksf
	e_u	=	1.39 ft	$q_{u,\,heel}$	=	0.95	ksf
	S	=	n/a	$q_{u,\,3}$	=	1.70	ksf

$(A_{S,\,3})_{required}$ = 0.88 in^2 / ft < $A_{S,\,3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, $A_{S,4}$, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon u}{\varepsilon u + \varepsilon t} \quad = \quad 0.015$$

$$\rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.001$$

$$M_{u,4} = \frac{\left(q_{u,4} + 2q_{u,toe}\right)bL_T^2}{6} - \frac{L_T^2}{2L}\gamma W_f \quad = \quad 23.21 \text{ ft-kips}$$

where	d	=	17.50 in
	$q_{u,\,4}$	=	1.869 ksf

$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} \quad = \quad 0.001$$

$(A_{S,\,4})_{required}$ = 0.30 in^2 / ft < $A_{S,\,4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (CBC 1807A.2.3)

$1.5\,(H_b + H_s + H_E)$ = 13.6 kips < $H_p + \mu\,\Sigma W$ = 13.76 kips

[Satisfactory]

Daniel T. Li
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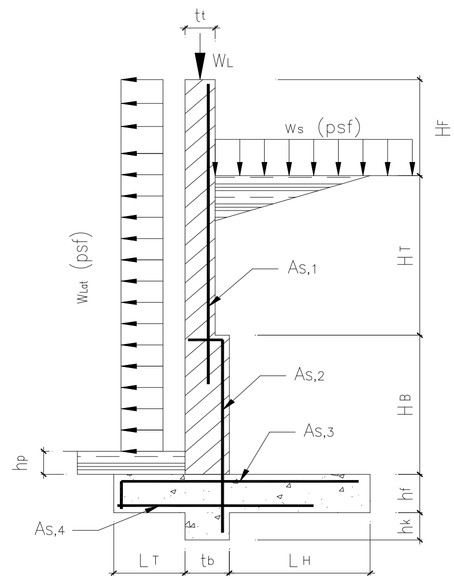
PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Retaining / Fence Wall Design Based on TMS 402-08 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

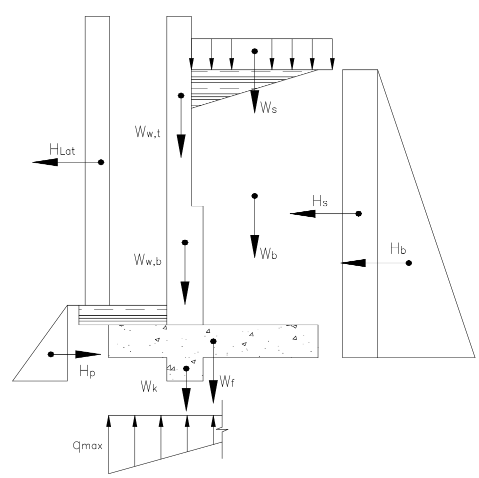


SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes		
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
CONCRETE STRENGTH	f_c'	=	4.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a	=	30	pcf (equivalent fluid pressure)		
PASSIVE PRESSURE	P_p	=	400	psf / ft		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SURCHARGE WEIGHT	w_s	=	100	psf		
WALL TOP LIVE LOAD	W_L	=	5000	lbs / ft		
SERVICE LATERAL FORCE	w_{Lat}	=	20	psf		
FRICTION COEFFICIENT	μ	=	0.3			
ALLOW SOIL PRESSURE	Q_a	=	3	ksf		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	12	in		
TOE WIDTH	L_T	=	3	ft		
HEEL WIDTH	L_H	=	6	ft		
HEIGHT OF FENCE STEM	H_F	=	4	ft		
HEIGHT OF TOP STEM	H_T	=	4	ft		
HEIGHT OF BOT. STEM	H_B	=	4	ft		
FOOTING THICKNESS	h_f	=	12	in		
KEY DEPTH	h_k	=	12	in		
SOIL OVER TOE	h_p	=	12	in		
TOP STEM REINF. ($A_{s,1}$)	#		7	@	16	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
BOT. STEM REINF. ($A_{s,2}$)	#		7	@	16	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
GROUTED CORES (0=fully, 1=at vertical rebars only)					0	solid
TOP REINF.OF FOOTING ($A_{s,3}$)	#		6	@	18	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)	#		5	@	14	in o.c.

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS



$H_b = 0.5\ P_a\ (H_T + H_B + h_f)^2$	=	1.22	kips
$H_s = w_s\ P_a\ (H_T + H_B + h_f) / \gamma_b$	=	0.25	kips
$H_p = 0.5\ P_p\ (h_p + h_f + h_k)^2$	=	1.80	kips
$H_{Lat} = w_{Lat}\ (H_F + H_T + H_B - h_p)$	=	0.22	kips
$W_s = w_s\ (L_H + t_b - t_t)$	=	0.63	kips
$W_b = [H_T\ (L_H + t_b - t_t) + H_B\ L_H]\ \gamma_b$	=	5.43	kips
$W_f = h_f\ (L_H + t_b + L_T)\ \gamma_c$	=	1.50	kips
$W_k = h_k\ t_b\ \gamma_c$	=	0.15	kips
$W_{w,t} = t_t\ (H_T + H_F)\ \gamma_m$	=	0.59	kips
$W_{w,b} = t_b\ H_B\ \gamma_m$	=	0.44	kips
W_L =	=	5.00	kips

FACTORED LOADS

$\gamma H_b = 1.6\ H_b$	=	1.94	kips
$\gamma H_s = 1.6\ H_s$	=	0.39	kips
$\gamma H_{Lat} = 1.6\ H_{Lat}$	=	0.35	kips
$\gamma W_s = 1.6\ W_s$	=	1.01	kips
$\gamma W_b = 1.2\ W_b$	=	6.51	kips
$\gamma W_f = 1.2\ W_f$	=	1.80	kips
$\gamma W_k = 1.2\ W_k$	=	0.18	kips
$\gamma W_{w,t} = 1.2\ W_{w,t}$	=	0.70	kips
$\gamma W_{w,b} = 1.2\ W_{w,b}$	=	0.53	kips
$\gamma W_L = 1.6\ W_L$	=	8.00	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	1.22	1.94	3.00	3.65	5.83
H_s	0.25	0.39	4.50	1.10	1.77
H_{Lat}	0.22	0.35	7.50	1.65	2.64
Σ	1.68	2.69		**6.40**	10.24

(cont'd)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.63	1.01	6.83	4.33	6.92
W_b	5.43	6.51	6.91	37.52	45.03
W_f	1.50	1.80	5.00	7.50	9.00
W_k	0.15	0.18	3.50	0.53	0.63
$W_{w,t}$	0.59	0.70	3.33	1.95	2.35
$W_{w,b}$	0.44	0.53	3.50	1.54	1.85
Σ	8.74	10.74		**53.37**	65.77

M_{HP} = 0.53 ft-kips/ft

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma W x + M_{HP}}{\left(H_b y + H_s y + H_{Lat} y\right)} = \quad 8.423 \quad > \quad 1.1$$

$$SF = \frac{\Sigma W x + M_{HP}}{\left(H_b y + H_s y\right)} = \quad 11.35 \quad > \quad 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318 15.2.2)

$$L = L_T + t_b + L_H \quad = \quad 10.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma W x - \Sigma H y - M_{HP}}{\Sigma W} \quad = \quad -0.32 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\left(\Sigma W + W_L\right)\left(1 + \dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\left(\Sigma W + W_L\right)}{3B(0.5L - e)}, & for \quad e > \dfrac{L}{6} \end{cases} = \quad 1.11 \text{ ksf} \quad < \quad Q_a$$

[Satisfactory]

CHECK FLEXURE CAPACITY FOR MASONRY STEM (TMS 402 2.3.3)

		At top stem	At base of bottom stem
$M = \dfrac{P_a y^3}{6} + \dfrac{P_a y^2 w_s}{2\gamma_b} + M_{Lat}$	=	1.18 ft-kips ,	4.86 ft-kips
$P = W_w$	=	0.59 kips ,	1.03 kips

$$M_{allowable} = MIN\left[\frac{1}{2} b_w k d F_b\left(d - \frac{kd}{3}\right) - P\left(d - \frac{t_e}{2}\right) \quad , \quad A_s F_s\left(d - \frac{kd}{3}\right) + P\left(\frac{t_e}{2} - \frac{kd}{3}\right)\right]$$

			At top stem	At base of bottom stem	
where	t_e	=	7.63 in ,	11.63 in	<== Based on effective section area.
	d	=	5.44 in ,	9.44 in	<== Based on TMS 402-08, 1.13.3.5
	b_w	=	12 in ,	12 in	
	F_b	=	0.495 ksi ,	0.495 ksi	
	F_s	=	24 ksi ,	24 ksi	
	A_s	=	0.45 in^2 ,	0.45 in^2	
	ρ	=	0.007 ,	0.004	
	E_m	=	1350 ksi ,	1350 ksi	
	E_s	=	29000 ksi ,	29000 ksi	
	n	=	21.48	21.48	
	k	=	0.42	0.34	
and	$\boldsymbol{M_{allowable}}$	=	2.54 ft-kips ,	6.28 ft-kips	
		>	***M***	> ***M***	
			[Satisfactory]	[Satisfactory]	

CHECK SHEAR CAPACITY FOR MASONRY STEM (TMS 402 2.3.5)

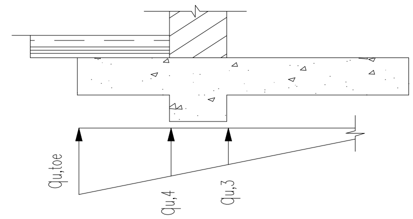
		At top stem	At base of bottom stem
$V = \dfrac{P_a y^2}{2} + \dfrac{w_s P_a y}{\gamma_b} + V_{Lat}$	=	0.51 kips ,	1.40 kips
$V_{allowable} = d\, b_w MIN\left(\sqrt{f'_m} \quad , \quad 50\right)$	=	2.53 kips ,	4.39 kips
		> ***V***	> ***V***
		[Satisfactory]	[Satisfactory]

(cont'd)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} \quad = \quad 0.023 \qquad\qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} \quad = \quad 0.001$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{\left(q_{u,3}+2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} \quad = \quad 0.90 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} \quad = \quad 0.000$$

where	d	=	9.63	in	$q_{u,\,toe}$	=	2.56	ksf
	e_u	=	0.61	ft	$q_{u,\,heel}$	=	1.18	ksf
	S	=	n/a		$q_{u,\,3}$	=	1.78	ksf

$(A_{S,\,3})_{required}$ = 0.13 in^2 / ft < $A_{S,\,3}$ [Satisfactory]

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} \quad = \quad 0.023 \qquad\qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.001$$

$$M_{u,4} = \frac{\left(q_{u,4}+2q_{u,toe}\right)bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f \quad = \quad 9.85 \text{ ft-kips}$$

where	d	=	8.69	in
	$q_{u,\,4}$	=	1.98	ksf

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} \quad = \quad 0.002$$

$(A_{S,\,4})_{required}$ = 0.26 in^2 / ft < $A_{S,\,4}$ [Satisfactory]

CHECK SLIDING CAPACITY (IBC 09 1807.2.3)

1.1 (H_b + H_s + H_{Lat}) =	1.85	kips	<	H_p + μ ΣW =	4.42	kips	[Satisfactory]
1.5 (H_b + H_s) =	2.19	kips	<	H_p + μ ΣW =	4.42	kips	[Satisfactory]

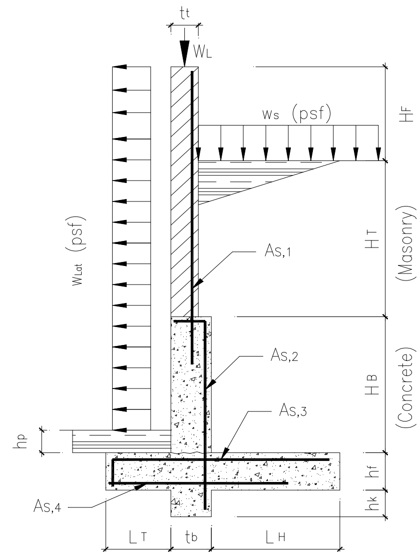
Daniel T. Li
Engineering International

PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Retaining Wall Design, for Masonry Top & Concrete Bottom, Based on TMS 402-08 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes		
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
CONCRETE STRENGTH	f_c'	=	4.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a	=	30	pcf (equivalent fluid pressure)		
PASSIVE PRESSURE	P_p	=	400	psf / ft		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SURCHARGE WEIGHT	w_s	=	100	psf		
WALL TOP LIVE LOAD	W_L	=	4000	lbs / ft		
SERVICE LATERAL FORCE	w_{Lat}	=	20	psf		
FRICTION COEFFICIENT	μ	=	0.3			
ALLOW SOIL PRESSURE	Q_a	=	3	ksf		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	10	in		
TOE WIDTH	L_T	=	3	ft		
HEEL WIDTH	L_H	=	6	ft		
HEIGHT OF FENCE STEM	H_F	=	4	ft		
HEIGHT OF TOP STEM	H_T	=	4	ft		
HEIGHT OF BOT. STEM	H_B	=	6	ft		
FOOTING THICKNESS	h_f	=	12	in		
KEY DEPTH	h_k	=	12	in		
SOIL OVER TOE	h_p	=	12	in		
TOP STEM REINF. ($A_{s,1}$)		#	7	@	16	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
MASONRY GROUTED CORES (0=fully, 1=at vertical rebars only)					0	solid
BOT. STEM REINF. ($A_{s,2}$)		#	7	@	16	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
TOP REINF.OF FOOTING ($A_{s,3}$)		#	6	@	18	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	14	in o.c.

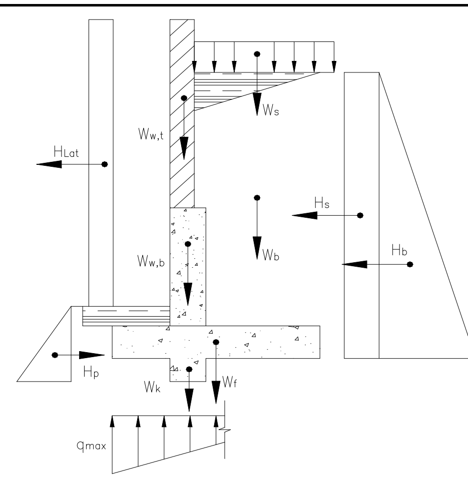


[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a\,(H_T + H_B + h_f)^2$	=	1.82	kips
$H_s = w_s\,P_a\,(H_T + H_B + h_f)\,/\,\gamma_b$	=	0.30	kips
$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$	=	1.80	kips
$H_{Lat} = w_{Lat}\,(H_F + H_T + H_B - h_p)$	=	0.26	kips
$W_s = w_s\,(L_H + t_b - t_t)$	=	0.62	kips
$W_b = [H_T\,(L_H + t_b - t_t) + H_B\,L_H]\,\gamma_b$	=	6.67	kips
$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$	=	1.48	kips
$W_k = h_k\;t_b\;\gamma_c$	=	0.13	kips
$W_{w,t} = t_t\;(H_T + H_F)\,\gamma_m$	=	0.59	kips
$W_{w,b} = t_b\;H_B\;\gamma_c$	=	0.00	kips
$W_L =$	=	4.00	kips



FACTORED LOADS

$\gamma H_b = 1.6\,H_b$	=	2.90	kips
$\gamma H_s = 1.6\,H_s$	=	0.48	kips
$\gamma H_{Lat} = 1.6\,H_{Lat}$	=	0.42	kips
$\gamma W_s = 1.6\,W_s$	=	0.99	kips
$\gamma W_b = 1.2\,W_b$	=	8.01	kips
$\gamma W_f = 1.2\,W_f$	=	1.77	kips
$\gamma W_k = 1.2\,W_k$	=	0.15	kips
$\gamma W_{w,t} = 1.2\,W_{w,t}$	=	0.70	kips
$\gamma W_{w,b} = 1.2\,W_{w,b}$	=	0.00	kips
$\gamma W_L = 1.6\,W_L$	=	6.40	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	1.82	2.90	3.67	6.66	10.65
H_s	0.30	0.48	5.50	1.65	2.64
H_{Lat}	0.26	0.42	8.50	2.21	3.54
Σ	2.38	3.80		**10.52**	16.82

(cont'd)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.62	0.99	6.75	4.16	6.66
W_b	6.67	8.01	6.80	45.38	54.45
W_f	1.48	1.77	4.92	7.25	8.70
W_k	0.13	0.15	3.42	0.43	0.51
$W_{w,t}$	0.59	0.70	3.33	1.95	2.35
$W_{w,b}$	0.00	0.00	3.42	0.00	0.00
Σ	9.48	11.62		**59.17**	72.67

M_{HP} = 0.53 ft-kips/ft

OVERTURNING FACTOR OF SAFETY

$$SF=\frac{\Sigma Wx+M_{HP}}{(H_b y+H_s y+H_{Lat} y)}= \quad 5.678 \quad > \quad 1.1$$

$$SF=\frac{\Sigma Wx+M_{HP}}{(H_b y+H_s y)}= \quad 7.189 \quad > \quad 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318 15.2.2)

$$L=L_T+t_b+L_H \quad = \quad 9.83 \text{ ft}$$

$$e=\frac{L}{2}-\frac{\Sigma Wx-\Sigma Hy-M_{HP}}{\Sigma W} \quad = \quad -0.16 \text{ ft}$$

$$q_{MAX}=\begin{cases}\dfrac{(\Sigma W+W_L)\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e\le\dfrac{L}{6}\\ \dfrac{2(\Sigma W+W_L)}{3B(0.5L-e)}, & for \quad e>\dfrac{L}{6}\end{cases} = \quad 1.24 \text{ ksf} \quad < \quad Q_a$$

[Satisfactory]

CHECK FLEXURE CAPACITY FOR MASONRY TOP STEM (TMS 402-08 2.3.3) `

$$M=\frac{P_a y^3}{6}+\frac{P_a y^2 w_s}{2\gamma_b}+M_{Lat} \quad = \quad 1.18 \text{ ft-kips ,}$$

$$P=W_w \quad = \quad 0.59 \text{ kips}$$

$$M_{allowable}=MIN\left[\frac{1}{2}b_w kd F_b\left(d-\frac{kd}{3}\right)-P\left(d-\frac{t_e}{2}\right) \;,\; A_s F_s\left(d-\frac{kd}{3}\right)+P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right]= \quad 2.54 \text{ ft-kips}$$

> ***M*** [Satisfactory]

where				
t_e = 7.63 in ,	<== Based on effective section area.			
d = 5.44 in ,	<== Based on TMS 402-08, 1.13.3.5			
b_w = 12 in ,	ρ =	0.007		
F_b = 0.495 ksi ,	E_m =	1350	ksi	
F_s = 24 ksi ,	E_s =	29000	ksi	
A_s = 0.45 in^2 ,	n =	21.48		k = 0.42

CHECK FLEXURE CAPACITY FOR CONCRETE BOTTOM STEM (ACI 318-08 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$M_u=\gamma\left(\frac{P_a y^3}{6}+\frac{P_a y^2 w_s}{2\gamma_b}+M_{Lat}\right) \quad = \quad 13.30 \text{ ft-kips}$$

$$P_u=\gamma W_w \quad = \quad 0.70 \text{ kips}$$

$$\phi M_n=\phi\left[A_S f_y\left(d-\frac{A_S f_y-P_u}{1.7bf'_c}\right)\right]= \quad 14.73 \text{ ft-kips} \quad > \quad \boldsymbol{M_u}$$

[Satisfactory]

where					
d	=	7.56	in ,	A_s =	0.45 in^2
b	=	12	in ,	ρ =	0.005
ϕ	=	0.9	(ACI 318 Fig R9.3.2)		

$$\rho_{MAX}=\frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}= \quad 0.023 \quad > \quad \rho$$

[Satisfactory]

$$\rho_{MIN}=0.0018\frac{t}{d}= \quad 0.002 \quad < \quad \rho$$

[Satisfactory]

CHECK SHEAR CAPACITY FOR MASONRY TOP STEM (TMS 402-08 2.3.5)

$$V=\frac{P_a y^2}{2}+\frac{w_s P_a y}{\gamma_b}+V_{Lat} \quad = \quad 0.51 \text{ kips ,}$$

$$V_{allowable}=d\,b_w MIN\left(\sqrt{f'_m} \;,\; 50\right) \quad = \quad 2.53 \text{ kips} \quad > \quad \boldsymbol{V}$$

[Satisfactory]

(cont'd)

CHECK SHEAR CAPACITY FOR CONCRETE BOTTOM STEM (ACI 318-08 SEC.15.5.2, 11.1.3.1, & 11.2)

$$V_u = \gamma\left(\frac{P_a y^2}{2} + \frac{w_s P_a y}{\gamma_b} + V_{Lat}\right) \quad = \quad 3.25 \text{ kips ,}$$

$$\phi V_n = 2\phi b d\sqrt{f'_c} \quad = \quad 9.13 \text{ kips} \quad > \quad V_u \quad \text{[Satisfactory]}$$

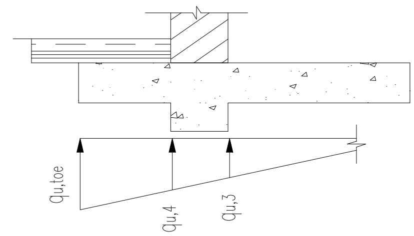
where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} \quad = \quad 0.023 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} \quad = \quad 0.001$$

$$M_{u,3} = \begin{cases} \dfrac{L_H}{2}\left(\gamma w_s + \gamma w_b + \dfrac{L_H}{L}\gamma w_f\right) - \dfrac{\left(q_{u,3}+2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{L_H}{2}\left(\gamma w_s + \gamma w_b + \dfrac{L_H}{L}\gamma w_f\right) - \dfrac{q_{u,3}bS^2}{6}, & for \quad e_u > \dfrac{L}{6} \end{cases} \quad = \quad 6.04 \text{ ft-kips}$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} \quad = \quad 0.001$$



where	d	=	9.63 in	$q_{u,\,toe}$	=	2.54 ksf
	e_u	=	0.63 ft	$q_{u,\,heel}$	=	1.12 ksf
	S	=	n/a	$q_{u,\,3}$	=	1.78 ksf

$(A_{S,3})_{required}$ = 0.14 in^2 / ft < $A_{S,3}$ [Satisfactory]

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} \quad = \quad 0.023 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \ \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.001$$

$$M_{u,4} = \frac{\left(q_{u,4}+2q_{u,toe}\right)bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f \quad = \quad 9.73 \text{ ft-kips}$$

where d = 8.69 in
$q_{u,\,4}$ = 1.95 ksf

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} \quad = \quad 0.002$$

$(A_{S,4})_{required}$ = 0.25 in^2 / ft < $A_{S,4}$ [Satisfactory]

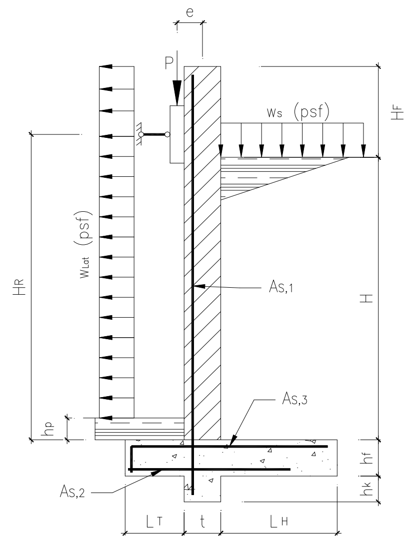
CHECK SLIDING CAPACITY (IBC 09 1807.2.3)

1.1 ($H_b + H_s + H_{Lat}$) =	2.61 kips	<	$H_p + \mu\,\Sigma W$ =	4.64 kips	[Satisfactory]
1.5 ($H_b + H_s$) =	3.17 kips	<	$H_p + \mu\,\Sigma W$ =	4.64 kips	[Satisfactory]

Restrained Retaining Masonry Wall Design Based on TMS 402-08 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes		
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a	=	45	pcf (equivalent fluid pressure)		
PASSIVE PRESSURE	P_p	=	350	psf / ft		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SURCHARGE WEIGHT	w_s	=	100	psf		
SERVICE LATERAL FORCE	w_{Lat}	=	30.1	psf		
SERVICE GRAVITY LOAD	P	=	0.386	kips / ft		
ECCENTRICITY	e	=	6	in		
FRICTION COEFFICIENT	μ	=	0.35			
ALLOW SOIL PRESSURE	Q_a	=	2.25	ksf		
THICKNESS OF STEM	t	=	8	in		
TOE WIDTH	L_T	=	0.917	ft		
HEEL WIDTH	L_H	=	0.917	ft		
HEIGHT OF FENCE STEM	H_F	=	5	ft		
HEIGHT OF STEM	H	=	6.83	ft		
RESTRAINED HEIGHT	H_R	=	9	ft		
FOOTING THICKNESS	h_f	=	12	in		
RESTRAINED BOTTOM ? (1=Yes, 0=No)			1	Yes		
KEY DEPTH	h_k	=	0	<=No ReqD		
SOIL OVER TOE	h_p	=	6	in		
STEM REINF. ($A_{s,1}$)		#	8	@	8	in o.c.
$A_{s,1}$ LOCATION (0=at inside face, 1=at middle, 2=at each face)					0	at inside face
BOT. REINF.OF FOOTING ($A_{s,2}$)		#	4	@	24	in
TOP REINF.OF FOOTING ($A_{s,3}$)		#	4	@	18	in

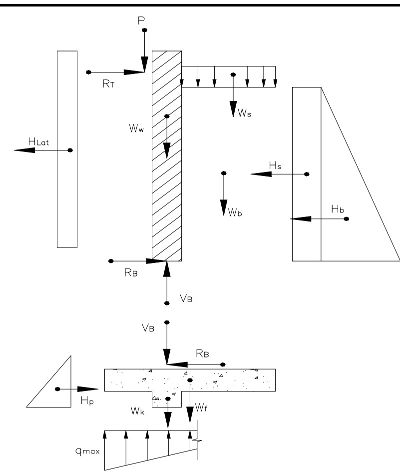


[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS

$H_b = 0.5\, P_a\, H^2$	=	1.05	kips
$H_s = w_s\, P_a\, H / \gamma_b$	=	0.28	kips
$H_p = 0.5\, P_p\, (h_p + h_f + h_k)^2$	=	0.39	kips
$H_{Lat} = w_{Lat}\,(H_F + H - h_p)$	=	0.34	kips
$W_s = w_s\, L_H$	=	0.09	kips
$W_b = H\, L_H\, \gamma_b$	=	0.69	kips
$W_f = h_f\,(L_H + t + L_T)\,\gamma_c$	=	0.38	kips
$W_k = h_k\; t\, \gamma_c$	=	0.00	kips
$W_w = t\,(H_F + H)\;\gamma_m$	=	0.87	kips
$R_T = 0.5H_{Lat}(H_F/H_R + h_p/H_R + H/H_R) + Pe/H_R + 0.5H_sH/H_R + H_bH/3H_R$	=	0.63	kips
$R_B = H_{Lat} + H_s + H_b - R_T$	=	1.04	kips
$V_B = W_w + P$	=	1.25	kips



FACTORED LOADS

$\gamma H_b = 1.6\, H_b$	=	1.68	kips
$\gamma H_s = 1.6\, H_s$	=	0.45	kips
$\gamma H_{Lat} = 1.6\, H_{Lat}$	=	0.55	kips
$\gamma W_s = 1.6\, W_s$	=	0.15	kips
$\gamma W_b = 1.2\, W_b$	=	0.83	kips
$\gamma W_f = 1.2\, W_f$	=	0.45	kips
$\gamma W_k = 1.2\, W_k$	=	0.00	kips
$\gamma W_w = 1.2\, W_w$	=	1.04	kips
$\gamma P = 1.6\, P$	=	0.62	kips

$\gamma R_T = 1.6\, R_T$	=	1.00	kips
$\gamma R_B = 1.6\, R_B$	=	1.67	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
R_B	1.04	1.67	1.00	1.04	1.67
Σ	1.04	1.67		**1.04**	1.67

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.09	0.15	2.04	0.19	0.30
W_b	0.69	0.83	2.04	1.41	1.69
W_f	0.38	0.45	1.25	0.47	0.56
W_k	0.00	0.00	1.25	0.00	0.00
P	0.39	0.62	1.25	0.48	0.77
W_w	0.87	1.04	1.25	1.08	1.30
Σ	2.41	3.08		**3.63**	4.62

OVERTURNING FACTOR OF SAFETY

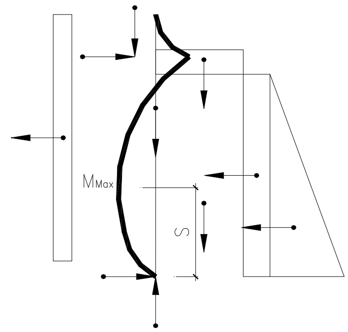
$$SF = \frac{\Sigma Wx}{\Sigma Hy} = \quad 3.48 \quad > \quad 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H$$ = 2.50 ft

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W}$$ = 0.18 ft



$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$ = 1.37 ksf

< Q_a

[Satisfactory]

CHECK FLEXURE CAPACITY FOR MASONRY STEM (TMS 402 2.3.3)

$S = P_a^{-1}\{ (P_aH + w_{Lat} + H_S / H) + [(P_aH + w_{Lat} + H_S / H)^2 - 2P_a(R_B + w_{Lat}\, h_p)]^{0.5} \}$

= 3.54 ft

$P = V_B - W_W\, S / (H + H_F)$ = 0.99 kips, @ M_{max} section

$M_{Max} = S\, R_B - 0.5\, H_S\, S^2 / H - P_a\, S^3/ 3 - P_a\, (H\text{-}S)\, S^2/2 - 0.5\, w_{Lat}\, (S - h_p)^2$ = 1.71 ft-kips

$$M_{allowable} = MIN\left[\frac{1}{2}b_w kd\, F_b\left(d-\frac{kd}{3}\right) - P\left(d-\frac{t_e}{2}\right) \quad , \quad A_s F_s\left(d-\frac{kd}{3}\right) + P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right] =$$ 3.21 ft-kips

> *M*

[Satisfactory]

where							
t_e	=	7.63	in	ρ	=	0.018	
b_w	=	12	in	E_m	=	1350	ksi
F_b	=	0.495	ksi	E_s	=	29000	ksi
F_s	=	24	ksi	n	=	21.48	
A_s	=	1.185	in^2	k	=	0.58	
d	=	5.38	in, (TMS 402-08, 1.13.3.5)				

CHECK SHEAR CAPACITY FOR MASONRY STEM (TMS 402 2.3.5)

		At restrained stem	At bottom of stem
$V = Max.\ \ Horiz.\ \ Shear$	=	0.54 kips ,	1.04 kips
$V_{allowable} = d\, b_w MIN\left(\sqrt{f'_m} \ \ , \ \ 50\right)$	=	1.97 kips ,	2.50 kips
		> *V* [Satisfactory]	> *V* [Satisfactory]

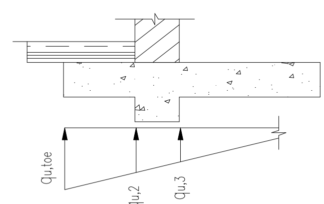
CHECK HEEL FLEXURE CAPACITY, $A_{S,3}$, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$$ = 0.013

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d}$$ = 0.001

$$M_{u,3} = \begin{cases} \dfrac{L_H}{2}\left(\gamma w_s + \gamma w_b + \dfrac{L_H}{L}\gamma w_f\right) - \dfrac{\left(q_{u,3} + 2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{L_H}{2}\left(\gamma w_s + \gamma w_b + \dfrac{L_H}{L}\gamma w_f\right) - \dfrac{q_{u,3}bS^2}{6}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$ = 0.29 ft-kips

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y}$$ = 0.000



where							
d	=	10.25	in	$q_{u,\,toe}$	=	2.09	ksf
e_u	=	0.29	ft	$q_{u,\,heel}$	=	0.37	ksf
S	=	n/a		$q_{u,\,3}$	=	0.94	ksf

$(A_{S,3})_{required}$ = 0.13 in^2 / ft < $A_{S,3}$ [Satisfactory]

CHECK TOE FLEXURE CAPACITY, $A_{S,2}$, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} = 0.013 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.000$$

$$M_{u,2} = \frac{\left(q_{u,4} + 2q_{u,toe}\right)bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 0.71 \text{ ft-kips}$$

where d = 8.75 in

$q_{u,2}$ = 1.43 ksf

$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,2}}{0.383 b d^2 f'_c}}\right)}{f_y} = 0.000$$

$(A_{S,2})_{required}$ = 0.02 in^2 / ft < $A_{S,2}$ [Satisfactory]

CHECK SLIDING CAPACITY

1.5 R_B = 1.57 kips < $H_p + \mu \Sigma W$ = N/A (Restrained)
[Satisfactory]

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.
3. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

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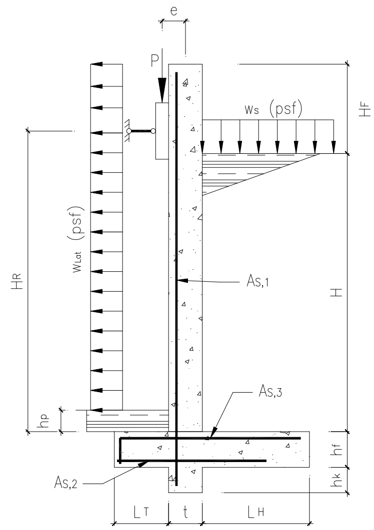
Restrained Retaining Concrete Wall Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	2.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
LATERAL SOIL PRESSURE	P_a	=	45	pcf (equivalent fluid pressure)
PASSIVE PRESSURE	P_p	=	350	psf / ft
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf
SURCHARGE WEIGHT	w_s	=	100	psf
SERVICE LATERAL FORCE	w_{Lat}	=	30.1	psf
SERVICE GRAVITY LOAD	P	=	0.386	kips / ft
ECCENTRICITY	e	=	6	in
FRICTION COEFFICIENT	μ	=	0.35	
ALLOW SOIL PRESSURE	Q_a	=	2.25	ksf
THICKNESS OF STEM	t	=	8	in
TOE WIDTH	L_T	=	0.917	ft
HEEL WIDTH	L_H	=	0.917	ft
HEIGHT OF FENCE STEM	H_F	=	5	ft
HEIGHT OF STEM	H	=	10	ft
RESTRAINED HEIGHT	H_R	=	9	ft
FOOTING THICKNESS	h_f	=	12	in
RESTRAINED BOTTOM ? (1=Yes, 0=No)			1	Yes
KEY DEPTH	h_k	=	0	<=No ReqD
SOIL OVER TOE	h_p	=	6	in



[THE WALL DESIGN IS ADEQUATE.]

STEM REINF. ($A_{s,1}$)	#	6	@	12	in o.c.
$A_{s,1}$ LOCATION (0=at inside face, 1=at middle, 2=at each face)				0	at inside face
BOT. REINF.OF FOOTING ($A_{s,2}$)	#	4	@	24	in
TOP REINF.OF FOOTING ($A_{s,3}$)	#	4	@	18	in

ANALYSIS

SERVICE LOADS

$H_b = 0.5\, P_a\, H^2$ = 2.25 kips

$H_s = w_s\, P_a\, H / \gamma_b$ = 0.41 kips

$H_p = 0.5\, P_p\, (h_p + h_f + h_k)^2$ = 0.39 kips

$H_{Lat} = w_{Lat}\, (H_F + H - h_p)$ = 0.44 kips

$W_s = w_s\, L_H$ = 0.09 kips

$W_b = H\, L_H\, \gamma_b$ = 1.01 kips

$W_f = h_f\, (L_H + t + L_T)\, \gamma_c$ = 0.38 kips

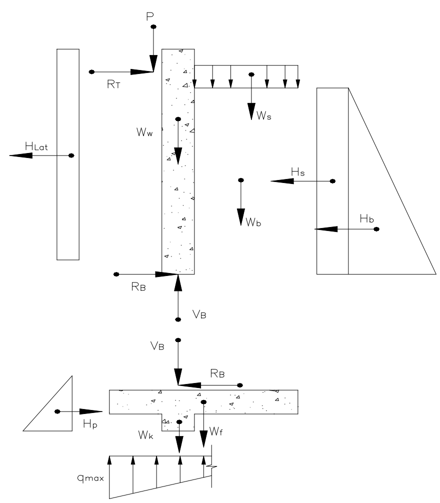
$W_k = h_k\, t\, \gamma_c$ = 0.00 kips

$W_w = t\, (H_F + H)\, \gamma_c$ = 1.50 kips

$R_T = 0.5 H_{Lat}(H_F/H_R + h_p/H_R + H/H_R) + Pe/H_R + 0.5 H_s H/H_R + H_b H/3H_R$ = 1.46 kips

$R_B = H_{Lat} + H_s + H_b - R_T$ = 1.64 kips

$V_B = W_w + P$ = 1.89 kips



FACTORED LOADS

$\gamma H_b = 1.6\, H_b$ = 3.60 kips

$\gamma H_s = 1.6\, H_s$ = 0.65 kips

$\gamma H_{Lat} = 1.6\, H_{Lat}$ = 0.70 kips

$\gamma W_s = 1.6\, W_s$ = 0.15 kips

$\gamma W_b = 1.2\, W_b$ = 1.21 kips

$\gamma W_f = 1.2\, W_f$ = 0.45 kips

$\gamma W_k = 1.2\, W_k$ = 0.00 kips

$\gamma W_w = 1.2\, W_w$ = 1.80 kips

$\gamma P = 1.6\, P$ = 0.62 kips

$\gamma R_T = 1.6\, R_T$ = 2.33 kips

$\gamma R_B = 1.6\, R_B$ = 2.62 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
R_B	1.64	2.62	1.00	1.64	2.62
Σ	1.64	2.62		**1.64**	2.62

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.09	0.15	2.04	0.19	0.30
W_b	1.01	1.21	2.04	2.06	2.47
W_f	0.38	0.45	1.25	0.47	0.56
W_k	0.00	0.00	1.25	0.00	0.00
P	0.39	0.62	1.25	0.48	0.77
W_w	1.50	1.80	1.25	1.88	2.25
Σ	3.36	4.22		**5.07**	6.36

OVERTURNING FACTOR OF SAFETY

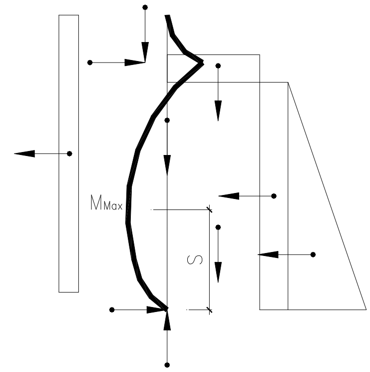
$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 3.10 > 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318 15.2.2)

$L = L_T + t_b + L_H$ = 2.50 ft

$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W}$ = 0.23 ft



$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$$ = 2.08 ksf

< Q_a

[Satisfactory]

CHECK FLEXURE CAPACITY FOR CONCRETE STEM

(ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$S = P_a^{-1}\{ (P_aH + w_{Lat} + H_S / H) + [(P_aH + w_{Lat} + H_S / H)^2 - 2P_a(R_B + w_{Lat}\, h_p)]^{0.5} \}$

= 3.79 ft

$P = V_B - W_w\, S / (H + H_F)$ = 1.51 kips, @ M_{max} section

$M_{Max} = S\, R_B - 0.5\, H_S\, S^2 / H - P_a\, S^3/ 3 - P_a\, (H-S)\, S^2/2 - 0.5\, w_{Lat}\, (S - h_p)^2$ = 2.93 ft-kips

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$$ = 0.013

$$\rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \ 0.0018\frac{t}{d}\right) =$$ 0.003

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y}$$ = 0.003

t	=	8.00	in
d	=	5.63	in
b	=	12	in
M_u	=	1.5 M_{Max} = 4.39	ft-kips
$A_{S,\,1}$	=	0.44	in^2 / ft

$(A_{S,\,1})_{required}$ = 0.18 in^2 / ft < $A_{S,\,1}$ [Satisfactory]

CHECK SHEAR CAPACITY (ACI 318 15.5.2, 11.1.3.1, & 11.2)

		At restrained section	At bottom of wall
$V = Max.\ \ Horiz.\ \ Shear$	=	1.28 kips ,	1.64 kips
$\phi V_n = 2\phi bd\sqrt{f'_c}$	=	5.06 kips ,	5.06 kips
		> V_u = 1.5 V	> V_u = 1.5 V
		[Satisfactory]	[Satisfactory]

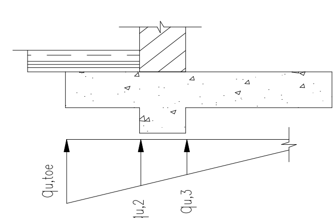
CHECK HEEL FLEXURE CAPACITY, $A_{S,3}$, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$$ = 0.013

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d}$$ = 0.001

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{\left(q_{u,3} + 2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases}$$ = 0.46 ft-kips

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y}$$ = 0.000



where							
	d	=	10.25 in	$q_{u,\,toe}$	=	3.17	ksf
	e_u	=	0.37 ft	$q_{u,\,heel}$	=	0.21	ksf
	S	=	n/a	$q_{u,\,3}$	=	1.28	ksf

$(A_{S,\,3})_{required}$ = 0.13 in^2 / ft < $A_{S,\,3}$ [Satisfactory]

CHECK TOE FLEXURE CAPACITY, $A_{S,2}$, FOR FOOTING (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} \quad = \quad 0.013 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.000$$

$$M_{u,2} = \frac{\left(q_{u,4} + 2q_{u,toe}\right) b L_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f \quad = \quad 1.10 \text{ ft-kips}$$

where d = 8.75 in

$q_{u,2}$ = 2.08 ksf

$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,2}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.000$$

$(A_{S,2})_{required}$ = 0.04 in^2 / ft < $A_{S,2}$ [Satisfactory]

CHECK SLIDING CAPACITY

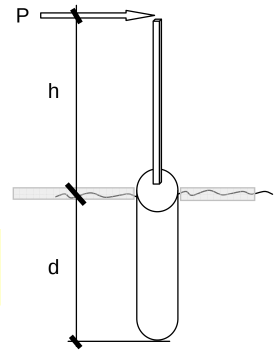
1.5 R_B = 2.46 kips < $H_P + \mu\,\Sigma W$ = N/A (Restrained)

[Satisfactory]

Flagpole Footing Design Based on Chapter 18 of IBC & CBC

INPUT DATA & DESIGN SUMMARY

IS FOOTING RESTRAINED @ GRADE LEVEL ? (1=YES,0=NO)		0	no
LATERAL FORCE @ TOP OF POLE	P =	100	k
HEIGHT OF POLE ABOVE GRADE	h =	62.11	ft
DIAMETER OF POLE FOOTING	b =	6	ft
LATERAL SOIL BEARING CAPACITY	S =	0.2	ksf / ft
ISOLATED POLE FACTOR (IBC 09 1806.3.4)	F =	2	
FIRST TRIAL DEPTH ===>	d =	8	ft



Use 6 ft dia x 30.55 ft deep footing unrestrained @ ground level

ANALYSIS

LATERAL BEARING @ BOTTOM : $S_3 = FS \quad Min(d \;,\; 12')$

LATERAL BEARING @ d/3 : $S_1 = FS \quad Min\left(\frac{d}{3} \;,\; 12'\right)$

$$A = \frac{2.34P}{bS_1}$$

REQUIRD DEPTH :
$$d = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25Ph}{bS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

		NONCONSTRAINED		CONSTRAINED	
LATERAL FORCE @ TOP OF POLE	P =>	100.00	k	100.00	k
HEIGHT OF POLE ABOVE GRADE	h =>	62.1	ft	62.1	ft
DIAMETER OF POLE FOOTING	b =>	6.00	ft	6.00	ft
LATERAL SOIL BEARING CAPACITY	FS =>	0.40	ksf / ft	0.40	ksf / ft
1ST TRIAL	TRY d_1 =>	8.00	ft	8.00	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	1.07	ksf	1.07	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	3.20	ksf	3.20	ksf
CONSTANT 2.34P/(bS_1)	A =>	36.56	-		-
REQD FOOTING DEPTH	RQRD d =>	71.29	ft	37.08	ft
2ND TRIAL :	TRY d_2=>	39.64	ft	22.54	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	4.80	ksf	3.01	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	4.80	ksf	4.80	ksf
CONSTANT 2.34P/(bS_1)	A =>	8.13	-		-
REQD FOOTING DEPTH	RQRD d =>	27.87	ft	30.27	ft
3RD TRIAL :	TRY d_3=>	33.75	ft	26.41	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	4.50	ksf	3.52	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	4.80	ksf	4.80	ksf
CONSTANT 2.34P/(bS_1)	A =>	8.67	-		-
REQD FOOTING DEPTH	RQRD d =>	28.94	ft	30.27	ft
4TH TRIAL :	TRY d_4=>	31.35	ft	28.34	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	4.18	ksf	3.78	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	4.80	ksf	4.80	ksf
CONSTANT 2.34P/(bS_1)	A =>	9.33	-		-
REQD FOOTING DEPTH	RQRD d =>	30.23	ft	30.27	ft
5TH TRIAL :	TRY d_5=>	30.79	ft	29.31	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	4.11	ksf	3.91	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	4.80	ksf	4.80	ksf
CONSTANT 2.34P/(bS_1)	A =>	9.50	-		-
REQD FOOTING DEPTH	RQRD d =>	30.55	ft	30.27	ft

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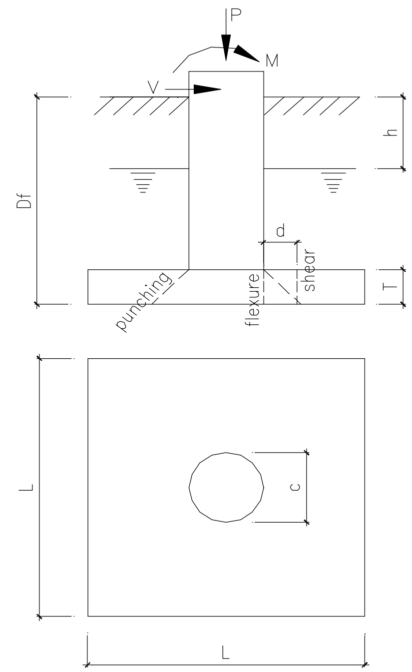
Deep Footing Design Based on ACI 318-08

INPUT DATA

PEDESTAL DIAMETER	c	=	60	in
SQUARE FOOTING LENGTH	L	=	18	ft
FOOTING EMBEDMENT DEPTH	D_f	=	10.5	ft
FOOTING THICKNESS	T	=	22	in
WATER TABLE	h	=	8	ft
CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	73.1	k
AXIAL LIVE LOAD	P_{LL}	=	50	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	1	Seismic,SD
SEISMIC AXIAL LOAD	P_{LAT}	=	-79.175	k, SD, uplift
SEISMIC MOMENT LOAD	M_{LAT}	=	120	ft-k, SD
SEISMIC SHEAR LOAD	V_{LAT}	=	11	k, SD
SURCHARGE	q_s	=	0.1	ksf
BACKFILL SOIL WEIGHT	w_s	=	0.1	kcf
ALLOW SOIL PRESSURE	Q_a	=	4	ksf
FOOTING REINFORCING SIZE		#	10	
PEDESTAL VERT. REINF. SIZE	22	#	10	vertical
PEDESTAL SHEAR. REINF.	#	4	spiral @ 3	in o.c.



DESIGN SUMMARY

TOP FOOTING REINF., E. WAY => 2 # 10

BOT. FOOTING REINF., E. WAY => 13 # 10 @ 17 in o.c.

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS AT TOP OF FOOTING (IBC SEC.1605.3.2 & ACI 318-08 SEC.9.2.1)

CASE 1:	DL + LL	P	=	123	kips	1.2 DL + 1.6 LL	Pu	=	168	kips
CASE 2:	DL + LL + E / 1.4	P	=	67	kips	1.2 DL + 1.0 LL + 1.0 E	Pu	=	59	kips
		M	=	154	ft-kips		Mu	=	215	ft-kips
		V	=	8	kips		Vu	=	11	kips
		e	=	2.3	ft, fr cl ftg		eu	=	3.7	ft, fr cl ftg
CASE 3:	0.9 DL + E / 1.4	P	=	9	kips	0.9 DL + 1.0 E	Pu	=	-13	kips
		M	=	154	ft-kips		Mu	=	215	ft-kips
		V	=	8	kips		Vu	=	11	kips
		e	=	16.7	ft, fr cl ftg		eu	=	-16.1	ft, fr cl ftg

CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 4.20042 **>** 1 / 0.9 **[Satisfactory]**

Where M_O = $M_{LAT} + V_{LAT} D_f - 0.5 P_{LAT} L$ = 948 k-ft

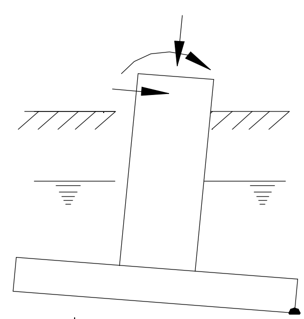
P_{ftg} = (0.15 kcf) $[T L^2 + (\pi c^2/ 4)(D_f - T)]$ = 114.63 k, footing weight

w_{sat} = w_s + 0.018 kcf = 0.118 kcf, saturated soil weight

w_{water} = 0.0625 kcf, water specifc weight

P_{soil} = $[w_s \text{ MIN}(h, D_f - T) + (w_{sat} - w_{water}) \text{ MAX}(D_f - T - h, 0)] (L^2 - \pi c^2/ 4)$ = 254.75 k, soil wt

M_R = $0.5 (P_{DL} + P_{ftg} + P_{soil}) L$ = 3982 k-ft



CHECK UPLIFT CAPACITY

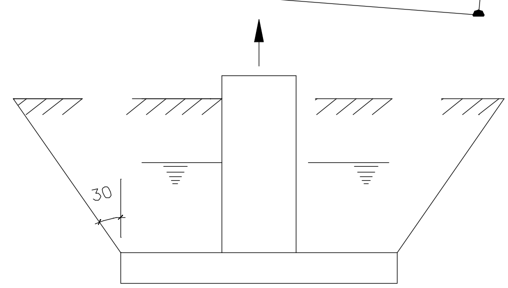
$F_{Gravity}$ / F_{Uplift} = 7.51183 **>** 1.0 **[Satisfactory]**

Where F_{Uplift} = $- P_{LAT}$ = 79.2 k

P_{ftg} = 114.63 k, footing weight

P_{soil} = 407.02 k, soil weight with 30° pyramid

$F_{Gravity}$ = $P_{DL} + P_{ftg} + P_{soil}$ = 594.75 k



(cont'd)

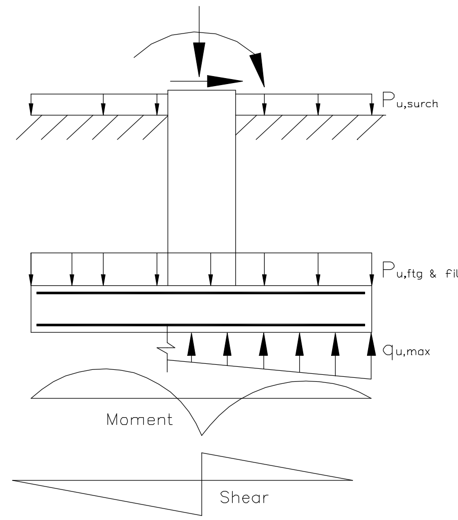
CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

Service Loads	CASE 1	CASE 2	CASE 3	
P	123.1	66.5	9.2	k
e	0.0	2.5	18.2	ft, (from center of footing)
$q_s L^2$	32.4	32.4	0.0	k, (surcharge load)
ΔP_{ftg}	38.2	38.2	34.4	k, (footing increased)
Σ P	193.7	137.2	43.6	k
Σ M	0.0	250.7	250.7	k-ft, (V_{Lat} included)
e	0.0 < L/6	1.8 < L/6	5.7 > L/6	ft
q_{max}	0.6	0.7	0.5	ksf
q_{allow}	4.0	5.3	5.3	ksf

Where

$$q_{\max}=\begin{cases}\dfrac{(\Sigma P)\left(1+\dfrac{6e}{L}\right)}{L^2}, & for\quad e\le\dfrac{L}{6}\\[2ex] \dfrac{2(\Sigma P)}{3L(0.5L-e)}, & for\quad e>\dfrac{L}{6}\end{cases}$$

[Satisfactory]



DESIGN FOOTING FLEXURE & CHECK FLEXURE SHEAR

(ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)

$$q_{u,\max}=\begin{cases}\dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{L^2}, & for\quad e_u\le\dfrac{L}{6}\\[2ex] \dfrac{2(\Sigma P_u)}{3L(0.5L-e_u)}, & for\quad e_u>\dfrac{L}{6}\end{cases}$$

$$\rho_{MAX}=\frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho=\frac{0.85f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2f'_c}}\right)}{f_y}$$

$$\rho_{MIN}=MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	167.7	58.5	-13.4	k
e_u	0.0	4.0	-17.6	ft, (at base, including V_u T / P_u)
$\gamma\, q_s L^2$	51.8	32.4	0.0	k, (factored surcharge load)
$\gamma\, P_{u,ftg\,\&\,fill}$	443.3	443.3	332.4	k, (factored footing & backfill loads)
Σ P_u	662.8	534.2	319.1	k
Σ M_u	0.0	351.0	351.0	k-ft, (V_{Lat} included)
e_u	0.0 < L/6	0.7 < L/6	1.1 < L/6	ft
$q_{u,\,max}$	2.046	2.010	1.346	ksf

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **1**

Section	0	0.09 L	0.18 L	0.27 L	Col_L	Col_R	0.73 L	0.82 L	0.91 L	L
X_u (ft, dist. from left of footing)	0	1.63	3.25	4.88	6.50	11.50	13.13	14.75	16.38	18.00
$M_{u,pedestal}$ (ft-k)	0	0	0	0	0	-419.3	-691.85	-964.39	-1236.9	-1509.5
$V_{u,pedestal}$ (k)	0	0.0	0.0	0.0	0.0	167.7	167.7	167.7	167.7	167.7
$P_{u,surch}$ (klf)	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88
$M_{u,surch}$ (ft-k)	0	-3.8	-15.2	-34.2	-60.8	-190.4	-248.1	-313.3	-386.1	-466.6
$V_{u,surch}$ (k)	0	4.7	9.4	14.0	18.7	33.1	37.8	42.5	47.2	51.8
$P_{u,ftg\,\&\,fill}$ (klf)	24.63	24.63	24.63	24.63	24.63	24.63	24.63	24.63	24.63	24.63
$M_{u,ftg\,\&\,fill}$ (ft-k)	0	-32.513	-130.05	-292.62	-520.21	-1628.3	-2121	-2678.8	-3301.5	-3989.3
$V_{u,ftg\,\&\,fill}$ (k)	0	40.0	80.0	120.0	160.1	283.2	323.2	363.2	403.2	443.3
$q_{u,soil}$ (ksf)	2.05	2.05	2.05	2.05	2.05	2.05	2.05	2.05	2.05	2.05
$M_{u,soil}$ (ft-k)	0	48.618	194.47	437.56	777.89	2434.9	3171.7	4005.7	4936.9	5965.3
$V_{u,soil}$ (k)	0	-59.837	-119.67	-179.51	-239.35	-423.46	-483.3	-543.14	-602.98	-662.81
Σ M_u (ft-k)	**0**	**12.3**	**49.2**	**110.7**	**196.8**	**196.8**	**110.7**	**49.2**	**12.3**	**0**
Σ V_u (kips)	**0**	**-15.1**	**-30.3**	**-45.4**	**-60.6**	**60.6**	**45.4**	**30.3**	**15.1**	**0**

(cont'd)

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **2**

Section	0	0.09 L	0.18 L	0.27 L	Col_L	Col_R	0.73 L	0.82 L	0.91 L	L
X_u (ft, dist. from left of footing)	0	1.63	3.25	4.88	6.50	11.50	13.13	14.75	16.38	18
$M_{u,pedestal}$ (ft-k)	0	0	0	0	0	204.64	109.5	14.366	-80.769	-175.91
$V_{u,pedestal}$ (k)	0	0.0	0.0	0.0	0.0	58.5	58.5	58.5	58.5	58.5
$P_{u,surch}$ (klf)	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.80
$M_{u,surch}$ (ft-k)	0	-2.4	-9.5	-21.4	-38.0	-119.0	-155.0	-195.8	-241.3	-291.6
$V_{u,surch}$ (k)	0	2.9	5.9	8.8	11.7	20.7	23.6	26.6	29.5	32.4
$P_{u,ftg\ \&\ fill}$ (klf)	24.63	24.63	24.63	24.63	24.63	24.63	24.63	24.63	24.63	24.63
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-32.513	-130.05	-292.62	-520.21	-1628.3	-2121	-2678.8	-3301.5	-3989.3
$V_{u,ftg\ \&\ fill}$ (k)	0	40.0	80.0	120.0	160.1	283.2	323.2	363.2	403.2	443.3
$q_{u,soil}$ (ksf)	1.29	1.35	1.42	1.48	1.55	1.75	1.81	1.88	1.94	2.01
$M_{u,soil}$ (ft-k)	0	31.1	126.5	289.4	522.7	1715.7	2268.5	2907.6	3636.0	4456.8
$V_{u,soil}$ (k)	0	-38.617	-79.142	-121.57	-165.91	-314.3	-366.42	-420.44	-476.36	-534.2
Σ M_u (ft-k)	**0**	**-3.7712**	**-13.019**	**-24.644**	**-35.547**	**172.96**	**101.94**	**47.373**	**12.36**	**0**
Σ V_u (kips)	**0**	**4.3**	**6.7**	**7.2**	**5.9**	**48.1**	**39.0**	**27.9**	**14.9**	**0**

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **3**

Section	0	0.09 L	0.18 L	0.27 L	Col_L	Col_R	0.73 L	0.82 L	0.91 L	L
X_u (ft, dist. from left of footing)	0	1.63	3.25	4.88	6.50	11.50	13.13	14.75	16.38	18.00
$M_{u,pedestal}$ (ft-k)	0	0	0	0	0	384.46	406.21	427.96	449.71	471.47
$V_{u,pedestal}$ (k)	0	0.0	0.0	0.0	0.0	-13.4	-13.4	-13.4	-13.4	-13.4
$P_{u,surch}$ (klf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{u,surch}$ (ft-k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$V_{u,surch}$ (k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{u,ftg\ \&\ fill}$ (klf)	18.47	18.47	18.47	18.47	18.47	18.47	18.47	18.47	18.47	18.47
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-24.385	-97.539	-219.46	-390.16	-1221.3	-1590.8	-2009.1	-2476.1	-2992
$V_{u,ftg\ \&\ fill}$ (k)	0	30.0	60.0	90.0	120.0	212.4	242.4	272.4	302.4	332.4
$q_{u,soil}$ (ksf)	0.62	0.69	0.75	0.82	0.88	1.09	1.15	1.22	1.28	1.35
$M_{u,soil}$ (ft-k)	0	15.337	63.416	147.33	270.19	925.34	1239	1607.4	2033.5	2520.5
$V_{u,soil}$ (k)	0	-19.195	-40.297	-63.306	-88.222	-176.85	-209.54	-244.14	-280.64	-319.06
Σ M_u (ft-k)	**0**	**-9.0**	**-34.1**	**-72.1**	**-120.0**	**88.5**	**54.5**	**26.3**	**7.1**	**0**
Σ V_u (kips)	**0**	**10.8**	**19.7**	**26.7**	**31.8**	**22.2**	**19.5**	**14.9**	**8.4**	**0**

DESIGN FLEXURE

Location	$M_{u,max}$	d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	use	ρ_{provD}
Top Longitudinal	-120.0 ft-k	19.37	0.0004	0.0003	0.0155	no limit	2 # 10	0.0006
Bottom Longitudinal	196.8 ft-k	18.37	0.0008	0.0006	0.0155	18	13 # 10 @ 17 in o.c.	0.0042

[Satisfactory]

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$	$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$	check $V_u < \phi\ V_c$
Longitudinal	60.6 k	326 k	**[Satisfactory]**

CHECK FOOTING PUNCHING SHEAR (ACI 318-08 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u[d+c]}{J} \qquad A_P = b_0 d \qquad \phi v_c(psi) = \phi(2+y)\sqrt{f_c'}$$

$$\gamma_v = 0.4$$

$$J = 0.5(d+c)\left[\pi d\left(\frac{d+c}{2}\right)^2 + \frac{d^3}{3}\right] \qquad A_f = L^2 \qquad y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

$$R = \frac{P_u \pi (d+c)^2}{4A_f} \qquad b_0 = \pi(c+d)$$

Case	P_u	M_u	b_0	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\ (psi)}$	$\phi\ v_c$
1	198.4	0.0	246.2	0.4	1.0	2.0	324.0	31.4	20.5	171.3	39.3	164.3
2	89.2	330.8	246.2	0.4	1.0	2.0	324.0	31.4	9.2	171.3	35.2	164.3
3	9.6	330.8	246.2	0.4	1.0	2.0	324.0	31.4	1.0	171.3	19.4	164.3

[Satisfactory]

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

(cont'd)

CHECK PEDESTAL REINF. LIMITATIONS

ρ_{max}	=	0.08	(ACI 318-08, Section 10.9)	ρ_{provd}	=	0.011		
ρ_{min}	=	0.01	(ACI 318-08, Section 10.9)					**[Satisfactory]**
s_{max}	=	3	(ACI 318-08, Section 7.10.4.3)	s_{provd}	=	3	in	
s_{min}	=	1	(ACI 318-08, Section 7.10.4.3)					**[Satisfactory]**

Soil Pressure Determination for Irregular Footing

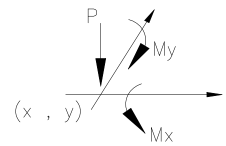
INPUT DATA & ANALYSIS RESULTS

FOOTING EDGE POINT & REACTION PRESSURE

EDGE POINT	X (ft)	Y (ft)	R (psf)
1	0	0	47
2	0	21.5	215
3	38	21.5	196
4	38	0	28

COLUMN LOCATION & BASE LOAD

COL. NO.	X (ft)	Y (ft)	P (kips)	M_x (ft-k)	M_y (ft-k)
1	9.33	4.47	10.6		
2	13	4.47	20		
3	25	4.47	20		
4	28.67	4.47	10.6		
5	17.6	10.5	7		
6	21.2	10.5	7		
7	9.33	19.73	20		
8	13	19.73	20		
9	25	19.73	10.6		
10	28.67	19.73	20		



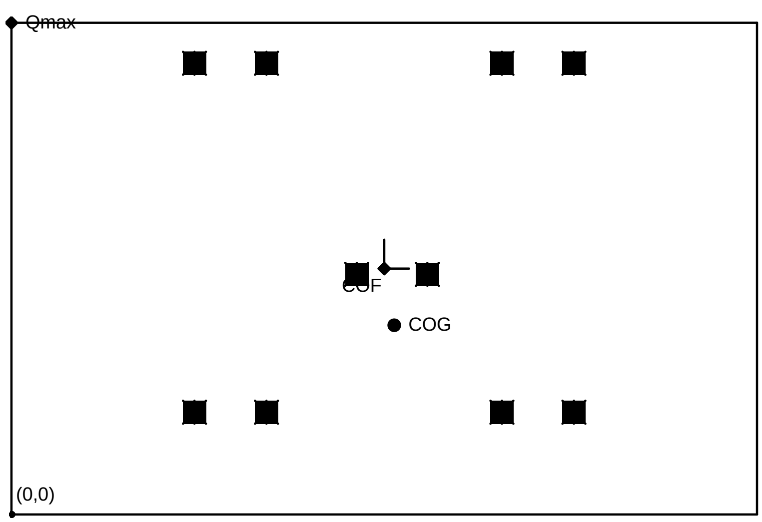
WALL LOCATION & UNIFORM LOAD

WALL NO.	START X (ft)	START Y (ft)	END X (ft)	END Y (ft)	w (k/ft)

NET PRESSURE OF FOOTING SELF WEIGHT = 0.3 k/ft^2

THE MAXIMUM SOIL PRESSURE **215** psf **@ POINT** **2**

ANALYSIS



Footing Area
$A =$ -817.0 ft^2

Centroid of Footing (COF)
$X_c =$ 19.0 ft
$Y_c =$ 10.8 ft

Center of Gravity (COG)
$X_g =$ 19.5 ft
$Y_g =$ 8.3 ft
$\Sigma P =$ -99.3 kips

Moment of Inertia
$I_{xc} =$ 31472 ft^4
$I_{yc} =$ 98312 ft^4

Moment of Inertia for Principle Axes
$I_u =$ 31472 ft^4
$I_v =$ 98312 ft^4
$\theta =$ 0.00 deg

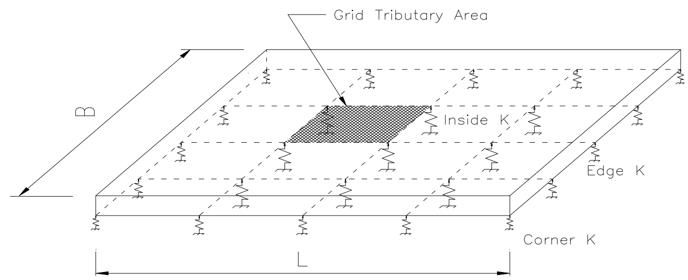
Notes:
1. Assume that the footing is rigid without any deformation.
2. The footing self pressure should be net pressure, (0.15 kcf - 0.11 kcf) (Thk.), to check allowable soil capacity.
3. Use two end columns, uplift & download, to input the shear wall bending load.
4. To design concrete, may use 1.5 time section forces of ADS level.

Mat Boundary Spring Generator

INPUT DATA & DESIGN SUMMARY

FOUNDATION LENGTH	L	=	55	ft
FOUNDATION WIDTH	B	=	31	ft
GRID TRIBUTARY AREA	A	=	1	ft^2
MODULUS OF SUBGRADE	K_1	=	100	lb / in^3

(Obtained from the soil report for 1' x 1' sf plate load test,
in the absence of a soil report obtain from table below)



INSIDE SPRING VALUE	**4.8**	**kips / inch, at each joint**
EDGE SPRING VALUE	**2.4**	**kips / inch, at each joint**
CORNER SPRING VALUE	**1.2**	**kips / inch, at each joint**

ANALYSIS

$$k_s = \begin{cases} k_{11} & (\text{ for } B = L) \\ k_{12} & (\text{ for } B < L) \end{cases} = 33.5 \ lb / in^3$$

$$k_{11} = k_1\left(\frac{B+1}{2B}\right)^2 = 26.6 \ lb / in3 = 46 \ k / ft^3$$

$$k_{12} = k_1\left(\frac{B+1}{2B}\right)^2\left(\frac{\frac{0.5L}{B}+1}{1.5}\right) = 33.5 \ lb / in3 = 58 \ k / ft^3$$

TYPICAL VALUES OF MODULUS OF SUBGRADE REACTIONS, K_1 (lb / in^3)

TYPE OF MATERIAL	MOISTURE CONTENT							
	1 to 4%	5 to 8%	9 to 12%	13 to 16%	17 to 20%	21 to 24%	25 to 28%	> 28%
Silts and clays (liquid limit >50) (OH, CH, MH)	-	175	150	125	100	75	50	25
Silts and clays (liquid limit <50) (OL, CL, ML)	-	200	175	150	125	100	75	50
Silty and clay sands (SM & SH)	300	250	225	200	150	-	-	-
Gravelly sands (SW & SP)	Over 300	300	250	-	-	-	-	-
Silty and clayey gravels (GM & GC)	Over 300	Over 300	300	-	-	-	-	-
Gravel and sand	Over 300	Over 300	-	-	-	-	-	-

Daniel T. Li
Engineering International

PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

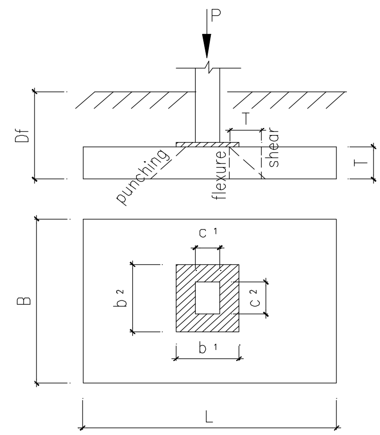
Plain Concrete Footing Design Based on ACI 318-08

INPUT DATA

COLUMN WIDTH	c_1	=	3	in
COLUMN DEPTH	c_2	=	3	in
BASE PLATE WIDTH	b_1	=	7	in
BASE PLATE DEPTH	b_2	=	4	in
FOOTING CONCRETE STRENGTH	fc'	=	2.5	ksi
REBAR YIELD STRESS	fy	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	2	k
AXIAL LIVE LOAD	P_{LL}	=	4.5	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	1	Seismic,SD
SEISMIC AXIAL LOAD	P_{LAT}	=	6.5	k, SD
SURCHARGE	qs	=	0	ksf
SOIL WEIGHT	ws	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	Df	=	0.50	ft
FOOTING THICKNESS	T	=	8	in
ALLOWABLE SOIL PRESSURE	Qa	=	1	ksf
FOOTING WIDTH	B	=	3	ft
FOOTING LENGTH	L	=	3	ft

DESIGN SUMMARY

FOOTING WIDTH	B	=	3.00	ft
FOOTING LENGTH	L	=	3.00	ft
FOOTING THICKNESS	T	=	8	in



THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS (IBC SEC.1605.3.2 & ACI 318-08 SEC.9.2.1)

CASE 1:	DL + LL	P	=	7	k	1.2 DL + 1.6 LL	Pu	=	10	k
CASE 2:	DL + LL + E / 1.4	P	=	11	k	1.2 DL + 1.0 LL + 1.0 E	Pu	=	13	k
CASE 3:	0.9 DL + E / 1.4	P	=	7	k	0.9 DL + 1.0 E	Pu	=	8	k

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

		CASE 1		CASE 2		CASE 3	
$q_{MAX}=\frac{P}{BL}$	=	0.75	ksf,	1.26	ksf,	0.74	ksf

q_{MAX} < k Q_a , [Satisfactory]

where k = 1 for gravity loads, 4/3 for lateral loads.

DESIGN FOR FLEXURE (ACI 318-08 SEC.22.5.1)

$$\phi M_n = MIN\left(5\phi\sqrt{f'_c}S,\ \ 0.85\phi f'_c S\right) \quad = \quad 4.80 \text{ ft-kips}$$

where ϕ = 0.6 (ACI 318-08, Section 9.3.5)

S = elastic section modulus of section = 384 in^3

$$M_u = \frac{\left(0.5L-0.25b_1-0.25c_1\right)^2 P_{u,\max}}{2L} \quad = \quad 3.73 \text{ ft-kips} \quad < \quad \phi\ M_n \qquad \text{[Satisfactory]}$$

CHECK FLEXURE SHEAR (ACI 318-08 SEC.22.5.4)

$$\phi V_n = \frac{4}{3}\phi\sqrt{f'_c}BT \quad = \quad 11.52 \text{ kips}$$

where ϕ = 0.6 (ACI 318-08, Section 9.3.5)

$$V_u = \left(0.5L-0.25b_1-0.25c_1-T\right)\frac{P_{u,\max}}{L} \quad = \quad 2.79 \text{ kips} \quad < \quad \phi\ V_n \qquad \text{[Satisfactory]}$$

CHECK PUNCHING SHEAR (ACI 318-08 SEC.22.5.4)

$$\phi V_n = MIN\left[\left(\frac{4}{3}+\frac{8}{3\beta_c}\right),\ \ 2.66\right]\phi\sqrt{f'_c}\left(c_1+c_2+b_1+b_2+4T\right)T \quad = \quad 31.28 \text{ kips}$$

where ϕ = 0.6 (ACI 318-08, Section 9.3.5)

β_c = ratio of long side to short side of concentrated load = 1.43

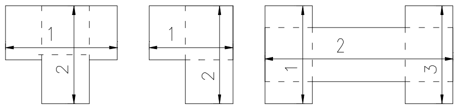
$$V_u = P_{u,\max}\left[1-\frac{1}{BL}\left(\frac{b_1+c_1}{2}+T\right)\left(\frac{b_2+c_2}{2}+T\right)\right] \quad = \quad 11.85 \text{ ft-kips} \quad < \quad \phi\ V_n \qquad \text{[Satisfactory]}$$

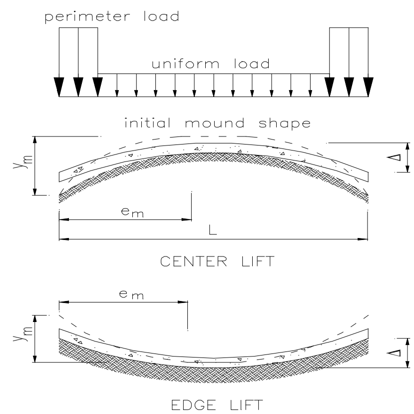
Design of Conventional Slabs on Expansive Soil Grade Based on ACI 360

1. DESIGN METHODS

1.1 DIVIDE AN IRREGULAR FOUNDATION PLAN INTO OVERLAPPING RECTANGLES AND USING THIS SPREADSHEET DESIGN EACH RECTANGULAR SECTION SEPARATELY.

1.2 THE POST-TENSION INSTITUTE (PTI) METHOD IS ACCEPTABLE FOR THE DESIGN OF NONPRESTRESSED SLAB ON GRADE (IBC 09 1808.6.2). THE DESIGNER MAY SELECT EITHER NONPRESTRESSED REINFORCEMENT USING THIS SPREADSHEET, OR POST-TENSIONED REINFORCEMENT IF REQUIRED (ACI 360, 9).





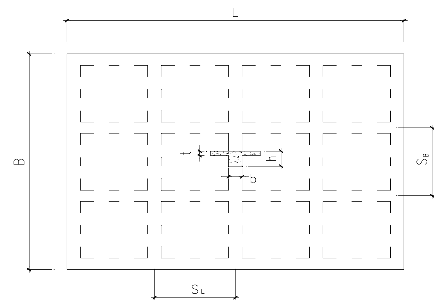
2. INPUT DATA & DESIGN SUMMARY

2.1 SOILS PROPERTIES

ALLOWABLE SOIL-BEARING PRESSURE	q_{allow}	=	2000	psf
EDGE MOISTURE VARIATION DISTANCE	e_m	=	4	ft, for center lift
		=	4.5	ft, for edge lift
DIFFERENTIAL SOIL MOVEMENT	y_m	=	2.68	in, for center lift
		=	0.3	in, for edge lift

2.2 STRUCTURAL DATA AND MATERIALS PROPERTIES

SLAB LENGTH	L	=	164	ft
SLAB WIDTH	B	=	125	ft
SLAB THICKNESS	t	=	5	in
PERIMETER LOADING	P	=	270	plf
MAX BEARING LOADING ON THE SLAB	P_b	=	270	plf
ADDED DEAD LOAD	DL	=	50	psf
LIVE LOAD	LL	=	125	psf
AVERAGE STIFFENING BEAM SPACING, L DIRECTION	S_L	=	30	ft
AVERAGE STIFFENING BEAM SPACING, B DIRECTION	S_B	=	30	ft
STIFFENING BEAM DEPTH	h	=	24	in
STIFFENING BEAM WIDTH	b	=	20	in
CONCRETE STRENGTH	f'_c	=	3	ksi
REINFORCEMENT IN THE BOTTOM OF STIFFENING BEAM	2	#	6	
SLAB REINFORCEMENT #	4	@	18	in o.c., with 1.5 in clear from top of slab, each way.



THE DESIGN IS ADEQUATE.

3. ASSUME A TRIAL SECTION

3.1 ASSUME BEAM DEPTH AND SPACING

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR CENTER LIFT, AT L DIRECTION

Δ_{allow} = 12 MIN(L, 6β) / C_Δ = 1.60 in
Where β = 8 ft
C_Δ = 360

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR EDGE LIFT, AT L DIRECTION

Δ_{allow} = 12 MIN(L, 6β) / C_Δ = 0.80 in
Where β = 8 ft
C_Δ = 720

BEAM DEPTH, FOR CENTER LIFT, AT L DIRECTION

$h = [(y_m\,L)^{0.205}\,S_B^{1.059}\,P^{0.523}\,e_m^{1.296} / 380\,\Delta_{allow}]^{0.824}$ = 13.56 in

BEAM DEPTH, FOR EDGE LIFT, AT L DIRECTION

$h = [L^{0.35}\,S_B^{0.88}\,e_m^{0.74}\,y_m^{0.76} / 15.9\,\Delta_{allow}\,P^{0.01}]^{1.176}$ = 8.47 in

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR CENTER LIFT, AT B DIRECTION

Δ_{allow} = 12 MIN(B, 6β) / C_Δ = 1.60 in
Where β = 8 ft
C_Δ = 360

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR EDGE LIFT, AT B DIRECTION

Δ_{allow} = 12 MIN(B, 6β) / C_Δ = 0.80 in
Where β = 8 ft
C_Δ = 720

BEAM DEPTH, FOR CENTER LIFT, AT B DIRECTION

$h = [(y_m\,B)^{0.205}\,S_L^{1.059}\,P^{0.523}\,e_m^{1.296} / 380\,\Delta_{allow}]^{0.824}$ = 12.95 in

BEAM DEPTH, FOR EDGE LIFT, AT B DIRECTION

$h = [B^{0.35}\,S_L^{0.88}\,e_m^{0.74}\,y_m^{0.76} / 15.9\,\Delta_{allow}\,P^{0.01}]^{1.176}$ = 7.58 in

GOVERNING h = 13.56 in < **ACTUAL h = 24.00 in [Satisfactory]**

3.2 DETERMINE SECTION PROPERTIES

L DIRECTION

A_s	=	17	in^2	n	=	6	beams
E_s / E_c	=	9.29		y_b	=	18.75	in
CGS	=	21.75	in	S_t	=	64268	in^3
A	=	9935	in^2	S_b	=	17995	in^3
I	=	337410	in^4				

B DIRECTION

A_s	=	22	in^2	n	=	7	beams
E_s / E_c	=	9.29		y_b	=	19.00	in
CGS	=	22.25	in	S_t	=	80834	in^3
A	=	12703	in^2	S_b	=	21276	in^3
I	=	404232	in^4				

4. CALCULATE MAXIMUM APPLIED SERVICE MOMENTS

4.1 CENTER LIFT MOMENT AT L DIRECTION

$M_L = A_0\,(B\,e_m^{1.238} + C)$ = 4.96 ft-kips / ft

Where $A_0 = (L^{0.013}\,S_B^{0.306}\,h^{0.688}\,P^{0.534}\,y_m^{0.193}) / 727$ = 0.891

B = 1, for e_m < 5
B = MIN[(y_m - 1) / 3, 1], for e_m > 5
= 1.00

C = 0, for e_m < 5
C = MAX{[8 - (P - 613) / 255] (4 - y_m) / 3], 0}, for e_m > 5
= 0.00

CENTER LIFT MOMENT AT B DIRECTION

$M_B = (58 + e_m)\,M_L / 60$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1
= 5.12 ft-kips / ft

4.2 EDGE LIFT MOMENT AT L DIRECTION

$M_L = S_B^{0.10}\,(h\,e_m)^{0.78}\,y_m^{0.66} / (7.2\,L^{0.0065}\,P^{0.04})$ = 2.63 ft-kips / ft

EDGE LIFT MOMENT AT B DIRECTION

$M_B = h^{0.35}\,(19 + e_m)\,M_L / 57.75$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1
= 3.25 ft-kips / ft

5. CHECK FLEXURAL CONCRETE STRESSES

5.1 ALLOWABLE CONCRETE STRESSES

FLEXURAL TENSILE STRESS $f_{t,allow} = -6\,(f_c')^{0.5} =$ -0.329 ksi

FLEXURAL COMPRESSIVE STRESS $f_{c,allow} = -0.45\,f_c' =$ 1.350 ksi

5.2 TOP STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = -M_L / S_t =$ -0.116 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

TOP STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = -M_B / S_t =$ -0.125 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

5.3 BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = M_L / S_b =$ 0.413 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = M_B / S_b =$ 0.474 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

5.4 TOP STRESS, FOR EDGE LIFT MOMENT, AT L DIRECTION

$f = -M_L / S_b =$ -0.219 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

TOP STRESS, FOR EDGE LIFT MOMENT, AT B DIRECTION

$f = -M_B / S_b =$ -0.301 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

5.5 BOTTOM STRESS, FOR EDGE LIFT MOMENT, AT L DIRECTION

$f = M_L / S_t =$ 0.061 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

BOTTOM STRESS, FOR EDGE LIFT MOMENT, AT B DIRECTION

$f = M_B / S_t =$ 0.079 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

6. CHECK DIFFERENTIAL DEFLECTIONS

6.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$\beta = (E_c\, I / E_s)^{1/4} / 12 =$ 12.624 ft

Where $E_c = (0.5)\,57000\,(f_c')^{0.5} =$ 1561009 psi

$E_s =$ 1000 psi, soil

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$\beta = (E_c\, I / E_s)^{1/4} / 12 =$ 13.208 ft

Where $E_c = (0.5)\,57000\,(f_c')^{0.5} =$ 1561009 psi

$E_s =$ 1000 psi, soil

6.2 ALLOWABLE DIFFERENTIAL DEFLECTION AT L DIRECTION

FOR CENTER LIFT

$\Delta_{allow} = 12\,\mathrm{MIN}(L, 6\beta) / C_\Delta =$ 2.64 in

Where $C_\Delta =$ 360

FOR EDGE LIFT

$\Delta_{allow} = 12\,\mathrm{MIN}(L, 6\beta) / C_\Delta =$ 1.32 in

Where $C_\Delta =$ 720

ALLOWABLE DIFFERENTIAL DEFLECTION AT B DIRECTION

FOR CENTER LIFT

$\Delta_{allow} = 12\,\mathrm{MIN}(B, 6\beta) / C_\Delta =$ 2.52 in

Where $C_\Delta =$ 360

FOR EDGE LIFT

$\Delta_{allow} = 12\,\mathrm{MIN}(B, 6\beta) / C_\Delta =$ 1.26 in

Where $C_\Delta =$ 720

6.3 EXPECTED DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

FOR CENTER LIFT, AT L DIRECTION

$\Delta_0 = (y_m\, L)^{0.205}\, S_B^{1.059}\, P^{0.523}\, e_m^{1.296} / (380\, h^{1.214}) =$ 0.80 in < Δ_{allow}

[Satisfactory]

FOR EDGE LIFT, AT L DIRECTION

$\Delta_0 = L^{0.35}\, y_m^{0.76}\, S_B^{0.88}\, e_m^{0.74} / (15.9\, h^{0.85}\, P^{0.01}) =$ 0.58 in < Δ_{allow}

[Satisfactory]

EXPECTED DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

FOR CENTER LIFT, AT B DIRECTION

$\Delta_0 = (y_m\, B)^{0.205}\, S_L^{1.059}\, P^{0.523}\, e_m^{1.296} / (380\, h^{1.214}) =$ 0.76 in < Δ_{allow}

[Satisfactory]

FOR EDGE LIFT, AT B DIRECTION

$\Delta_0 = B^{0.35}\, y_m^{0.76}\, S_L^{0.88}\, e_m^{0.74} / (15.9\, h^{0.85}\, P^{0.01}) =$ 0.53 in < Δ_{allow}

[Satisfactory]

7. CHECK SHEAR CAPACITY

7.1 APPLIED SERVICE LOAD SHEAR AT L DIRECTION

FOR CENTER LIFT

$V_L = L^{0.09}\, S_B^{0.71}\, h^{0.43}\, P^{0.44}\, y_m^{0.16}\, e_m^{0.93} / 1940 =$ 1.786 kips/ft

FOR EDGE LIFT

$V_L = L^{0.07}\, h^{0.4}\, P^{0.03}\, y_m^{0.67}\, e_m^{0.16} / (3.0\, S_B^{0.015}) =$ 1.084 kips/ft

APPLIED SERVICE LOAD SHEAR AT B DIRECTION

FOR CENTER LIFT

$V_B = B^{0.19}\, S_L^{0.45}\, h^{0.20}\, P^{0.54}\, y_m^{0.04}\, e_m^{0.97} / 1350 =$ 1.327 kips/ft

FOR EDGE LIFT

$V_B = B^{0.07}\, h^{0.4}\, P^{0.03}\, y_m^{0.67}\, e_m^{0.16} / (3.0\, S_L^{0.015}) =$ 1.063 kips/ft

7.2 ALLOWABLE CONCRETE SHEAR STRESS, AT L DIRECTION

$v_c = 2\,(f_c')^{0.5} =$ 0.110 ksi

ALLOWABLE CONCRETE SHEAR STRESS, AT B DIRECTION

$v_c = 2\,(f_c')^{0.5} =$ 0.110 ksi

7.3 SHEAR STRESS OF RIBBED FOUNDATION, AT L DIRECTION

FOR CENTER LIFT

$v = V\,B / (n\,h\,b) =$ 0.078 ksi < v_c

[Satisfactory]

FOR EDGE LIFT

$v = V\,B / (n\,h\,b) =$ 0.047 ksi < v_c

[Satisfactory]

SHEAR STRESS OF RIBBED FOUNDATION, AT B DIRECTION

FOR CENTER LIFT

$v = V\,L / (n\,h\,b) =$ 0.065 ksi < v_c

[Satisfactory]

FOR EDGE LIFT

$v = V\,L / (n\,h\,b) =$ 0.052 ksi < v_c

[Satisfactory]

8. CHECK SOIL BEARING

8.1 APPLIED LOADING

SLAB WEIGHT	150 L B t	=	1281250	lbs
ADDED DL	DL L B	=	1025000	lbs
LIVE LOAD	LL L B	=	2562500	lbs
BEAM WEIGHT	150 (h-t) b (Total Length)	=	708146	lbs
PERIMETER LOAD	P (2L + 2B)	=	156060	lbs

BEAM BEARING AREA (b)(Total Length) = 3028.333333 ft^2

SOIL PRESSURE q = Total Load / THE AREA = 1893 psf < q_{allow}

[Satisfactory]

9. CHECK SLAB STRESS DUE TO LOAD-BEARING PARTITIONS

9.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$M_{max} = P_b\, \beta / 4 =$ 2.03 ft-kips / ft

Where $\beta = \mathrm{MIN}[(E_c\, t^3 / 3\, k_s)^{0.25}, S_B] =$ 30 ft

$k_s =$ 4 lb / in^3

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$M_{max} = P_b\, \beta / 4 =$ 2.03 ft-kips / ft

Where $\beta = \mathrm{MIN}[(E_c\, t^3 / 3\, k_s)^{0.25}, S_L] =$ 30 ft

$k_s =$ 4 lb / in^3

9.2 TENSILE STRESS AT L DIRECTION

$f = -M_{max} / 2\, t^2 =$ -0.041 ksi

> $f_{t,allow}$ **[Satisfactory]**

TENSILE STRESS AT B DIRECTION

$f = -M_{max} / 2\, t^2 =$ -0.041 ksi

> $f_{t,allow}$ **[Satisfactory]**

Technical References:

1. "Design of Slabs on Grade, ACI Committee 360R-06", American Concrete Institute, 2006.
2. "Design and Construction of Post-Tensioned Slab-on-Ground, Second Edition", The Post-Tensioning Institute, 2004.
3. "1997 Uniform Building Code, Volume 2", International Conference of Building Officials, 1997.

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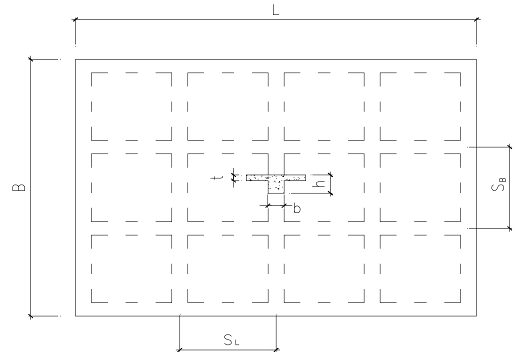
JOB NO. :	DATE :	REVIEW BY :	

Design of Conventional Slabs on Compressible Soil Grade Based on ACI 360

1. INPUT DATA & DESIGN SUMMARY



1.1 SOILS PROPERTIES

ALLOWABLE SOIL-BEARING PRESSURE	q_{allow}	=	2000	psf
EXPECTED SETTLEMENT BY GEOTECHNICAL ENR	δ	=	0.75	in

1.2 STRUCTURAL DATA AND MATERIALS PROPERTIES

SLAB LENGTH	L	=	164	ft
SLAB WIDTH	B	=	125	ft
SLAB THICKNESS	t	=	5	in
PERIMETER LOADING	P	=	270	plf
MAX BEARING LOADING ON THE SLAB	P_b	=	270	plf
ADDED DEAD LOAD	DL	=	50	psf
LIVE LOAD	LL	=	125	psf
AVERAGE STIFFENING BEAM SPACING, L DIRECTION	S_L	=	30	ft
AVERAGE STIFFENING BEAM SPACING, B DIRECTION	S_B	=	30	ft
STIFFENING BEAM DEPTH	h	=	24	in
STIFFENING BEAM WIDTH	b	=	20	in
CONCRETE STRENGTH	f'$_c$	=	3	ksi

REINFORCEMENT IN THE BOTTOM OF STIFFENING BEAM 2 # 6

SLAB REINFORCEMENT # 4 @ 18 in o.c., with 1.5 in clear from top of slab, each way.

THE DESIGN IS ADEQUATE.

2. DETERMINE SECTION PROPERTIES

L DIRECTION

n =	6		I =	338782	in^4
A_s =	17	in^2	y_b =	18.80	in
E_s / E_c =	18.58		S_t =	65100	in^3
CGS =	21.75	in	S_b =	18024	in^3
A =	10090	in^2			

B DIRECTION

n =	7		I =	406345	in^4
A_s =	22	in^2	y_b =	19.05	in
E_s / E_c =	18.58		S_t =	82096	in^3
CGS =	22.25	in	S_b =	21330	in^3
A =	12906	in^2			

3. CALCULATE MAXIMUM APPLIED SERVICE MOMENTS

L DIRECTION

$M_{csL} = (\delta / \Delta_{nsL})^{0.5}\, M_{nsL}$ = 0.83 ft-kips / ft

Where $M_{nsL} = h^{1.35}\, S_B^{0.36} / 80\, L^{0.12}\, P^{0.10}$ = 0.96 ft-kips / ft

$\Delta_{nsL} = L^{1.28}\, S_B^{0.80} / 133\, h^{0.28}\, P^{0.62}$ = 1.00

B DIRECTION

$M_{csS} = M_{csL}\, (970\text{-}h) / 880$ = 0.90 ft-kips / ft

$\Delta_{nsL} = L^{1.28}\, S_B^{0.80} / 133\, h^{0.28}\, P^{0.62}$ = 0.70

4. CHECK FLEXURAL CONCRETE STRESSES

4.1 ALLOWABLE CONCRETE STRESSES

FLEXURAL TENSILE STRESS $f_{t,allow} = - 6\, (f_c')^{0.5}$ = -0.329 ksi

FLEXURAL COMPRESSIVE STRESS $f_{c,allow} = - 0.45\, f_c'$ = 1.350 ksi

4.2 TOP STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = M_L / S_t$ = 0.019 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

TOP STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = M_B / S_t$ = 0.021 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

4.3 BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = - M_L / S_b$ = -0.069 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = - M_B / S_b$ = -0.083 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**; f < $f_{c,allow}$ **[Satisfactory]**

5. CHECK DIFFERENTIAL DEFLECTIONS

5.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$\beta = (E_c\, I\, \Delta_{nsL} / E_s\, \delta)^{1/4} / 12$ = 13.571 ft

Where E_c = (0.5) 57000 $(f_c')^{0.5}$ = 1561009 psi

E_s = 1000 psi, soil

I = 338782 in^4

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$\beta = (E_c\, I\, \Delta_{nsB} / E_s\, \delta)^{1/4} / 12$ = 13.020 ft

Where E_c = (0.5) 57000 $(f_c')^{0.5}$ = 1561009 psi

E_s = 1000 psi, soil

I = 406345 in^4

5.2 ALLOWABLE DIFFERENTIAL DEFLECTION AT L DIRECTION

$\Delta_{allow} = 12\, \text{MIN}(L, 6\beta) / C_\Delta$ = 0.51 in

Where C_Δ = 1920

ALLOWABLE DIFFERENTIAL DEFLECTION AT B DIRECTION

$\Delta_{allow} = 12\, \text{MIN}(B, 6\beta) / C_\Delta$ = 0.49 in

Where C_Δ = 1920

5.3 DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

$\Delta_{cs} = \delta\, e_n^{0(1.78 - 0.103 h - 1.65E\text{-}03 P + 3.95E\text{-}07 P P)}$ = 0.25 in < Δ_{allow}

[Satisfactory]

DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

$\Delta_{cs} = \delta\, e_n^{0(1.78 - 0.103 h - 1.65E\text{-}03 P + 3.95E\text{-}07 P P)}$ = 0.25 in < Δ_{allow}

[Satisfactory]

6. CHECK SHEAR CAPACITY

6.1 APPLIED SERVICE LOAD SHEAR AT L DIRECTION

$V_{csL} = (\delta / \Delta_{nsL})^{0.30}\, V_{nsL}$ = 0.260 kips/ft

Where $V_{nsL} = h^{0.90}\, (PS_B)^{0.30} / 550\, L^{0.10}$ = 0.284 kips/ft

APPLIED SERVICE LOAD SHEAR AT B DIRECTION

$V_{csS} = V_{csL}\, (116\text{-}h) / 94$ = 0.255 kips/ft

6.2 ALLOWABLE CONCRETE SHEAR STRESS, AT L DIRECTION

$v_c = 2\, (f_c')^{0.5}$ = 0.110 ksi

ALLOWABLE CONCRETE SHEAR STRESS, AT B DIRECTION

$v_c = 2\, (f_c')^{0.5}$ = 0.110 ksi

6.3 SHEAR STRESS OF RIBBED FOUNDATION, AT L DIRECTION

$v = V\, B / (n\, h\, b)$ = 0.010 ksi < v_c

[Satisfactory]

SHEAR STRESS OF RIBBED FOUNDATION, AT B DIRECTION

$v = V\, L / (n\, h\, b)$ = 0.015 ksi < v_c

[Satisfactory]

(cont'd)

7. CHECK SOIL BEARING

7.1 APPLIED LOADING

SLAB WEIGHT	150 L B t	=	1281250	lbs
ADDED DL	DL L B	=	1025000	lbs
LIVE LOAD	LL L B	=	2562500	lbs
BEAM WEIGHT	150 (h-t) b (Total Length)	=	708146	lbs
PERIMETER LOAD	P (2L + 2B)	=	156060	lbs

BEAM BEARING AREA	(b)(Total Length) =	2982	ft^2
SOIL PRESSURE	q = Total Load / THE AREA =	1923	psf
		<	q_{allow}
		[Satisfactory]	

8. CHECK SLAB STRESS DUE TO LOAD-BEARING PARTITIONS

8.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$M_{max} = P_b \beta / 4 =$ 2.03 ft-kips / ft

Where $\beta = MIN[(E_c h^3 / 3 k_s)^{0.25}, S_B] =$ 30.000 ft

$k_s =$ 4 lb / in^3

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$M_{max} = P_b \beta / 4 =$ 2.03 ft-kips / ft

Where $\beta = MIN[(E_c h^3 / 3 k_s)^{0.25}, S_B] =$ 30.000 ft

$k_s =$ 4 lb / in^3

8.2 TENSILE STRESS AT L DIRECTION

$f = - M_{max} / 2 t^2 =$ -0.041 ksi

> $f_{t,allow}$ **[Satisfactory]**

TENSILE STRESS AT B DIRECTION

$f = - M_{max} / 2 t^2 =$ -0.041 ksi

> $f_{t,allow}$ **[Satisfactory]**

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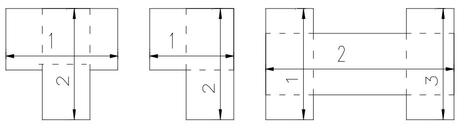
Design of PT Slabs on Expansive Soil Ground Based on Standards of PTI 3rd Edition

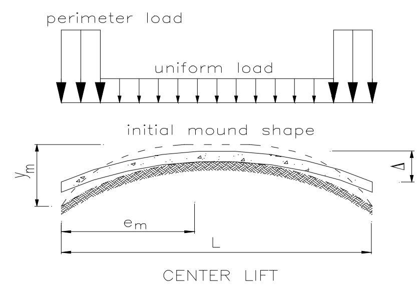
1. DESIGN METHODS

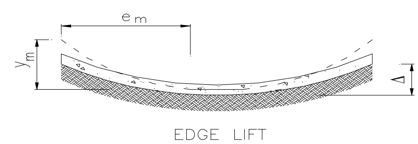
1.1 MAKE SURE THAT THE FOUNDATION PLAN SATISFIES THE SHAPE FACTOR REQUIREMENT:
SF = (ENTIRE FOUNDATION PERIMETER) / (ENTIRE FOUNDATION AREA) ≤ 24 , (Sec. 4.5.1)

1.2 DIVIDE AN IRREGULAR FOUNDATION PLAN INTO OVERLAPPING RECTANGLES AND USING THIS SPREADSHEET DESIGN EACH RECTANGULAR SECTION SEPARATELY.

1.3 ONCE THE RIBBED FOUNDATION HAS BEEN DESIGNED TO SATISFY MOMENT, SHEAR, AND DIFFERENTIAL DEFLECTION REQUIREMENTS, IT MAY BE CONVERTED TO AN EQUIVALENT UNIFORM THICKNESS FOUNDATION.







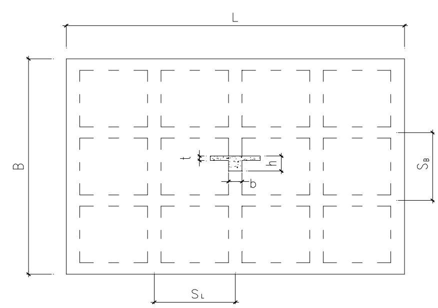
2. INPUT DATA & DESIGN SUMMARY

2.1 SOILS PROPERTIES (FROM SOIL REPORT / GEOTECHNICAL INVESTIGATION)

ALLOWABLE SOIL-BEARING PRESSURE	q_{allow}	=	2000	psf
EDGE MOISTURE VARIATION DISTANCE	e_m	=	9	ft, for center lift
		=	5.2	ft, for edge lift
DIFFERENTIAL SOIL MOVEMENT	y_m	=	0.07	in, for center lift
		=	0.46	in, for edge lift
SLAB-SUBGRADE FRICTION COEFFICIENT	μ	=	0.75	

2.2 STRUCTURAL DATA AND MATERIALS PROPERTIES

SLAB LENGTH	L	=	42	ft
SLAB WIDTH	B	=	24	ft
SLAB THICKNESS	t	=	4	in
PERIMETER LOADING	P	=	695	plf
MAX BEARING LOADING ON THE SLAB	P_b	=	2700	plf
ADDED DEAD LOAD	DL	=	15	psf
LIVE LOAD	LL	=	40	psf
AVERAGE STIFFENING BEAM SPACING, L DIRECTION	S_L	=	12	ft
AVERAGE STIFFENING BEAM SPACING, B DIRECTION	S_B	=	12	ft
STIFFENING BEAM DEPTH	h	=	14.5	in
STIFFENING BEAM WIDTH	b	=	12	in
CONCRETE STRENGTH	f'_c	=	3	ksi
SLAB PRESTRESSING TENDONS, L DIRECTION	5	tendons w/	0.153	in^2 at each tendon.
SLAB PRESTRESSING TENDONS, B DIRECTION	9	tendons w/	0.153	in^2 at each tendon.
TENDON IN THE BOTTOM OF EACH BEAM	1	tendons w/	0.153	in^2 (only for edge lift governing required)
EFFECTIVE PRESTRESS AFTER ALL LOSSES EXCEPT SG	f_e	=	174	ksi
CONVERT UNIFORM THICKNESS (Sec. 6.12) ?	No			



THE DESIGN IS ADEQUATE.
SUGGESTED RATIO OF EXPECTED ELONGATION IS 0.00777

3. DETERMINE SECTION PROPERTIES

L DIRECTION

n	=	3		y_b	=	10.71	in
A	=	1530	in^2	S_t	=	5267	in^3
I	=	19969	in^4	S_b	=	1865	in^3
CGS	=	9.03	in	e	=	-1.68	in

B DIRECTION

n	=	5		y_b	=	10.77	in
A	=	2646	in^2	S_t	=	9046	in^3
I	=	33706	in^4	S_b	=	3129	in^3
CGS	=	9.20	in	e	=	-1.58	in

4. CALCULATE MAXIMUM APPLIED SERVICE MOMENTS (Sec. 6.8)

4.1 CENTER LIFT MOMENT AT L DIRECTION

For e_m = 9 ft

$M_L = A_0 (B\, e_m^{1.238} + C)$ = 2.05 ft-kips / ft

Where $A_0 = (L^{0.013} S_B^{0.306} h^{0.688} P^{0.534} y_m^{0.193}) / 727$ = 0.383

$B = 1$, for $e_m < 5$
$B = \mathrm{MIN}[(y_m - 1) / 3, 1]$, for $e_m > 5$ = -0.31

$C = 0$, for $e_m < 5$
$C = \mathrm{MAX}\{[8 - (P - 613) / 255] (4 - y_m) / 3], 0\}$, for $e_m > 5$ = 10.06

For e_m = 5 ft (Sec. 4.3.2)

$M_L = A_0 (B\, e_m^{1.238} + C)$ = 2.81 ft-kips / ft

Where $A_0 = (L^{0.013} S_B^{0.306} h^{0.688} P^{0.534} y_m^{0.193}) / 727$ = 0.383

B = 1.00

C = 0.00

USE M_L = 2.81 ft-kips / ft

CENTER LIFT MOMENT AT B DIRECTION

For e_m = 9 ft

$M_B = (58 + e_m) M_L / 60$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1
= 2.29 ft-kips / ft

For e_m = 5 ft (Sec. 4.3.2)

$M_B = (58 + e_m) M_L / 60$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1
= 2.95 ft-kips / ft

USE M_B = 2.95 ft-kips / ft

4.2 EDGE LIFT MOMENT AT L DIRECTION

$M_L = S_B^{0.10} (h\, e_m)^{0.78} y_m^{0.66} / (7.2\, L^{0.0065} P^{0.04})$ = 2.33 ft-kips / ft

EDGE LIFT MOMENT AT B DIRECTION

$M_B = h^{0.35} (19 + e_m) M_L / 57.75$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1
= 2.49 ft-kips / ft

5. CHECK FLEXURAL CONCRETE STRESSES (Sec. 6.5)

5.1 ALLOWABLE CONCRETE STRESSES

FLEXURAL TENSILE STRESS	$f_{t,allow} = - 6 (f_c')^{0.5}$ =	-0.329	ksi
FLEXURAL COMPRESSIVE STRESS	$f_{c,allow} = - 0.45 f_c'$ =	1.350	ksi

(cont'd)

5.2 TOP STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = P_r / A - M_L / S_t + P_r e / S_t =$ -0.092 ksi

Where $P_r = P_e - SG =$ 182.71 kips

$P_e = f_e A_{ps} =$ 212.98 kips

$SG = W_{slab} \mu / 2000 =$ 30.27 kips

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

TOP STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = P_r / A - M_B / S_t + P_r e / S_t =$ -0.095 ksi

Where $P_r = P_e - SG =$ 342.44 kips

$P_e = f_e A_{ps} =$ 372.71 kips / ft

$SG = W_{slab} \mu / 2000 =$ 30.27 kips

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

5.3 BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = P_r / A + M_L / S_b - P_r e / S_b =$ 0.718 ksi

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = P_r / A + M_B / S_b - P_r e / S_b =$ 0.778 ksi

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

5.4 TOP STRESS, FOR EDGE LIFT MOMENT, AT L DIRECTION

$f = P_r / A - M_L / S_b - P_r e / S_b =$ -0.077 ksi

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

TOP STRESS, FOR EDGE LIFT MOMENT, AT B DIRECTION

$f = P_r / A - M_B / S_b - P_r e / S_b =$ -0.100 ksi

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

5.5 BOTTOM STRESS, FOR EDGE LIFT MOMENT, AT L DIRECTION

$f = P_r / A + M_L / S_t + P_r e / S_t =$ 0.189 ksi

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

BOTTOM STRESS, FOR EDGE LIFT MOMENT, AT B DIRECTION

$f = P_r / A + M_B / S_t + P_r e / S_t =$ 0.209 ksi

Then f > $f_{t,allow}$ [Satisfactory]

< $f_{c,allow}$ [Satisfactory]

6. CHECK MINIMUM FOUNDATION STIFFNESS (Sec. 6.10)

6.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$\beta = (E_{cr}\, I / E_S)^{1/4} / 12 =$ 6.323 ft

Where $E_{cr} = (0.5)\, 33\, w^{1.5} (f_c')^{0.5} =$ 1660280 psi

$E_S =$ 1000 psi

$w =$ 150 pcf

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$\beta = (E_{cr}\, I / E_S)^{1/4} / 12 =$ 7.208 ft

Where $E_{cr} = (0.5)\, 33\, w^{1.5} (f_c')^{0.5} =$ 1660280 psi

$E_S =$ 1000 psi

6.2 CHECK MINIMUM FOUNDATION STIFFNESS AT L DIRECTION

FOR CENTER LIFT

$12000\, M_L\, B\, C_\Delta\, z_L / E_{cr} =$ 6660 in^4 < I_L

Where C_Δ = 360 [Satisfactory]

$z_L = \min(L, 6\beta) =$ 37.94 ft

$I_L =$ 19969 in^4

FOR EDGE LIFT

$12000\, M_L\, B\, C_\Delta\, z_L / E_{cr} =$ 11060 in^4 < I_L

Where C_Δ = 720 [Satisfactory]

CHECK MINIMUM FOUNDATION STIFFNESS AT B DIRECTION

FOR CENTER LIFT

$12000\, M_B\, L\, C_\Delta\, z_B / E_{cr} =$ 7741 in^4 < I_B

Where C_Δ = 360 [Satisfactory]

$z_B = \min(B, 6\beta) =$ 24.00 ft

$I_B =$ 33706 in^4

FOR EDGE LIFT

$12000\, M_B\, L\, C_\Delta\, z_B / E_{cr} =$ 13082 in^4 < I_B

Where C_Δ = 720 [Satisfactory]

7. CHECK SHEAR CAPACITY (Sec. 6.11)

7.1 APPLIED SERVICE LOAD SHEAR AT L DIRECTION

FOR CENTER LIFT

$V_L = L^{0.09} S_B^{0.71} h^{0.45} P^{0.44} y_m^{0.16} e_m^{0.93} / 1940 =$ 1.260 kips/ft

FOR EDGE LIFT

$V_L = L^{0.07} h^{0.4} P^{0.03} y_m^{0.67} e_m^{0.16} / (3.0\, S_B^{0.015}) =$ 1.145 kips/ft

APPLIED SERVICE LOAD SHEAR AT B DIRECTION

FOR CENTER LIFT

$V_B = B^{0.19} S_L^{0.45} h^{0.20} P^{0.54} y_m^{0.04} e_m^{0.97} / 1350 =$ 1.836 kips/ft

FOR EDGE LIFT

$V_B = B^{0.07} h^{0.4} P^{0.03} y_m^{0.67} e_m^{0.16} / (3.0\, S_L^{0.015}) =$ 1.101 kips/ft

7.2 ALLOWABLE CONCRETE SHEAR STRESS, AT L DIRECTION

$v_c = 2.4\, (f_c')^{0.5} + 0.2\, f_p =$ 0.155 ksi

Where $f_p =$ 0.119 ksi > 50 psi

[Satisfactory]

ALLOWABLE CONCRETE SHEAR STRESS, AT B DIRECTION

$v_c = 2.4\, (f_c')^{0.5} + 0.2\, f_p =$ 0.157 ksi

Where $f_p =$ 0.129 ksi > 50 psi

[Satisfactory]

7.3 SHEAR STRESS OF RIBBED FOUNDATION, AT L DIRECTION

FOR CENTER LIFT

$v = V\, B / (n\, h\, b) =$ 0.058 ksi < v_c

[Satisfactory]

FOR EDGE LIFT

$v = V\, B / (n\, h\, b) =$ 0.053 ksi < v_c

[Satisfactory]

SHEAR STRESS OF RIBBED FOUNDATION, AT B DIRECTION

FOR CENTER LIFT

$v = V\, L / (n\, h\, b) =$ 0.089 ksi < v_c

[Satisfactory]

FOR EDGE LIFT

$v = V\, L / (n\, h\, b) =$ 0.053 ksi < v_c

[Satisfactory]

8. CHECK SOIL BEARING (Sec. 4.5)

8.1 APPLIED LOADING

SLAB WEIGHT	150 L B t	=	50400	lbs
ADDED DL	DL L B	=	15120	lbs
LIVE LOAD	LL L B	=	40320	lbs
BEAM WEIGHT	150 (h-t) b (Total Length)	=	30319	lbs
PERIMETER LOAD	P (2L + 2B)	=	91740	lbs

RIB BEARING AREA (Sec. 4.5.2.3) = 811.44 ft^2

SOIL PRESSURE q = Total Load / THE AREA = 281 psf < q_{allow}

[Satisfactory]

9. CHECK SLAB STRESS DUE TO LOAD-BEARING PARTITIONS (Sec. 6.14)

9.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$M_{max} = P_b\, \beta / 4 =$ 8.10 ft-kips / ft

Where $\beta = \mathrm{MIN}[(E_c\, t^3 / 3\, k_s)^{0.25}, S_B] =$ 12 ft

$k_s =$ 4 lb / in^3

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$M_{max} = P_b\, \beta / 4 =$ 8.10 ft-kips / ft

Where $\beta = \mathrm{MIN}[(E_c\, t^3 / 3\, k_s)^{0.25}, S_L] =$ 12 ft

$k_s =$ 4 lb / in^3

9.2 TENSILE STRESS AT L DIRECTION

$f = P_r / A - M_{max} / 2\, t^2 =$ -0.134 ksi

> $f_{t,allow}$ [Satisfactory]

TENSILE STRESS AT B DIRECTION

$f = P_r / A - M_{max} / 2\, t^2 =$ -0.124 ksi

> $f_{t,allow}$ [Satisfactory]

(cont'd)

10. CHECK CRACKED SECTION CAPACITY (Sec. 4.5.7 & 6.12)

10.1 CHECK CRACKED SECTION CAPACITY AT L DIRECTION

FOR CENTER LIFT

M_{cr} = F (h - 2" - 0.5a) =		130.6	ft-kips	**>**	0.9 M_L
Where	F =	133.11	kips	**[Satisfactory]**	
	a = F / 0.85 f_c' b =	1.45	in		
	0.9 M_L =	60.7	ft-kips, total		

FOR EDGE LIFT

M_{cr} = F (h - 3" - 0.5a) =		76.2	ft-kips	**>**	0.9 M_L
Where	F =	79.87	kips	**[Satisfactory]**	
	a = F / 0.85 f_c' b =	0.11	in		
	0.9 M_L =	50.4	ft-kips, total		

CHECK CRACKED SECTION CAPACITY AT B DIRECTION

FOR CENTER LIFT

M_{cr} = F (h - 2" - 0.5a) =		233.9	ft-kips	**>**	0.9 M_B
Where	F =	239.60	kips	**[Satisfactory]**	
	a = F / 0.85 f_c' b =	1.57	in		
	0.9 M_B =	111.6	ft-kips, total		

FOR EDGE LIFT

M_{cr} = F (h - 3" - 0.5a) =		127.0	ft-kips	**>**	0.9 M_B
Where	F =	133.11	kips	**[Satisfactory]**	
	a = F / 0.85 f_c' b =	0.10	in		
	0.9 M_B =	94.3	ft-kips, total		

11. CONVERT UNIFORM THICKNESS (Sec. 6.12)

H= MAX[$(I / L)^{1/3}$, $(I / B)^{1/3}$] = N/A (Does not apply for beam tendons)

12. SUGGEST RATIO OF EXPECTED ELONGATION

r = f_e / 0.8 E_{ps} = 7.77E-03

Where E_{ps} = 28000 ksi

Technical References:

1. "Design of Post-Tensioned Slab-on-Ground, Third Edition", Post-Tensioning Institute, 2004.
2. "Addendum No.1 to The 3RD Edition of Design of Post-Tensioned Slab-on-Ground", Post-Tensioning Institute, May 2007.
3. "Addendum No.2 to The 3RD Edition of Design of Post-Tensioned Slab-on-Ground", Post-Tensioning Institute, May 2008.

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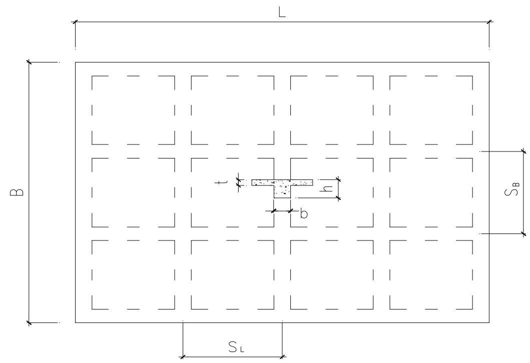
Design of PT Slabs on Compressible Soil Ground Based on Standards of PTI 2nd Edition

1. INPUT DATA & DESIGN SUMMARY



1.1 SOILS PROPERTIES

ALLOWABLE SOIL-BEARING PRESSURE	q_{allow}	=	1500	psf
EXPECTED SETTLEMENT BY GEOTECHNICAL ENR	δ	=	0.75	in
SLAB-SUBGRADE FRICTION COEFFICIENT	μ	=	0.75	

1.2 STRUCTURAL DATA AND MATERIALS PROPERTIES

SLAB LENGTH	L	=	40	ft
SLAB WIDTH	B	=	38	ft
SLAB THICKNESS	t	=	4	in
PERIMETER LOADING	P	=	840	plf
MAX BEARING LOADING ON THE SLAB	P_b	=	2700	plf
ADDED DEAD LOAD	DL	=	15	psf
LIVE LOAD	LL	=	40	psf
AVERAGE STIFFENING BEAM SPACING, L DIRECTION	S_L	=	13.333	ft
AVERAGE STIFFENING BEAM SPACING, B DIRECTION	S_B	=	12.667	ft
STIFFENING BEAM DEPTH	h	=	24	in
STIFFENING BEAM WIDTH	b	=	10	in
CONCRETE STRENGTH	f'_c	=	3	ksi

SLAB PRESTRESSING TENDONS, L DIRECTION	8	tendons w/	0.153	in^2 at each tendon.
SLAB PRESTRESSING TENDONS, B DIRECTION	8	tendons w/	0.153	in^2 at each tendon.
TENDON IN THE BOTTOM OF EACH BEAM	0	tendons w/	0	in^2
EFFECTIVE PRESTRESS AFTER ALL LOSSES EXCEPT SG	f_e	=	174	ksi

THE DESIGN IS ADEQUATE.
SUGGESTED RATIO OF EXPECTED ELONGATION IS 0.00777
CONVERTED UNIFORM THICKNESS IS 14.22 inch

2. DETERMINE SECTION PROPERTIES

L DIRECTION						B DIRECTION					
n =	4		y_b =	18.34	in	n =	4		y_b =	18.47	in
A =	2624	in^2	S_t =	19294	in^3	A =	2720	in^2	S_t =	19992	in^3
I =	109177	in^4	S_b =	5952	in^3	I =	110544	in^4	S_b =	5985	in^3
CGS =	22.00	in	e =	3.66	in	CGS =	22.00	in	e =	3.53	in

3. CALCULATE MAXIMUM APPLIED SERVICE MOMENTS

L DIRECTION

$M_{csL} = (\delta / \Delta_{nsL})^{0.5} M_{nsL} =$ 3.20 ft-kips / ft

Where $M_{nsL} = h^{1.35} S_B^{0.36} / 80\ L^{0.12} P^{0.10} =$ 0.75 ft-kips / ft

$\Delta_{nsL} = L^{1.28} S_B^{0.80} / 133\ h^{0.28} P^{0.62} =$ 0.04

B DIRECTION

$M_{csS} = M_{csL}\ (970\text{-}h) / 880 =$ 3.44 ft-kips / ft

$\Delta_{nsL} = L^{1.28} S_B^{0.80} / 133\ h^{0.28} P^{0.62} =$ 0.04

4. CHECK FLEXURAL CONCRETE STRESSES

4.1 ALLOWABLE CONCRETE STRESSES

FLEXURAL TENSILE STRESS $f_{t,allow} = -6\ (f_c')^{0.5} =$ -0.329 ksi

FLEXURAL COMPRESSIVE STRESS $f_{c,allow} = -0.45\ f_c' =$ 1.350 ksi

4.2 TOP STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = P_r / A + M_L / S_t + P_r\ e / S_t =$ 0.168 ksi

Where $P_r = P_e - SG =$ 161.14 kips

$P_e = f_e A_{ps} =$ 213 kips

$SG = W_{slab}\ \mu / 2000 =$ 51.83 kips

Then f > $f_{t,allow}$ **[Satisfactory]**
< $f_{c,allow}$ **[Satisfactory]**

TOP STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = P_r / A + M_B / S_t + P_r\ e / S_t =$ 0.170 ksi

Where $P_r = P_e - SG =$ 161.14 kips

$P_e = f_e A_{ps} =$ 213 kips

$SG = W_{slab}\ \mu / 2000 =$ 51.83 kips

Then f > $f_{t,allow}$ **[Satisfactory]**
< $f_{c,allow}$ **[Satisfactory]**

4.3 BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = P_r / A - M_L / S_b - P_r\ e / S_b =$ -0.283 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**
< $f_{c,allow}$ **[Satisfactory]**

BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = P_r / A - M_B / S_b - P_r\ e / S_b =$ -0.312 ksi

Then f > $f_{t,allow}$ **[Satisfactory]**
< $f_{c,allow}$ **[Satisfactory]**

5. CHECK DIFFERENTIAL DEFLECTIONS

5.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$\beta = (E_c\ I\ \Delta_{nsL} / E_s\ \delta)^{1/4} / 12 =$ 4.666 ft

Where $E_{cr} = (0.5)\ 33\ w^{1.5} (f_c')^{0.5} =$ 1660280 psi

$E_s =$ 1000 psi

I = 109177 in^4

w = 150 pcf

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$\beta = (E_c\ I\ \Delta_{nsB} / E_s\ \delta)^{1/4} / 12 =$ 4.652 ft

Where $E_{cr} = (0.5)\ 33\ w^{1.5} (f_c')^{0.5} =$ 1660280 psi

$E_s =$ 1000 psi

I = 110544 in^4

5.2 ALLOWABLE DIFFERENTIAL DEFLECTION AT L DIRECTION

$\Delta_{allow} = 12\ MIN(L, 6\beta) / C_\Delta =$ 0.17 in

Where C_Δ = 1920

ALLOWABLE DIFFERENTIAL DEFLECTION AT B DIRECTION

$\Delta_{allow} = 12\ MIN(B, 6\beta) / C_\Delta =$ 0.17 in

Where C_Δ = 1920

5.3 DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

$\Delta_{cs} = \delta\ e_n^{0(1.78 - 0.103\ h - 1.65E\text{-}03\ P + 3.95E\text{-}07\ P\ P)} =$ 0.12 in

< Δ_{allow}

[Satisfactory]

DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

$\Delta_{cs} = \delta\ e_n^{0(1.78 - 0.103\ h - 1.65E\text{-}03\ P + 3.95E\text{-}07\ P\ P)} =$ 0.12 in

< Δ_{allow}

[Satisfactory]

6. CHECK SHEAR CAPACITY

6.1 APPLIED SERVICE LOAD SHEAR AT L DIRECTION

$V_{csL} = (\delta / \Delta_{nsL})^{0.30} V_{nsL} =$ 0.850 kips/ft

Where $V_{nsL} = h^{0.90} (PS_B)^{0.30} / 550 L^{0.10} =$ 0.355 kips/ft

APPLIED SERVICE LOAD SHEAR AT B DIRECTION

$V_{csS} = V_{csL} (116\text{-}h) / 94 =$ 0.832 kips/ft

6.2 ALLOWABLE CONCRETE SHEAR STRESS, AT L DIRECTION

$v_c = 2.4 (f_c')^{0.5} + 0.2 f_p =$ 0.144 ksi

Where $f_p =$ 0.061 ksi > 50 psi

[Satisfactory]

ALLOWABLE CONCRETE SHEAR STRESS, AT B DIRECTION

$v_c = 2.4 (f_c')^{0.5} + 0.2 f_p =$ 0.143 ksi

Where $f_p =$ 0.059 ksi > 50 psi

[Satisfactory]

6.3 SHEAR STRESS OF RIBBED FOUNDATION, AT L DIRECTION

$v = V B / (n h b) =$ 0.034 ksi < v_c

[Satisfactory]

SHEAR STRESS OF RIBBED FOUNDATION, AT B DIRECTION

$v = V L / (n h b) =$ 0.035 ksi < v_c

[Satisfactory]

7. CHECK SOIL BEARING

7.1 APPLIED LOADING

SLAB WEIGHT	150 L B t	=	76000	lbs
ADDED DL	DL L B	=	22800	lbs
LIVE LOAD	LL L B	=	60800	lbs
BEAM WEIGHT	150 (h-t) b (Total Length)	=	62222	lbs
PERIMETER LOAD	P (2L + 2B)	=	131040	lbs

BEAM BEARING AREA	(b)(Total Length) =	249	ft^2
SOIL PRESSURE	q = Total Load / THE AREA =	1418	psf
		<	q_{allow}
		[Satisfactory]	

8. CHECK SLAB STRESS DUE TO LOAD-BEARING PARTITIONS

8.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$M_{max} = P_b \beta / 4 =$ 8.55 ft-kips / ft

Where $\beta = MIN[(E_c h^3 / 3 k_s)^{0.25}, S_B] =$ 12.667 ft

$k_s =$ 4 lb / in^3

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$M_{max} = P_b \beta / 4 =$ 9.00 ft-kips / ft

Where $\beta = MIN[(E_c h^3 / 3 k_s)^{0.25}, S_B] =$ 13.333 ft

$k_s =$ 4 lb / in^3

8.2 TENSILE STRESS AT L DIRECTION

$f = P_r / A - M_{max} / 2 t^2 =$ -0.206 ksi

> $f_{t,allow}$ **[Satisfactory]**

TENSILE STRESS AT B DIRECTION

$f = P_r / A - M_{max} / 2 t^2 =$ -0.206 ksi

> $f_{t,allow}$ **[Satisfactory]**

9. CONVERT UNIFORM THICKNESS

$H = MAX[(I / L)^{1/3}, (I / B)^{1/3}] =$ 14.22 in

10. SUGGEST RATIO OF EXPECTED ELONGATION

$r = f_e / 0.8 E_{ps} =$ 7.77E-03

Where $E_{ps} =$ 28000 ksi

Technical References:

1. "Design of Post-Tensioned Slab-on-Ground, Second Edition", Post-Tensioning Institute, 1996.
2. "1997 Uniform Building Code, Volume 2, Chapter 18", International Conference of Building Officials, 1997.

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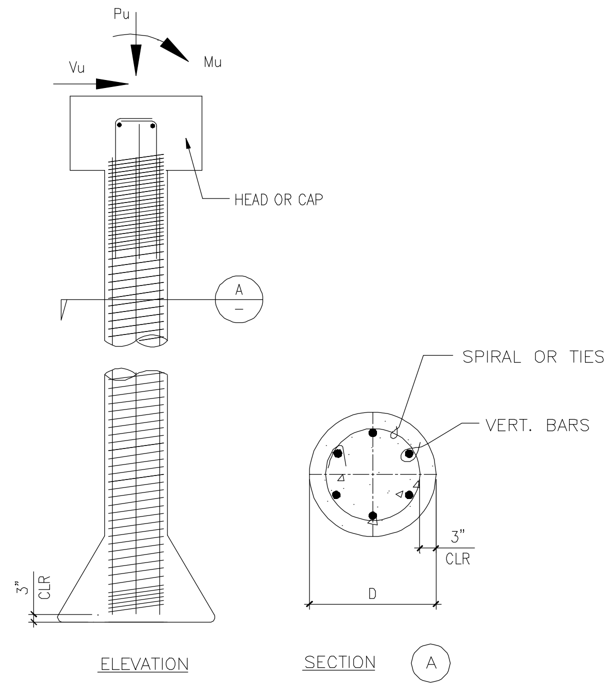
Concrete Pier (Isolated Deep Foundation) Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PIER DIAMETER	D	=	24	in
PIER LENGTH	L	=	10	ft
FACTORED AXIAL LOAD	P_u	=	100	k
FACTORED MOMENT LOAD	M_u	=	200	ft-k
FACTORED SHEAR LOAD	V_u	=	20	k
PIER VERT. REINF.	8	#	7	
SEISMIC DESIGN (ACI 21.12.4) ?			Yes	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	# 4	@	6	in o.c.

(spacing 3.0 in o.c. at top end of 10.0 ft.)
(IBC 09 1810.3.9 & ACI 21.12.4)



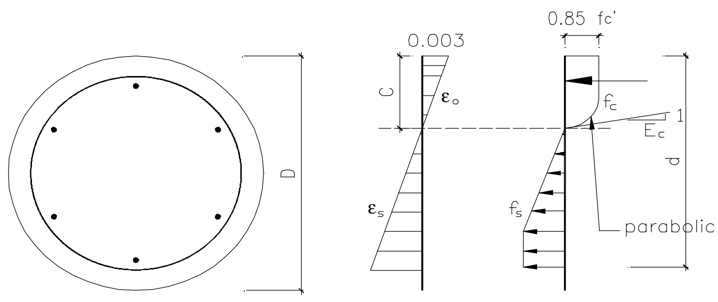
THE PIER DESIGN IS ADEQUATE.

ANALYSIS

CHECK PIER LIMITATIONS

f_c' =	3	ksi	>	2.5	ksi
		[Satisfactory]		(IBC 09 Table 1808.8.1)	
D =	24	in	>	MAX(L / 12 , 24 in)	
		[Satisfactory]		(IBC 09 1810.2.2)	

CHECK FLEXURAL & AXIAL CAPACITY



STRAIN DIAGRAM FORCE DIAGRAM

$$\varepsilon_o = \frac{2\left(0.85 f_c'\right)}{E_c} \;,\; E_c = 57\sqrt{f_c'} \;,\; E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f_c' \left[2\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right) - \left(\dfrac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] , & for \;\; 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f_c' \;,\; for \;\; \varepsilon_c \geq \varepsilon_o \end{cases}$$

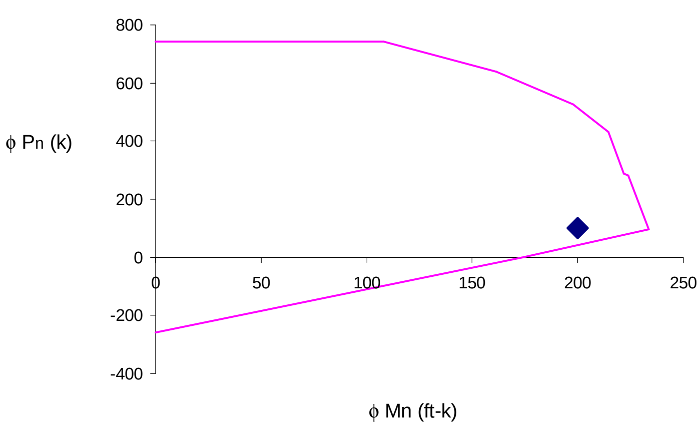
$$f_s = \begin{cases} \varepsilon_s E_s \;,\; for \;\; \varepsilon_s \leq \varepsilon_y \\ f_y \;,\; for \;\; \varepsilon_s > \varepsilon_y \end{cases}$$

$\phi P_{max} = F\ \phi\, [\, 0.85\, f_c' \,(A_g - A_{st}) + f_y A_{st}]$ = 743.26 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.2)

where F = 0.8 , ACI 318-08, Sec. 10.3.6.1 or 10.3.6.2

ϕ = 0.65 (ACI 318-08, Sec.9.3.2.2) > Pu **[Satisfactory]**

A_g = 452 in^2. A_{st} = 4.80 in^2.



	ϕ Pn (kips)	ϕ Mn (ft-kips)
AT COMPRESSION ONLY	743	0
AT MAXIMUM LOAD	743	108
AT 0 % TENSION	639	161
AT 25 % TENSION	527	198
AT 50 % TENSION	432	215
AT ε_t = 0.002	288	222
AT BALANCED CONDITION	282	224
AT ε_t = 0.005	96	234
AT FLEXURE ONLY	0	174
AT TENSION ONLY	-259	0

(cont'd)

$a = C_b\beta_1$ = 10 in (at balanced strain condition, ACI 10.3.2)

ϕ = 0.75 + (ε_t - 0.002) (50), for Spiral; 0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-08, Fig. R9.3.2)

where C_b = d ε_c / (ε_c + ε_s) = 12 in ε_t = 0.002069 ε_c = 0.003

d = 20.1 in, (ACI 7.7.1) β_1 = 0.85 (ACI 318-08, Sec. 10.2.7.3)

$\phi M_n = 0.9 M_n$ = 174 ft-kips @ P_n = 0, (ACI 318-08, Sec. 9.3.2) ,& $\varepsilon_{t,max}$ = 0.004, (ACI 318-08, Sec. 10.3.5)

ϕM_n = 234 ft-kips @ P_u = 100 kips > M_u **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.011

ρ_{min} = 0.005 (IBC 09, 1810.3.9.4.2) **[Satisfactory]**

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c)$ = 86 kips, (ACI 318-08 Sec. 11.1.1)

> V_u **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3)

A_0 = 316 in^2. A_v = 0.40 in^2. f_y = 60 ksi

V_c = $2 (f_c')^{0.5} A_0$ = 34.6 kips, (ACI 318-08 Sec. 11.2.1, 11.2.1.3)

V_s = MIN (d f_y A_v / s , 8 $(f_c')^{0.5} A_0$) = 80.3 kips, (ACI 318-08 Sec. 11.4.7.2 & 11.4.7.9)

s_{max} = 10.5 (IBC 09 1810.3.9.4.2) s_{provd} = 6 in

s_{min} = 1 **[Satisfactory]**

ρ_s = 0.12 f_c' / f_{yt} = 0.006 < $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-08 Sec. 21.12.4.4 & 21.6.4.1)

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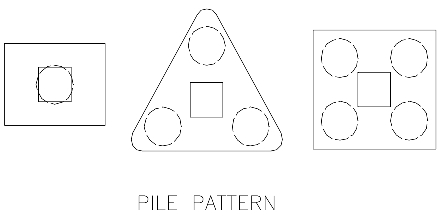
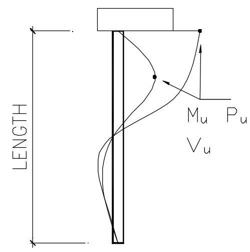
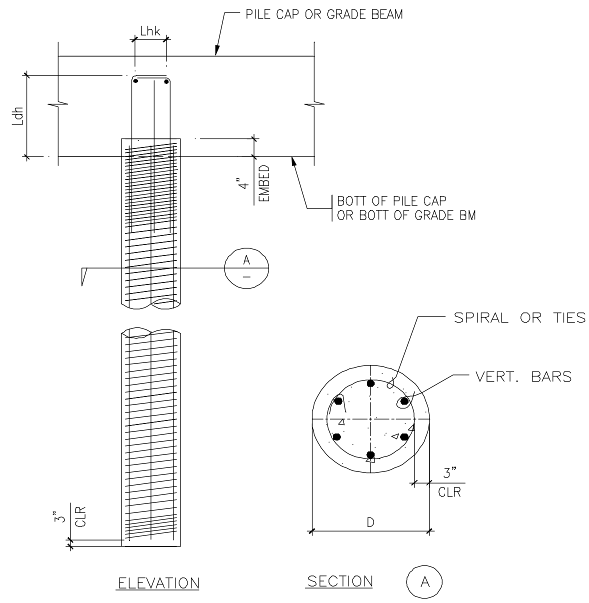
Drilled Cast-in-place Pile Design Based on ACI 318-08

DESIGN CRITERIA

1. ASSUME FIX HEAD CONDITION IF L_{dh} & L_{hk} COMPLY WITH THE TENSION DEVELOPMENT. OTHERWISE PINNED AT TOP.
2. FROM PILE CAP BALANCED LOADS & REACTIONS, DETERMINE MAX SECTION FORCES OF SINGLE PILE, P_u, M_u, & V_u.







INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	4	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	24	in
PILE LENGTH	L	=	35	ft
FACTORED AXIAL LOAD	P_u	=	100	k
FACTORED MOMENT LOAD	M_u	=	200	ft-k
FACTORED SHEAR LOAD	V_u	=	20	k
PILE VERT. REINF.	8	#	7	
SEISMIC DESIGN (ACI 21.12.4) ?			Yes	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	# 4	@	6	in o.c.

(spacing 3.0 in o.c. at top end of 10.0 ft.)
(IBC 09 1810.3.9 & ACI 21.12.4)

(L_{dh} = 9 in & L_{hk} = 14 in)

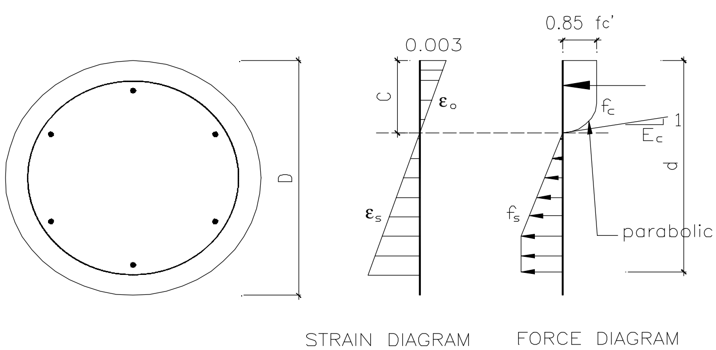
THE PILE DESIGN IS ADEQUATE.

ANALYSIS

CHECK PILE LIMITATIONS

f_c' =	4	ksi	>	4	ksi	**[Satisfactory]**	(IBC 09 Table 1808.8.1)
D =	24	in	>	MAX(L / 30 , 12 in)		**[Satisfactory]**	(IBC 09 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY



$$\varepsilon_o = \frac{2\left(0.85 f_c^{'}\right)}{E_c} \ , \ E_c = 57\sqrt{f_c^{'}} \ , \ E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f_c^{'} \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] \ , \ for \ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f_c^{'} \ , \ for \ \ \varepsilon_c \geq \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s \ , \ for \ \ \varepsilon_s \leq \varepsilon_y \\ f_y \ , \ for \ \ \varepsilon_s > \varepsilon_y \end{cases}$$

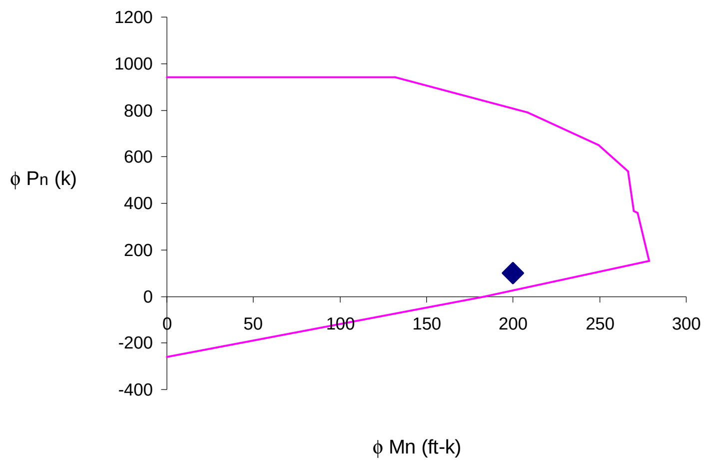
$\phi P_{max} = F \ \phi \ [\ 0.85\ f_c'\ (A_g - A_{st}) + f_y\ A_{st}]$ = 941.1 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.2)

where F = 0.8 , ACI 318-08, Sec. 10.3.6.1 or 10.3.6.2

ϕ = 0.65 (ACI 318-08, Sec.9.3.2.2) **>** Pu **[Satisfactory]**

A_g = 452 in^2. A_{st} = 4.80 in^2.

(cont'd)



	ϕ Pn (kips)	ϕ Mn (ft-kips)
AT COMPRESSION ONLY	941	0
AT MAXIMUM LOAD	941	132
AT 0 % TENSION	789	208
AT 25 % TENSION	651	250
AT 50 % TENSION	536	267
AT $\varepsilon_t = 0.002$	367	270
AT BALANCED CONDITION	360	272
AT $\varepsilon_t = 0.005$	153	279
AT FLEXURE ONLY	0	183
AT TENSION ONLY	-259	0

$a = C_b\beta_1$ = 10 in (at balanced strain condition, ACI 10.3.2)

ϕ = $0.75 + (\varepsilon_t - 0.002)(50)$, for Spiral; $0.65 + (\varepsilon_t - 0.002)(250/3)$, for Ties = 0.656 (ACI 318-08, Fig. R9.3.2)

where $C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 12 in ε_t = 0.002069 ε_c = 0.003

d = 20.1 in, (ACI 7.7.1) β_1 = 0.85 (ACI 318-08, Sec. 10.2.7.3)

$\phi M_n = 0.9 M_n$ = 183 ft-kips @ Pn = 0, (ACI 318-08, Sec. 9.3.2) ,& et,max = 0.004, (ACI 318-08, Sec. 10.3.5)

ϕM_n = 246 ft-kips @ P_u = 100 kips > Mu **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.011

ρ_{min} = 0.005 (IBC 09 1810.3.9.4.2) **[Satisfactory]**

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c)$ = 90 kips, (ACI 318-08 Sec. 11.1.1)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3)

A_0 = 316 in^2. A_v = 0.40 in^2. f_y = 60 ksi

V_c = $2 (f_c')^{0.5} A_0$ = 40.0 kips, (ACI 318-08 Sec. 11.2.1, 11.2.1.3)

V_s = MIN $(d\, f_y A_v / s\ ,\ 8 (f_c')^{0.5} A_0)$ = 80.3 kips, (ACI 318-08 Sec. 11.4.7.2 & 11.4.7.9)

s_{max} = 10.5 (IBC 09 1810.3.9.4.2) s_{provd} = 6 in

s_{min} = 1 **[Satisfactory]**

ρ_s = $0.12 f_c' / f_{yt}$ = 0.008 < $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-08 Sec. 21.12.4.4 & 21.6.4.1)

DETERMINE FIX HEAD CONDITION

$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f_c'}}\ ,\ 8d_b\ ,\ 6\ in\right) = 11\ d_b = 9\ in$$

(ACI 318-08 12.5.2)

L_{hk} = 14 in, (ACI 318-08, Fig. R12.5)

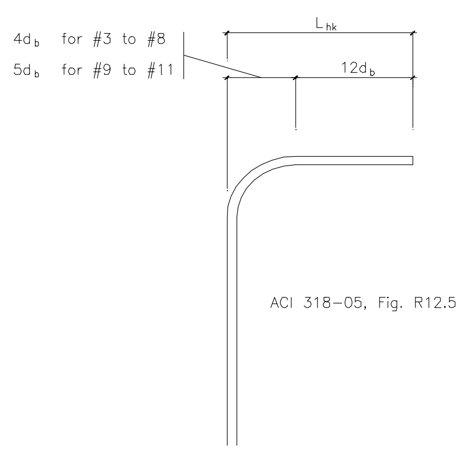
where d_b = 0.88 in

$\rho_{required} / \rho_{provided}$ = 0.8 ($A_{s,reqd} / A_{s,provd}$, ACI 318, 12.2.5)

ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-08 12.2.4)

λ = 1.0 (normal weight)

η = 0.7 (#11 or smaller, cover > 2.5" & side >2.0", ACI 318-08 12.5.3)



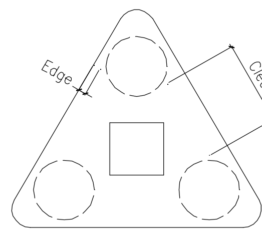
Pile Cap Design for 3-Piles Pattern Based on ACI 318-08

DESIGN CRITERIA

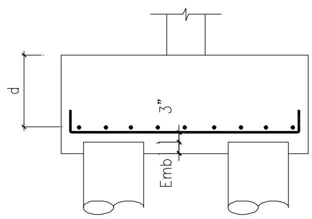
1. FROM PILE DESIGN & SOIL REPORT, DETERMINE SINGLE PILE MAX LOADS OR CAPACITY AT CAP BOTTOM FACE, ϕP_n, ϕM_n, & ϕV_n.
2. THE MAXIMUM COLUMN CAPACITY AT COLUMN BASE, $\phi P_{n,col}$, $\phi M_{n,col}$, $\phi V_{n,col}$, MAY BE BASED ON PILE CAP BALANCED LOADS. USER CAN CHANGE THE YELLOW CELLS TO MODIFY THEM FOR DIFFERENT CASES.
3. PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH Min(0.25, S_{DS}/10) TIMES AXIAL VERT COLUMN LOADING. (IBC 09 1810.3.13)

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	20	in
COLUMN SIZE (SHORT SIDE)	C	=	24	in
SINGLE PILE MAX LOADS OR CAPACITY	ϕP_n	=	130	k
(at the section of cap bottom face)	ϕM_n	=	400	ft-k
	ϕV_n	=	35	k
PILE CLEAR DIST. (24" min, 2D reqd)	Clear	=	40	in
EDGE DISTANCE (9" min)	Edge	=	12	in
EFFECTIVE DEPTH	d	=	53	in
CAP BOTTOM REINFORCING	# 9	@	16	in o.c., each way



PILE PATTERN



SECTION

The Column Can Support Max Loads:

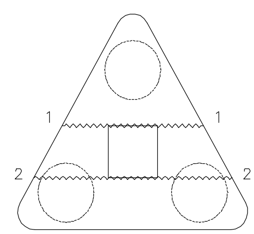
$P_{u,col} \le \phi P_{n,col}$ =	390	kips
$M_{u,col} \le \phi M_{n,col}$ =	710.0	ft-kips
$V_{u,col} \le \phi V_{n,col}$ =	105	kips

THE PILE CAP DESIGN IS ADEQUATE.

ANALYSIS

CHECK FLEXURE CAPACITY AT COLUMN FACE (ACI 318-08, 10.2, 10.3, 10.5, 7.12.2)

Pile Spacing = 60 in
Cap Edge Length = 136.2 in
L_{1-1} = 77.0 in, length of section 1-1
L_{2-2} = 104.7 in, length of section 2-2
$M_{u,\ 1-1}$ = 113.4 ft-kips **/ ft**, (to middle of cap elevation)
$M_{u,\ 2-2}$ = 123.7 ft-kips **/ ft**, (to middle of cap elevation)
ρ_{provD} = 0.0012



FLEXURE CRITICAL SECTION

$$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\frac{M_{u,\max}}{0.383 b d^2 f_c'}}\right)}{f_y} = 0.0008 < \rho_{provD}$$ **[Satisfactory]**

$$\rho_{MAX} = \frac{0.85\beta_1 f_c'}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = 0.0206 > \rho_{provD}$$ **[Satisfactory]**

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) = 0.0011 < \rho_{provD}$$ **[Satisfactory]**

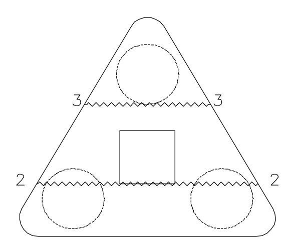
CHECK ONE WAY SHEAR CAPACITY AT THE FACE OF COLUMN & PILE

(ACI 318-08, Chapter 11, Except 11.1.3.1)

L_{3-3} = 62.4 in, length of section 3-3
$V_{u,\ 2-2} = 2\ (\phi P_n) / L_{2-2}$ = 0.0 kips **/ ft** (No shear at "d" offset.)
$V_{u,\ 3-3} = (\phi P_n) / L_{3-3}$ = 0.0 kips **/ ft** (No shear at "d" offset.)

$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$ = 60.3 kips **/ ft** > $V_{u,\ max}$ **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)



ONE WAY SHEAR CRITICAL SECTION

CHECK COLUMN PUNCHING CAPACITY (ACI 318-08, 11.4.7.4, 11.11.1.2, 11.11.6 & 13.5.3.2)

$$\phi v_c(psi) = \phi(2+y)\sqrt{f_c'} = 190 \text{ ksi} > v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} = 31 \text{ ksi}$$ **[Satisfactory]**

where

β_c = 1.00
b_1 = (C + d) = 77 in
b_2 = (C + d) = 77 in
b_0 = $2b_1 + 2b_2$ = 308 in
A_p = b_0 d = 16324 in^2

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = 0.4$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right) = 2.0$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1+\left(\frac{d}{b_1}\right)^2+3\left(\frac{b_2}{b_1}\right)\right] = 870 \text{ ft}^4$$

(cont'd)

CHECK SINGLE PILE PUNCHING CAPACITY (ACI 318-08, 11.4.7.4, 11.11.1.2, 11.11.6 & 13.5.3.2)

$\phi v_c(psi) = \phi(2+y)\sqrt{f_c^{'}} =$ 95 ksi **>** $v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} =$ 49 ksi **[Satisfactory]**

where $\alpha =$ 134.1 deg

$b_1 =$ $(\alpha / 360)(D\pi / 4 + d) =$ 26 in

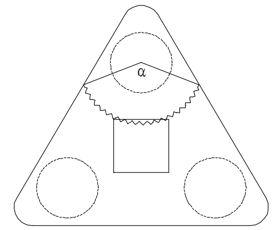
$b_2 =$ $(\alpha / 360)(D\pi / 4 + d) =$ 26 in

$b_0 =$ $(\alpha / 360)(D + d)\pi =$ 85 in, conservative value

$A_p =$ $b_0 d$ = 4529 in^2

$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] =$ 59 ft^4

$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} =$ 0.4



PILE PUNCHING CRITICAL SECTION

y = 0 , conservative value as one way shear

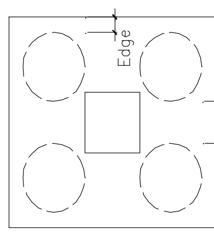
Pile Cap Design for 4-Piles Pattern Based on ACI 318-08

DESIGN CRITERIA

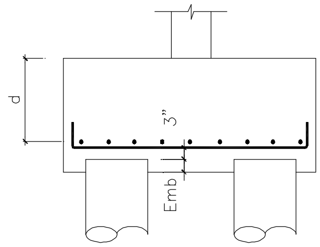
1. FROM PILE DESIGN & SOIL REPORT, DETERMINE SINGLE PILE MAX LOADS OR CAPACITY AT CAP BOTTOM FACE, ϕP_n, ϕM_n, & ϕV_n.
2. THE MAXIMUM COLUMN CAPACITY AT COLUMN BASE, $\phi P_{n,col}$, $\phi M_{n,col}$, $\phi V_{n,col}$, MAY BE BASED ON PILE CAP BALANCED LOADS. USER CAN CHANGE THE YELLOW CELLS TO MODIFY THEM FOR DIFFERENT CASES.
3. PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH Min(0.25, S_{DS}/10) TIMES AXIAL VERT COLUMN LOADING. (IBC 09 1810.3.13)

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	20	in
COLUMN SIZE (SHORT SIDE)	C	=	24	in
SINGLE PILE MAX LOADS OR CAPACITY	ϕP_n	=	130	k
(at the section of cap bottom face)	ϕM_n	=	400	ft-k
	ϕV_n	=	35	k
PILE CLEAR DIST. (24" min, 2D reqd)	Clear	=	40	in
EDGE DISTANCE (9" min)	Edge	=	12	in
EFFECTIVE DEPTH	d	=	53	in
CAP BOTTOM REINFORCING #	9	@	12	in o.c., each way



PILE PATTERN



SECTION

The Column Can Support Max Loads:

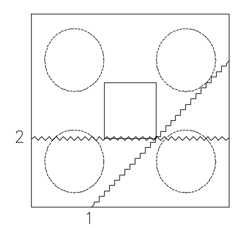
$P_{u,col} \le \phi P_{n,col}$ =	520	kips
$M_{u,col} \le \phi M_{n,col}$ =	946.7	ft-kips
$V_{u,col} \le \phi V_{n,col}$ =	140	kips

THE PILE CAP DESIGN IS ADEQUATE.

ANALYSIS

CHECK FLEXURE CAPACITY AT COLUMN FACE (ACI 318-08, 10.2, 10.3, 10.5, 7.12.2)

Pile Spacing = 60 in
Cap Edge Length = 104.0 in
L_{1-1} = 113.1 in, length of section 1-1
L_{2-2} = 104.0 in, length of section 2-2
$M_{u,\,1-1}$ = 80.3 ft-kips **/ ft**, (to middle of cap elevation)
$M_{u,\,2-2}$ = 156.2 ft-kips **/ ft**, (to middle of cap elevation)
ρ_{provD} = 0.0016



FLEXURE CRITICAL SECTION

$$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\frac{M_{u,\max}}{0.383 b d^2 f_c'}}\right)}{f_y} = \quad 0.0010 \quad < \quad \rho_{provD} \qquad \textbf{[Satisfactory]}$$

$$\rho_{MAX} = \frac{0.85\beta_1 f_c'}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = \quad 0.0206 \quad > \quad \rho_{provD} \qquad \textbf{[Satisfactory]}$$

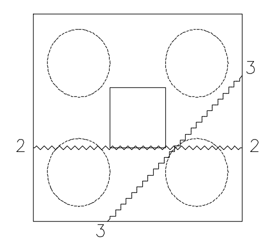
$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) = \quad 0.0014 \quad < \quad \rho_{provD} \qquad \textbf{[Satisfactory]}$$

CHECK ONE WAY SHEAR CAPACITY AT THE FACE OF COLUMN & PILE

(ACI 318-08, Chapter 11, Except 11.1.3.1)

L_{3-3} = 82.2 in, length of section 3-3
$V_{u,\,2-2} = 2\,(\phi P_n) / L_{2-2}$ = 0.0 kips **/ ft** (No shear at "d" offset.)
$V_{u,\,3-3} = (\phi P_n) / L_{3-3}$ = 0.0 kips **/ ft** (No shear at "d" offset.)

$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$ = 60.3 kips **/ ft** > $V_{u,\,max}$ **[Satisfactory]**
where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)



ONE WAY SHEAR CRITICAL SECTION

CHECK COLUMN PUNCHING CAPACITY (ACI 318-08, 11.4.7.4, 11.11.1.2, 11.11.6 & 13.5.3.2)

$$\phi v_c(psi) = \phi(2+y)\sqrt{f_c'} = \ 190 \text{ ksi} \quad > \quad v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} = \ 42 \text{ ksi} \qquad \textbf{[Satisfactory]}$$

where β_c = 1.00

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = 0.4 \qquad y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right) = 2.0$$

b_1 = (C + d) = 77 in
b_2 = (C + d) = 77 in
b_0 = $2b_1 + 2b_2$ = 308 in
A_p = b_0 d = 16324 in^2

$$J = \left(\frac{d b_1^3}{6}\right)\left[1+\left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] = 870 \ ft^4$$

(cont'd)

CHECK SINGLE PILE PUNCHING CAPACITY (ACI 318-08, 11.4.7.4, 11.11.1.2, 11.11.6 & 13.5.3.2)

$\phi v_c(psi) = \phi(2+y)\sqrt{f_c^{'}} =$ 95 ksi **>** $v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} =$ 40 ksi **[Satisfactory]**

where $\alpha =$ 164.1 deg

$b_1 =$ $(\alpha / 360)(D\pi / 4 + d) =$ 31 in

$b_2 =$ $(\alpha / 360)(D\pi / 4 + d) =$ 31 in

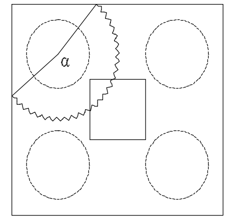
$b_0 =$ $(\alpha / 360)(D + d)\pi =$ 105 in, conservative value

$A_p =$ $b_0 d$ = 5542 in^2

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] =$$ 90 ft^4

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} =$$ 0.4

y = 0 , conservative value as one way shear



PILE PUNCHING CRITICAL SECTION

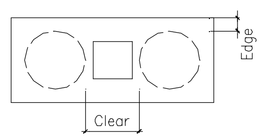
Pile Cap Design for 2-Piles Pattern Based on ACI 318-08

DESIGN CRITERIA

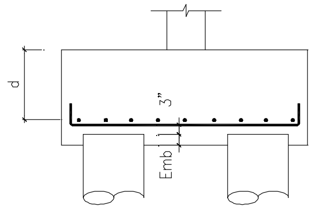
1. FROM PILE DESIGN & SOIL REPORT, DETERMINE SINGLE PILE MAX LOADS OR CAPACITY AT CAP BOTTOM FACE, ϕP_n, ϕM_n, & ϕV_n.
2. THE MAXIMUM COLUMN CAPACITY AT COLUMN BASE, $\phi P_{n,col}$, $\phi M_{n,col}$, $\phi V_{n,col}$, MAY BE BASED ON PILE CAP BALANCED LOADS. USER CAN CHANGE THE YELLOW CELLS TO MODIFY THEM FOR DIFFERENT CASES.
3. PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH Min(0.25, S_{DS}/10) TIMES AXIAL VERT COLUMN LOADING. (IBC 09 1810.3.13)

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	20	in
COLUMN SIZE (SHORT SIDE)	C	=	24	in
SINGLE PILE MAX LOADS OR CAPACITY	ϕP_n	=	130	k
(at the section of cap bottom face)	ϕM_n	=	400	ft-k
	ϕV_n	=	35	k
PILE CLEAR DIST. (24" min, 2D reqd)	Clear	=	40	in
EDGE DISTANCE (9" min)	Edge	=	12	in
EFFECTIVE DEPTH	d	=	53	in
CAP BOTTOM REINFORCING	# 9	@	12	in o.c., each way



PILE PATTERN



SECTION

The Column Can Support Max Loads:

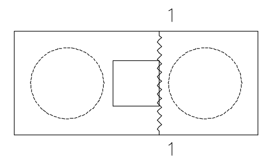
$P_{u,col} \le \phi P_{n,col}$ =	260	kips
$M_{u,col} \le \phi M_{n,col}$ =	473.3	ft-kips
$V_{u,col} \le \phi V_{n,col}$ =	70	kips

THE PILE CAP DESIGN IS ADEQUATE.

ANALYSIS

CHECK FLEXURE CAPACITY AT COLUMN FACE (ACI 318-08, 10.2, 10.3, 10.5, 7.12.2)

Pile Spacing = 60 in
Cap Edge Length = 104.0 in
L_{1-1} = 44.0 in, length of section 1-1
$M_{u,\ 1-1}$ = 104.4 ft-kips **/ ft**, (to middle of cap elevation)
ρ_{provD} = 0.0016



FLEXURE CRITICAL SECTION

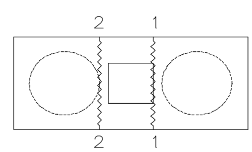
$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,\max}}{0.383 b d^2 f'_c}}\right)}{f_y} = \quad 0.0007 \ < \ \rho_{provD}$$ **[Satisfactory]**

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = \quad 0.0206 \ > \ \rho_{provD}$$ **[Satisfactory]**

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) = \quad 0.0009 \ < \ \rho_{provD}$$ **[Satisfactory]**

CHECK ONE WAY SHEAR CAPACITY AT THE FACE OF COLUMN & PILE
(ACI 318-08, Chapter 11, Except 11.1.3.1)

L_{2-2} = 44.0 in, length of section 2-2
$V_{u,\ 1-1} = (\phi P_n) / L_{1-1}$ = 0.0 kips **/ ft** (No shear at "d" offset.)
$V_{u,\ 2-2} = (\phi P_n) / L_{2-2}$ = 0.0 kips **/ ft** (No shear at "d" offset.)



ONE WAY SHEAR CRITICAL SECTION

$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$ = 60.3 kips **/ ft** > $V_{u,\ max}$ **[Satisfactory]**
where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

CHECK COLUMN PUNCHING CAPACITY (ACI 318-08, 11.4.7.4, 11.11.1.2, 11.11.6 & 13.5.3.2)

$$\phi v_c(psi) = \phi(2+y)\sqrt{f'_c} = \ 190 \text{ ksi} \quad > \quad v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} = \ 31 \text{ ksi}$$ **[Satisfactory]**

where

β_c = 1.00
b_1 = (C + d) = 77 in
b_2 = [(C + min(Edge , d)] = 36 in
$b_0 = 2b_1 + 2b_2$ = 226 in
$A_p = b_0\ d$ = 11978 in^2

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = 0.4937$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right) = 2.0$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1+\left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] = 559 \ ft^4$$

(cont'd)

CHECK SINGLE PILE PUNCHING CAPACITY (ACI 318-08, 11.4.7.4, 11.11.1.2, 11.11.6 & 13.5.3.2)

$\phi v_c(psi) = \phi(2+y)\sqrt{f'_c} =$ 95 ksi **>** $v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} =$ 67 ksi **[Satisfactory]**

where $\alpha =$ 94.3 deg

$b_1 =$ $(\alpha / 360)\ (D\pi / 4 + d) =$ 18 in

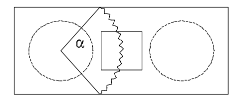
$b_2 =$ $(\alpha / 360)\ (D\pi / 4 + d) =$ 18 in

$b_0 =$ $(\alpha / 360)\ (D + d)\ \pi =$ 60 in, conservative value

$A_p =$ $b_0\ d$ = 3185 in^2

$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] =$ 31 ft^4

$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} =$ 0.4



PILE PUNCHING CRITICAL SECTION

y = 0 , conservative value as one way shear

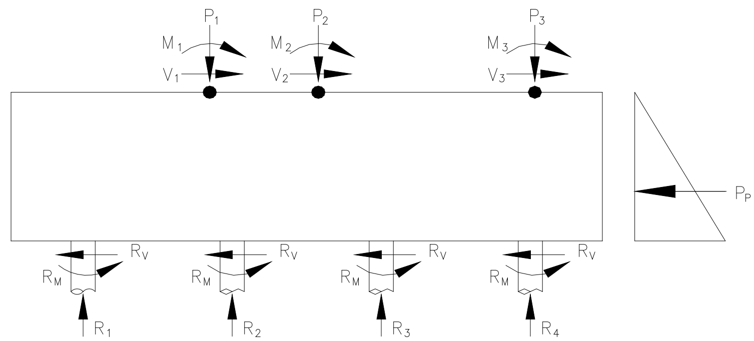
Daniel T. Li
Engineering International

PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Determination of Pile Cap Balanced Loads and Reactions

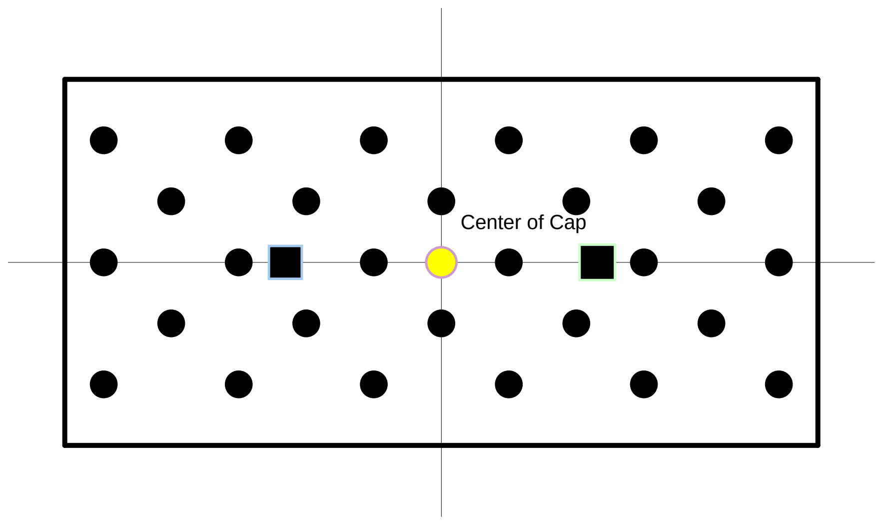
DESIGN CRITERIA

1. PILE TOP SHEAR, R_V, & MOMENT, R_M, RELATIONSHIP MUST BE FROM SOIL REPORT (R_V vs R_M) DIAGRAM. THEY ARE NON-LINEAR SET AND EQUAL AT ALL TOP OF PILES FOR RIGID PILE CAP. USING LINEAR SPRINGS TO MODEL THEM IS INADEQUATE.
2. PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH Min(S_{DS}/10, 0.25) TIMES AXIAL VERT COLUMN LOADING. (IBC 09 1810.3.13). TO CONSIDER CONCRETE TENSION CREAKED, THE TIE BEAM SHOULD NOT BE LATERAL REACTION MEMBER.



INPUT DATA & DESIGN SUMMARY

NUMBER OF HORIZONTAL PILE ROWS		5	
NUMBER OF VERTICAL PILE ROWS		11	
PILES STAGGERED ?		Yes	
PILE DIAMETER	D =	24	in
PILE CLEAR DISTANCE	Clear =	48	in
EDGE DISTANCE	Edge =	12	in
PILE CAP HEIGHT	H =	72	in
PASSIVE SOIL PRESSURE	P_P =	300	pcf



PILE LOCATION TO CENTER OF PILE CAP AND VERTICAL REACTIONS

Pile	X (in)	Y (in)	R (kips)
1	-312	72	-684.1
2	-312	0	-684.1
3	-312	-72	-684.1
4	-249	36	-585.3
5	-249	-36	-585.3
6	-187	72	-486.4
7	-187	0	-486.4
8	-187	-72	-486.4
9	-125	36	-387.5
10	-125	-36	-387.5
11	-62	72	-288.7
12	-62	0	-288.7
13	-62	-72	-288.7
14	0	36	-189.8
15	0	-36	-189.8
16	62	72	-91.0
17	62	0	-91.0
18	62	-72	-91.0
19	125	36	7.9
20	125	-36	7.9
21	187	72	106.8
22	187	0	106.8
23	187	-72	106.8
24	249	36	205.6
25	249	-36	205.6
26	312	72	304.5
27	312	0	304.5
28	312	-72	304.5
Group Center	0	0	

INPUT POINT LOADS ON TOP OF CAP

LOAD	X (in)	Y (in)	P (k)	Vx (k)	Vy (k)	My (ft-k)	Mx (ft-k)
1	-144	0	-8507.55	700			
2	144	0	3192.483	700			
3							
4							
5							
6							
7							
8							
9							
10							

PILE CAP SIZE

X = 672 in
Y = 192 in
H = 72 in

TOTAL LOADS ON PILE CENTER OF CAP BOTTOM FACE

P	-5315.1	kips			
V_x =	1313.6	kips	V_y =	0.0	kips
M_y =	148973.1	ft-kips	M_x =	0.0	ft-kips

$R_V = (V_x^2 + V_y^2)^{0.5}$ / No. = ***46.9*** kips $R_{M,x}$ = 170.0 ft-kips

R_M = ***170*** ft-kips, (from Soil Report per R_V above) $R_{M,y}$ = 0.0 ft-kips

DETERMINE MAXIMUM PILE VERTICAL REACTIONS

A = 28 piles I_x = 75168 piles-in^2 I_y = 1127520 piles-in^2

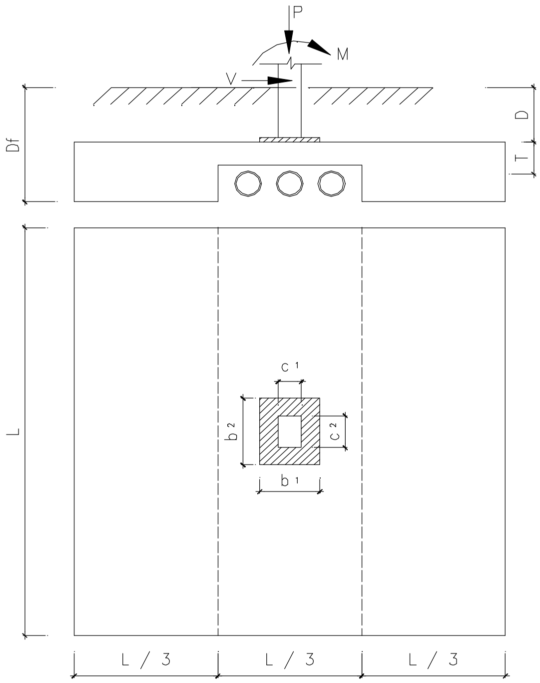
R_{max} = ***304.5*** kips R_{min} = ***-684.1*** kips, (Tension)

(The Bold Italic Red values are for pile and pile cap design.)

Design of Footing at Piping Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

COLUMN WIDTH	c_1	=	5	in
COLUMN DEPTH	c_2	=	5	in
BASE PLATE WIDTH	b_1	=	16	in
BASE PLATE DEPTH	b_2	=	16	in
FOOTING CONCRETE STRENGTH	f_c'	=	2.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	40	k
AXIAL LIVE LOAD	P_{LL}	=	25	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	1	Seismic,SD
SEISMIC AXIAL LOAD	P_{LAT}	=	20	k, SD
SEISMIC MOMENT LOAD	M_{LAT}	=	96	ft-k, SD
SEISMIC SHEAR LOAD	V_{LAT}	=	2	k, SD
SURCHARGE	q_s	=	0.1	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	3	ft
FOOTING MIDDLE THICKNESS	T	=	18	in
SOIL COVER THICKNESS	D	=	12	in
ALLOW SOIL PRESSURE	Q_a	=	3	ksf
SQUARE FOOTING LENGTH	L	=	7	ft
REINFORCING SIZE		#	5	



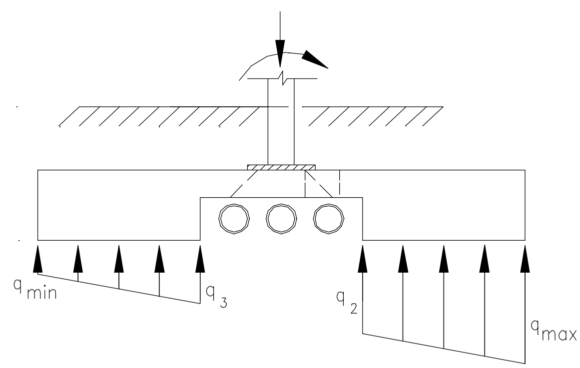
MIDDLE BOTTOM EACH WAY : 9 # 5 @ 9 in o.c.

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS (IBC SEC.1605.3.2 & ACI 318-08 SEC.9.2.1)

CASE 1:	DL + LL	P	=	65	kips
		M	=	0	ft-kips
CASE 2:	DL + LL + E / 1.4	P	=	79	kips
		M	=	69	ft-kips
CASE 3:	0.9 DL + E / 1.4	P	=	50	kips
		M	=	69	ft-kips



CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 6.4 **>** F = 0.75 / 0.9 **[Satisfactory]**

Where M_O = $M_{LAT} + V_{LAT}\, D_f - 0.5\, P_{LAT}\, L$ = 32 k-ft

P_{conc} = (0.15 kcf) L^2 [T + 2 (D_f - D - T) /3] = 13.48 k, footing wt

P_{soil} = $w_s\, D\, L^2$ = 5.39 k, soil weight

M_R = $0.5\, P_{DL} L + 0.5\, (P_{conc} + P_{soil})\, L$ = 206 k-ft

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

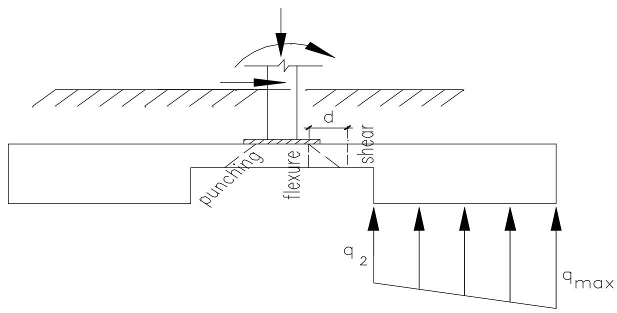
Service Loads	CASE 1	CASE 2	CASE 3	
P	65.0	79.3	50.3	k
$q_s L^2$	4.9	4.9	0.0	k, (surcharge load)
$P_{conc - soil}$	3.6	3.6	3.2	k, (footing increased)
Σ P	73.5	87.8	53.5	k
Σ M	0.0	68.6	68.6	ft - k
q_{min}	2.250 > 0	1.441 > 0	0.393 > 0	ksf, net pressure
q_3	2.250	2.272	1.223	ksf, net pressure
q_2	2.250	3.102	2.054	ksf, net pressure
q_{max}	2.250	3.933	2.884	ksf, net pressure
q_{allow}	3.0	4.0	4.0	ksf

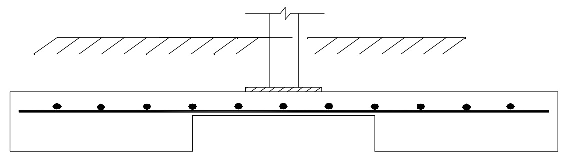
(cont'd)

Where
$$\begin{cases} q_{\max} = 0.5\left(\dfrac{3\Sigma P}{L^2} + \dfrac{162\Sigma M}{13L^3}\right) \\ q_{\min} = 0.5\left(\dfrac{3\Sigma P}{L^2} - \dfrac{162\Sigma M}{13L^3}\right) \end{cases} \qquad \begin{cases} q_2 = \dfrac{2}{3}q_{\max} + \dfrac{1}{3}q_{\min} \\ q_3 = \dfrac{1}{3}q_{\max} + \dfrac{2}{3}q_{\min} \end{cases}$$

[Satisfactory]

DESIGN FLEXURE & CHECK FLEXURE SHEAR (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)





Service Loads	CASE 1	CASE 2	CASE 3	
V	36.7	57.5	40.3	k, flexure shear
M	69.7	111.6	79.1	ft - k, flexure moment

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} \qquad \rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y} \qquad \rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

DESIGN FLEXURE

Location	$M_{u,max}$ = 1.5 M	d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	use	ρ_{provD}
Middle Bottom Each Way	167.3 ft-k	14.69	0.0022	0.0021	0.0129	18	9 # 5 @ 9 in o.c.	0.0023

[Satisfactory]

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$ = 1.5 V	$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	check $V_u < \phi\, V_c$
Pipe Direction	86.2 k	93 k	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318-08 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J} \qquad A_P = 2(b_1 + b_2)d \qquad \phi v_c(psi) = \phi(2 + y)\sqrt{f'_c}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] \qquad \gamma_v = 1 - \frac{1}{1 + \dfrac{2}{3}\sqrt{\dfrac{b_1}{b_2}}} \qquad y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

$$R = 0 \qquad A_f = \frac{2}{3}L^2 \qquad b_0 = \frac{A_P}{d},\ b_1 = (0.5c_1 + 0.5b_1 + d),\ b_2 = (0.5c_2 + 0.5b_2 + d)$$

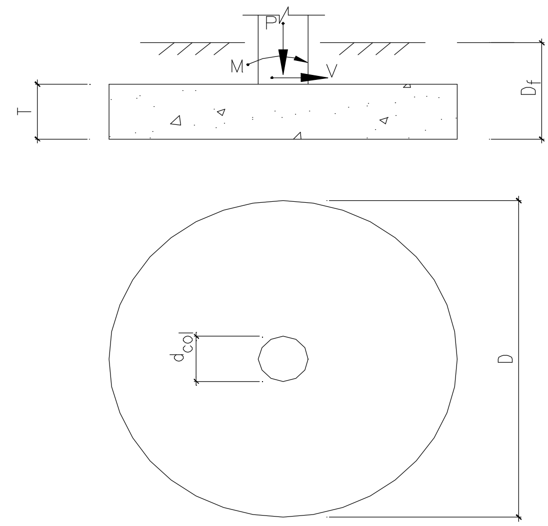
Case	P_u	M_u	b_1	b_2	b_0	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\,(psi)}$	$\phi\, v_c$
1	97.5	0.0	25.2	25.2	100.8	0.4	1.0	2.0	32.7	10.3	0.0	8.2	65.9	150.0
2	118.9	102.9	25.2	25.2	100.8	0.4	1.0	2.0	32.7	10.3	0.0	8.2	117.0	150.0
3	75.4	102.9	25.2	25.2	100.8	0.4	1.0	2.0	32.7	10.3	0.0	8.2	87.6	150.0

[Satisfactory]

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)
P_u = 1.5 P_{col}
M_u = 1.5 M_{col}

Circular Footing Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY



COLUMN DIAMETER	d_{col} =	12	in		
COLUMN DEAD LOAD	P_{DL} =	40	kips		
COLUMN LIVE LOAD	P_{LL} =	38	kips		
LATERAL LOAD (0=Wind, 1=Seismic)		0	Wind,ASD		
WIND AXIAL LOAD	P_{LAT} =	5	k, ASD		
WIND MOMENT LOAD	M_{LAT} =	39.5	ft-k, ASD		
WIND SHEAR LOAD	V_{LAT} =	0.15	k, ASD		
SOIL WEIGHT	w_S =	0.11	kcf		
FOOTING EMBEDMENT DEPTH	D_f =	2	ft		
FOOTING THICKNESS	T =	18	in		
ALLOW SOIL PRESSURE	Q_a =	2.5	ksf		
FOOTING DIAMETER	D =	7.5	ft		
CONCRETE STRENGTH	f_c' =	3	ksi		
REBAR YIELD STRESS	f_y =	60	ksi		
FOOTING TOP REBAR	#	4	@	48	in o.c., each way
FOOTING BOTTOM REBAR	#	6	@	18	in o.c., each way

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 9.4 **>** F = 1.6 / 0.9 = 1.78 **[Satisfactory]**

Where M_O = M_{LAT} + V_{LAT} T - P_{LAT}(0.5 D) = 21 k-ft

M_R = (P_{DL}+ P_{ftg} + P_{soil}) (0.5 D) = 196 k-ft

P_{ftg} = (0.15 kcf) π T D^2/ 4 = 9.94 k, footing weight

P_{soil} = w_S (D_f - T) π D^2/ 4 = 2.43 k, soil weight

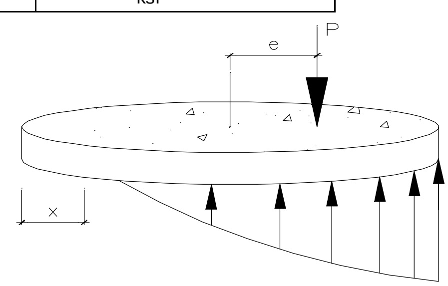
COMBINED LOADS AT TOP FOOTING (IBC 1605.3.2 & ACI 318-08 9.2.1)

CASE 1:	DL + LL	P =	78.0	kips	1.2 DL + 1.6 LL	Pu	=	108.8	kips
CASE 2:	DL + LL + 1.3 W	P =	84.5	kips	1.2 DL + LL + 1.6 W	Pu	=	94.0	kips
		M =	51	ft-kips		Mu	=	63	ft-kips
		V =	0.2	kips		Vu	=	0.2	kips
		e =	0.6	ft, fr cl ftg		eu	=	0.7	ft, fr cl ftg
CASE 3:	DL + LL + 0.65 W	P =	81.3	kips	0.9 DL+ 1.6 W	Pu	=	44.0	kips
		M =	26	ft-kips		Mu	=	63	ft-kips
		V =	0.1	kips		Vu	=	0.2	kips
		e =	0.3	ft, fr cl ftg		eu	=	1.4	ft, fr cl ftg

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

Service Loads	CASE 1	CASE 2	CASE 3	
P	78.0	84.5	81.3	k
e	0.0	0.6	0.3	ft (from center of footing)
P_{ftg} - P_{soil}	7.5	7.5	6.8	k, (footing increasing)
Σ P	85.5	92.0	88.0	k, (net loads)
e	0.0	0.6	0.3	ft
q_{min}	1.94	0.83	1.37	ksf
x		@ 0.00 ft, from edge	@ 0.00 ft, from edge	
q_{max}	1.94	3.33	2.62	ksf
$q_{allowable}$	2.50	3.33	3.33	ksf

[Satisfactory]



CHECK FLEXURE & SHEAR OF FOOTING
(ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) \qquad \rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

(cont'd)

FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	108.8	94.0	44.0	k
e_u	0.0	0.7	1.4	ft
$\gamma[0.15T + w_s(D_f - T)]\ A$	14.8	14.8	11.1	k, (factored footing & backfill)
$\Sigma\ P_u$	123.6	108.8	55.1	k
e_u	0.0	0.6	1.2	ft
$q_{u,\ min}$	2.80	0.93	0.00	ksf
x		@ 0.00 ft, from edge	@ 0.75 ft, from edge	
$q_{u,\ max}$	2.80	4.00	2.81	ksf

FOOTING MOMENT & SHEAR FOR CASE **1**

Section	0	1/10 D	2/10 D	3/10 D	4/10 D	Center	6/10 D	7/10 D	8/10 D	9/10 D	D
X_u (ft, dist. from left of footing)	0	0.75	1.50	2.25	3.00	3.75	4.50	5.25	6.00	6.75	7.50
Tangent (ft)	0.00	4.50	6.00	6.87	7.35	7.50	7.35	6.87	6.00	4.50	0.00
TA (ft^2)	0.00	2.30	3.99	4.86	5.36	11.17	5.36	4.86	3.99	2.30	0.00
$M_{u,col}$ (ft-k)	0	0	0	0	0	0	81.6	163.2	244.8	326.4	408
$V_{u,col}$ (k)	0	0	0	0	0	54.4	108.8	108.8	108.8	108.8	108.8
$q_{u,ftg\ \&\ fill}$ (ksf)	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34
$M_{u,ftg\ \&\ fill}$ (ft-k)	0.00	0.00	0.58	2.16	4.97	9.13	16.11	24.43	33.98	44.53	55.67
$V_{u,ftg\ \&\ fill}$ (k)	0.00	0.39	1.44	2.93	4.65	7.42	10.20	11.91	13.40	14.46	14.84
$q_{u,soil}$ (ksf)	-2.80	-2.80	-2.80	-2.80	-2.80	-2.80	-2.80	-2.80	-2.80	-2.80	-2.80
$M_{u,soil}$ (ft-k)	0	0	-4.8262	-18.029	-41.427	-76.066	-134.16	-203.5	-283.03	-370.93	-463.67
$V_{u,soil}$ (k)	0	-3.2175	-12.02	-24.401	-38.691	-61.822	-84.953	-99.243	-111.62	-120.43	-123.64
$\Sigma\ M_u$ (ft-k)	**0**	**-2.1234**	**-4.2468**	**-15.865**	**-36.454**	**-66.934**	**-36.454**	**-15.865**	**-4.2468**	**-2.1234**	**0**
$\Sigma\ V_u$ (kips)	**0**	**-2.8312**	**-10.577**	**-21.471**	**-34.046**	**0**	**34.046**	**21.471**	**10.577**	**2.8312**	**0**

FOOTING MOMENT & SHEAR FOR CASE **2**

Section	0	1/10 D	2/10 D	3/10 D	4/10 D	Center	6/10 D	7/10 D	8/10 D	9/10 D	D
X_u (ft, dist. from left of footing)	0	0.75	1.50	2.25	3.00	3.75	4.50	5.25	6.00	6.75	7.50
Tangent (ft)	0.00	4.50	6.00	6.87	7.35	7.50	7.35	6.87	6.00	4.50	0.00
TA (ft^2)	0.00	2.30	3.99	4.86	5.36	11.17	5.36	4.86	3.99	2.30	0.00
$M_{u,col}$ (ft-k)	0	0	0	0	0	-31.84	6.82	77.32	147.82	218.32	288.82
$V_{u,col}$ (k)	0	0	0	0	0	47	94.0	94.0	94.0	94.0	94.0
$q_{u,ftg\ \&\ fill}$ (ksf)	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34
$M_{u,ftg\ \&\ fill}$ (ft-k)	0.00	0.00	0.58	2.16	4.97	9.13	16.11	24.43	33.98	44.53	55.67
$V_{u,ftg\ \&\ fill}$ (k)	0.00	0.39	1.44	2.93	4.65	7.42	10.20	11.91	13.40	14.46	14.84
$q_{u,soil}$ (ksf)	0.93	1.24	1.54	1.85	2.16	2.46	2.77	3.08	3.39	3.69	4.00
$M_{u,soil}$ (ft-k)	0	0	-2.0199	-8.4177	-21.204	-42.205	-82.787	-133.92	-195.69	-267.07	-344.49
$V_{u,soil}$ (k)	0	-1.42	-5.9175	-13.486	-23.752	-43.292	-64.476	-79.37	-93.6	-104.6	-108.84
$\Sigma\ M_u$ (ft-k)	**0**	**-0.7203**	**-1.4405**	**-6.2532**	**-16.231**	**-64.913**	**-59.861**	**-32.173**	**-13.894**	**-6.9468**	**0**
$\Sigma\ V_u$ (kips)	**0**	**-1.0337**	**-4.4745**	**-10.557**	**-19.107**	**11.13**	**39.723**	**26.544**	**13.801**	**3.8584**	**0**

FOOTING MOMENT & SHEAR FOR CASE **3**

Section	0	1/10 D	2/10 D	3/10 D	4/10 D	Center	6/10 D	7/10 D	8/10 D	9/10 D	D
X_u (ft, dist. from left of footing)	0	0.75	1.50	2.25	3.00	3.75	4.50	5.25	6.00	6.75	7.50
Tangent (ft)	0.00	4.50	6.00	6.87	7.35	7.50	7.35	6.87	6.00	4.50	0.00
TA (ft^2)	0.00	2.30	3.99	4.86	5.36	11.17	5.36	4.86	3.99	2.30	0.00
$M_{u,col}$ (ft-k)	0	0	0	0	0	-31.84	-30.68	2.32	35.32	68.32	101.32
$V_{u,col}$ (k)	0	0	0	0	0	22	44.0	44.0	44.0	44.0	44.0
$q_{u,ftg\ \&\ fill}$ (ksf)	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
$M_{u,ftg\ \&\ fill}$ (ft-k)	0.00	0.00	0.43	1.62	3.73	6.85	12.08	18.32	25.48	33.40	41.75
$V_{u,ftg\ \&\ fill}$ (k)	0.00	0.29	1.08	2.20	3.48	5.57	7.65	8.94	10.05	10.84	11.13
$q_{u,soil}$ (ksf)	0.00	0.12	0.55	0.98	1.41	1.84	2.28	2.71	3.14	3.57	2.81
$M_{u,soil}$ (ft-k)	0	0	-0.1226	-1.2188	-4.4261	-10.982	-26.656	-47.721	-74.601	-107.02	-143.07
$V_{u,soil}$ (k)	0	-0.0937	-0.932	-3.2909	-7.4659	-16.999	-28.094	-36.663	-45.346	-52.357	-55.133
$\Sigma\ M_u$ (ft-k)	**0**	**0.156**	**0.312**	**0.4045**	**-0.696**	**-35.973**	**-45.256**	**-27.078**	**-13.797**	**-6.8985**	**0**
$\Sigma\ V_u$ (kips)	**0**	**0.196**	**0.1502**	**-1.0938**	**-3.9821**	**10.567**	**23.555**	**16.273**	**8.7048**	**2.4859**	**0**

(cont'd)

FOOTING MOMENT & SHEAR SUMMARY

Section			0	1/10 D	2/10 D	3/10 D	4/10 D	Center	6/10 D	7/10 D	8/10 D	9/10 D	D
X_u (ft, dist. from left of footing)			0	0.75	1.50	2.25	3.00	3.75	4.50	5.25	6.00	6.75	7.50
Tangent (ft)			0.00	4.50	6.00	6.87	7.35	7.50	7.35	6.87	6.00	4.50	0.00
Uniform Loads	**Case 1**	M_u, (ft-k / ft)	0	-0.4719	-0.7078	-2.308	-4.9607	-8.9245	-4.9607	-2.308	-0.7078	-0.4719	0
		V_u, (k / ft)	0	-0.6292	-1.7628	-3.1236	-4.6331	0	4.6331	3.1236	1.7628	0.6292	0
	Case 2	M_u, (ft-k / ft)	0	-0.1601	-0.2401	-0.9097	-2.2087	-8.6551	-8.146	-4.6804	-2.3156	-1.5437	0
		V_u, (k / ft)	0	-0.2297	-0.7458	-1.5358	-2.6001	1.484	5.4056	3.8616	2.3002	0.8574	0
	Case 3	M_u, (ft-k / ft)	0	0.0347	0.052	0.0589	-0.0947	-4.7964	-6.1586	-3.9393	-2.2995	-1.533	0
		V_u, (k / ft)	0	0.0435	0.025	-0.1591	-0.5419	1.409	3.2054	2.3673	1.4508	0.5524	0

CHECK FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	ρ_{provD}
Top Slab	0.1	ft-k / ft	15.75	0.0000	0.0000	0.0155	no limit	0.0003
Bottom Slab	-8.9	ft-k / ft	14.63	0.0010	0.0008	0.0155	18	0.0017

[Satisfactory]

CHECK FLEXURE SHEAR

$V_{u,max}$	$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	check $V_u < \phi\, V_c$
5.4 k / ft	14 k	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318-08 SEC.15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J}$$

$$A_P = 2(b_1 + b_2)d$$

$$\phi v_c(psi) = \phi(2 + y)\sqrt{f_c'}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right]$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

$$R = \frac{P_u b_1 b_2}{A_f}$$

$$A_f = \frac{\pi D^2}{4}$$

$$b_0 = \pi(d_{col} + d) \;,\; b_1 = b_2 = \frac{b_0}{4}$$

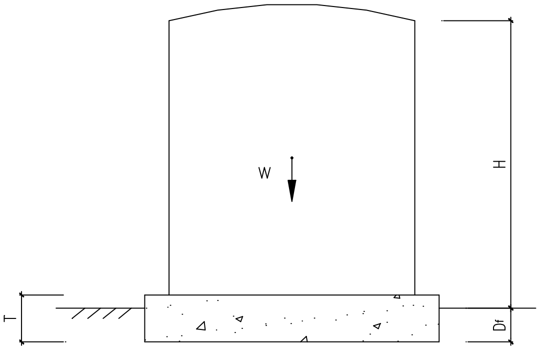
Case	P_u	M_u	b_1	b_2	b_0	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\,(psi)}$	$\phi\, v_c$
1	108.8	0.0	21.4	21.4	7.1	0.4	1.0	2.0	44.2	9.0	7.8	5.4	77.9	164.3
2	94.0	63.7	21.4	21.4	7.1	0.4	1.0	2.0	44.2	9.0	6.7	5.4	67.5	164.3
3	44.0	63.7	21.4	21.4	7.1	0.4	1.0	2.0	44.2	9.0	3.2	5.4	31.7	164.3

[Satisfactory]

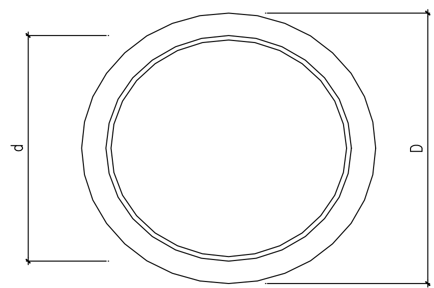
where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

Tank Footing Design Based on ACI 318-19, ASCE 7-16 & AWWA D103-19

INPUT DATA



TANK HEIGHT	H =	24.17	ft
TANK DIAMETER	d =	38.67	ft
TANK THICKNESS	t =	2	in
WT OF TANK & MAX CONTENTS	W =	1920.1	kips
SOIL WEIGHT	w_s =	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f =	1	ft
FOOTING THICKNESS	T =	18	in
ALLOW SOIL PRESSURE	Q_a =	2	ksf
FOOTING DIAMETER	D =	40.67	ft
TOTAL ANCHORAGE POINTS	n =	26	(@ 56" o.c. along perimeter.)
ANCHOR BOLT DIAMETER	ϕ =	3/4	in
CONCRETE STRENGTH	f_c' =	3	ksi
REBAR YIELD STRESS	f_y =	60	ksi
FOOTING REBAR	2 #	6	
	@	18	in o.c. each way, at top & bot.



DESIGN SUMMARY

FOOTING 40.67 ft DIA x 18 in THK. w/ #6 @ 18" o.c. EACH WAY, AT TOP & BOT.

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE LATERAL LOADS

$$T = 2\pi\sqrt{\frac{d}{3.68g\tanh\left(\frac{3.68H}{d}\right)}} =$$ 0.020 sec, (AWWA D103-19 Eq 14-14)

$$T = 7.65\times10^{-6}\left(\frac{L}{D}\right)^2\left(\frac{wD}{t}\right)^{1/2} =$$ 0.013 sec, (SEAOC IBC 06 Manual I, page 188)

< 0.06 sec
(rigid nonbuilding structure, ASCE 7-16, 15.4.2)

Where L = 24.17 ft w = W / L = 79439 plf D = 38.67 ft

$V = (S_{DS}\ I_E\ W / 1.4)$ 0.30 = 0.12 W = 221.35 kips, ASD **(for IBC, Seismic)**

Where S_{DS} = 0.538 (ASCE 7-16, 11.4.4)
I_E = 1.00 (IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)
Factor = 0.30 (ASCE 7-16, 15.4.2)

$V = (V_i + V_c) / 1.4 = (S_{DS}\ I_E\ W_i) / (1.4\ R) + (S_{DS}\ I_E\ W_c) / (1.4 \times 1.5) =$ 269.42 kips, ASD **(for ASCE 7-16 15.7.6, Seismic)**

Where R = 3 (ASCE 7-16, Table 15.4-2) *($S_{ai} = S_{ac} = S_{DS}$, conservatively assumed.)*
W_c = 183.23 kips, the portion of liquid weight sloshing $W_i = W - W_c =$ 1736.83 kips

$V = (C_a\ I_E\ W / 1.4)$ 0.7 = 0.14 W = 268.81 kips, ASD **(for UBC, Seismic)**

Where C_a = 0.28 (2001 CBC / UBC 97 1634.3)
I_E = 1.00 (2001 CBC / UBC 97 Table 16-K)
Factor = 0.7 (2001 CBC / UBC 97 1634.3)

$V = (2 / 3)\ P\ A =$ 0.01 W = 15.58 kips, ASD **(for Wind)**
Where A = 934.65 ft^2, (projected area)
P = 25 psf, (wind pressure)
Circular Factor = 2/3

CONSIDERING SLOSHING EFFECTS, USE 296.36 kips.

COMBINED LOADS AT TOP FOOTING (IBC 1605.2 & ACI 318 5.3)

CASE 1:	DL + LL	P	=	1920	kips	1.2 DL + 1.6 LL	Pu	=	2897	kips
		M	=	0	ft-kips		Mu	=	0	ft-kips
		e	=	0.0	ft, fr cl ftg		eu	=	0.0	ft, fr cl ftg
CASE 2:	DL + LL + E / 1.4	P	=	1920	kips	1.2 DL + 1.0 LL + 1.0 E	Pu	=	2007	kips
		M	=	3656	ft-kips		Mu	=	5118	ft-kips
		e	=	1.9	ft, fr cl ftg		eu	=	2.5	ft, fr cl ftg
CASE 3:	0.9 DL + E / 1.4	P	=	393	kips	0.9 DL + 1.0 E	Pu	=	393	kips
		M	=	831	ft-kips		Mu	=	1164	ft-kips
		e	=	2.1	ft, fr cl ftg		eu	=	3.0	ft, fr cl ftg

(cont'd)

CHECK OVERTURNING FACTOR AT WIND LOAD WITHOUT CONTENTS (IBC 1605.2.1, 1808.3.1, & ASCE 7 12.13.4)

M_R / M_O = 55.6 **>** 1.1667 **[Satisfactory]**

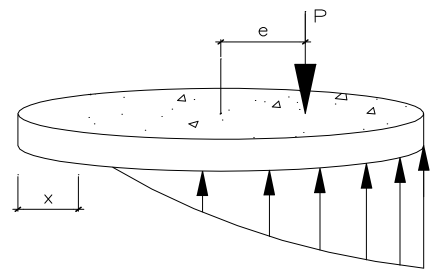
Where M_O = V_{wind} (2 / 3) (H + T) = 267 k-ft , M_R = (P_{DL} + P_{ftg}) 0.5 D = 14822 k-ft

P_{ftg} = (0.15 kcf) T D^2 π / 4 = 292.29 k, footing weight. F = 1.5, for wind

CHECK SOIL BEARING CAPACITY (ACI 318 13.3.1.1)

Service Loads	CASE 1	CASE 2	CASE 3	
P	1920.1	1920.1	392.9	k
e	0.0	2.1	2.9	ft (from center of footing)
P_{ftg} - P_{soil}	149.4	149.4	134.5	k, (footing increasing)
Σ P	2069.4	2069.4	527.4	k, (net loads)
e	0.0	1.9	2.1	ft
q_{min}	1.6	1.0	0.2	ksf
x		@ 0.00 ft from edge	@ 0.00 ft from edge	
q_{max}	1.59	2.22	0.58	ksf
$q_{allowable}$	2.00	2.67	2.67	ksf

[Satisfactory]



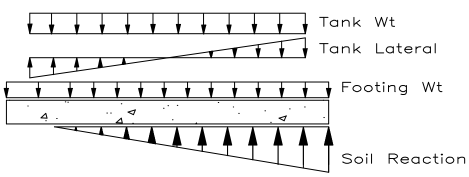
CHECK ENTIRE FLEXURE & SHEAR OF FOOTING
(ACI 318 13, 21, & 22)

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) \qquad \rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	2897.4	2007.4	392.9	k
e_u	0.0	2.8	4.0	ft
γ (0.15 T) A	350.8	350.8	263.1	k, (factored footingloads)
Σ P_u	3248.2	2358.1	656.0	k
e_u	0.0	2.3	2.4	ft
$q_{u, min}$	2.50	0.94	0.25	ksf
x		@ 0.00 ft from edge	@ 0.00 ft from edge	
$q_{u, max}$	2.50	2.69	0.76	ksf



FOOTING MOMENT & SHEAR FOR CASE **1**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	1.00	5.83	10.67	15.50	20.34	25.17	30.00	34.84	39.67	40.67
Tangent (ft)	0.00	12.60	28.51	35.78	39.50	40.67	39.50	35.78	28.51	12.60	0.00
$q_{u,tank}$ (ksf)	0.00	2.47	2.47	2.47	2.47	2.47	2.47	2.47	2.47	2.47	0.00
$M_{u,tank}$ (ft-k)	0	0	298	2,076	5,793	11,676	19,798	30,087	42,314	56,022	58,920
$V_{u,tank}$ (k)	0	0	62	368	769	1,680	2,129	2,530	2,836	2,897	2,897
$q_{u,ftg}$ (ksf)	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
$M_{u,ftg}$ (ft-k)	0	2	58	297	763	1,481	2,459	3,688	5,145	6,784	7,133
$V_{u,ftg}$ (k)	0	2	12	49	96	202	254	301	339	349	351
$q_{u,soil}$ (ksf)	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50
$M_{u,soil}$ (ft-k)	0	-15.907	-541.36	-2749.3	-7068.6	-13719	-22770	-34151	-47644	-62820	-66052
$V_{u,soil}$ (k)	0	-16	-109	-457	-894	-1,872	-2,355	-2,791	-3,139	-3,232	-3,248
Σ M_u (ft-k)	**0**	**-14**	**-185**	**-376**	**-513**	**-561**	**-513**	**-376**	**-185**	**-14**	**0**
Σ V_u (kips)	**0**	**-14**	**-35**	**-40**	**-28**	**10**	**28**	**40**	**35**	**14**	**0**

(cont'd)

FOOTING MOMENT & SHEAR FOR CASE **2**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	1.00	5.83	10.67	15.50	20.34	25.17	30.00	34.84	39.67	40.67
Tangent (ft)	0.00	12.60	28.51	35.78	39.50	40.67	39.50	35.78	28.51	12.60	0.00
$q_{u,tank}$ (ksf)	0.00	0.69	1.05	1.30	1.56	1.81	2.07	2.33	2.58	2.73	0.00
$M_{u,tank}$ (ft-k)	0	0	78.228	749.6	2386.7	5315	9795.3	15990	23906	33283	35287
$V_{u,tank}$ (k)	0	0	16	139	339	928	1,284	1,640	1,943	2,007	2,007
$q_{u,ftg}$ (ksf)	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
$M_{u,ftg}$ (ft-k)	0	2	58	297	763	1,481	2,459	3,688	5,145	6,784	7,133
$V_{u,ftg}$ (k)	0	2	12	49	96	202	254	301	339	349	351
$q_{u,soil}$ (ksf)	-0.94	-0.98	-1.19	-1.40	-1.61	-1.82	-2.02	-2.23	-2.44	-2.65	-2.69
$M_{u,soil}$ (ft-k)	0	-5.9898	-211.63	-1221.1	-3416.5	-7116.8	-12568	-19915	-29156	-40048	-42420
$V_{u,soil}$ (k)	0	-6	-43	-209	-454	-1,128	-1,520	-1,912	-2,253	-2,352	-2,358
Σ M_u (ft-k)	**0**	**-4**	**-75**	**-175**	**-267**	**-320**	**-314**	**-237**	**-105**	**-19**	**0**
Σ V_u (kips)	**0**	**-4**	**-15**	**-20**	**-18**	**3**	**18**	**30**	**29**	**4**	**0**

FOOTING MOMENT & SHEAR FOR CASE **3**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	1.00	5.83	10.67	15.50	20.34	25.17	30.00	34.84	39.67	40.67
Tangent (ft)	0.00	12.60	28.51	35.78	39.50	40.67	39.50	35.78	28.51	12.60	0.00
$q_{u,tank}$ (ksf)	0.00	0.04	0.15	0.22	0.29	0.36	0.44	0.51	0.58	0.63	0.00
$M_{u,tank}$ (ft-k)	0	0	4.7788	90.133	333.6	812.69	1595.9	2732.3	4237.4	6064.2	6411.6
$V_{u,tank}$ (k)	0	0	1	18	50	162	235	312	379	393	393
$q_{u,ftg}$ (ksf)	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
$M_{u,ftg}$ (ft-k)	0	1	44	223	572	1,111	1,844	2,766	3,859	5,088	5,349
$V_{u,ftg}$ (k)	0	1	9	37	72	152	191	226	254	262	263
$q_{u,soil}$ (ksf)	-0.25	-0.27	-0.33	-0.39	-0.45	-0.50	-0.56	-0.62	-0.68	-0.74	-0.76
$M_{u,soil}$ (ft-k)	0	-1.6264	-57.572	-334.14	-938.13	-1959.4	-3467.7	-5505	-8071.8	-11101	-11761
$V_{u,soil}$ (k)	0	-2	-12	-57	-125	-312	-421	-531	-627	-654	-656
Σ M_u (ft-k)	**0**	**0**	**-9**	**-21**	**-32**	**-36**	**-28**	**-7**	**24**	**51**	**0**
Σ V_u (kips)	**0**	**0**	**-2**	**-3**	**-2**	**2**	**5**	**7**	**6**	**0**	**0**

FOOTING MOMENT & SHEAR SUMMARY

		Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
		X_u (ft, dist. from left of footing)	0	1.00	5.83	10.67	15.50	20.34	25.17	30.00	34.84	39.67	40.67
		Tangent (ft)	0.00	12.60	28.51	35.78	39.50	40.67	39.50	35.78	28.51	12.60	0.00
Uniform Loads	**Case 1**	M_u, (ft-k / ft)	0.0	-1.1	-6.5	-10.5	-13.0	-13.8	-13.0	-10.5	-6.5	-1.1	0.0
		V_u, (k / ft)	0.0	-1.1	-1.2	-1.1	-0.7	0.2	0.7	1.1	1.2	1.1	0.0
	Case 2	M_u, (ft-k / ft)	0.0	-0.3	-2.6	-4.9	-6.7	-7.9	-7.9	-6.6	-3.7	-1.5	0.0
		V_u, (k / ft)	0.0	-0.3	-0.5	-0.6	-0.5	0.1	0.5	0.8	1.0	0.3	0.0
	Case 3	M_u, (ft-k / ft)	0.0	0.0	-0.3	-0.6	-0.8	-0.9	-0.7	-0.2	0.9	4.0	0.0
		V_u, (k / ft)	0.0	0.0	-0.1	-0.1	-0.1	0.0	0.1	0.2	0.2	0.0	0.0

CHECK FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	ρ_{provD}
Top Slab	4.0	ft-k / ft	15.63	0.0004	0.0003	0.0155	no limit	0.0016
Bottom Slab	-13.8	ft-k / ft	14.63	0.0016	0.0012	0.0155	18	0.0017

[Satisfactory]

CHECK FLEXURE SHEAR

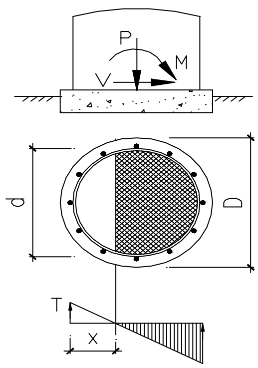
$V_{u,max}$	$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$	check $V_u < \phi\ V_c$
1.2 k / ft	14 k	**[Satisfactory]**

Technical References:

1. "Seismic Design Manual (UBC 97) - Volume 1, Code Application Examples", Structural Engineers Association of California, 1999.
2. "2006 IBC Structural/Seismic Design Manual - Volume 1, Code Application Examples", Structural Engineers Association of California, 2007.

Purchaser's LOGO (Please email us your LOGO at purchase.)	PROJECT :			PAGE :	
	CLIENT :			DESIGN BY :	
	JOB NO. :		DATE :	REVIEW BY :	

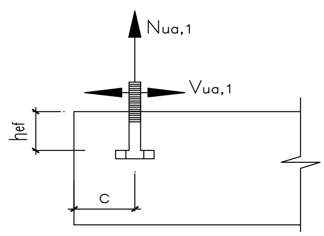
Tank Anchorage Design Based on ACI 318-19 & AWWA D103-19



INPUT DATA & DESIGN SUMMARY

SERVICE (ASD level) LOADS	P =	1920.1	*kips, (8541.0 kN), compression*
	M =	42885.4	*ft-kips, (58144.1 kN-m)*
	V =	296.36	*kips, (1318.3 kN), compression*
TANK DIAMETER	d =	38.67	*ft, (11.79 m)*
FOOTING DIAMETER	D =	40.67	*ft, (12.40 m)*
CONCRETE STRENGTH	f_c' =	5	*ksi, (34 MPa)*
SPECIFIED STRENGTH OF FASTENER	f_{uta} =	60	*ksi, (414 MPa)*

(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)



EFFECTIVE EMBEDMENT DEPTH	h_{ef} =	8	*in, (203 mm)*
FASTENER SPACING	s =	24	*in o.c., (610 mm)*
FASTENER DIAMETER	d_b =	1	*in, (25 mm)*
FASTENER HEAD TYPE		3	Hex

(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)

SEISMIC LOAD ? (ACI 318 17.2.3) Yes

THE ANCHORAGE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE MAX FACTORED DESIGN LOADS (LRFD level)

c = 10.50 in $N_{ua,1}$ = 1.5 x 5.15 kips = 7.72 k, *the max capacity*

x = Compression Control $V_{ua,1}$ = 1.5 x 4.85 kips = 7.27 k

M ***allowable*** = 80224.48 ft-k > M = 42885.434 ft-k [Satisfactory]

EFFECTIVE AREA OF FASTENER $A_{se,N}$ = 0.606 in^2

BEARING AREA OF HEAD A_b = 1.163 in^2, (or determined from manufacture's catalogs.)

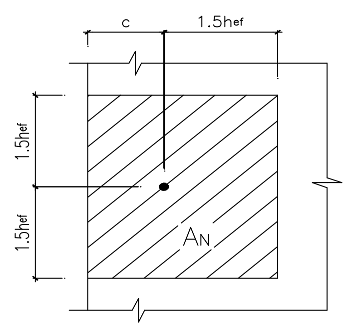
CHECK FASTENER TENSILE STRENGTH (ACI 318 17.4.1.2)

$$\phi N_{sa} = \phi n A_{se,N}\left(f_{uta}\right)$$ = 27.270 k > N_{ua} = 19.296 k [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75 , (ACI 318-19 17.2.3.4.4)

Ω_0 = 2.50 , ASCE 7-10 Tab. 12.2-1 or 15.4-1

CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318 17.4.2.1)



$$\phi N_{cb} = \phi \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{\left(9h_{ef}^2\right)} \left(0.7 + \frac{0.3c}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f_c'} h_{ef}^{1.5}\right)$$

= 19.491 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK PULLOUT STRENGTH : (ACI 318 17.4.3.1)

$$\phi N_{pn} = \phi \psi_{cp,N}\left(A_b 8 f_c'\right)$$ = 26.168 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK SIDE-FACE BLOWOUT STRENGTH : (ACI 318 17.4.4.1)

$$\phi N_{sb} = \phi\left(160 c_{a1}\sqrt{A_b}\sqrt{f_c'}\right)$$ = 72.062 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

DETERMINE DESIGN TENSILE STRENGTH :

$$\phi N_n = \min\left(\phi N_{sa}, \phi N_{cb}, \phi N_{pn}, \phi N_{sb}\right)$$ = 19.491 K

CHECK FASTENER SHEAR STRENGTH : (ACI 318 17.5.1.2)

$$\phi V_s = \phi n 0.6 A_{se,N} f_{ut}$$ = 14.180 k = V_{ua} = 14.180 k [Satisfactory]

where : ϕ = 0.65 x 1 = 0.65

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 17.5.1.3)

(Cont'd)

CHECK CONCRETE BREAKOUT STRENGTH FOR SHEAR LOAD : (ACI 318 17.5.1.2)

$$\phi V_{cb} = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} \left(7 \left(\frac{l}{d} \right)^{0.2} \sqrt{d} \sqrt{f'_c} c^{1.5} \right)$$ = 19.145 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

A_V / A_{Vo} and $\psi_{cd,V}$ terms are 1.0 for single shear fastener not influenced by more than one free edge.

l term is load bearing length of the anchor for shear, not to exceed 8d.

CHECK PRYOUT STRENGTH FOR SHEAR LOAD : (ACI 318 17.5.3.1)

$$\phi V_{cp} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{\left(9 h_{ef}^2\right)} \left(0.7 + \frac{0.3c}{1.5 h_{ef}} \right) \psi_{c,N} \left(24 \sqrt{f'_c} h_{ef}^{1.5} \right)$$

= 51.975 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

k_{cp} = 2.0 for $h_{ef} > 2.5$ in.

DETERMINE DESIGN SHEAR STRENGTH :

$$\phi V_n = \min\left(\phi V_{sa} \;, \phi V_{cb} \;, \phi V_{cp} \right)$$ = 14.180 K

CHECK TENSION AND SHEAR INTERACTION : (ACI 318 17.6)

Since $N_{ua,1}$ > $0.2\,\phi\,N_n$ and

$V_{ua,1}$ > $0.2\,\phi\,V_n$ the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua,1}}{\phi N_n} + \frac{V_{ua,1}}{\phi V_n} =$$ 0.91 < 1.2 [Satisfactory]

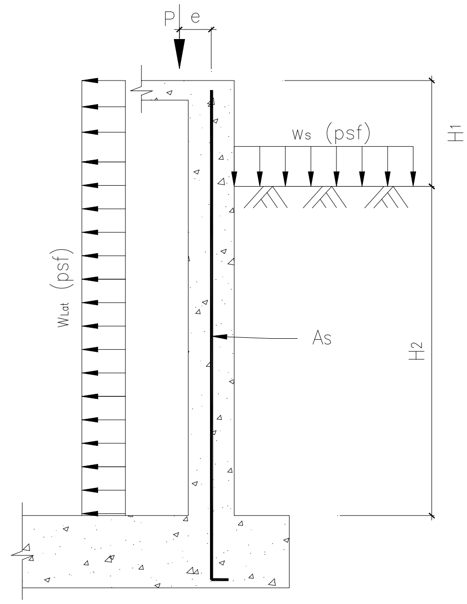
Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in^2)	Effective Area of Threaded Fastener (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Basement Concrete Wall Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a	=	45	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SURCHARGE WEIGHT	w_s	=	100	psf		
WALL LATERAL FORCE, ASD	w_{Lat}	=	25	psf		
SERVICE GRAVITY LOAD	P	=	20	kips / ft		
ECCENTRICITY	e	=	6	in		
SEISMIC GROUND SHAKING	P_E	=	48	psf / ft (for H > 12 ft, CBC 07 1806A.1)		
HEIGHT ABOVE GROUND	H_1	=	1.4	ft		
HEIGHT UNDER GROUND	H_2	=	12.5	ft		
THICKNESS OF WALL	t	=	10	in		
WALL VERT. REINF. (A_s)		#	5	@	18	in o.c.
A_s LOCATION (1=at middle, 2=at each face)			2	at each face		
WALL HORIZ. REINF.	2	#	5	@	18	in o.c.



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

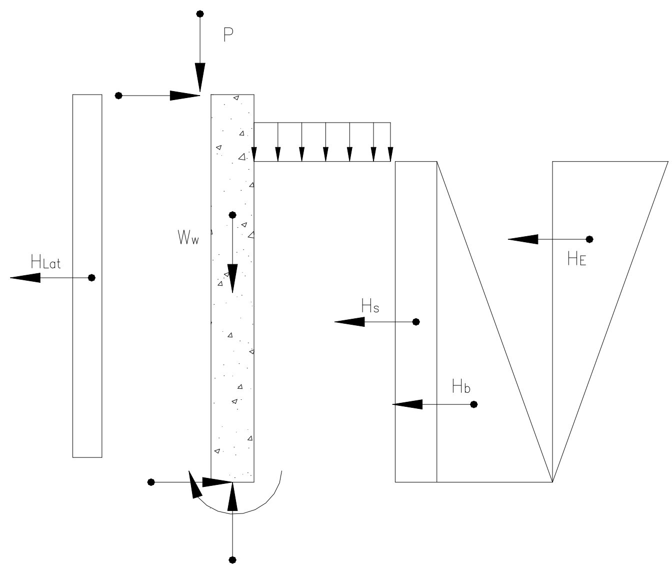
Case A: Fixed Bottom & Pinned Top, with Lateral Soil Pressure Increasing Uniformly to Bottom

SERVICE LOADS

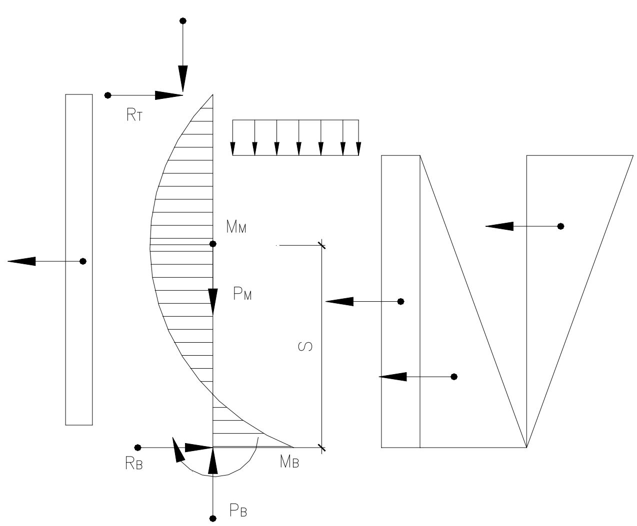
$H_b = 0.5\ P_a\ H_2^2$	=	3.52	kips / ft
$H_s = w_s\ P_a\ H_2 / \gamma_b$	=	0.51	kips / ft
$H_{Lat} = w_{Lat}\ (H_1 + H_2)$	=	0.35	kips / ft
$H_E = 0.5\ P_E\ (H_2)^2$	=	3.75	kips / ft
$W_w = t\ (H_1 + H_2)\ \gamma_c$	=	1.74	kips / ft

FACTORED LOADS

$\gamma H_b = 1.6\ H_b$	=	5.63	kips / ft
$\gamma H_s = 1.6\ H_s$	=	0.82	kips / ft
$\gamma H_{Lat} = 1.6\ H_{Lat}$	=	0.56	kips / ft
$\gamma H_E = 1.6\ H_E$	=	6.00	kips / ft
$\gamma W_w = 1.2\ W_w$	=	2.09	kips / ft
$\gamma P = 1.6\ P$	=	32.00	kips / ft



DETERMINE FACTORED SECTION FORCES

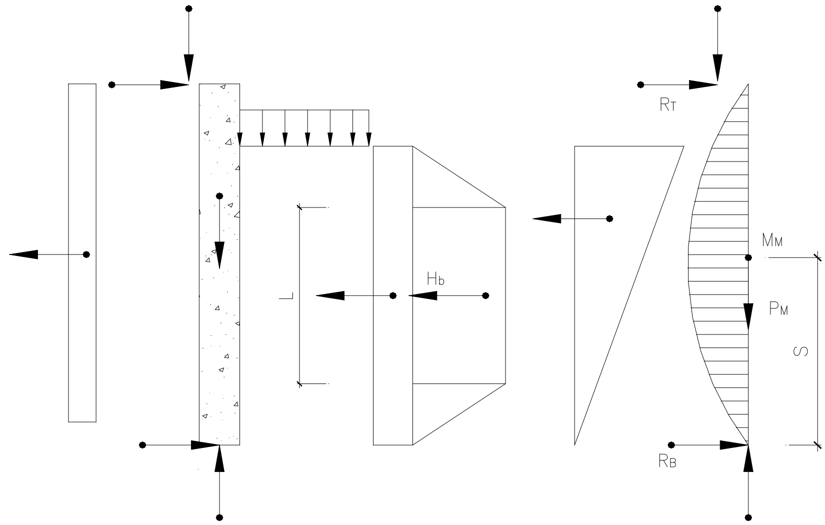


R_T =	5.89	kips / ft
R_B =	7.11	kips / ft
P_B =	34.09	kips / ft
M_B =	16.53	ft-kips / ft
S =	7.00	ft, at max moment
P_M =	33.04	kips / ft
M_M =	8.32	ft-kips / ft

(cont'd)

Case B: Pinned both Bottom & Top, with Lateral Soil Pressure Trapezium Distributed

L	=	0.6	H2
Hb	=	1.13	kips / ft
γHb	=	1.80	kips / ft
R_T	=	6.20	kips / ft
R_B	=	2.97	kips / ft
S	=	6.13	ft, at max moment
P_M	=	33.16	kips / ft
M_M	=	13.82	ft-kips / ft



CHECK MINIMUN HORIZ. REINF.

ρ_{ProvD} = 0.00344

> ρ_{MIN} = 0.002

(ACI 318-08 14.3.3)

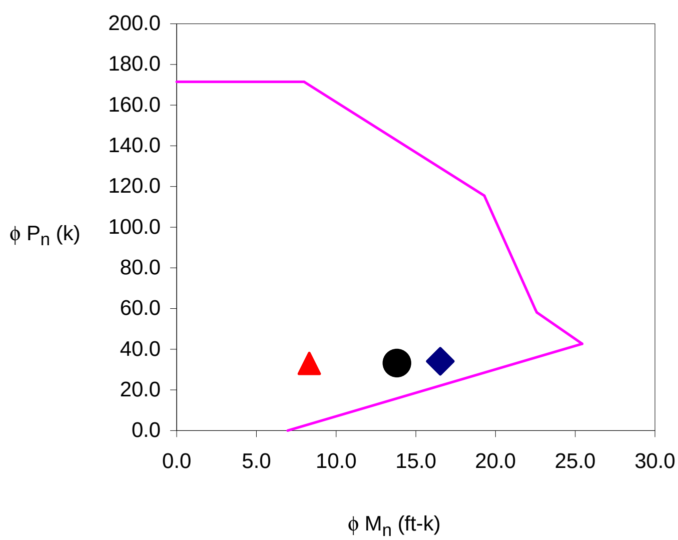
[Satisfactory]

CHECK VERT. FLEXURE CAPACITY

ρ_{ProvD} = 0.00224 **<** ρ_{MAX} = 0.04 (tension face only, ACI 318-05 10.3.5 or 10.9.1)

> ρ_{MIN} = 0.00075 (tension face only, ACI 318-05 10.5.1, 10.5.3 or 14.3.2)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	171.5	0.0
AT MAXIMUM LOAD	171.5	8.0
AT MIDDLE	115.5	19.3
AT ε_t = 0.002	59.5	22.5
AT BALANCED	58.1	22.6
AT ε_t = 0.005	42.7	25.4
AT FLEXURE ONLY	0.0	7.0

(Note: For middle reforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

	Case A		***Case B***
	at bottom	at middle	
P_u	34.09	33.04	33.16
M_u	16.53	8.32	13.82

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-08 SEC.15.5.2, 11.1.3.1, & 11.2)

$V_u = Max.\ \ Horiz.\ \ Shear$ = 7.11 kips, at bottom

$\phi V_n = 2\phi bd\sqrt{f'_c}$ = 7.58 kips **>** ***V_u*** **[Satisfactory]**

Daniel T. Li Engineering International	PROJECT :		PAGE :
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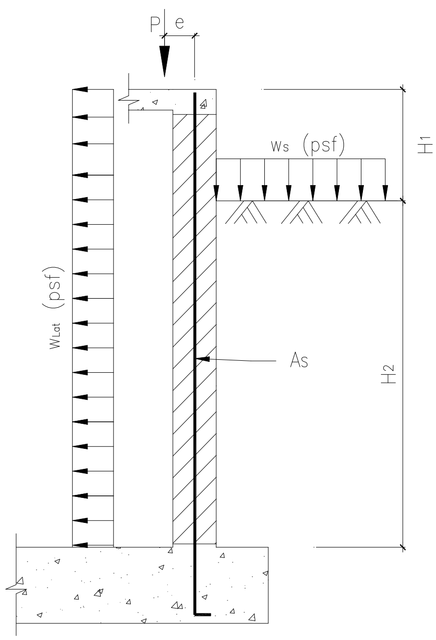
Basement Masonry Wall Design Based on TMS 402-08

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes		
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a	=	40	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SURCHARGE WEIGHT	w_s	=	100	psf		
WALL LATERAL FORCE, ASD	w_{Lat}	=	25	psf		
SERVICE GRAVITY LOAD	P	=	20	kips / ft		
ECCENTRICITY	e	=	6	in		
SEISMIC GROUND SHAKING	P_E	=	20	psf / ft (for H > 12 ft, CBC 07 1806A.1)		
HEIGHT ABOVE GROUND	H_1	=	1.4	ft		
HEIGHT UNDER GROUND	H_2	=	12.5	ft		
THICKNESS OF WALL	t	=	12	in		
WALL VERT. REINF. (A_s)		#	8	@	8	in o.c.
A_S LOCATION (1=at middle, 2=at each face)			2	at each face		
WALL HORIZ. REINF.	2	#	4	@	32	in o.c.



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

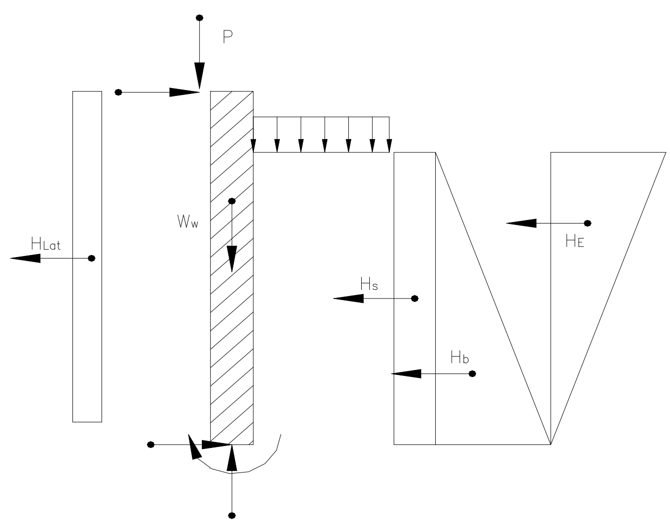
Case A: Fixed Bottom & Pinned Top, with Lateral Soil Pressure Increasing Uniformly to Bottom

SERVICE LOADS

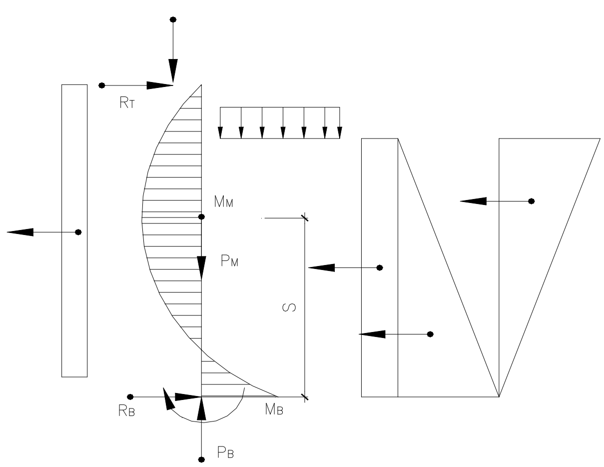
$H_b = 0.5\, P_a\, H_2^2$	=	3.13	kips / ft
$H_s = w_s\, P_a\, H_2 / \gamma_b$	=	0.45	kips / ft
$H_{Lat} = w_{Lat}\,(H_1 + H_2)$	=	0.35	kips / ft
$H_E = 0.5\, P_E\, (H_2)^2$	=	1.56	kips / ft
$W_w = t\,(H_1 + H_2)\,\gamma_m$	=	1.81	kips / ft

ALLOWABLE STRESS DESIGN LOADS

$\gamma H_b = 1.0\, H_b$	=	3.13	kips / ft
$\gamma H_s = 1.0\, H_s$	=	0.45	kips / ft
$\gamma H_{Lat} = 1.0\, H_{Lat}$	=	0.35	kips / ft
$\gamma H_E = 1.0\, H_E$	=	1.56	kips / ft
$\gamma W_w = 1.0\, W_w$	=	1.81	kips / ft
$\gamma P = 1.0\, P$	=	20.00	kips / ft



DETERMINE FACTORED SECTION FORCES

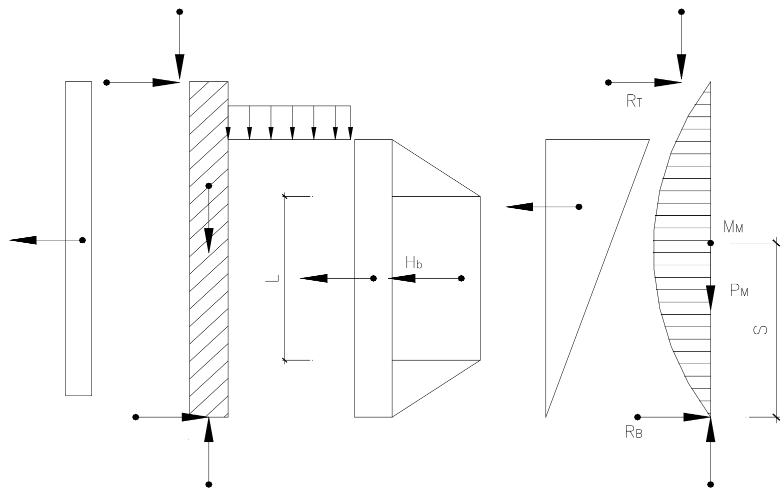


R_T	=	2.59	kips / ft
R_B	=	2.90	kips / ft
P_B	=	21.81	kips / ft
M_B	=	5.29	ft-kips / ft
S	=	7.25	ft, at max moment
P_M	=	20.86	kips / ft
M_M	=	0.47	ft-kips / ft

(cont'd)

Case B: Pinned both Bottom & Top, with Lateral Soil Pressure Trapezium Distributed

L = 0.6 H_2
H_b = 1.00 kips / ft
γH_b = 1.00 kips / ft
R_T = 2.48 kips / ft
R_B = 0.88 kips / ft
S = 3.82 ft, at max moment
P_M = 21.31 kips / ft
M_M = 3.78 ft-kips / ft



CHECK MINIMUN HORIZ. REINF.

ρ_{ProvD} = 0.00104
> ρ_{MIN} = 0.0007
(TMS 402-08 1.13.6.3)
[Satisfactory]

CHECK VERT. FLEXURE CAPACITY (TMS 402 2.3.3)

ρ_{ProvD} = 0.01039 **>** ρ_{SUM} = 0.0010 (horizontal and vertical at least 0.002, TMS 402-08 1.13.6.3)
> ρ_{MIN} = 0.0007 (TMS 402-08 1.13.6.3)
[Satisfactory]

$$M_m = MIN\left[\frac{1}{2}b_w kd F_b\left(d-\frac{kd}{3}\right)-P\left(d-\frac{t_e}{2}\right)\ ,\ A_s F_s\left(d-\frac{kd}{3}\right)+P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right]$$, allowable moment

Loads	Case A		Case B
	at bottom	at middle	
P	21.81	20.86	21.31
M	5.29	0.47	3.78
M_m	5.30	5.58	5.44

where

A_s = 1.185 in^2 E_s = 29000 ksi
d = 9.380 in n = 21.5
b_w = 12 in ρ = 0.01053
t_e = 11.63 in   SF = 1.33
E_m = 1350 ksi k = 0.4834
F_b = 0.66 ksi F_s = 32 ksi

[Satisfactory]

CHECK SHEAR CAPACITY (TMS 402 2.3.5)

$$f_v = \frac{(Max.\ \ Horiz.\ \ Shear)}{b_w t_e} =$$ 20.8 psi, at bottom

< $F_v = (SF)\,MIN\left(\sqrt{f'_m}\ ,\ 50\right) =$ 51.6 psi **[Satisfactory]**

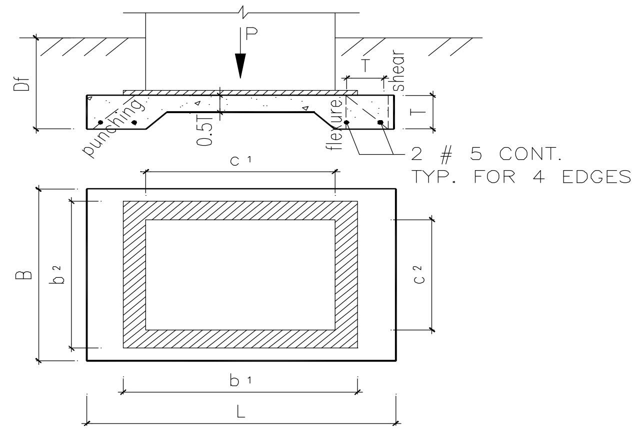
Temporary Tank Footing Design Based on ACI 318-08

INPUT DATA

TANK SIZE	c_1	=	40	ft
	c_2	=	20	ft
BASE PLATE EDGE SIZE	b_1	=	40.5	ft
	b_2	=	20.5	ft
FOOTING CONCRETE STRENGTH	fc'	=	2	ksi
REBAR YIELD STRESS	fy	=	60	ksi
TOTAL DEAD LOAD	P_{DL}	=	23	kips
LIVE LOAD	P_{LL}	=	1450	kips
SURCHARGE	qs	=	0.1	ksf
SOIL WEIGHT	ws	=	0.11	pcf
FOOTING EMBEDMENT DEPTH	Df	=	0.50	ft
FOOTING EDGE THICKNESS	T	=	10	in
ALLOW SOIL PRESSURE	Qa	=	2	ksf
FOOTING WIDTH	B	=	22	ft
FOOTING LENGTH	L	=	42	ft

DESIGN SUMMARY

FOOTING WIDTH	B	=	22.00	ft
FOOTING LENGTH	L	=	42.00	ft
FOOTING EDGE THICKNESS	T	=	10	in



THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS (ACI 318-08 SEC.9.2.1)

ASD:	DL + LL	P	=	1582	kips
SD:	1.2 DL + 1.6 LL	Pu	=	2189	kips

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

$$q_{MAX} = \frac{P}{BL}$$ = ASD 1.71 ksf, < Q_a **[Satisfactory]**

DESIGN FOR FLEXURE (ACI 318-08 SEC.22.5.1)

$$\phi M_n = MIN\left(5\phi\sqrt{f'_c}S,\ \ 0.85\phi f'_c S\right)$$ = 49.19 ft-kips

where ϕ = 0.6 (ACI 318-08, Section 9.3.5)
S = elastic section modulus of section = 4400 in^3

$$M_u = \frac{(0.5L - 0.25b_1 - 0.25c_1)^2 P_{u,\max}}{2L}$$ = 19.95 ft-kips < $\phi\ M_n$ **[Satisfactory]**

CHECK FLEXURE SHEAR (ACI 318-08 SEC.22.5.4)

$$\phi V_n = \frac{4}{3}\phi\sqrt{f'_c}BT$$ = 94.45 kips

where ϕ = 0.6 (ACI 318-08, Section 9.3.5)

$$V_u = (0.5L - 0.25b_1 - 0.25c_1 - T)\frac{P_{u,\max}}{L}$$ = 2.17 kips < $\phi\ V_n$ **[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318-08 SEC.22.5.4)

$$\phi V_n = MIN\left[\left(\frac{4}{3} + \frac{8}{3\beta_c}\right),\ \ 2.66\right]\phi\sqrt{f'_c}\,(c_1 + c_2 + b_1 + b_2 + 4T)T$$ = 1064.92 kips

where ϕ = 0.6 (ACI 318-08, Section 9.3.5)
β_c = ratio of long side to short side of concentrated load = 1.99

$$V_u = P_{u,\max}\left[1 - \frac{1}{BL}\left(\frac{b_1 + c_1}{2} + T\right)\left(\frac{b_2 + c_2}{2} + T\right)\right]$$ = 136.98 ft-kips < $\phi\ V_n$ **[Satisfactory]**

Daniel T. Li
Engineering International

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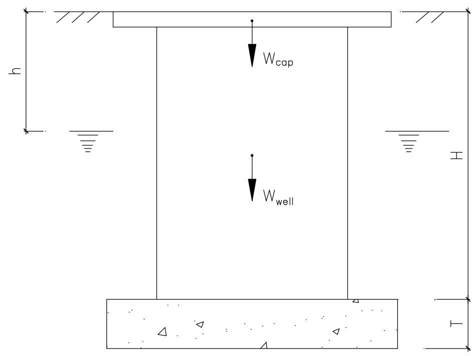
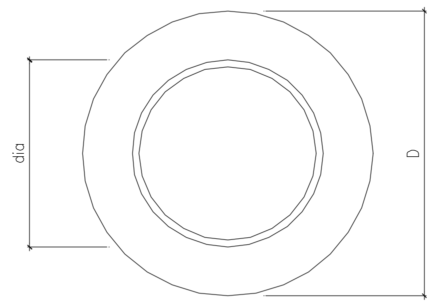
UnderGround Well Design Based on ACI 350-06 & ACI 318-08

INPUT DATA

WELL DEPTH	H =	28.5	ft
WELL OUTSIDE DIAMETER	dia =	8.3333	ft
WELL CONCRETE THICKNESS	t =	8	in
FOOTING DIAMETER	D =	16	ft
FOOTING THICKNESS	T =	18	in
WATER TABLE LOCATION	h =	0	ft
WT OF WELL & MAX CONTENTS	W_{well} =	171.61	kips
WT OF TOP CAP	W_{cap} =	18.869	kips
ALLOWABLE SOIL PRESSURE	Q_a =	3	ksf
SOIL WEIGHT	γ_s =	0.11	kcf
LATERAL SOIL PRESSURE	P_a =	45	pcf (equivalent fluid pressure)
PASSIVE PRESSURE	P_p =	450	psf / ft
SEISMIC GROUND SHAKING	P_E =	30	psf / ft
CONCRETE STRENGTH	f_c' =	4	ksi
REBAR YIELD STRESS	f_y =	60	ksi

FOOTING REBAR 2 Layers # 6
@ 16 in o.c. each way, at top & bot.
DOWEL / WELL VERTICAL REBARS # 5 @ 8 in o.c.
DOWEL LOCATION (1=at middle, 2=at each face) 2 at each face
WELL HORIZONTAL REBARS # 4 @ 18 in o.c.





DESIGN SUMMARY

THE FOOTING DESIGN IS ADEQUATE.

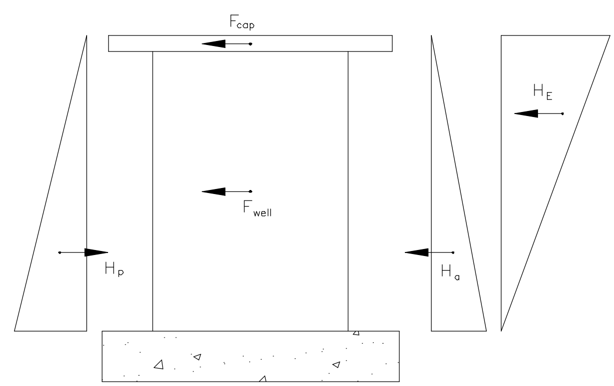
FOOTING 16 ft DIA x 18 in THK. w/ # 6 @ 16" o.c. EACH WAY, AT TOP & BOT.
CONCRETE WELL 8 in THK. w/ # 5 @ 8 in o.c.DOWEL / VERT. BARS AT AT EACH FACE

ANALYSIS

DETERMINE LATERAL LOADS

F = (S_{DS} I_E W / 1.4) 0.30 = 0.12 W, ASD **(for IBC, Seismic)**

Where S_{DS} = 0.538 (ASCE 7-05, 11.4.4)
I_E = 1.00 (IBC 09 Tab 1604.5 & ASCE 7-05 Tab 11.5-1)
Factor = 0.30 , (ASCE 7-05, 15.4.2)



F_{cap} = 0.12 W_{cap} = 2.18 kips, at ground
F_{well} = 0.12 W_{well} = 19.78 kips, at mid of H
H_a = 0.5 P_a H^2 (2/3 d) = 101.53 kips, at 1/3 H
H_E = 0.5 P_E H^2 (2/3 d) = 67.69 kips, at 2/3 H
H_p = MIN [0.5 P_p H^2 (2/3 d) , Σ(F + H)] = 191.18 kips, at 1/3 H

V = 0.00 kips, total shear at top of footing
M = 778 ft-kips, total moment at top of footing

COMBINED LOADS AT TOP FOOTING (IBC 1605.3.2 & ACI 318-08 9.2.1)

CASE 1:	DL + LL	P	=	190	kips	1.2 DL + 1.6 LL	Pu	=	269	kips
		M	=	0	ft-kips		Mu	=	0	ft-kips
		e	=	0.0	ft, fr cl ftg		eu	=	0.0	ft, fr cl ftg
CASE 2:	DL + LL + E / 1.4	P	=	190	kips	1.2 DL + 1.0 LL + 1.0 E	Pu	=	175	kips
		M	=	778	ft-kips		Mu	=	1090	ft-kips
		e	=	4.1	ft, fr cl ftg		eu	=	6.2	ft, fr cl ftg
CASE 3:	0.9 DL + E / 1.4	P	=	13	kips	0.9 DL + 1.0 E	Pu	=	13	kips
		M	=	755	ft-kips		Mu	=	1057	ft-kips
		e	=	56.2	ft, fr cl ftg		eu	=	78.7	ft, fr cl ftg

CHECK OVERTURNING FACTOR AT FOOTING EDGE BOTTOM (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 2.4 **>** 1.5 **[Satisfactory]**

Where M_O = M + V T = 778 k-ft , M_R = Σ(W) 0.5 D = 1886 k-ft

W_{ftg} = (0.15 kcf) T D^2 π / 4 = 45.24 kips, footing weight.

(cont'd)

CHECK UPLIFT CAPACITY UNDER MINIMUM CONTENTS & TOP OPENING

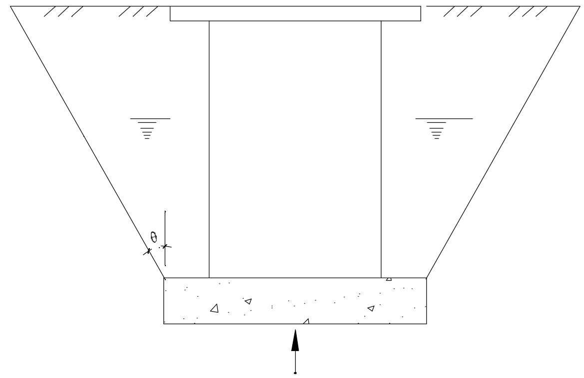
$W_{Gravity} / F_{Uplift}$ = 43.784 **>** 1.5 **[Satisfactory]**

Where F_{Uplift} = $(d^2 \pi / 4)\ (H + T - h)\ \gamma_{water}$ = 102.1 kips

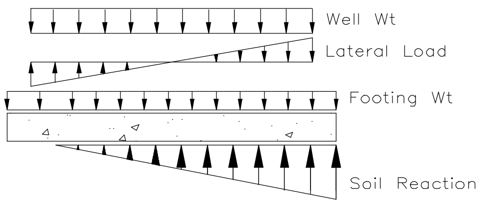
θ = 30 ^{0} , from soil report

W_{soil} = 2645 kips, cone soil weight

$W_{Gravity} = W_{ftg} + W_{soil} + W_{concrete}$ = 4470 kips

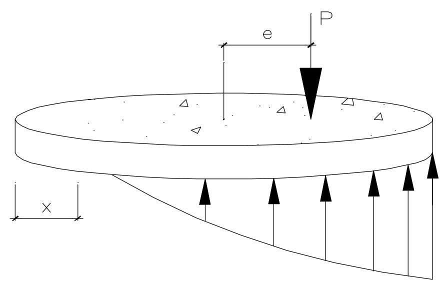


CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)



Service Loads	CASE 1	CASE 2	CASE 3	
P	190.5	190.5	13.4	k
e	0.0	4.1	56.2	ft (from center of footing)
$P_{ftg} - P_{soil}$	12.1	12.1	10.9	k, (footing increasing)
Σ P	202.5	202.5	24.3	k, (net loads)
e	0.0	3.8	31.1	ft
q_{min}	1.0	0.0	0.0	ksf
x		@ 4.80 ft from edge	@ 8.00 ft from edge	
q_{max}	1.01	3.47	3.85	ksf
$q_{allowable}$	3.00	4.00	4.00	ksf

[Satisfactory]



CHECK ENTIRE FLEXURE & SHEAR OF FOOTING

(ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) \qquad \rho_{MAX} = \frac{0.85\beta_1 f_c^{'}}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho = \frac{0.85 f_c^{'}\left(1-\sqrt{1-\frac{M_u}{0.383 b d^2 f_c^{'}}}\right)}{f_y}$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	268.6	174.6	13.4	k
e_u	0.0	6.2	78.7	ft
γ (0.15 T) A	54.3	54.3	40.7	k, (factored footingloads)
Σ P_u	322.9	228.9	54.1	k
e_u	0.0	4.8	19.5	ft
$q_{u, min}$	1.61	0.00	0.00	ksf
x		@ 6.40 ft from edge	@ 8.00 ft from edge	
$q_{u, max}$	1.61	4.93	5.32	ksf

FOOTING MOMENT & SHEAR FOR CASE **1**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	3.83	4.88	5.92	6.96	8.00	9.04	10.08	11.13	12.17	16.00
Tangent (ft)	0.00	13.66	14.73	15.45	15.86	16.00	15.86	15.45	14.73	13.66	0.00
$q_{u,tank}$ (ksf)	0.00	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	0.00
$M_{u,tank}$ (ft-k)	0	0	21	67	149	269	429	627	860	1,119	2,149
$V_{u,tank}$ (k)	0	0	20	45	78	153	190	224	249	269	269
$q_{u,ftg}$ (ksf)	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
$M_{u,ftg}$ (ft-k)	0	24	38	56	79	105	135	169	208	250	434
$V_{u,ftg}$ (k)	0	6	14	18	21	29	33	37	40	48	54
$q_{u,soil}$ (ksf)	-1.61	-1.61	-1.61	-1.61	-1.61	-1.61	-1.61	-1.61	-1.61	-1.61	-1.61
$M_{u,soil}$ (ft-k)	0	-139.95	-226.35	-335.03	-467.1	-623.16	-803.44	-1007.7	-1235.4	-1485.3	-2583.1
$V_{u,soil}$ (k)	0	-37	-83	-104	-127	-173	-196	-219	-240	-286	-323
Σ M_u (ft-k)	**0**	**-116**	**-168**	**-211**	**-240**	**-250**	**-240**	**-211**	**-168**	**-116**	**0**
Σ V_u (kips)	**0**	**-30**	**-49**	**-42**	**-27**	**9**	**27**	**42**	**49**	**30**	**0**

(cont'd)

FOOTING MOMENT & SHEAR FOR CASE **2**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	3.83	4.88	5.92	6.96	8.00	9.04	10.08	11.13	12.17	16.00
Tangent (ft)	0.00	13.66	14.73	15.45	15.86	16.00	15.86	15.45	14.73	13.66	0.00
$q_{u,tank}$ (ksf)	0.00	-16.90	25.12	30.14	35.17	40.19	45.22	50.25	55.27	23.31	0.00
$M_{u,tank}$ (ft-k)	0	0	-1.587	-0.151	6.0352	18.418	38.115	65.779	101.36	143.6	307.08
$V_{u,tank}$ (k)	0	0	-6	6	24	77	109	140	166	175	175
$q_{u,ftg}$ (ksf)	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
$M_{u,ftg}$ (ft-k)	0	24	38	56	79	105	135	169	208	250	434
$V_{u,ftg}$ (k)	0	6	14	18	21	29	33	37	40	48	54
$q_{u,soil}$ (ksf)	0.00	0.00	0.00	0.000	-0.29	-0.82	-1.36	-1.89	-2.43	-2.96	-4.93
$M_{u,soil}$ (ft-k)	0	0	0	0	0	-5.4543	-26.673	-73.698	-155.76	-280.68	-741.37
$V_{u,soil}$ (k)	0	0	0	0	0	-20	-45	-79	-120	-229	-229
Σ M_u (ft-k)	**0**	**24**	**36**	**56**	**85**	**118**	**147**	**162**	**153**	**113**	**0**
Σ V_u (kips)	**0**	**6**	**8**	**23**	**46**	**86**	**97**	**98**	**86**	**-6**	**0**

FOOTING MOMENT & SHEAR FOR CASE **3**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	3.83	4.88	5.92	6.96	8.00	9.04	10.08	11.13	12.17	16.00
Tangent (ft)	0.00	13.66	14.73	15.45	15.86	16.00	15.86	15.45	14.73	13.66	0.00
$q_{u,tank}$ (ksf)	0.00	-19.26	21.51	26.39	31.26	36.14	41.02	45.90	50.77	19.75	0.00
$M_{u,tank}$ (ft-k)	0	0	-0.1566	-0.089	0.3388	1.2438	2.7185	4.8192	7.5464	10.803	-949.9
$V_{u,tank}$ (k)	0	0	-1	0	2	6	8	11	13	13	13
$q_{u,ftg}$ (ksf)	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
$M_{u,ftg}$ (ft-k)	0	18	29	42	59	79	101	127	156	187	326
$V_{u,ftg}$ (k)	0	5	10	13	16	22	25	28	30	36	41
$q_{u,soil}$ (ksf)	0.00	0.00	0.00	0.00	0.00	0.00	-0.69	-1.38	-2.08	-2.77	-5.32
$M_{u,soil}$ (ft-k)	0	0	0	0	0	0	-4E-15	-4.0895	-16.144	-39.588	624.18
$V_{u,soil}$ (k)	0	0	0	0	0	0	-4	-12	-23	-54	-54
Σ M_u (ft-k)	**0**	**18**	**28**	**42**	**59**	**80**	**104**	**128**	**147**	**159**	**0**
Σ V_u (kips)	**0**	**5**	**10**	**13**	**18**	**28**	**29**	**27**	**21**	**-5**	**0**

FOOTING MOMENT & SHEAR SUMMARY

		Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
		X_u (ft, dist. from left of footing)	0	3.83	4.88	5.92	6.96	8.00	9.04	10.08	11.13	12.17	16.00
		Tangent (ft)	0.00	13.66	14.73	15.45	15.86	16.00	15.86	15.45	14.73	13.66	0.00
Uniform Loads	**Case 1**	M_u, (ft-k / ft)	0.0	-8.5	-11.4	-13.7	-15.1	-15.6	-15.1	-13.7	-11.4	-8.5	0.0
		V_u, (k / ft)	0.0	-2.2	-3.3	-2.7	-1.7	0.6	1.7	2.7	3.3	2.2	0.0
	Case 2	M_u, (ft-k / ft)	0.0	1.7	2.5	3.6	5.3	7.4	9.2	10.5	10.4	8.2	0.0
		V_u, (k / ft)	0.0	0.4	0.5	1.5	2.9	5.4	6.1	6.3	5.9	-0.4	0.0
	Case 3	M_u, (ft-k / ft)	0.0	1.3	1.9	2.7	3.7	5.0	6.6	8.3	10.0	11.6	0.0
		V_u, (k / ft)	0.0	0.3	0.7	0.9	1.1	1.7	1.8	1.7	1.4	-0.3	0.0

CHECK FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	ρ_{provD}
Top Slab	11.6	ft-k / ft	15.63	0.0012	0.0009	0.0206	no limit	0.0018
Bottom Slab	-15.6	ft-k / ft	14.63	0.0018	0.0014	0.0206	18	0.0019

[Satisfactory]

CHECK FLEXURE SHEAR

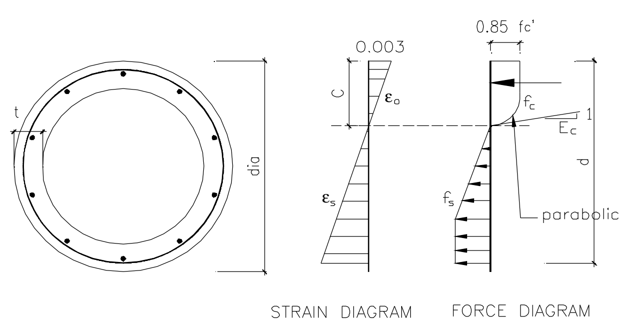
$V_{u,max}$	$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	check $V_u < \phi\, V_c$
6.3 k / ft	17 k	**[Satisfactory]**

CHECK WELL CONCRETE FLEXURAL & AXIAL CAPACITY

$$\varepsilon_O = \frac{2\left(0.85 f_C^{'}\right)}{E_C} \quad , \quad E_C = 57\sqrt{f_C^{'}} \quad , \quad E_S = 29000 ksi$$

$$f_C = \begin{cases} 0.85 f_C^{'}\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right] \; , & for \;\; 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f_C^{'} \; , & for \;\; \varepsilon_C \ge \varepsilon_O \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S \; , & for \;\; \varepsilon_S \le \varepsilon_y \\ f_y \; , & for \;\; \varepsilon_S > \varepsilon_y \end{cases}$$



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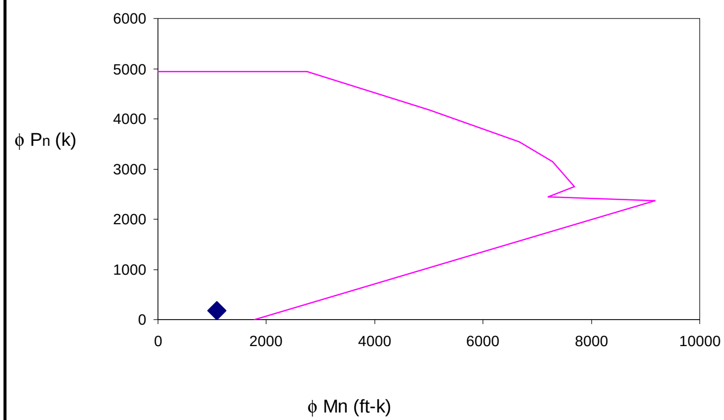


	φ Pn (kips)	φ Mn (ft-kips)
AT AXIAL LOAD ONLY	4939	0
AT MAXIMUM LOAD	4939	2748
AT 0 % TENSION	4184	5008
AT 25 % TENSION	3543	6674
AT 50 % TENSION	3151	7288
AT ε_t = 0.002	2644	7682
AT BALANCED CONDITION	2449	7206
AT ε_t = 0.005	2365	9175
AT FLEXURE ONLY	0	1779

P_u = 175 kips
M_u = 1090 ft-kips, max at bottom
(from load combinations)

$\phi P_{max} = 0.85\ \phi\ [\,0.85 f_c' (A_g - A_{st}) + f_y A_{st}]$ = 4938.6 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.1)
where ϕ = 0.70 (ACI 318-08, Sec.9.3.2.2) > P_u **[Satisfactory]**
A_g = 2312.2 in^2. in^2. A_{st} = 7.75 in^2.
$a = C_b \beta_1$ 49 in (at balanced strain condition, ACI 10.3.2)

ϕ = 0.7 + (ε_t - 0.002) (200 / 3), for Spiral
0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-08, Fig. R9.3.2)

where C_b = d ε_c / (ε_c + ε_s) = 58 in ε_t = 0.002069 ε_c = 0.003
d = 97.563 in, (ACI 7.7.1) β_1 = 0.85 (ACI 318-08, Sec. 10.2.7.3)
$\phi M_n = 0.9 M_n$ = 1779 ft-kips @ Pn = 0, (ACI 318-08, Sec. 9.3.2) ,& et,min = 0.004, (ACI 318-08, Sec. 10.3.5)
ϕM_n = 2325 ft-kips @ P_u = 175 kips > M_u **[Satisfactory]**
ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.003
ρ_{min} = 0.003 (ACI 318-08, Section 10.5.1 or 10.9.1) **[Satisfactory]**

CHECK WELL CONCRETE SHEAR CAPACITY

$\phi V_n = \phi\ (V_c)$ = 183 kips, (ACI 318-08 Sec. 11.1.1)
> V_u = 1.4 V = 3.0455 kips, max at top **[Satisfactory]**
where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3)
A_0 = 1934 in^2.
V_c = 2 $(f_c')^{0.5} A_0$ = 244.6 kips, (ACI 318-08 Sec. 11.2.1)

CHECK WELL LOCAL SHEAR STRESS ON A SQUARE FOOT

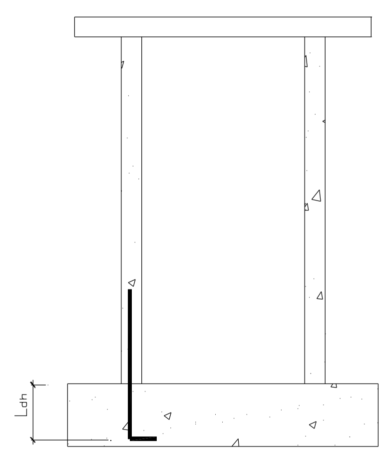
$\phi v_n = \phi\ (v_c)$ = 40.73 kips, (ACI 318-08 Sec. 11.1.1)
> v_u = 1.4 P_{max} = 3.38 kips **[Satisfactory]**
where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3)
A_0 = 4 x (1'-0") x (0.5 T) = 192 in^2.
v_c = 4 $(f_c')^{0.5} A_0$ = 54.3 kips, (ACI 318-08 Sec. 11.11)
P_{max} = 2415 psf, (the max perpendicular wall pressure)

CHECK DOWEL DEVELOPMENT

$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f_c'}}\ ,\ 8d_b\ ,\ 6\ in\right) =$$ 13 d_b = 8 in, (ACI 318-08 12.5.2) < 14.375 in

[Satisfactory]

where Bar size # 5
d_b = 0.625 in
$\rho_{required} / \rho_{provided}$ = 1 (A s,reqd / A s,provd , ACI 318-08, 12.2.5)
f_y = 60 ksi
f'_c = 4 ksi
ψ_t = 1.0
ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-08 12.2.4)
ψ_s = 0.8 (0.8 for # 6 or smaller, 1.0 for other)
λ = 1.0
c = 3.3 in, min(d' , 0.5s), (ACI 318-08, 12.2.4)
K_{tr} = ($A_{tr} f_{yt}$ / 1500 s n) = 0 (ACI 318-08, 12.2.3)
(c + K_{tr}) / d_b = 2.5 < 2.5 , (ACI 318-08, 12.2.3)
η = 0.7 (#11 or smaller, cover > 2.5" & side >2.0", ACI 318-08 12.5.3)



Daniel T. Li
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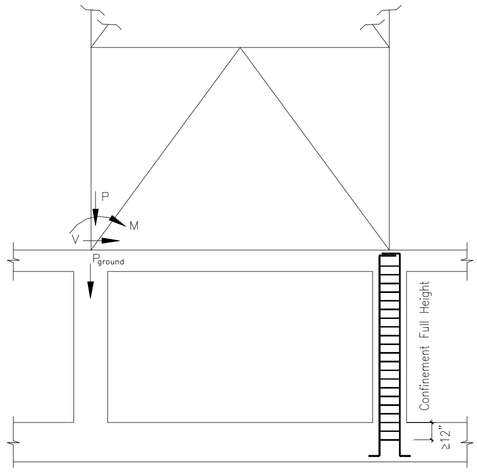
PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Basement Column Supporting Lateral Resisting Frame Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	4.5	ksi
REBAR YIELD STRESS	f_y =	60	ksi
COLUMN CLEAR HEIGHT	h =	14	ft
COLUMN SIZE	c_1 =	24	in
	c_2 =	24	in
AMPLIFICATION FACTOR (ASCE 7 Tab 12.2-1)	Ω_0 =	2.2	
DESIGN LEVEL BASEMENT DISPLACEMENT	Δ_S =	0	in



LOADS, ASD (ft-kips, kips)

	P	V	M	P_{ground}
DL	225			157.5
LL	270			90
E / 1.4	-333.3	131	0.1	

(- P for uplift)

LONGITUDINAL REINFORCING

SECTION	TOP	BOTTOM
LEFT	5 # 10 (d = 21.37 in) (1 Layer)	5 # 10 (d = 21.37 in) (1 Layer)
RIGHT	5 # 10 (d = 21.37 in) (1 Layer)	5 # 10 (d = 21.37 in) (1 Layer)

THE COLUMN DESIGN IS ADEQUATE.

TRANSVERSE REINFORCEMENT FOR CONFINEMENT

4 Legs # 4 @ 5 in, o.c., full height (ACI 318-08 4.4.5)

ANALYSIS

DESIGN CRITERIA

1. Since the column supported reaction from lateral resisting frame, ASCE 7-05 12.3.3.3 apply.
2. Since the column is not part of the lateral force resisting system, ACI 318-08 21.13 apply.
3. Since the transverse reinforcement required 12" at least into footing per ACI 318-08 21.6.4.6 a condition of pinned top & fixed bottom should be used.

DESIGN LOADS AT TOP OF COLUMN

$U_1 = (1.2\pm0.2S_{DS})\ D + f_1\ L + 1.0\ \Omega_0\ E_h$, (ACI 318-05 21.13)

P_u = -527.9 kips M_u = 0.3 ft-kips

V_u = 403.5 kips f_1 = 0.5

S_{DS} = 1.246

$U_2 = (0.9\pm0.2S_{DS})\ D \pm 1.0\ \Omega_0\ E_h$

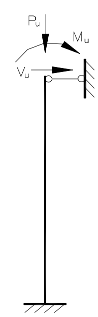
P_u = -777.6 kips M_u = 0.3 ft-kips

V_u = 403.5 kips

$U_3 = 1.2\ D + 1.6\ L$, (ACI 318-05 9.2.1)

P_u = 891.0 kips M_u = 0.0 ft-kips

V_u = 0.0 kips



CHECK CAPACITY SUBJECTED TO BENDING AND AXIAL LOAD

LOADING	U_1,top	U_1,bot	U_2,top	U_2,bot	U_3,top	U_3,bot
P_u (kips)	-527.9	-517.8	-777.6	-770.1	891.0	902.8
M_u (ft-kips)	0.3	0.2	0.3	0.2	0.0	0.0
$\delta_{ns} = C_m/[1-P_u/(0.75P_c)]$	1.000	1.000	1.000	1.000	1.148	1.150
$\delta_{ns}M_u$ (ft-kips)	0.3	0.2	0.3	0.2	0.0	0.0
ϕM_n (ft-kips) @ P_u	426.7	822.4	822.4	822.4	822.3	822.3

where $EI = 0.4E_cI_g / (1+\beta_d) = 0.25\ E_cI_g$

$P_c = \pi^2 EI / (kL_u)^2$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES (for ACI 318-08 10.2 & 10.3 only)

CAPACITY	**ϕ P_n (kips)**	**ϕ M_n (ft-kips)**
AT AXIAL LOAD ONLY	1739	0
AT MAXIMUM LOAD	1739	281
AT 0 % TENSION	1384	487
AT 25 % TENSION	1148	580
AT 50 % TENSION	941	648
AT ε_t = 0.002	605	743
AT BALANCED CONDITION	585	751
AT ε_t = 0.005	258	943
AT FLEXURE ONLY	0	822
AT TENSION ONLY	-1097	0

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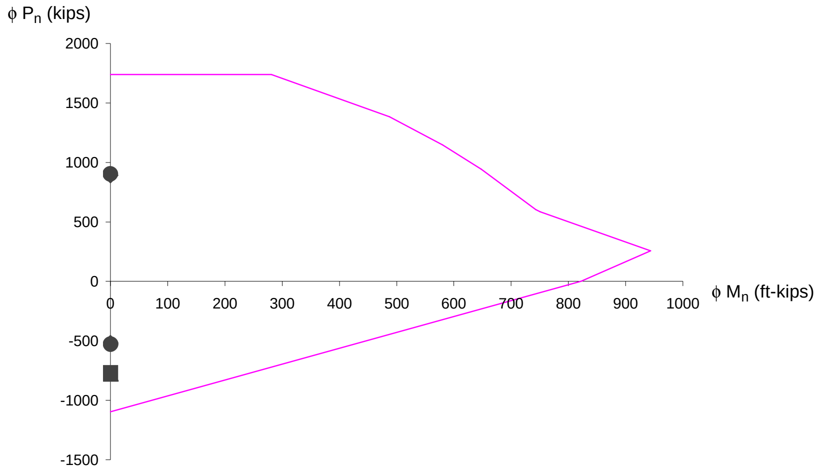
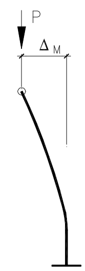


All load points to be within capacity diagram. [Satisfactory]



DETERMINE INDUCED MOMENT IN THE COLUMN

$$M_{\Delta} = \frac{3E_C I_C \delta_u}{h^2} + P\Delta = \quad 168.0906 \text{ ft-kips}$$

where $E_c = 57000\ (f_c')^{0.5} =$ 3824 ksi, ACI 318-08 8.5.1

$I_g = c_1\ c_2^{\,3} / 12 =$ 27648 in^4

$I_c =$ 0.7 $I_g =$ 19354 in^4 , ACI 318-08 9.5.2.3 & 10.11.1

$C_d =$ 5.0 $\delta_{xe} =$ 0.1 in

$\delta_u = C_d\ \delta_{xe} / I =$ 0.50 $I =$ 1.0

$\Delta = 0.5\ \delta_u =$ 0.25 in, ACI 318-08 21.13.3

$P = 0.9\ P_{DL} =$ 202.5 kips, ACI 318-08 21.13.3

CHECK REQUIREMENTS OF NOT PART OF THE LATERAL RESISTING SYSTEM

$M_u = 1.2\ M_{DL} + 1.0\ M_{LL} + M_{\Delta} =$ 168.0906 ft-kips

< $\phi\ M_n =$ 822.3 kips [Satisfactory]

$P_{u,\ max} =$ 902.8 kips > $0.1A_g f_c' =$ 259.2 kips [Satisfactory]

Per ACI 318-08 21.13.4.3, the column shall satisfy ACI 318-08 21.6.3, 21.6.4, 21.6.5, and 21.7.3.1.

CHECK SECTION REQUIREMENTS (ACI 318-08 21.6.1)

$c_{min} = MIN(c_1, c_2) =$ 24 in > 12 in [Satisfactory]

$c_{min} / c_{max} =$ 1.00 > 0.4 [Satisfactory]

CHECK TRANSVERSE REINFORCING AT BOTTOM OF COLUMN (ACI 318-08 21.6.4)

$A_{sh} =$ 0.80 in^2 > $MAX[\ 0.09 s h_c f_c' / f_{yh}\ ,\ 0.3 s h_c (A_g/A_{ch}-1) f_c' / f_{yh}\ ] =$ 0.71 in^2

[Satisfactory] where $s = MAX[MIN(c_1/4,\ 6d_b,\ 4+(14-h_x)/3,\ 6),\ 4] =$ 5 in

$h_c = c_1 - 2Cover - d_t =$ 20.5 in

$A_{ch} = (c_1-3)(c_2-3) =$ 441.0 in^2

CHECK FLEXURAL REINFORCING (ACI 318-08 21.6.1.1)

$\rho_{total} =$ 0.040 > $\rho_{min} =$ 0.010 [Satisfactory]

< $\rho_{max} =$ 0.060 [Satisfactory]

CHECK SHEAR STRENGTH (ACI 318-08 21.6.4.6)

$V_e = MAX[\ (M_{pr,\ left,\ top} + M_{pr,\ right,bot}) / h\ ,\ V_{u,max}] =$ 73.4 kips

< $8\phi(f_c')^{0.5} c_2 d =$ 206.4 kips [Satisfactory]

< $\phi[2(f_c')^{0.5} c_2 d + A_v f_y d / s] =$ 205.4 kips [Satisfactory]

where $\rho_{top,left} =$ 0.012 > $\rho_{min} = MIN[3(f_c')^{0.5}/f_y,\ 200/f_y] =$ 0.003 [Satisfactory]

$\rho_{bot,left} =$ 0.012 > $\rho_{min} =$ 0.003 [Satisfactory]

$M_{pr,\ left,\ top} =$ 0 ft-kips $\phi =$ 0.75

$M_{pr,\ right,\ bot} = 1.25 M_{n,col,max} =$ 1028 ft-kips $A_v =$ 0.8 in^2

$V_{u,\ max} =$ 0.033 ft-kips

DETERMINE SEISMIC TENSION DEVELOPMENT, Ld, INTO THE FOOTING PER ACI 318-08 21.6.4.6

$$L_{dh} = MAX\left(\frac{d_b f_y}{65\sqrt{f_c'}}\ ,\ 8d_b\ ,\ 6\ \ in\right) = \quad 14\ d_b \quad = \quad 17 \text{ in, (ACI 318-08 21.7.5.1)}$$

$$L_d = MAX\left(3.5 L_{dh}\beta\ ,\ 12\ \ in\right) = \quad 48\ d_b \quad = \quad 61 \text{ in, (ACI 318-08 21.7.5.2)}$$

where d_b = 1.27 in

β = 1.0 , (1.2 for epoxy-coated, ACI 318-08 21.7.5.4 & 12.2.4)

(cont'd)

CHECK FLEXURE CAPACITY, AS,1, FOR STEM (TMS SEC.402 2.3.3)

$$M = \frac{w_{Lat} H^2}{2} = 1.47 \text{ ft-kips / ft} \qquad P = W_w = 0.78 \text{ kips / ft}$$

$$M_{allowable} = MIN\left[\frac{1}{2} b_w kd F_b \left(d - \frac{kd}{3}\right) - P\left(d - \frac{t_e}{2}\right) \ , \ A_s F_s \left(d - \frac{kd}{3}\right) + P\left(\frac{t_e}{2} - \frac{kd}{3}\right)\right]$$

where	t_e	=	7.63	in ,	<== Based on effective section area.				
	d	=	3.82	in ,	<== Based on TMS 402-08, 1.13.3.5				
	b_w	=	12	in		E_m	=	1350	ksi
	F_b	=	0.495	ksi		E_s	=	29000	ksi
	F_s	=	24	ksi		n	=	21.48	
	A_s	=	0.66	in^2		k	=	0.54	
	ρ	=	0.014						

and $\boldsymbol{M_{allowable}}$ = 1.59 ft-kips , **>** ***M*** **[Satisfactory]**

CHECK SHEAR CAPACITY FOR MASONRY STEM (TMS SEC.402 2.3.5)

$$V = H_{Lat} = 0.33 \text{ kips / ft}$$

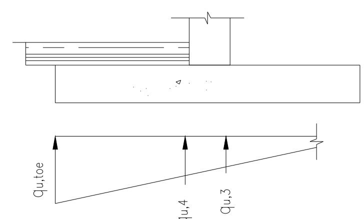
$$V_{allowable} = d\, b_w MIN\left(\sqrt{f'_m} \ , \ 50\right) = 1.77 \text{ kips / ft}$$ **>** ***V*** **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,3}$, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon u}{\varepsilon u + \varepsilon t} = 0.015 \qquad \rho_{MIN} = \frac{0.0018}{2} \frac{h_f}{d} = 0.001$$

$$M_{u,3} = \begin{cases} \frac{(L-t)}{4}\left(0.5\gamma W_s + \frac{(L-t)}{2L}\gamma W_f\right) - \frac{(q_{u,3} + 2q_{u,heel})b}{6}\left(\frac{L-t}{2}\right)^2, & for \ \ e_u \le \frac{L}{6} \\ \frac{(L-t)}{4}\left(0.5\gamma W_s + \frac{(L-t)}{2L}\gamma W_f\right) - \frac{q_{u,3} b S^2}{6}, & for \ \ e_u > \frac{L}{6} \end{cases} = 2.534 \text{ ft-kips}$$

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} = 0.00045$$



where	d	=	10.19	in	$q_{u, toe}$	=	4.90	ksf
	e_u	=	1.49	ft	$q_{u, heel}$	=	n/a	ksf
	S	=	-1.29	ft	$q_{u, 3}$	=	-7.99	ksf

$(A_{S,3})_{required}$ = 0.13 in^2 / ft **<** $A_{S,3}$ = 0.21 in^2 / ft **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,2}$, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon u}{\varepsilon u + \varepsilon t} = 0.015 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2} \frac{h_f}{d}\right) = 0.001$$

$$M_{u,2} = \frac{(q_{u,4} + 2q_{u,toe})b}{6}\left(\frac{L-t}{2}\right)^2 - \frac{\gamma W_f}{2L}\left(\frac{L-t}{2}\right)^2 = 1.80 \text{ ft-kips}$$

where	d	=	8.69	in
	$q_{u, 4}$	=	-3.86	ksf

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,2}}{0.383 b d^2 f'_c}}\right)}{f_y} = 0.00044$$

(cont'd)

$(A_{S,2})_{required}$ = 0.06 in^2 / ft **<** $A_{S,2}$ = 0.21 in^2 / ft **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 09 1807.2.3, CBC 07 1806A.1)

$1.5 (H_{Lat})$ = 0.49 kips / ft **<** $H_p + \mu \Sigma W$ = 1.27 kips / ft
[Satisfactory]

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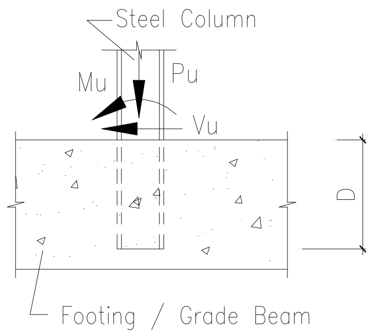
Fixed Moment Condition Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

COLUMN SHAPE (Tube, Pipe, or WF) & SIZE	W24X192	<==	W Shape	d	A	b_f
				25.5	56.3	13.0

CONCRETE STRENGTH	f_c' =	3	ksi
FACTORED SHEAR LOAD	V_u =	205	kips
FACTORED MOMENT	M_u =	750	ft-kips
FACTORED VERTICAL LOAD (negative for uplift)	P_u =	691	kips
EMBEDMENT DEPTH	D =	48	in



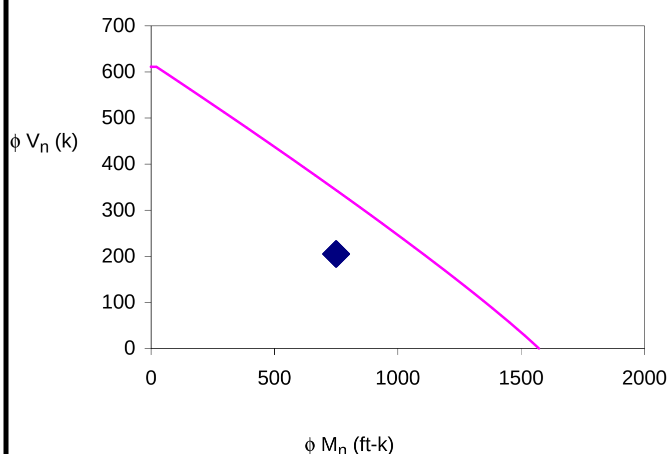
THE FIXED MOMENT DESIGN IS ADEQUATE.

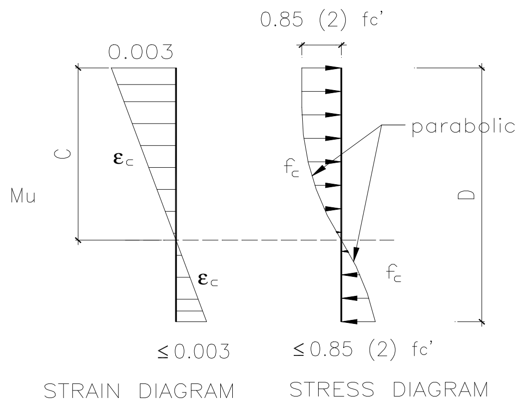
(A_{vf} = **16.0** in^2 , Required Area of Shear Studs or Welded Reinforcement)

(Edge of Concrete Footing / Grade Beam must be wider than "b_f")

ANALYSIS

CHECK BASE FLEXURAL & SHEAR CAPACITY (ACI 318-08 Sec 9 & Sec 10)





$$\varepsilon_o = \frac{2f_c'}{E_c} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right)\ ,\ E_c = 57\sqrt{f_c'}$$

$$f_c = \begin{cases} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f_c' \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]\ ,\ for\ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f_c'\ ,\ for\ \ \varepsilon_c \geq \varepsilon_o \end{cases}$$

ϕM_n = 1103 ft-kips @ V_u = 205 kips

> M_u = 750 ft-kips [Satisfactory]

$\phi V_{n,max}$ = 611.08 kips, when C = 33.0 in

> V_u = 205 ft-kips [Satisfactory]

where ϕ = 0.65 , (ACI 318-08 Sec 9.3.2.4)

Bearing factor = 2, (ACI 318-08 Sec 22.5.5)

b = effective bearing width = 95% b_f = 12.35 in

CHECK VERTICAL CAPACITY

ϕP_n = End Bering + Friction = 1295.4 kips > P_u = 691 kips [Satisfactory]

where End Bering = 0.65 (2) 0.85 f_c' A = 186.6 kips, (ACI 318-08 Sec 22.5.5)

Friction = 0.75 MAX(0.2f_c' A_c , 800 A_c) = 1108.8 kips, (ACI 318-08 Sec 11.7.5)

A = 56 in^2, end bearing area

A_c = 0.5 (2d + 2b_f) D = 1848 in^2, (0.5 for concrete cracked)

A_{vf} = $P_{u,Friction}$ / (ϕ f_y μ) = 16.0 in^2 , Required Area of Shear Studs or Welded Reinforcement

where ϕ = 0.75 , (ACI 318-08 Sec 9.3.2.3)

μ = 0.70 , (ACI 318-08 Sec 11.7.4.3)

f_y = 60 ksi, use 30% f_y for DSA / OSHPD seismic shear studs (CBC 07 2204A.1.2).

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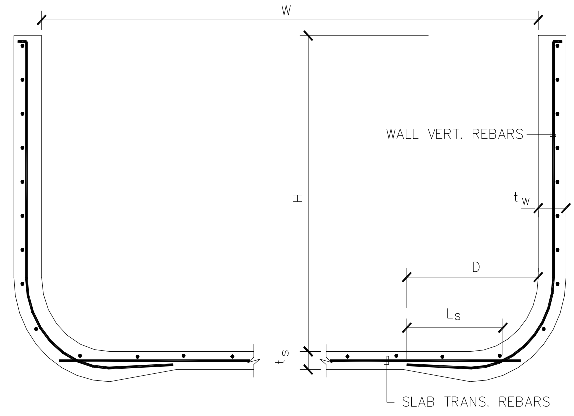
Concrete Floodway Design Based on ACI 350-06 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
LATERAL SOIL PRESSURE	P_a	=	45	pcf
	(equivalent fluid pressure)			
BACKFILL WEIGHT	γ_b	=	110	pcf
SURCHARGE WEIGHT	w_s	=	50	psf
SEISMIC GROUND SHAKING	P_E	=	20	psf /ft, ASD
(soil pressure, if no report $35S_{DS}$ suggested.)				
CHANNEL DEPTH	H	=	6	ft
THICKNESS OF WALL	t_w	=	8	in
THICKNESS OF SLAB	t_s	=	6	in
SLAB TRANS REBARS	# 5	@	10	in o.c. at mid
WALL VERTICAL REBARS	# 5	@	8	in o.c.
WALL BAR LOCATION (1=at middle, 2=at each face)			1	at middle
LAP LENGTH	L_s	=	36	in
SLAB THICKER DISTANCE	D	=	4	ft



[THE CHANNEL DESIGN IS ADEQUATE.]

ANALYSIS

DESIGN CRITERIA

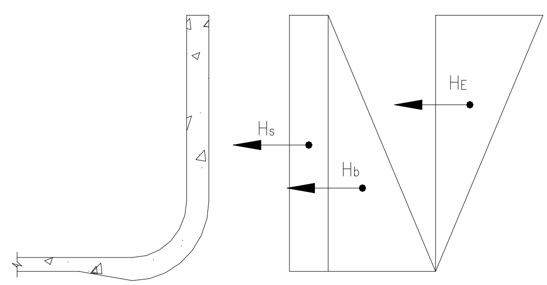
1. THE CRITERICAL DESIGN, FOR REBAR AT MIDDLE OR EQUAL OF EACH FACE, IS CHANNEL WALL AT INWARD SOIL PRESSURE BEFORE RESTRAINED AT TOP AND CHANNEL FILLED.
2. SINCE THE WALL AXIAL LOAD SMALL AND SECTIONS UNDER TENSION-CONTROLLED (ACI 318-08, 10.3.4), ONLY CHECK WALL FLEXURAL CAPACITIES ARE ADEQUATE.
3. SINCE THE SLAB AT FLEXURAL & AXIAL LOADS, THE COMBINED CAPACITY OF FLEXURAL & AXIAL MUST BE CHECKED.

SERVICE LOADS

$H_b = 0.5\, P_a\, (H + t_s)^2$ = 0.95 kips / ft

$H_s = w_s\, P_a\, (H + t_s) / \gamma_b$ = 0.13 kips / ft

$H_E = 0.5\, P_E\, (H + t_s)^2$ = 0.42 kips / ft



FACTORED LOADS

$\gamma H_b = 1.6\, H_b$ = 1.52 kips / ft

$\gamma H_s = 1.6\, H_s$ = 0.21 kips / ft

$\gamma H_E = 1.6\, H_E$ = 0.68 kips / ft

CHECK WALL FLEXURE CAPACITY (ACI 318-08, 15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$M_u = (0.5\,\gamma\, H_s + 0.33\,\gamma\, H_b + 0.67\,\gamma\, H_E)\, H =$ 6.38 ft-kips / ft, (entire lateral loads used conservatively)

$P_u =$ 1.19 kips / ft, (concrete wall self weight)

d = 4.00 in, b = 12 in, A_s = 0.465 in^2 / ft

$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7 b f_c'}\right)\right]$$ = 7.46 ft-kips / ft > M_u **[Satisfactory]**

ρ_{ProvD} = 0.010 < ρ_{MAX} = 0.015

\> ρ_{MIN} = 0.004 **[Satisfactory]**

CHECK WALL SHEAR CAPACITY (ACI 318-08, 15.5.2, 11.1.3.1, & 11.2)

$V_u = \gamma\, H_s + \gamma\, H_b + \gamma\, H_E$ = 2.41 kips / ft, (entire lateral loads used conservatively)

$$\phi V_n = 2\phi b d \sqrt{f_c'}$$ = 3.94 kips / ft > V_u **[Satisfactory]**

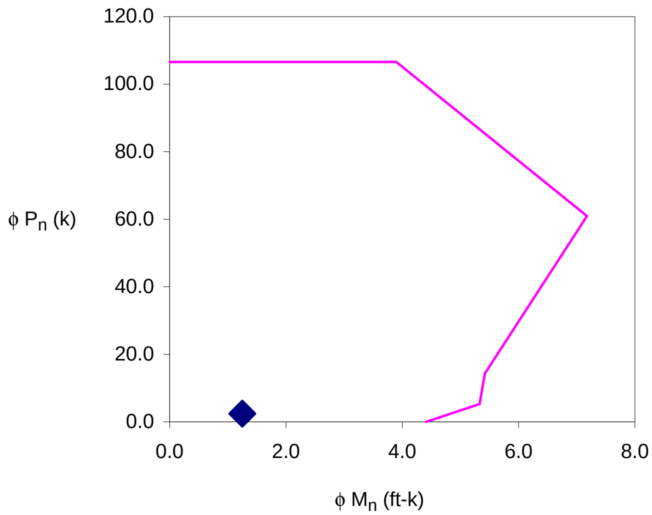
(cont'd)

CHECK SLAB COMBINED CAPACITY OF FLEXURE & AXIAL (ACI 318-08, 10)

ρ_{ProvD} = 0.01033 **<** ρ_{MAX} = 0.08 (for compression, ACI 318-08, 10.9.1)

> ρ_{MIN} = 0.00333 (for flexural, ACI 318-08, 10.5.1)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	106.6	0.0
AT MAXIMUM LOAD	106.6	3.9
AT MIDDLE	60.9	7.2
AT ε_t = 0.002	15.3	5.5
AT BALANCED	14.4	5.4
AT ε_t = 0.005	5.3	5.3
AT FLEXURE ONLY	0.0	4.4

(Note: For middle reforming the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

P_u =	2.41	kips / ft
M_u =	1.25	ft-kips / ft

[Satisfactory]

CHECK REBAR DEVELOPMENT

$$L_d = MAX\left(\frac{\rho_{requird}}{\rho_{provided}}\frac{0.075\psi_t\psi_e\psi_s d_b f_y}{\lambda\sqrt{f'_c}\left(\frac{c+K_{tr}}{d_b}\right)},\ 12\ in\right) =$$ 26 d_b = 16 in, (ACI 318-08, 12.2.3)

< L_s **[Satisfactory]**

where Bar size # 5 , (governing size)

d_b = 0.625 in

$\rho_{required} / \rho_{provid}$ = 1 (As,reqd / As,provd, ACI 318-08, 12.2.5)

ψ_t = 1.0 (1.3 for bottom cover more than 12", ACI 318-08, 12.2.4)

ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-08, 12.2.4)

ψ_s = 0.8 (0.8 for # 6 or smaller, 1.0 for other)

λ = 1.0 (0.7 for light weight)

c = 3.3 in, min(d' , 0.5s), (ACI 318-08, 12.2.4)

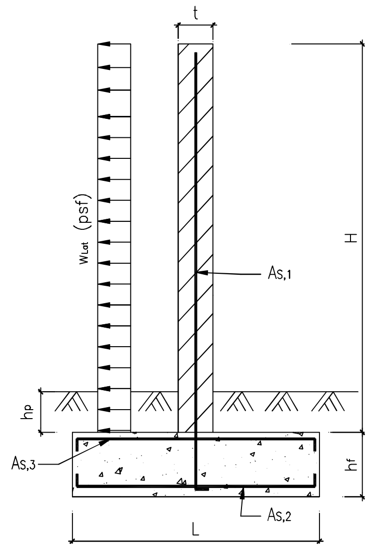
K_{tr} = (A_{tr} f_{yt} / 1500 s n) = 0 (ACI 318-08, 12.2.4)

(c + K_{tr}) / d_b = 2.5 < 2.5 , (ACI 318-08, 12.2.3)

Free Standing Masonry Wall Design Based on TMS 402-08 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes		
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
CONCRETE STRENGTH	f_c'	=	3	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
PASSIVE SOIL PRESSURE	P_p	=	350	pcf (equivalent fluid pressure)		
ALLOW SOIL PRESSURE	Q_a	=	2	ksf		
FRICTION COEFFICIENT	μ	=	0.35			
SOIL SPECIFIC WEIGHT	γ_s	=	110	pcf		
SOIL OVER	h_p	=	12	in		
WALL LATERAL FORCE, ASD	w_{Lat}	=	36.4	psf		
HEIGHT OF STEM	H	=	9	ft		
THICKNESS OF WALL	t	=	8	in		
WALL VERT. REINF. ($A_{s,1}$)		#	6	@	8	in o.c.
$A_{s,1}$ LOCATION (1=at middle, 2=at each face)			1	at middle		
FOOTING WIDTH	L	=	3.5	ft		
FOOTING THICKNESS	h_f	=	12	in		
BOT. REINF.OF FOOTING ($A_{s,2}$)		#	5	@	18	in o.c.
TOP. REINF.OF FOOTING ($A_{s,3}$)		#	5	@	18	in o.c.



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS

$H_{Lat} = w_{Lat}\,H$ = 0.33 kips / ft

$H_p = 0.5\,P_p\,(h_p + h_f)^2$ = 0.70 kips / ft

$W_w = t\ H\,\gamma_m$ = 0.78 kips / ft

$W_f = h_f\,L\,\gamma_c$ = 0.53 kips / ft

$W_s = h_p\,(L - t)\,\gamma_s$ = 0.31 kips / ft

FACTORED LOADS

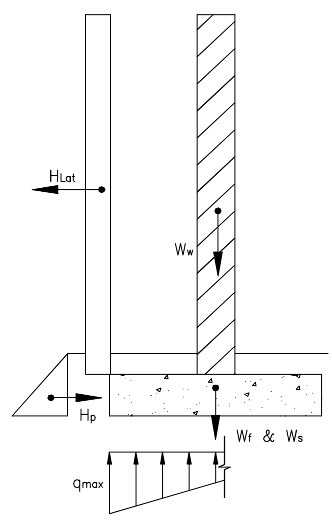
$\gamma H_{Lat} = 1.6\,H_{Lat}$ = 0.52 kips / ft

$\gamma H_p = 0.0\,H_p$ = 0.00 kips / ft

$\gamma W_w = 1.2\,W_w$ = 0.94 kips / ft

$\gamma W_f = 1.2\,W_f$ = 0.63 kips / ft

$\gamma W_s = 1.2\,W_s$ = 0.37 kips / ft



OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_{Lat}	0.33	0.52	5.5	1.80	2.88
Σ	0.33	0.52		**1.80**	2.88

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.31	0.37	1.75	0.55	0.65
W_f	0.53	0.63	1.75	0.92	1.10
W_w	0.78	0.94	1.75	1.37	1.64
Σ	1.62	1.94		**2.83**	3.40

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 1.57 > 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W}$$ = 1.11 ft > L / 4

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for\quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L-e)}, & for\quad e > \dfrac{L}{6} \end{cases}$$ = 1.70 ksf < Q_a **[Satisfactory]**

(cont'd)

CHECK FLEXURE CAPACITY, AS,1, FOR STEM (TMS SEC.402 2.3.3)

$$M=\frac{w_{Lat}H^2}{2}$$ = 1.47 ft-kips / ft $\qquad P=W_w$ = 0.78 kips / ft

$$M_{allowable}=MIN\left[\frac{1}{2}b_w kd F_b\left(d-\frac{kd}{3}\right)-P\left(d-\frac{t_e}{2}\right)\ ,\ A_sF_s\left(d-\frac{kd}{3}\right)+P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right]$$

where	t_e	=	7.63	in ,	<== Based on effective section area.			
	d	=	3.82	in ,	<== Based on TMS 402-08, 1.13.3.5			
	b_w	=	12	in	E_m	=	1350	ksi
	F_b	=	0.495	ksi	E_s	=	29000	ksi
	F_s	=	24	ksi	n	=	21.48	
	A_s	=	0.66	in^2	k	=	0.54	
	ρ	=	0.014					

and $M_{allowable}$ = 1.59 ft-kips , > M **[Satisfactory]**

CHECK SHEAR CAPACITY FOR MASONRY STEM (TMS SEC.402 2.3.5)

$$V=H_{Lat}$$ = 0.33 kips / ft

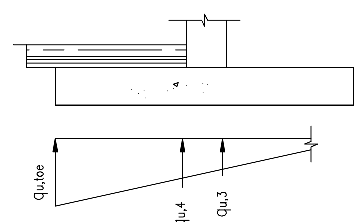
$$V_{allowable}=d\,b_w MIN\left(\sqrt{f'_m}\ ,\ 50\right)$$ = 1.77 kips / ft > V **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,3}$, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX}=\frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$ = 0.015 $\qquad \rho_{MIN}=\frac{0.0018}{2}\frac{h_f}{d}$ = 0.001

$$M_{u,3}=\begin{cases}\frac{(L-t)}{4}\left(0.5\gamma W_s+\frac{(L-t)}{2L}\gamma W_f\right)-\frac{(q_{u,3}+2q_{u,heel})b}{6}\left(\frac{L-t}{2}\right)^2, & for\quad e_u\le\frac{L}{6}\\ \frac{(L-t)}{4}\left(0.5\gamma W_s+\frac{(L-t)}{2L}\gamma W_f\right)-\frac{q_{u,3}bS^2}{6}, & for\quad e_u>\frac{L}{6}\end{cases}$$ = 2.534 ft-kips

$$\rho=\frac{0.85f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2f'_c}}\right)}{f_y}$$ = 0.00045



where	d	=	10.19	in	$q_{u,\,toe}$	=	4.90	ksf
	e_u	=	1.49	ft	$q_{u,\,heel}$	=	n/a	ksf
	S	=	-1.29	ft	$q_{u,\,3}$	=	-7.99	ksf

$(A_{S,3})_{required}$ = 0.13 in^2 / ft < $A_{S,3}$ = 0.21 in^2 / ft **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,2}$, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX}=\frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$ = 0.015 $\qquad \rho_{MIN}=MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right)$ = 0.001

$$M_{u,2}=\frac{(q_{u,4}+2q_{u,toe})b}{6}\left(\frac{L-t}{2}\right)^2-\frac{\gamma W_f}{2L}\left(\frac{L-t}{2}\right)^2$$ = 1.80 ft-kips

where d = 8.69 in

$q_{u,\,4}$ = -3.86 ksf

$$\rho=\frac{0.85f'_c\left(1-\sqrt{1-\frac{M_{u,2}}{0.383bd^2f'_c}}\right)}{f_y}$$ = 0.00044

(cont'd)

$(A_{S,2})_{required}$ = 0.06 in^2 / ft < $A_{S,2}$ = 0.21 in^2 / ft **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 09 1807.2.3, CBC 07 1806A.1)

1.5 (H_{Lat}) = 0.49 kips / ft < $H_p + \mu\,\Sigma W$ = 1.27 kips / ft
[Satisfactory]

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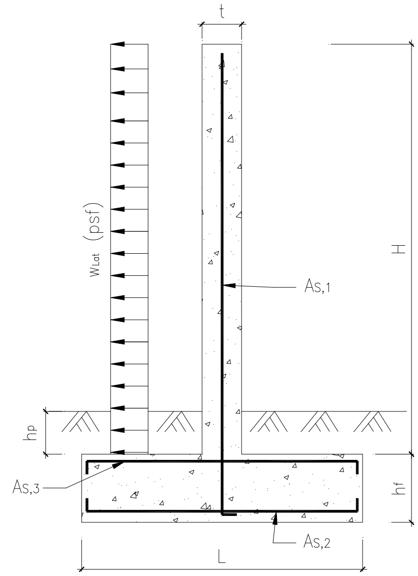
Free Standing Concrete Wall Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
PASSIVE SOIL PRESSURE	P_p	=	350	pcf (equivalent fluid pressure)		
ALLOW SOIL PRESSURE	Q_a	=	2	ksf		
FRICTION COEFFICIENT	μ	=	0.35			
SOIL SPECIFIC WEIGHT	γ_s	=	110	pcf		
SOIL OVER	h_p	=	12	in		
WALL LATERAL FORCE, ASD	w_{Lat}	=	42	psf		
HEIGHT OF STEM	H	=	21	ft		
THICKNESS OF WALL	t	=	12	in		
WALL VERT. REINF. ($A_{s,1}$)		#	6	@	12	in o.c.
$A_{s,1}$ LOCATION (1=at middle, 2=at each face)			2	at each face		
FOOTING WIDTH	L	=	9	ft		
FOOTING THICKNESS	h_f	=	18	in		
BOT. REINF.OF FOOTING ($A_{s,2}$)		#	6	@	18	in o.c.
TOP. REINF.OF FOOTING ($A_{s,3}$)		#	6	@	18	in o.c.



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS

$H_{Lat} = w_{Lat}\,H$ = 0.88 kips / ft

$H_p = 0.5\,P_p\,(h_p + h_f)^2$ = 1.09 kips / ft

$W_w = t\,H\,\gamma_c$ = 3.15 kips / ft

$W_f = h_f\,L\,\gamma_c$ = 2.03 kips / ft

$W_s = h_p\,(L - t)\,\gamma_s$ = 0.88 kips / ft

FACTORED LOADS

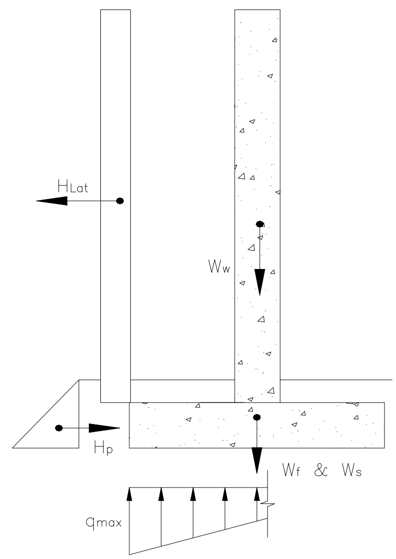
$\gamma H_{Lat} = 1.6\,H_{Lat}$ = 1.41 kips / ft

$\gamma H_p = 0.0\,H_p$ = 0.00 kips / ft

$\gamma W_w = 1.2\,W_w$ = 3.78 kips / ft

$\gamma W_f = 1.2\,W_f$ = 2.43 kips / ft

$\gamma W_s = 1.2\,W_s$ = 1.06 kips / ft



OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_{Lat}	0.88	1.41	12	10.58	16.93
Σ	0.88	1.41		**10.58**	16.93

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.88	1.06	4.5	3.96	4.75
W_f	2.03	2.43	4.5	9.11	10.94
W_w	3.15	3.78	4.5	14.18	17.01
Σ	6.06	7.27		**27.25**	32.70

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 2.574 > 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 1.75 \text{ ft} \quad < \quad L / 4$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 1.47 \text{ ksf} \quad < \quad Q_a$$

[Satisfactory]

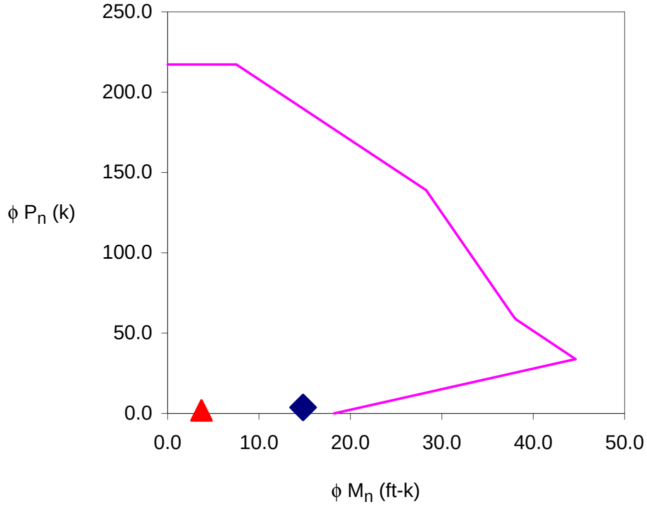
(cont'd)

CHECK FLEXURE CAPACITY, $A_{S,1}$, FOR STEM (ACI 318-08 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

ρ_{ProvD} = 0.00381 **<** ρ_{MAX} = 0.04 (tension face only, ACI 318-08 10.3.5 or 10.9.1)

> ρ_{MIN} = 0.00075 (tension face only, ACI 318-08 10.5.1, 10.5.3 or 14.3.2)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	217.2	0.0
AT MAXIMUM LOAD	217.2	7.5
AT MIDDLE	139.0	28.3
AT ε_t = 0.002	60.8	37.8
AT BALANCED	58.6	38.1
AT ε_t = 0.005	33.8	44.6
AT FLEXURE ONLY	0.0	18.2

(Note: For middle reforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

	at bottom	at middle
P_u	3.78	1.89
M_u	14.82	3.70

[Satisfactory]

CHECK STEM SHEAR CAPACITY (ACI 318-08 SEC.15.5.2, 11.1.3.1, & 11.2)

$V_u = Max.\ \ Horiz.\ \ Shear$ = 1.41 kips / ft, at bottom

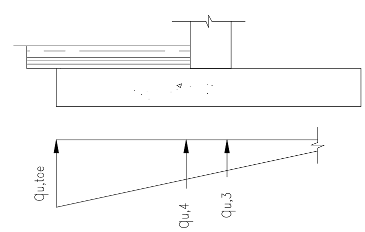
$\phi V_n = 2\phi bd\sqrt{f'_c}$ = 9.49 kips / ft **>** $\boldsymbol{V_u}$ **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,3}$, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = 0.015 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.001$$

$$M_{u,3} = \begin{cases} \frac{(L-t)}{4}\left(0.5\gamma W_s + \frac{(L-t)}{2L}\gamma W_f\right) - \frac{\left(q_{u,3}+2q_{u,heel}\right)b}{6}\left(\frac{L-t}{2}\right)^2, & for\ \ e_u \le \frac{L}{6} \\ \frac{(L-t)}{4}\left(0.5\gamma W_s + \frac{(L-t)}{2L}\gamma W_f\right) - \frac{q_{u,3}bS^2}{6}, & for\ \ e_u > \frac{L}{6} \end{cases} = 3.02 \text{ ft-kips}$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.00023$$



where	d	=	15.63 in	$q_{u, toe}$	=	2.23	ksf
	e_u	=	2.33 ft	$q_{u, heel}$	=	n/a	ksf
	S	=	1.51 ft	$q_{u, 3}$	=	0.52	ksf

$(A_{S,3})_{required}$ = 0.19 in^2 / ft **<** $A_{S,3}$ = 0.29 in^2 / ft **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,2}$, FOR FOOTING (ACI 318-08 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = 0.015 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.001$$

(cont'd)

$$M_{u,2} = \frac{(q_{u,4}+2q_{u,toe})b}{6}\left(\frac{L-t}{2}\right)^2 - \frac{\gamma W_f}{2L}\left(\frac{L-t}{2}\right)^2 \quad = \quad 12.04 \text{ ft-kips}$$

where d = 14.63 in

$q_{u,4}$ = 0.861 ksf

$$\rho = \frac{0.85 f_c^{'}\left(1-\sqrt{1-\frac{M_{u,2}}{0.383bd^2 f_c^{'}}}\right)}{f_y} \quad = \quad 0.00105$$

$(A_{S,2})_{required}$ = 0.19 in^2 / ft < $A_{S,2}$ = 0.29 in^2 / ft **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 09 1807.2.3, CBC 07 1806A.1)

1.5 (H_{Lat}) = 1.32 kips / ft < $H_p + \mu\, \Sigma W$ = 3.21 kips / ft

[Satisfactory]

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Lateral Earth Pressure of Rigid Wall Based on AASHTO 17th & 2018 IBC

INPUT DATA & DESIGN SUMMARY

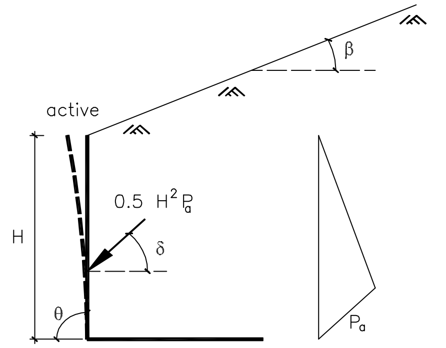
SOIL SPECIFIC WEIGHT	γ_b	=	110	pcf
SOIL INTERNAL FRICTION ANGLE	ϕ	=	30	deg
SLOPE OF BACKFILL	β	=	15	deg
EXTERNAL FRICTION ANGLE	δ	=	17	deg
RACK ANGLE OF WALL FACE	θ	=	90	deg

The Active Earth Pressure: P_a = 41 psf / ft

$P_{a,\,h}$ = 39 pcf (horizontal equivalent fluid pressure)

The At-rest Earth Pressure: P_0 = 55 psf / ft

The Passive Earth Pressure: P_p = 330 psf / ft



ANALYSIS

DETERMINE ACTIVE EARTH PRESSURE

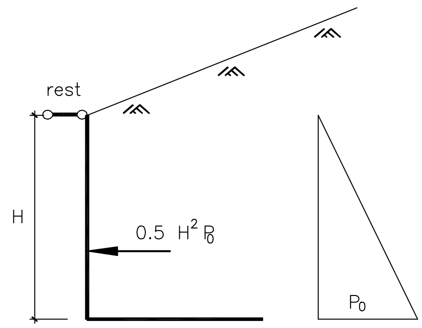
$P_a = \gamma_b K_a =$ 41 psf / ft

$P_{a,\,h} = P_a \sin(\theta - \delta) =$ 39 pcf (horizontal equivalent fluid pressure)

where $$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2} = 0.372$$

(Coulomb, AASHTO Figure 5.5.2A)

The total active resultant, $(0.5\ H^2\ P_a)$, acts H/3 above the base.

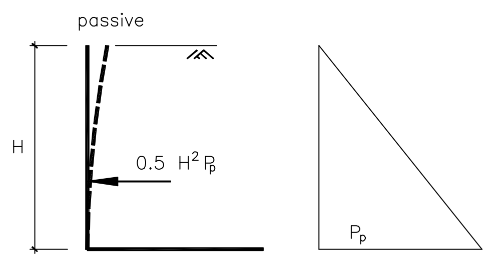


DETERMINE AT-REST EARTH PRESSURE

$P_0 = \gamma_b K_0 =$ 55 psf / ft

where $K_0 = 1 - \sin\phi =$ 0.500 , (AASHTO 5.5.2-2)

The total horizontal resultant at rest, $(0.5\ H^2\ P_0)$, acts H/3 above the base.



DETERMINE PASSIVE EARTH PRESSURE

$P_p = \gamma_b K_p =$ 330 psf / ft

where $$K_p = \frac{1+\sin\phi}{1-\sin\phi} = 3.000$$, (Rankine, AASHTO Figure 5.5.2D)

$\delta =$ 0 deg

$\theta =$ 90 deg

The total horizontal resultant at rest, $(0.5\ H^2\ P_p)$, acts H/3 above the base.

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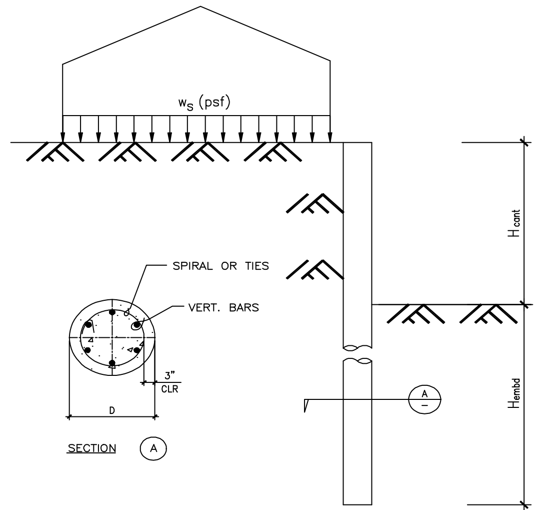
Sheet Pile Wall Design Based on 2015 IBC / 2016 CBC / ACI 318-14

INPUT DATA & DESIGN SUMMARY

HEIGHT OF CANTILEVER	H_{cant}	=	10	ft
SURCHARGE WEIGHT	w_s	=	500	psf
ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT				
	P_p	=	300	psf / ft
LATERAL SOIL PRESSURE	P_a	=	35	pcf (equivalent fluid pressure)
SEISMIC GROUND SHAKING	P_E	=	450	psf / ft (for H_{cant} > 12 ft only)
SOIL SPECIFIC WEIGHT	γ_b	=	110	pcf
CONCRETE STRENGTH	f_c'	=	4	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	44	in
PILE SPACING	S	=	4.0333	ft, o.c.
PILE VERT. REINF.	16	#	11	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	# 6	@	8	in o.c.



THE SHORING DESIGN IS ADEQUATE.

(H_{embd} = 20.51 ft. Min. Req'D)

ANALYSIS

DETERMINE PILE SECTION FORCES AT CANTILEVER BOTTOM

$H_b = 0.5\ S\ P_a\ (H_{cant})^2 =$ 7.06 kips, ASD

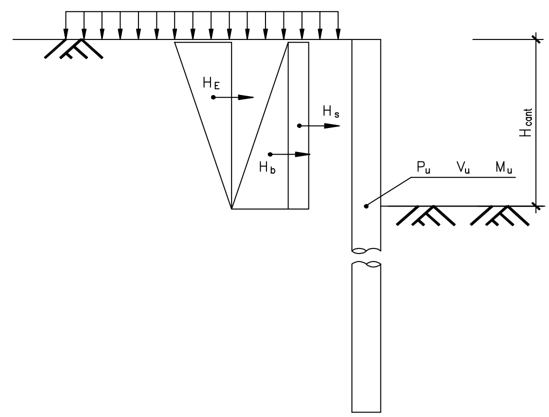
$H_s = w_s\ S\ P_a\ (H_{cant})\ /\ \gamma_b =$ 6.42 kips, ASD

$H_E = 0.5\ S\ P_E\ (H_{cant})^2 =$ 90.75 kips, ASD

$P = S\ D\ w_s + 0.25\ \pi\ \gamma_c\ D^2\ H_{cant} =$ 23.23 kips, ASD

$V = H_b + H_E + H_s =$ 104.23 kips, ASD

$M = (H_b/3 + 2H_E/3 + H_s/2)\ H_{cant} =$ 520.1 ft-kips, ASD



$P_u = 1.2\ P =$ 27.88 kips, SD

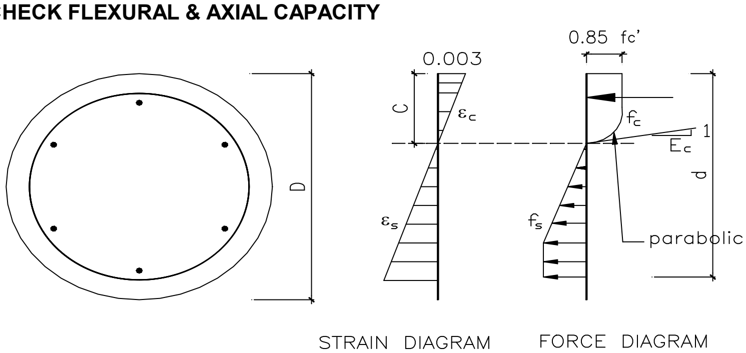
$V_u = 1.6\ V =$ 166.76 kips, SD

$M_u = 1.6\ M =$ 832.1 ft-kips, SD

CHECK PILE LIMITATIONS

f_c' =	4	ksi	>	4 ksi	**[Satisfactory]**	(IBC Table 1808.8.1)
D =	44	in	>	MAX[(H_{cant} + H_{embd}) / 30 , 12 in]	**[Satisfactory]**	(IBC 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY

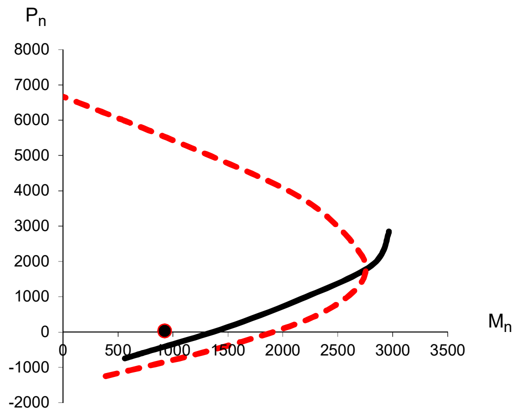


$$\varepsilon_o = or\left[\frac{2\left(\beta f_c'\right)}{E_c}\ ,\ \varepsilon_{\max}\right]\ ,\ E_c = 57\sqrt{f_c'}\ ,\ E_s = 29000 ksi$$

$$f_c = \begin{cases} \beta f_c'\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] & ,\ for\ \ 0 < \varepsilon_c < \varepsilon_o \\ \beta f_c'\ ,\ for\ \ \varepsilon_c \ge \varepsilon_o & \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \le \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$

(cont'd)



$P_u / \phi =$ 31.0 *kips, (138 kN)*

M_{nc} @ $P_u / \phi =$ 1386.5 *ft-kips, (1880 kN-m)*

> $M_u / \phi =$ 924.5 *ft-kips, (1254 kN-m)*

[Satisfactory]

where $\phi =$ 0.900 , (ACI 318-14 21.2)

d = 39.545 in, (ACI 20.6)

Solid Line - Tension Controlled

Dash Line - Compression Controlled

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c) =$ 312 kips, (ACI 318-14 22.5)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318 21.2)

A_0 = 1228 in^2. A_v = 0.88 in^2. f_y = 60 ksi

V_c = $2 (f_c')^{0.5} A_0$ = 155.4 kips, (ACI 318-14 22.5)

V_s = MIN ($d f_y A_v / s$, $8 (f_c')^{0.5} A_0$) = 261.0 kips, (ACI 318-14 22.5.1)

s_{max} = 12 (IBC 1810.3.9.4.2) s_{provd} = 8 in

s_{min} = 1 **[Satisfactory]**

ρ_s = $0.12 f_c' / f_{yt}$ = 0.008 < $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-14 18.13.4.3 & 18.7.5.1)

DETERMINE PILE EMBEDMENT LENGTH, H_{embd} , (IBC 1807.3)

By trials, use pole depth, $d = H_{embd}$ = 20.51 ft

Lateral bearing @ bottom, $S_3 = 2 P_p$ *Min(d , 12')* = 7.20 ksf

Lateral bearing @ *d / 3*, $S_1 = 2 P_p$ *Min(d / 3 , 12')* = 4.10 ksf

Require Depth is given by

$$d = \frac{A}{2}\left[1 + \sqrt{1 + \frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad =$$ 20.51 ft **[Satisfactory]**

Where *P* = *V* = 104.23 kips

A = $2.34 P / (D S_1)$ = 16.21

h = *M / V* = 4.99 ft

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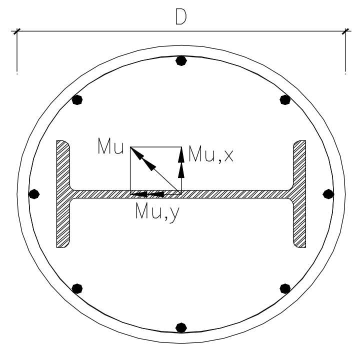
Composite Element Design Based on AISC 360-05 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
REBAR STEEL YIELD STRESS	$f_{y,\,Bar}$	=	60	ksi
VERTICAL REINFORCING	8	#	8	
COVER TO VERTICAL REBAR	C_c	=	3.25	in
WF STEEL YIELD STRESS	$f_{y,\,WF}$	=	50	ksi
WF SECTION			W16X67	

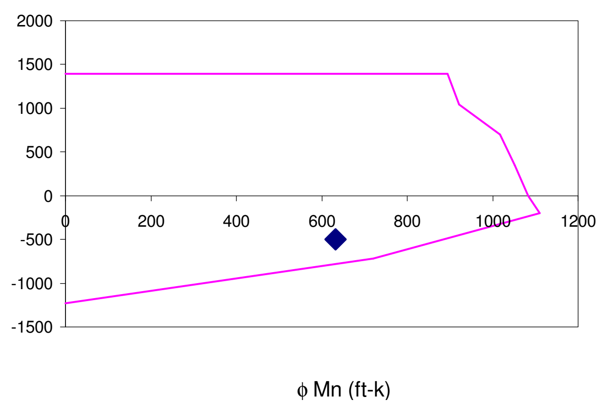
t_f	b_f	t_w	d	A
0.665	10.2	0.395	16.3	19.7

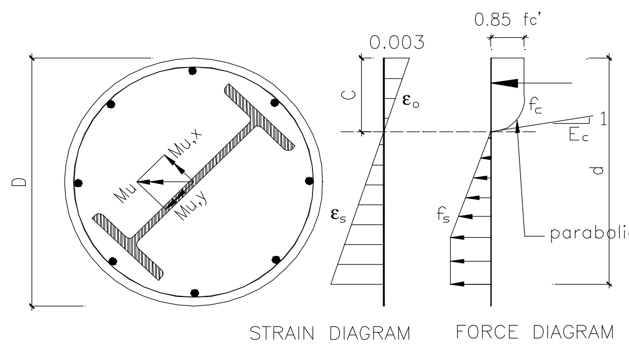
ELEMENT DIAMETER	D	=	30	in
FACTORED AXIAL LOAD	P_u	=	-500	k, tension
FACTORED MOMENT	$M_{u,x}$	=	600	ft-k
	$M_{u,y}$	=	200	ft-k
FACTORED SHEAR LOAD	$V_{u,x}$	=	68	k
	$V_{u,y}$	=	50	k



THE SECTION DESIGN IS ADEQUATE.

ANALYSIS





Capacity Drawings	ϕ	ϕ Pn (k)	ϕ Mn (ft-k)
AT AXIAL LOAD ONLY	0.75	1394	0
AT MAXIMUM LOAD	0.75	1394	894
AT 75% MAXIMUM LOAD	0.75	1045	922
AT 50% MAXIMUM LOAD	0.81	697	1017
AT 25% MAXIMUM LOAD	0.84	348	1052
AT FLEXURE ONLY	0.87	0	1084
AT STEEL STRAIN 0.005	0.9	-202	1110
AT AXIAL LOAD -715 k	0.9	-715	721
AT PURE TENSION	0.9	-1228	0

$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c}\ ,\quad E_c = 57\sqrt{f'_c}\ ,\quad E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] & ,\ for\quad 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c\ ,\ for\quad \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\quad \varepsilon_s \le \varepsilon_t \\ f_y\ ,\ for\quad \varepsilon_s > \varepsilon_t \end{cases}$$

CHECK FLEXURAL & AXIAL CAPACITY

$\phi P_{max} = \phi_c P_n$ = 1394 kips, (AISC 360-05 I2-2 & I2-3)

> P_u **[Satisfactory]**

where	ϕ_c	=	0.75	(AISC 360-05 I2.1b & ACI 318-08 9.3.2.2)				
	A_c	=	707	in^2	I_c	=	39642	in^4
	$A_{s,\,Bar}$	=	6.32	in^2	I_s	=	119	in^4

(cont'd)

C_1 = 0.154 , (AISC 360-05 I2-7) $E\,I_{eff}$ = 28093060 ksi-in^4, (AISC 360-05 I2-6)

KL = 30 ft, effective length

P_e = 2139 kips, (AISC 360-05 I2-5) P_o = 4368 kips, (AISC 360-05 I2-4)

Balanced : ϕ = 0.75 (AISC 360-05 I2.1b & ACI 318-08 Fig. R9.3.2)

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 16.7 in $\varepsilon_{t,\,min}$ = 0.0017 ε_c = 0.003

d = 26.3 in, (ACI 7.7.1) D = 30.0 in

Critical Points	ϕ	ϕ **Pn (k)**	ϕ **Mn (ft-k)**
AT AXIAL LOAD ONLY	0.75	1394	0
AT MAXIMUM LOAD	0.75	1394	894
AT 0 % TENSION	0.75	2174	650
AT 25 % TENSION	0.75	1853	781
AT 50 % TENSION	0.75	1577	859
AT STEEL STRAIN 0.002	0.75	984	933
AT BALANCED CONDITION	0.75	1117	925
AT STEEL STRAIN 0.005	0.9	-202	1110
AT FLEXURE ONLY	0.8736	0	1084

$\phi\,M_n$ = 884 ft-kips @ P_u = -500 kips **>** M_u = 632 ft-kips **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08 10.9) ρ_{provd} = 0.037

ρ_{min} = 0.01 (AISC 360-05 I2.1a & ACI 318-08 10.9) **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC I2.1d & ACI 318-08 11.1 & 11.2)

$\phi\,V_{nx} = \phi\,(V_{cx})$ **>** V_{ux} (ACI 318-08 11.1.1)

[Satisfactory]

$\phi_v\,V_{ny} = \phi_v\,(V_{ny})$ **>** V_{uy} (AISC 360-05 G2.1)

where ϕ = 0.75 (ACI 318-08 9.3.2.3)

ϕ_V = 1.00 (AISC 360-05 G2.1)

	d	A_0	A_w	$V_c = 2\,(f_c')^{0.5} A_0$	$V_n = 0.6\,f_y\,A_w\,C_v$	$\phi\,V_n$
x	26	656		92.8		70
y	10		14		407.0	407

Note:

1. The minimum Stud Shear Connectors (not shown on this spreadsheet) are 3/4" ϕ @ 12" O.C. in both directions of vertical and horizontal around WF steel shape. (AISC 360-05 I2.1g)

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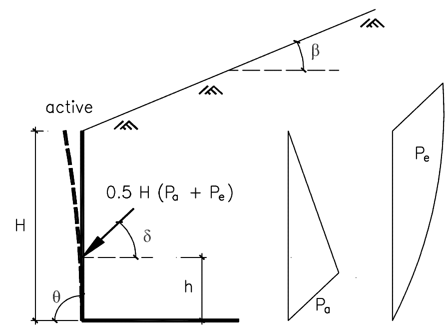
Seismic Earth Pressure of Deep Stiff Wall Based on FEMA P-750 & AASHTO/IBC

INPUT DATA & DESIGN SUMMARY

HORIZONTAL ACCELERATION (in g)	k_h	=	0.35	g
WALL HEIGHT	H	=	20	ft
SOIL SPECIFIC WEIGHT	γ_b	=	110	pcf
SOIL INTERNAL FRICTION ANGLE	ϕ	=	30	deg
SLOPE OF BACKFILL	β	=	15	deg
EXTERNAL FRICTION ANGLE	δ	=	17	deg
RACK ANGLE OF WALL FACE	θ	=	90	deg



The Seismic Earth Pressure: P_e = 29 psf / ft

The Active Earth Pressure: P_a = 41 psf / ft

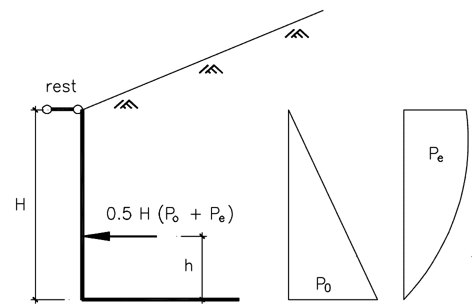
$0.5\ H\ (P_a + P_e)$ = 698 psf, *Total*

h = 8.87 ft

The At-rest Earth Pressure: P_0 = 55 psf / ft

$0.5\ H\ (P_o + P_e)$ = 839 psf, *Total*

h = 8.50 ft



The Passive Earth Pressure: P_p = 330 psf / ft

$0.5\ H\ (P_p + P_e)$ = 3589 psf, *Total*

h = 7.10 ft

ANALYSIS

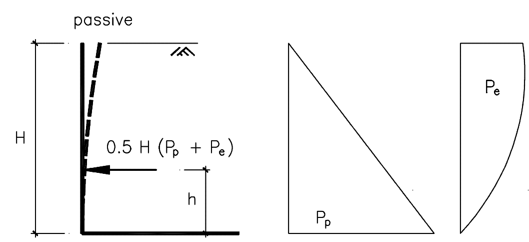
DETERMINE ACTIVE EARTH PRESSURE

$P_e = 0.75\ k_h\ \gamma_b$ = 29 psf / ft, (FEMA P-750 Page 356)

$P_a = \gamma_b\ K_a$ = 41 psf / ft

where $$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2} = 0.372$$

(Coulomb, AASHTO Figure 5.5.2A)



$h = (1/3\ P_a + 0.6\ P_e)\ H / (P_a + P_e)$ = 8.87 ft

DETERMINE AT-REST EARTH PRESSURE

$P_0 = \gamma_b\ K_0$ = 55 psf / ft

where $K_0 = 1 - \sin\phi$ = 0.500 , (AASHTO 5.5.2-2)

$h = (1/3\ P_o + 0.6\ P_e)\ H / (P_o + P_e)$ = 8.50 ft

DETERMINE PASSIVE EARTH PRESSURE

$P_p = \gamma_b\ K_p$ = 330 psf / ft

where $K_p = \frac{1+\sin\phi}{1-\sin\phi}$ = 3.000 , (Rankine, AASHTO Figure 5.5.2D)

δ = 0 deg

θ = 90 deg

$h = (1/3\ P_p + 0.6\ P_e)\ H / (P_p + P_e)$ = 7.10 ft

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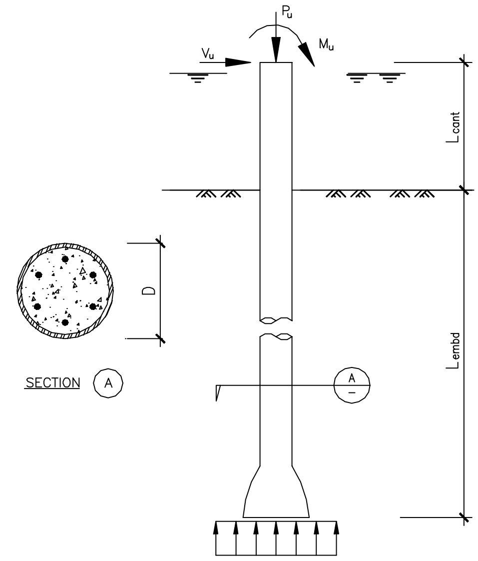
Caisson Design Based on 2015 IBC & 2016 CBC

DESIGN CRITERIA

1. The caisson is designed to rest on an underlying stratum of rock or satisfactory soil and is used when unsatisfactory soil/water exists. If caisson bottom end not belled, the caisson has to be drilled into hard stratum 3'-0' minimum.
2. All vertical/axial loads are supported by the bottom end hard stratum. The lateral loads are supported by the embeded soil, L_{embd}, based on IBC/CBC Chapter 18.



INPUT DATA & DESIGN SUMMARY

DIMENTION	L_{embd} =	24	ft, (7.32 m)
	L_{cant} =	10	ft, (3.05 m)
	D =	48	in, (1219 mm)
STEEL PLATE THICKNESS	t_s =	0.375	in, (10 mm), zero for no tube.
STEEL YIELD STRESS	F_y =	50	ksi, (345 MPa)
CONCRETE STRENGTH	f_c' =	5	ksi, (34 MPa)
THE MAX CONCRETE STRESS	β =	0.85	f_c' (0.85 on ACI, 1.0 on AISC)
VERTICAL REINFORCEMENT		12	# 11
		(zero for no bars.)	
TOP LOADS (LRFD / SD level)	M_u =	450	ft-kips, (610 kN-m)
	P_u =	260	kips, (1157 kN)
	V_u =	80	kips, (356 kN)

THE DESIGN IS ADEQUATE.

ALLOW SOIL PRESSURE IN BOTTOM END STRATUM Q_a = 18 ksf, (861.84 kN/m^2)

ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT P_p = 0.25 ksf / ft, (39.272 $[kN/m^2]/m$)

ANALYSIS

CHECK SOIL END BEARING CAPACITY (ACI 318-14 13.3.1.1)

$0.7 P_u + 0.25\,\pi\,D^2 (\gamma_c - \gamma_s)(L_{embd} + L_{cant})$ = 216.18 kips, (962 kN)

< $0.25\,\pi\,D^2 Q_a$ = 226.19 kips, (1006 kN) **[Satisfactory]**

Where γ_c = 0.15 kcf, one cubic foot of caisson weight γ_s = 0.07 kcf, soil/water weight

CHECK CAISSON EMBEDMENT LENGTH, L_{embd}, (IBC/CBC 1807.3)

By trials, use pole depth, $d = L_{embd}$ = 24.00 ft, (7.32 m)

Lateral bearing @ bottom, $S_3 = 2\,P_p\,Min(d\,,\,12')$ = 6.00 ksf, (287.28 kN/m^2)

Lateral bearing @ $d/3$, $S_1 = 2\,P_p\,Min(d/3\,,\,12')$ = 3.21 ksf, (153.64 kN/m^2)

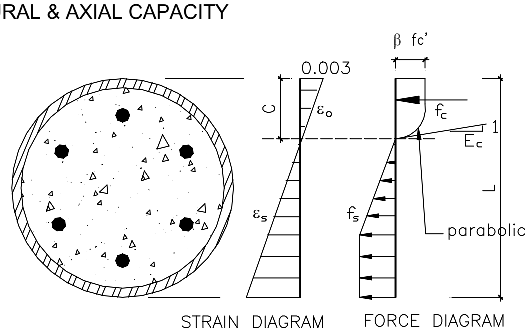
Require Depth is given by

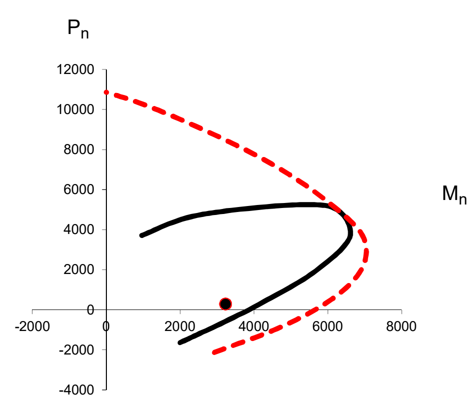
$$d = \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad =$$ 19.24 ft, (5.87 m) < L_{embd} **[Satisfactory]**

Where $P = 0.7\,V_u$ = 56 kips, (249 kN) $A = 2.34\,P/(D\,S_1)$ = 10.209

$h = M_u / V_u + L_{cont}$ = 15.625 ft, (4.76 m)

CHECK FLEXURAL & AXIAL CAPACITY





P_u / ϕ = 288.9 kips, (1285 kN) , ϕ = 0.9

M_{nc} @ P_u / ϕ = 4177.8 ft-kips, (5664 kN-m)

> $[V_u\,(L_{cant} + 0.67\,L_{embd}) + M_u\,]\,/\,\phi$ = 3228.9 ft-kips, (4378 kN-m)

[Satisfactory]

Solid Line - Tension Controlled

Dash Line - Compression Controlled

CHECK SHEAR CAPACITY (ACI 318 13.2.7.2 & 22.5)

V_u / ϕ = 106.7 kips, (474 kN) < $2\,A_c\,(f_c')^{0.5}$ = 248.0 kips, (1103 kN) **[Satisfactory]**

where A_c = 1753.5 in^2 (1131256 mm^2) ϕ = 0.75

Purchaser's LOGO

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Rectangular Machine or Tank Footing Design Based on ACI 318-14

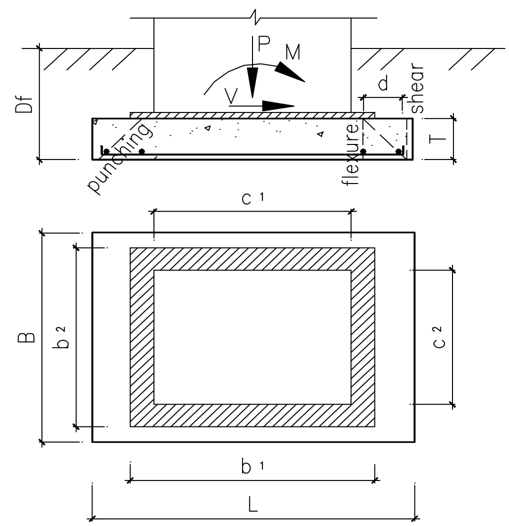
INPUT DATA

MACHINE WIDTH	c_1	=	40	ft
MACHINE DEPTH	c_2	=	20	ft
BASE PLATE WIDTH	b_1	=	42	ft
BASE PLATE DEPTH	b_2	=	22	ft
FOOTING CONCRETE STRENGTH	f_c'	=	2.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	23	k
AXIAL LIVE LOAD	P_{LL}	=	1450	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	0	Wind,SD
WIND AXIAL LOAD	P_{LAT}	=	4.6	k, SD
WIND MOMENT LOAD	M_{LAT}	=	11.5	ft-k, SD
WIND SHEAR LOAD	V_{LAT}	=	4.6	k, SD
SURCHARGE	q_s	=	0.1	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	2.5	ft
FOOTING THICKNESS	T	=	18	in
ALLOW SOIL PRESSURE	Q_a	=	2	ksf
FOOTING WIDTH	B	=	24	ft
FOOTING LENGTH	L	=	44	ft
REINFORCING SIZE		#	8	

THE FOOTING DESIGN IS ADEQUATE.

DESIGN SUMMARY

FOOTING WIDTH	B	=	24.00	ft
FOOTING LENGTH	L	=	44.00	ft
FOOTING THICKNESS	T	=	18	in
LONGITUDINAL REINF., TOP	1 # 8			
LONGITUDINAL REINF., BOT.	17 # 8 @ 17 in o.c.			
TRANSVERSE REINF., BOT.	30 # 8 @ 18 in o.c.			



ANALYSIS

DESIGN LOADS AT TOP OF FOOTING (IBC 1605.3.2 & ACI 318 5.3)

CASE 1:	DL + LL	P	=	1473	kips	1.2 DL + 1.6 LL	Pu	=	2348	kips
		M	=	0	ft-kips		Mu	=	0	ft-kips
		e	=	0.0	ft, fr cl ftg		eu	=	0.0	ft, fr cl ftg
CASE 2:	DL + LL + 0.6(1.3) W	P	=	1477	kips	1.2 DL + LL + 1.0 W	Pu	=	1482	kips
		M	=	13	ft-kips		Mu	=	12	ft-kips
		V	=	4	kips		Vu	=	5	kips
		e	=	0.0	ft, fr cl ftg		eu	=	0.0	ft, fr cl ftg
CASE 3:	DL + LL + 0.6(0.65) W	P	=	1475	kips	0.9 DL+ 1.0 W	Pu	=	25	kips
		M	=	8	ft-kips		Mu	=	12	ft-kips
		V	=	2	kips		Vu	=	5	kips
		e	=	0.0	ft, fr cl ftg		eu	=	0.5	ft, fr cl ftg

CHECK OVERTURNING FACTOR (2015 IBC 1605.2.1, 1808.3.1, & ASCE 7-10 12.13.4)

M_R / M_O = 8E+11 **>** F = 1.0 / 0.9 = 1.11 **[Satisfactory]**

Where M_O = $M_{LAT} + V_{LAT}\, T - P_{LAT} L_2$ = 0 k-ft

P_{ftg} = (0.15 kcf) T B L = 237.60 k, footing weight

P_{soil} = $w_s\,(D_f - T)$ B L = 116.16 k, soil weight

M_R = $P_{DL} L_2 + 0.5\,(P_{ftg} + P_{soil})$ L = 8289 k-ft

FOR REVERSED LATERAL LOADS,

M_R / M_O = 8E+11 **>** F = 1.0 / 0.9 **[Satisfactory]**

Where M_O = $M_{LAT} + V_{LAT}\, D_f - P_{LAT} L_1$ = 0 k-ft

M_R = $P_{DL} L_1 + 0.5\,(P_{ftg} + P_{soil})$ L = 8289 k-ft

CHECK SLIDING (2015 IBC 1807.2.3)

1.5 ($V_{Lat,\,ASD}$) = 4.14 kips < $\mu\, \Sigma W$ = 104.24 kips **[Satisfactory]**

Where ` μ = 0.4

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318 13.3.1.1)

Service Loads	CASE 1	CASE 2	CASE 3	
P	1473.0	1476.6	1474.8	k
e	0.0	0.0	0.0	ft (from center of footing)
q_s B L	105.6	105.6	63.4	k, (surcharge load)
$(0.15\text{-}w_s)$T B L	63.4	63.4	38.0	k, (footing increased)
Σ P	1642.0	1645.5	1576.2	k
e_L	0.0 < L/6	0.0 < L/6	0.0 < L/6	ft
e_B	0.0 < B/6	0.0 < B/6	0.0 < B/6	ft
q_L	37.3	37.5	35.9	k / ft
q_{max}	1.6	1.6	1.5	ksf
q_{allow}	2.0	2.7	2.7	ksf

Where

$$q_L = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e_L}{L}\right)}{L}, & for \quad e_L \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3(0.5L - e_L)}, & for \quad e_L > \dfrac{L}{6} \end{cases} \qquad q_{MAX} = \begin{cases} \dfrac{q_L\left(1+\dfrac{6e_B}{B}\right)}{B}, & for \quad e_B \le \dfrac{B}{6} \\ \dfrac{2q_L}{3(0.5B - e_B)}, & for \quad e_B > \dfrac{B}{6} \end{cases}$$

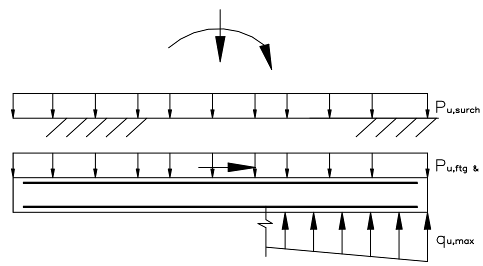
[Satisfactory]

DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318 13, 21, & 22)

$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L - e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

$$\rho_{MAX} = \frac{0.85\beta_1 f_c^{'}}{f_y}\,\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$$



$$\rho = \frac{0.85 f_c^{'}\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f_c^{'}}}\right)}{f_y}$$

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	2347.6	1482.2	25.3	k
e_u	0.0	0.0	0.7	ft
γ q_s B L	169.0	105.6	0.0	k, (factored surcharge load)
$\gamma[0.15T + w_s(D_f - T)]BL$	424.5	424.5	318.4	k, (factored footing & backfill loads)
Σ P_u	2941.1	2012.3	343.7	k
e_u	0.0 < L/6	0.0 < L/6	0.1 < L/6	ft
$q_{u,\ max}$	2.785	1.908	0.328	ksf

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **1**

Section	0	0.125 L	0.25 L	0.375 L	Cen_L	Cen_R	0.625 L	0.75 L	0.875 L	L
X_u (ft, dist. from left of footing)	0	5.50	11.00	16.50	22.00	22.00	27.50	33.00	38.50	44.00
$M_{u,machine}$ (ft-k)	0	-806.99	-3228	-7262.9	-12912	-12912	-20175	-29052	-39542	-51647
$V_{u,machine}$ (k)	0	293.5	586.9	880.4	1173.8	1173.8	1467.3	1760.7	2054.2	2347.6
$P_{u,surch}$ (klf)	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84
$M_{u,surch}$ (ft-k)	0	-58.1	-232.3	-522.7	-929.3	-929.3	-1452.0	-2090.9	-2845.9	-3717.1
$V_{u,surch}$ (k)	0	21.1	42.2	63.4	84.5	84.5	105.6	126.7	147.8	169.0
$P_{u,ftg\ \&\ fill}$ (klf)	9.65	9.65	9.65	9.65	9.65	9.65	9.65	9.65	9.65	9.65
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-145.9	-583.7	-1313.3	-2334.8	-2334.8	-3648.2	-5253.3	-7150.4	-9339.3
$V_{u,ftg\ \&\ fill}$ (k)	0	53.1	106.1	159.2	212.3	212.3	265.3	318.4	371.4	424.5
$q_{u,soil}$ (ksf)	2.79	2.79	2.79	2.79	2.79	2.79	2.79	2.79	2.79	2.79
$M_{u,soil}$ (ft-k)	0	1011.0	4044.0	9098.9	16175.9	16175.9	25274.8	36395.8	49538.7	64703.6
$V_{u,soil}$ (k)	0	-367.6	-735.3	-1102.9	-1470.5	-1470.5	-1838.2	-2205.8	-2573.4	-2941.1
Σ M_u (ft-k)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
Σ V_u (kips)	**0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0**

(cont'd)

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **2**

Section	0	0.125 L	0.25 L	0.375 L	Cen_L	Cen_R	0.625 L	0.75 L	0.875 L	L
X_u (ft, dist. from left of footing)	0	5.50	11.00	16.50	22.00	22.00	27.50	33.00	38.50	44.00
$M_{u,machine}$ (ft-k)	0	-509.22	-2036.9	-4583	-8147.5	-8147.5	-12730	-18332	-24952	-32590.0
$V_{u,machine}$ (k)	0	185.3	370.6	555.8	741.1	741.1	926.4	1111.7	1296.9	1482.2
$P_{u,surch}$ (klf)	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40
$M_{u,surch}$ (ft-k)	0	-36.3	-145.2	-326.7	-580.8	-580.8	-907.5	-1306.8	-1778.7	-2323.2
$V_{u,surch}$ (k)	0	13.2	26.4	39.6	52.8	52.8	66.0	79.2	92.4	105.6
$P_{u,ftg\ \&\ fill}$ (klf)	9.65	9.65	9.65	9.65	9.65	9.65	9.65	9.65	9.65	9.65
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-145.9	-583.7	-1313.3	-2334.8	-2334.8	-3648.2	-5253.3	-7150.4	-9339.3
$V_{u,ftg\ \&\ fill}$ (k)	0	53.1	106.1	159.2	212.3	212.3	265.3	318.4	371.4	424.5
$q_{u,soil}$ (ksf)	1.90	1.90	1.90	1.91	1.91	1.91	1.91	1.91	1.91	1.91
$M_{u,soil}$ (ft-k)	0	690.9	2764.1	6219.8	11058.5	11058.5	17280.7	24886.8	33877.3	44252.5
$V_{u,soil}$ (k)	0	-251.3	-502.6	-754.0	-1005.5	-1005.5	-1257.1	-1508.8	-1760.5	-2012.3
Σ M_u (ft-k)	**0**	**-0.5**	**-1.7**	**-3.2**	**-4.6**	**-4.6**	**-5.4**	**-5.2**	**-3.5**	**0**
Σ V_u (kips)	**0**	**0.3**	**0.5**	**0.6**	**0.6**	**0.6**	**0.6**	**0.5**	**0.3**	**0**

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **3**

Section	0	0.125 L	0.25 L	0.375 L	Cen_L	Cen_R	0.625 L	0.75 L	0.875 L	L
X_u (ft, dist. from left of footing)	0	5.50	11.00	16.50	22.00	22.00	27.50	33.00	38.50	44.00
$M_{u,machine}$ (ft-k)	0	-8.4094	-33.638	-75.684	-134.55	-134.55	-210.23	-302.74	-412.06	-538.2
$V_{u,machine}$ (k)	0	3.2	6.3	9.5	12.7	12.7	15.8	19.0	22.1	25.3
$P_{u,surch}$ (klf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{u,surch}$ (ft-k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$V_{u,surch}$ (k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{u,ftg\ \&\ fill}$ (klf)	7.24	7.24	7.24	7.24	7.24	7.24	7.24	7.24	7.24	7.24
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-109.4	-437.8	-985.0	-1751.1	-1751.1	-2736.1	-3940.0	-5362.8	-7004.4
$V_{u,ftg\ \&\ fill}$ (k)	0	39.8	79.6	119.4	159.2	159.2	199.0	238.8	278.6	318.4
$q_{u,soil}$ (ksf)	0.32	0.32	0.32	0.32	0.33	0.33	0.33	0.33	0.33	0.33
$M_{u,soil}$ (ft-k)	0	117.4	469.7	1057.5	1881.1	1881.1	2941.0	4237.6	5771.3	7542.6
$V_{u,soil}$ (k)	0	-42.7	-85.5	-128.3	-171.2	-171.2	-214.2	-257.3	-300.4	-343.7
Σ M_u (ft-k)	**0**	**-0.5**	**-1.7**	**-3.2**	**-4.6**	**-4.6**	**-5.4**	**-5.2**	**-3.5**	**0**
Σ V_u (kips)	**0**	**0.3**	**0.5**	**0.6**	**0.6**	**0.6**	**0.6**	**0.5**	**0.3**	**0**

DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	use	ρ_{provD}
Top Longitudinal	5	ft-k	15.50	0.0000	0.0000	0.0129	no limit	1 # 8	0.0002
Bottom Longitudinal	33.4	ft-k	14.50	0.0002	0.0001	0.0129	18	17 # 8 @ 17 in o.c.	0.0032
Bottom Transverse	1	ft-k / ft	14.00	0.0002	0.0001	0.0129	18	30 # 8 @ 18 in o.c.	0.0032

[Satisfactory]

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$		$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$		check $V_u < \phi\ V_c$
Longitudinal	0.0	k	313	k	**[Satisfactory]**
Transverse	0.0	k / ft	13	k / ft	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J} \qquad A_P = 2(b_1 + b_2)d \qquad \phi v_c(psi) = \phi(2+y)\sqrt{f'_c}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] \qquad \gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} \qquad y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

$$R = \frac{P_u b_1 b_2}{A_f} \qquad A_f = BL \qquad b_0 = \frac{A_P}{d}, b_1 = (0.5c_1 + 0.5b_1 + d), b_2 = (0.5c_2 + 0.5b_2 + d)$$

Case	P_u	M_u	b_1	b_2	b_0	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\ (psi)}$	$\phi\ v_c$
1	2347.6	0.0	506.0	266.0	10.7	0.5	1.9	2.0	1056.0	150.1	2077.9	37580.2	12.5	150.0
2	1482.2	11.5	506.0	266.0	10.7	0.5	1.9	2.0	1056.0	150.1	1311.9	37580.2	7.9	150.0
3	25.3	11.5	506.0	266.0	10.7	0.5	1.9	2.0	1056.0	150.1	22.4	37580.2	0.1	150.0

[Satisfactory]

where ϕ = 0.75 , (ACI 318 21.2)

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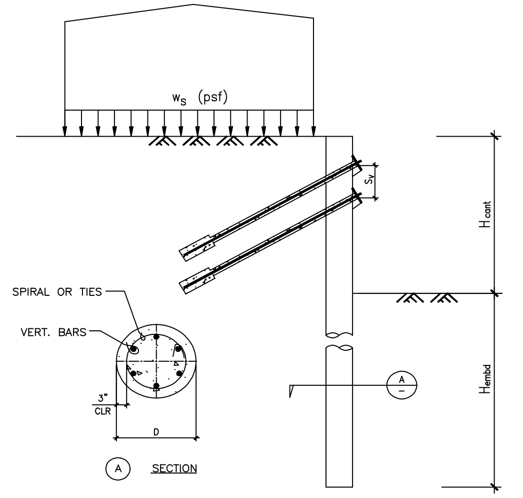
Sheet Pile Wall, with Tieback Anchors, Design Based on AASHTO (HB-17), 2015 IBC & ACI 318-14

INPUT DATA & DESIGN SUMMARY



HEIGHT OF CANTILEVER	H_{cant}	=	18	ft
SURCHARGE WEIGHT	w_s	=	500	psf
ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT				
	P_p	=	300	psf / ft
LATERAL SOIL PRESSURE	P_a	=	35	pcf (equivalent fluid pressure)
SEISMIC GROUND SHAKING	P_E	=	450	psf / ft (for H_{cant} > 12 ft only)
SOIL SPECIFIC WEIGHT	γ_b	=	110	pcf
CONCRETE STRENGTH	f_c'	=	4	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	44	in
PILE SPACING	S	=	4.0333	ft, o.c.
PILE VERT. REINF.	16	#	11	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	# 6	@	8	in o.c.

TIEBACK ANCHOR α = 20 deg, *Slope to horizontal*

T_b = 230 kips / anchor, SD, *from Soil Report, or AASHTO Tab 5.7.6.2*

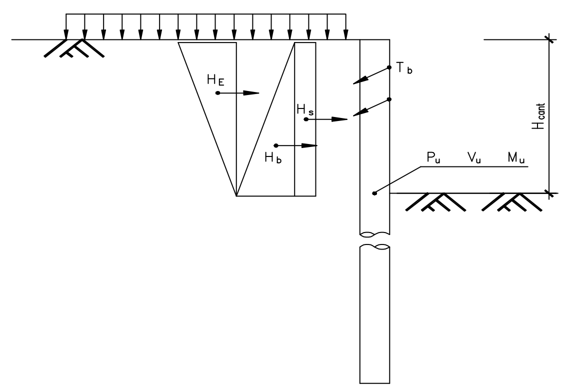
S_v = 9 ft, o.c., *vertical* , S_h = 4 ft, o.c., *horizontal*

THE SHORING DESIGN IS ADEQUATE.

(H_{embd} : 16.77 ft. Min. Req'D)

ANALYSIS

DETERMINE PILE SECTION FORCES AT CANTILEVER BOTTOM



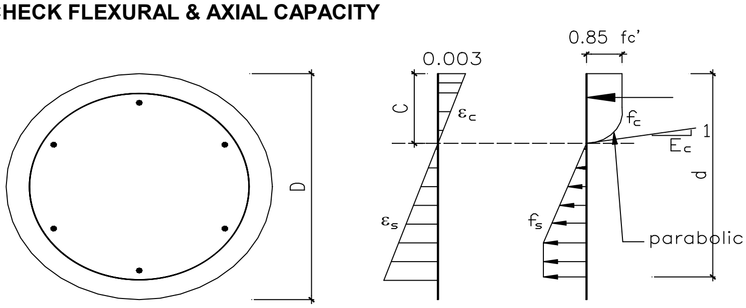
$H_b = 0.5\ S\ P_a\ (H_{cant})^2$ =	22.87	kips, ASD
$H_s = w_s\ S\ P_a\ (H_{cant}) / \gamma_b$ =	11.55	kips, ASD
$H_E = 0.5\ S\ P_E\ (H_{cant})^2$ =	294.03	kips, ASD
$H_{Tieback} = T_b\ (Cos\ \alpha)\ S\ (H_{cant}) / (S_v\ Sh)$ =	435.86	kips, SD
$P = S\ D\ w_s + 0.25\ \pi\ \gamma_c\ D^2\ H_{cant}$ =	35.90	kips, ASD
$V = H_b + H_E + H_s$ =	328.45	kips, ASD
$M = (H_b /3 + 2H_E /3 + H_s /2\ )\ H_{cant}$ =	2922.1	ft-kips, ASD
$P_u = 1.2\ P$ =	43.09	kips, SD
V_u = MAX (0, 1.6 V - $H_{Tieback}$) =	89.66	kips, SD
M_u = MAX (0, 1.6 M - 0.5 $H_{Tieback}$)=	752.6	ft-kips, SD

CHECK PILE LIMITATIONS

f_c' = 4 ksi > 4 ksi **[Satisfactory]** (IBC Table 1808.8.1)

D = 44 in > MAX[($H_{cant} + H_{embd}$) / 30 , 12 in] **[Satisfactory]** (IBC 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY



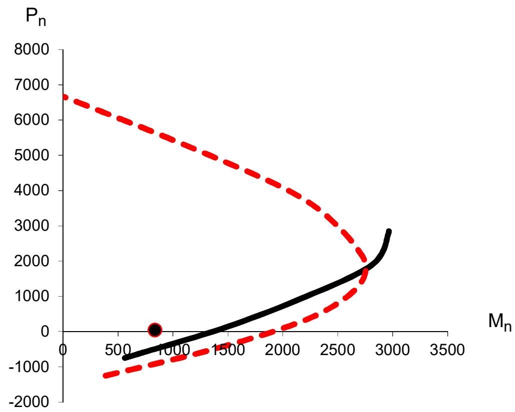
STRAIN DIAGRAM FORCE DIAGRAM

$$\varepsilon_O = or\left[\frac{2\left(\beta f_C'\right)}{E_C}\ ,\ \varepsilon_{max}\right]\ ,\ E_C = 57\sqrt{f_C'}\ ,\ E_S = 29000 ksi$$

$$f_C = \begin{cases} \beta f_C'\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right]\ , & for\ \ 0 < \varepsilon_C < \varepsilon_O \\ \beta f_C'\ ,\ for\ \ \varepsilon_C \ge \varepsilon_O & \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S\ ,\ for\ \ \varepsilon_S \le \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_S > \varepsilon_t \end{cases}$$

(cont'd)



$P_u / \phi =$ 47.9 *kips, (213 kN)*

M_{nc} @ $P_u / \phi =$ 1405.7 *ft-kips, (1906 kN-m)*

> $M_u / \phi =$ 836.2 *ft-kips, (1134 kN-m)*

[Satisfactory]

where $\phi =$ 0.900 , (ACI 318-14 21.2)

d = 39.545 in, (ACI 20.6)

Solid Line - Tension Controlled

Dash Line - Compression Controlled

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c) =$ 312 kips, (ACI 318-14 22.5)

> Vu **[Satisfactory]**

where $\phi =$ 0.75 (ACI 318 21.2)

A_0 = 1228 in^2. A_v = 0.88 in^2. f_y = 60 ksi

V_c = $2 (f_c')^{0.5} A_0 =$ 155.4 kips, (ACI 318-14 22.5)

V_s = MIN $(d\ f_y\ A_v / s\ ,\ 8\ (f_c')^{0.5} A_0)$ = 261.0 kips, (ACI 318-14 22.5.1)

s_{max} = 12 (IBC 1810.3.9.4.2) s_{provd} = 8 in

s_{min} = 1 **[Satisfactory]**

ρ_s = $0.12\ f_c' / f_{yt} =$ 0.008 < $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-14 18.13.4.3 & 18.7.5.1)

DETERMINE PILE EMBEDMENT LENGTH, H_{embd} , (IBC 1807.3)

By trials, use pole depth, $d = H_{embd}$ = 16.77 ft

Lateral bearing @ bottom, $S_3 = 2 P_p\ Min(d\ ,\ 12')$ = 7.20 ksf

Lateral bearing @ $d/3$, $S_1 = 2 P_p\ Min(d/3\ ,\ 12')$ = 3.37 ksf

Require Depth is given by

$$d = \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad = \quad 16.77 \text{ ft} \qquad \textbf{[Satisfactory]}$$

Where $P = V =$ 328.45 kips

$A = 2.34 P / (D S_1) =$ 10.63

$h = M / V =$ 8.90 ft

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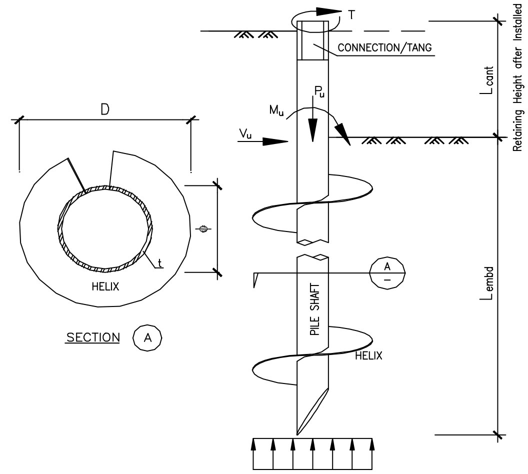
Screw Pile Design Based on 2018 IBC, ACI 318-14 & AISC 360-16



DESIGN CRITERIA

1. Steel screw piles do not cost time waiting for concrete strength and have less disturbed soil, so they can be used near tree roots and other foundations.
2. This is capacity design, not included environmental corrosion parameters.

INPUT DATA & DESIGN SUMMARY

DIMENTION L_{embd} = 50 *ft, (15.24 m)*

L_{cant} = 10 *ft, (3.05 m)*

D = 48 *in, (1219 mm)*

PILE WEB THICKNESS t = 0.375 *in, (10 mm), zero for no tube.*

ϕ = 8 *in, (203 mm)*

STEEL YIELD STRESS F_y = 50 *ksi, (345 MPa)*

MAX LOADS *(LRFD / SD level)* M_u = 73 *ft-kips, (99 kN-m)*

P_u = 80 *kips, (356 kN)*

V_u = 80 *kips, (356 kN)*

TOP INSTALLATION TORSION *(ASD level)* T = 289 *ft-kips, (392 kN-m)*

THE DESIGN IS ADEQUATE.

ALLOW SOIL PRESSURE IN BOTTOM END STRATUM Q_a = 18 *ksf, (861.84 kN/m^2)*

ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT P_p = 0.25 *ksf / ft, (39.272 $[kN/m^2]/m$)*

ANALYSIS

CHECK SOIL END BEARING CAPACITY (ACI 318-14 13.3.1.1)

$0.7\,P_u + t\,\pi\,\phi\,(\gamma_{pile} - \gamma_{soil})\,(L_{embd} + L_{cant})$ = 57.65 *kips, (256 kN)*

< $0.25\,\pi\,D^2\,Q_a$ = 226.19 *kips, (1006 kN)* **[Satisfactory]**

Where γ_{pile} = 0.49 *kcf, one cubic foot of pile weight* γ_{soil} = 0.07 *kcf, soil/water weight*

CHECK STEEL PILE EMBEDMENT LENGTH, L_{embd} , (IBC/CBC 1807.3)

By trials, use pole depth, $d = L_{embd}$ = 50.00 *ft, (15.24 m)*

Lateral bearing @ bottom, $S_3 = 2\,P_p\,Min(\,d\,,\,12'\,)$ = 6.00 *ksf, (287.28 kN/m^2)*

Lateral bearing @ $d/3$, $S_1 = 2\,P_p\,Min(\,d/3\,,\,12'\,)$ = 6.00 *ksf, (287.28 kN/m^2)*

Require Depth is given by

$$d = \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad =$$ 42.03 *ft, (12.81 m)* < L_{embd} **[Satisfactory]**

Where $P = 0.7\,V_u$ = 56 *kips, (249 kN)* $A = 2.34\,P\,/\,(\,\phi\,S_1\,)$ = 32.76

$h = M_u\,/\,V_u + L_{cont}$ = 10.913 *ft, (3.33 m)*

CHECK TORSIONAL CAPACITY (AISC 360 H3.1)

$$T_c = \frac{1}{\Omega_T}T_n = \frac{1}{\Omega_T}\frac{\pi(\phi-t)^2 t}{2}Max\left[\frac{1.23E}{\sqrt{\frac{L}{\phi}}\left(\frac{\phi}{t}\right)^{(5/4)}}\ ,\ \frac{0.60E}{\left(\frac{\phi}{t}\right)^{(3/2)}}\right] =$$ 301.78 *ft-kips, (409 kN-m)* > T **[Satisfactory]**

Where Ω_T = 1.67

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360 H1)

$$\begin{cases} \frac{P_u}{P_c}+\frac{8}{9}\left(\frac{M_u}{M_c}\right) & ,\ for\ \frac{P_u}{P_c}\ge 0.2 \\ \frac{P_u}{2P_c}+\left(\frac{M_u}{M_c}\right) & ,\ for\ \frac{P_u}{P_c}< 0.2 \end{cases} =$$ 1.28 < 1.3 **[Satisfactory]**
(2018 IBC, 1605.3.2)

Where $P_c = P_n\,\phi_c$ = 182 x 0.9 = 163.81 kips, (AISC 360 Chapter E)

> P_u **[Satisfactory]** *, where kL =* 0.5 L_{embd}

$M_c = M_n\,\phi_b$ = 90.92 x 0.9 = 81.83 ft-kips, (AISC 360 Chapter F)

> M_u **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360 G2)

$V_n\,\phi_v$ = 269.5 x 0.9 = 242.5 kips > V_u **[Satisfactory]**

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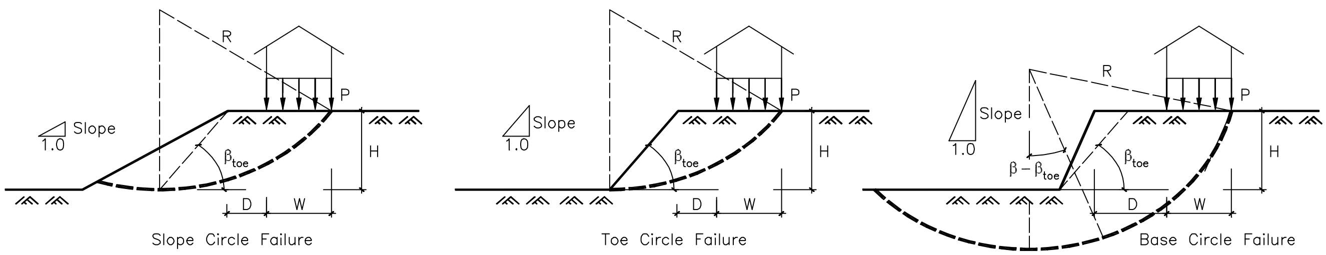
Slope Stability Analysis Based on AASHTO 17th & 2018 IBC

DESIGN CRITERIA

1. Assume that building/structure load, P, may cause three kind of failures: Slope Circle Failure, Toe Circle Failure, or Base Circle Failure. All failures started at the edge of building.
2. For wild fired mountain (tree total lost slope), the effective unit weight of soil, γ_{eff}, should input the both values of ($\gamma_{saturated}$ - γ_{water}) and $\gamma_{saturated}$ no matter if current soil is saturated or not, to check the worse condition.
3. The default cut angle of toe circle failure, β_{toe}, is 53°. When slope angle greater than it, the failure will be base circle failure, but less will be slope circle failure. User may change the default 53°, and input P zero load with different D and W values, to check general slope stability.



INPUT DATA & DESIGN SUMMARY

SOIL EFFECTIVE UNIT WEIGHT	γ_{eff} =	38	pcf, (609 kg/m³)
SOIL INTERNAL FRICTION ANGLE	ϕ =	30	deg
SOIL EFFECTIVE COHESION	c =	0.4	ksf, (19 kPa)
BUILDING PRESSURE (LOAD)	P =	4.2	ksf, (201 kPa)

DEMENSION H = 40 ft, (12.19 m) D = 10 ft, (3.05 m)

W = 30 ft, (9.14 m) R = *81.50* ft, (24.84 m)

SLOPE (Vertical : Horizontal Ratio) 1.05 : 1.0 (β = *46.397* < β_{toe} = **53** *deg , Slope Circle Failure apply.)*

THE DESIGN IS ADEQUATE.

Factor of Safety, ***F* = *1.51***

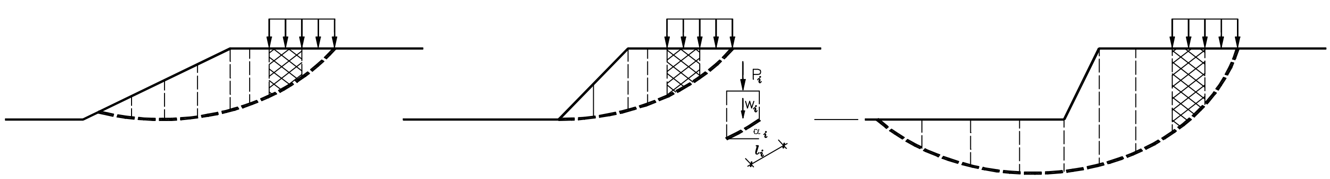
ANALYSIS

(Modified Bishop's Method of Analysis)

$$F = \frac{\sum_i \dfrac{c l_i + (w_i - u_i l_i)\tan\phi}{\cos\alpha_i + \dfrac{\sin\alpha_i \tan\phi}{F}}}{\sum_i w_i \sin\alpha_i} = \frac{\sum_i \dfrac{c l_i \cos\alpha_i + (w_i + P_i)\tan\phi}{Max\left(\cos\alpha_i + \dfrac{\sin\alpha_i \tan\phi}{F}\ ,\ 0.2\right)}}{\sum_i (w_i + P_i)\sin\alpha_i} = 1.51$$

> 1.5

[Satisfactory] (2018 IBC 1807.2.3 & AASHTO 17th 5.2.2.3)



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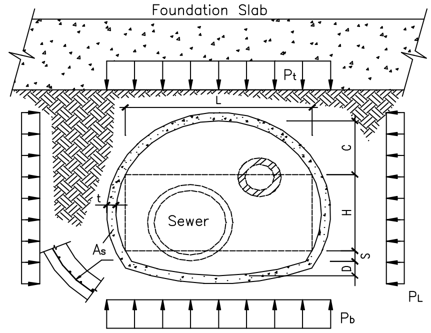
Underground Utilities Way Design Based on AASHTO-17th & 2018 IBC

DESIGN CRITERIA

1. The design that directly put utilities (sewer pipe, water main, or gas line) under foundation (PT/conventional slab on grade, mat footing) is inadequate. Even later the method of Trenchless Technology can be used to change/upgrade pipes, the leaking water and disturbed soil will reduce foundation capacity. For no California Basement, this software design combined utility (sewer, water, and gas) pipe way, which can be tunnel-shaped or full circler section (D + S = C).
2. No matter how big structural actual loads, if the foundation design adequate, the maximum vertical pressure of utility pipe way is allowable soil capacity plus soil, weight, and the maximum lateral pressure is allowable soil capacity.



INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	5	ksi, (34 MPa)
REBAR YIELD STRESS	f_y =	60	ksi, (414 MPa)
SECTION DIMENSIONS	L =	1.8	ft, (0.55 m)
	H =	1	ft, (0.30 m)
	C =	0.53	ft, (0.16 m)
	S =	2	in, (51 mm)
	D =	3	in, (76 mm)
	t =	4	in, (102 mm)

REINFORCING (A_s)

1 layer # 3 @ 8 in o.c., (203 mm), (curvered)

Concrete Cover = 0.75 in, (19 mm)

MAX TUNNEL DEEP UNDERGROUND	EMB =	3	ft, (0.91 m)
ALLOWABLE SOIL PRESSURE	Q_a =	2.2	ksf, (105 kPa), ASD
SOIL SPECIFIC WEIGHT	γ_b =	110	pcf, (1762 kg/m^3)

[THE DESIGN IS ADEQUATE.]

ANALYSIS

DETERMINE LOADS

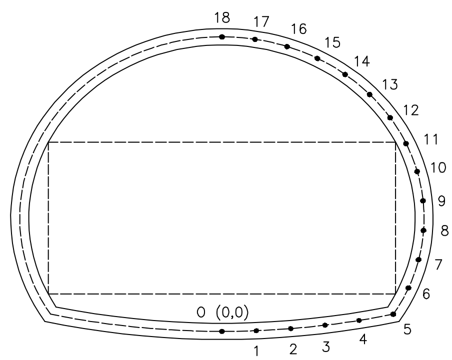
$P_b = 1.5\ Q_a$ + EMB γ_b = 3630 psf, (165 kPa)

$P_t = P_b$ - Wt = 3444 psf, (174 kPa)

P_L = (check from γ_b to Q_a) = 110 ft-pcf, (psf), (5 kPa)

DETERMINE TUNNEL WEB FORCES BY FINITE ELEMENT METHOD

Point	X (ft)	Y (ft)	P_u (k)	M_u (ft-k)	V_u (k)
0	0.00	0.00	0.12	1.28	0.00
1	0.19	0.01	0.29	1.22	0.98
2	0.38	0.05	0.61	1.03	1.57
3	0.56	0.11	1.07	0.73	2.06
4	0.74	0.19	1.63	0.33	2.42
5	0.90	0.29	2.79	-0.13	1.98
6	1.02	0.46	3.43	-0.55	1.65
7	1.12	0.65	3.91	-0.89	1.11
8	1.17	0.85	4.18	-1.13	0.44
9	1.20	1.06	4.19	-1.22	-0.27
10	1.18	1.27	3.96	-1.17	-0.95
11	1.13	1.48	3.51	-0.96	-1.51
12	1.04	1.67	2.90	-0.65	-1.88
13	0.92	1.84	2.20	-0.25	-2.01
14	0.78	1.99	1.50	0.17	-1.90
15	0.60	2.12	0.89	0.57	-1.55
16	0.41	2.21	0.43	0.90	-1.02
17	0.21	2.26	0.18	1.11	-0.35
18	0.00	2.28	0.18	1.19	0.00



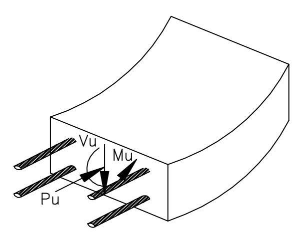
R_t =	1.20	ft, (top circle radius)
R_b =	1.52	ft, (bottom circle radius)
θ_t =	131.36	deg, (between 5 & 18)
θ_b =	36.13	deg, (between 0 & 5)

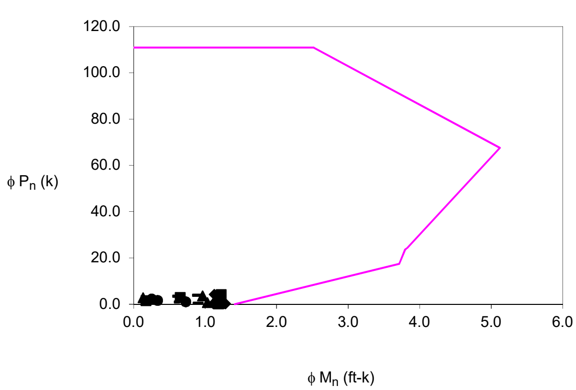
CHECK AXIAL & FLEXURE CAPACITY

ρ_{ProvD} = 0.006875 < ρ_{MAX} = 0.0800 (tension face only, ACI 318-14 7.3.3 or R21.2.2)

> ρ_{MIN} = 0.0012 (tension face only, ACI 318-14 9.6.1.2, 9.6.1.3 or Table 11.6.1)

[Satisfactory]





	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	110.9	0.0
AT MAXIMUM LOAD	110.9	2.5
AT MIDDLE	67.6	5.1
AT ε_t = 0.002	24.2	3.8
AT BALANCED	23.7	3.8
AT ε_t = 0.005	17.4	3.7
AT FLEXURE ONLY	0.0	1.4

(Note: For middle reinforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-14 9.6.3.1)

$\phi V_n = 2\phi bd\sqrt{f_c'}$ = 2.55 kips / ft > $V_{u,\,max}$ = 2.42 kips / ft **[Satisfactory]**

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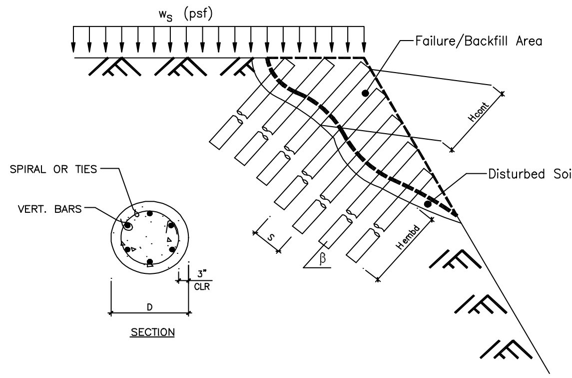
Landslide Repair Design Based on 2018 IBC & AASHTO 17th

DESIGN CRITERIA

1. There are many slope failure repair options. This software is only to design Sloped Retaining Piles (SRP), which have Soil Nail frictions and not have to be anchored in bedrock as tension Soil Anchors.
2. The SPR construction can be done, from bottom to top, by one layer backfill with one level piles. The link beams between piles, in each ways, may be required to hold on backfill soil.
3. The top SPR in backfill can be reinforcing concrete, WF/HSS steel members, or even wood poles.



INPUT DATA & DESIGN SUMMARY

THE MAX HEIGHT OF CANTILEVER	H_{cant}	=	6	ft
SURCHARGE WEIGHT	w_s	=	100	psf
ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT				
(including pile group reduction)	P_p	=	332	psf / ft
LATERAL SOIL PRESSURE (equivalent fluid pressure)	P_a	=	35	pcf
SEISMIC GROUND SHAKING	P_E	=	450	psf / ft
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf
CONCRETE STRENGTH	f_c'	=	4	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	22	in
PILE SPACING (Each Way)	S	=	8	ft, o.c. Each Way
PILE VERT. REINF.	11	#	9	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	# 5	@	8	in o.c.
PILE SLOPE ANGLE	β	=	42	°, deg

THE SHORING DESIGN IS ADEQUATE.

(H_{embd} = 18.78 ft. Min. Req'D)

ANALYSIS

DETERMINE PILE SECTION FORCES AT CANTILEVER BOTTOM

Step 1: Determine One Sheet Vertical Piles Loads:

$H_b = 0.5\ S\ P_a\ (H_{cant})^2 =$ 5.04 kips, ASD

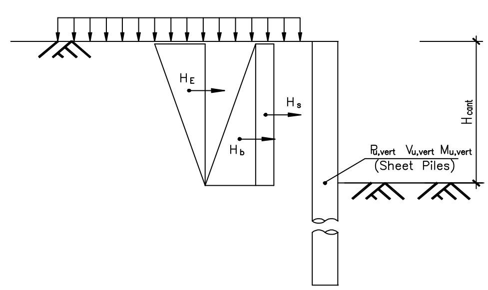
$H_s = w_s\ S\ P_a\ (H_{cant})\ /\ \gamma_b =$ 1.53 kips, ASD

$H_E = 0.5\ S\ P_E\ (H_{cant})^2 =$ 64.80 kips, ASD

$P = S\ D\ w_s + 0.25\ \pi\ \gamma_c\ D^2\ H_{cant} =$ 3.84 kips, ASD

$V = H_b + H_E + H_s =$ 71.37 kips, ASD

$M = (H_b/3 + 2H_E/3 + H_s/2)\ H_{cant} =$ 210.6 ft-kips, ASD



$P_{u,\ vert} = 1.2\ P =$ 4.61 kips, SD

$V_{u,\ vert} = 1.6\ V =$ 114.19 kips, SD

$M_{u,\ vert} = 1.6\ M =$ 336.94 ft-kips, SD

Step 2: Determine Single Slope Pile Loads:

$P_u = (P_{u,\ vert} \sin\beta + V_{u,\ vert} \cos\beta)\ /\ S + 1.2\ H_{cant} S^2\ \gamma_b)\ /\ \sin\beta =$ 86.74 kips, SD

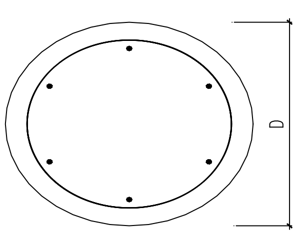
$V_u = (P_{u,\ vert} \cos\beta - V_{u,\ vert} \sin\beta)\ /\ S + 1.2\ H_{cant} S^2\ \gamma_b)\ /\ \cos\beta =$ 59.08 kips, SD

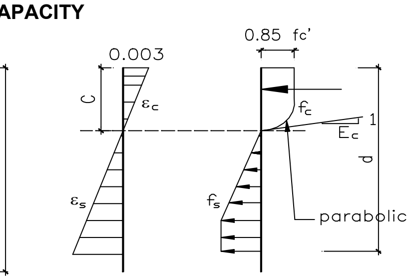
$M_u = (M_{u,\ vert})\ /\ S + 0.6\ H^2_{cant}\ S^2\ \gamma_b)\ /\ \cos\beta =$ 246.74 ft-kips, SD

CHECK PILE LIMITATIONS

f_c' =	4	ksi	>	4 ksi	**[Satisfactory]**	(IBC Table 1808.8.1)
D =	22	in	>	MAX[(H_{cant} + H_{embd}) / 30 , 12 in]	**[Satisfactory]**	(IBC 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY





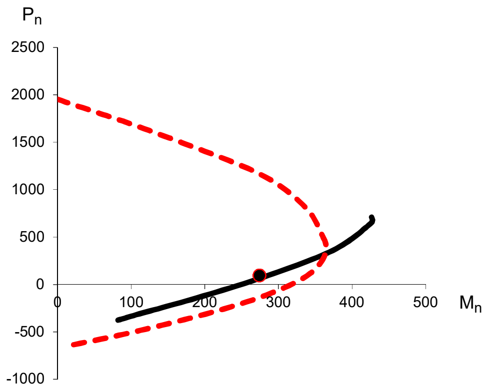
STRAIN DIAGRAM FORCE DIAGRAM

$$\varepsilon_o = or\left[\frac{2\left(\beta f_c'\right)}{E_c}\ ,\ \varepsilon_{max}\right]\ ,\ E_c = 57\sqrt{f_c'}\ ,\ E_s = 29000ksi$$

$$f_c = \begin{cases} \beta f_c'\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]\ ,\ for\ \ 0 < \varepsilon_c < \varepsilon_o \\ \beta f_c'\ ,\ for\ \ \varepsilon_c \geq \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \leq \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$

(cont'd)



$P_u / \phi =$ 96.4 *kips, (429 kN)*

M_{nc} @ $P_u / \phi =$ 288.5 *ft-kips, (391 kN-m)*

> $M_u / \phi =$ 274.2 *ft-kips, (372 kN-m)*

[Satisfactory]

where $\phi =$ 0.900 , (ACI 318-14 21.2)

d = 17.811 in, (ACI 20.6)

Solid Line - Tension Controlled

Dash Line - Compression Controlled

CHECK SHEAR CAPACITY

$\phi\ V_n = \phi\ (V_s + V_c) =$ 86 kips, (ACI 318-14 22.5)

> Vu **[Satisfactory]**

where $\phi =$ 0.75 (ACI 318 21.2)

A_0 = 249 in^2. A_v = 0.62 in^2. f_y = 60 ksi

V_c = $2\ (f_c')^{0.5} A_0 =$ 31.5 kips, (ACI 318-14 22.5)

V_s = MIN $(d\ f_y A_v / s\ ,\ 8\ (f_c')^{0.5} A_0)$ = 82.8 kips, (ACI 318-14 22.5.1)

s_{max} = 11 (IBC 1810.3.9.4.2) s_{provd} = 8 in

s_{min} = 1 **[Satisfactory]**

ρ_s = $0.12\ f_c' / f_{yt}$ = 0.008 **<** $\rho_{s,provd}$ = 0.010 **[Satisfactory]** (ACI 318-14 18.13.4.3 & 18.7.5.1)

DETERMINE PILE EMBEDMENT LENGTH, H_{embd} , (IBC 1807.3)

By trials, use pole depth, $d = H_{embd}$ = 18.78 ft

Lateral bearing @ bottom, $S_3 = 2\ P_p\ Min(\ d\ ,\ 12'\ ) =$ 7.97 ksf

Lateral bearing @ $d / 3$, $S_1 = 2\ P_p\ Min(\ d / 3\ ,\ 12'\ ) =$ 4.16 ksf

Require Depth is given by

$$d = \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad =$$ 18.78 ft **[Satisfactory]**

Where $P = V_u / 1.2 =$ 49.24 kips

$A = 2.34\ P / (D\ S_1) =$ 15.12

$h = M_u / V_u =$ 4.18 ft

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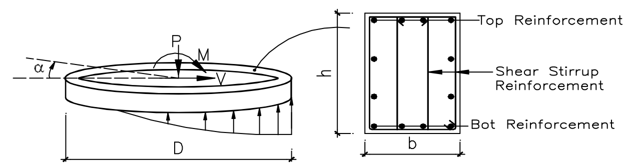
Ring Foundation Design Based on 2018 IBC & ACI 318-14

DESIGN CRITERIA

1. Assume that the ring foundation is rigid with linear distribution of soil reactions.
2. The ring foundation concrete design may be controlled by the maximum bending section, by the maximum shear force section, and/or by the maximum torsion section. Although the maximum force sections are not con-currently at the same location, this software is conservative to use all maximum forces to design entire sections.

INPUT DATA & DESIGN SUMMARY

TOTAL DEAD LOAD	P_{DL} =	40	*kips, (178 kN)*
TOTAL LIVE LOAD	P_{LL} =	38	*kips, (169 kN)*
LATERAL LOAD (0=Wind, 1=Seismic)		0	Wind,SD
WIND AXIAL LOAD	P_{LAT} =	5	*kips, (22 kN)*
WIND MOMENT LOAD	M_{LAT} =	39.5	*ft-kips, (54 kN-m)*
WIND SHEAR LOAD	V_{LAT} =	0.15	*kips, (1 kN)*
SOIL WEIGHT	w_s =	110	*pcf, (1762 kg/m^3)*
EMBEDMENT DEPTH (from bottom)	D_f =	2	*ft, (0.61 m)*
ALLOW SOIL PRESSURE	Q_a =	1.8	*ksf, (86 kPa)*
DIMENSIONS	D =	30	*ft, (9.14 m)*
	h =	48	*in, (1219 mm)*
	b =	24	*in, (610 mm)*
CONCRETE STRENGTH	f'_c =	4	*ksi, (28 MPa)*



THE DESIGN IS ADEQUATE.

REBAR STRENGTH	MAIN	f_y = 60	*ksi, (414 MPa)*
	STIRRUP	f_y = 60	*ksi, (414 MPa)*

TOP/BOT REINFORCEMENT 6 # 8

SHEAR REINFORCEMENT 4 legs # 5 @ 10 *in, (254 mm), o.c.*

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 1605.2.1, 1808.3.1, & ASCE 7 12.13.4)

M_R / M_O = 2183362.697 **>** F = 1.0 / 0.9 = 1.11 **[Satisfactory]**

Where M_O = $M_{LAT} + V_{LAT}\, h - P_{LAT}\,(0.5\, D)$ = 0 k-ft $M_R = (P_{DL} + P_{ftg})\,(0.5\, D)$ = 2183 k-ft

P_{ftg} = (0.15 kcf) h $\pi\,[D^2 - (D - 2b)^2]/ 4$ = 105.56 k, footing weight

P_{soil} = $w_s\,(D_f)\ \pi\,[D^2 - (D - 2b)^2]/ 4$ = 38.70 k, soil weight

COMBINED LOADS AT TOP FOOTING (IBC 1605.3.2 & ACI 318 5.3)

CASE 1:	DL + LL	P =	78.0	kips	1.2 DL + 1.6 LL	Pu =	108.8	kips
CASE 2:	DL + LL + 0.6(1.3) W	P =	81.9	kips	1.2 DL + LL + 1.0 W	Pu =	91.0	kips
		M =	31	ft-kips		Mu =	40	ft-kips
		V =	0.1	kips		Vu =	0.2	kips
		e =	0.4	ft, fr cl ftg		eu =	0.4	ft, fr cl ftg
CASE 3:	DL + LL + 0.6(0.65) W	P =	80.0	kips	0.9 DL+ 1.0 W	Pu =	41.0	kips
		M =	15	ft-kips		Mu =	40	ft-kips
		V =	0.1	kips		Vu =	0.2	kips
		e =	0.2	ft, fr cl ftg		eu =	1.0	ft, fr cl ftg

CHECK SOIL BEARING CAPACITY (ACI 318 13.3.1.1)

Service Loads	CASE 1	CASE 2	CASE 3	
P	78.0	81.9	80.0	k
e	0.0	0.4	0.2	ft (from center of footing)
$P_{ftg} - P_{soil}$	66.9	66.9	60.2	k, (footing increasing)
Σ P	144.9	148.8	140.1	k, (net loads)
e	0.0	0.2	0.1	ft
q_{min}	0.82	0.82	0.78	ksf
α °, (deg)		0.0	0.0	
q_{max}	0.82	0.87	0.81	ksf
$q_{allowable}$	1.80	2.40	2.40	ksf

[Satisfactory] Where Thicker Factor of Bottom Footing = 1.00 *, (including reversed T section.)*

DETERMINE RING FOOTING SECTION FORCES (ACI 318 13.3.1.1, 21, & 22)

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	108.8	91.0	41.0	k
e_u	0.0	0.4	1.0	ft
$\gamma\, P_{ftg}$	126.7	126.7	95.0	k, (factored footing & backfill)
$\Sigma\, P_u$	235.5	217.7	136.0	k
e_u	0.0	0.2	0.3	ft
$q_{u,\,min}$	1.34	1.20	0.74	ksf
α °, (deg)		0.0	0.0	
$q_{u,\,max}$	1.34	1.27	0.81	ksf
$M_{u,\,max}$		494.01	312.42	ft-k
α °, (deg)		90.0	90.0	
$T_{u,\,max}$		269.45	170.71	ft-k
α °, (deg)		140.0	140.0	
$V_{u,\,max}$		108.83	68.00	k

(Section forces, $M_{u,\,max}$, $T_{u,\,max}$, $V_{u,\,max}$, are only from soil reactions, very conservatively, without considered top uniform loads reduction.)

Use M_u = 494.01 *ft-kips, (670 kN-m)* V_u = 269.45 *kips, (1199 kN)* T_u = 108.83 *ft-kips, (148 kN-m)*

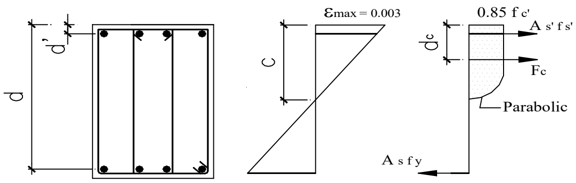
(cont'd)

CHECK FLEXURAL CAPACITY

$$\varepsilon_o = \varepsilon_{\max} \;\; or \;\; \frac{2\left(0.85 f'_c\right)}{E_c} \;\; , \;\; E_c = 57\sqrt{f'_c} \;\; , \;\; E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] \;\; , \;\; for \;\; 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c \;\; , \;\; for \;\; \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s \;\; , \;\; for \;\; \varepsilon_s \le \varepsilon_t \\ f_y \;\; , \;\; for \;\; \varepsilon_s > \varepsilon_t \end{cases}$$



Cover =	3	in, (ACI 318 20.6.1)	$\rho_{prov'd}$ =	0.0045	< ρ_{max} =	0.0207	, (ACI 318 9.3.3.1)
d =	43.88	in			> ρ_{min} =	0.0033	, (ACI 318 9.6.1)
d' =	4.13	in			**[Satisfactory]**		
ϕ =	0.90	, (ACI 318-14 21.2)	c =	7.96	in, by pure math method		
$\varepsilon_{c,max}$ =	0.0011		F_c =	211.07	kips		
$\varepsilon_{s,max}$ =	0.0050	, (ACI 318-14 21.2.2)	d_c =	2.89	in		

ϕM_n = 912.81 ft-k > M_u **[Satisfactory]**

CHECK SHEAR CAPACITY

Check section limitation (ACI 22.5.5 & 22.5.1.2)

$$V_u \le 10\phi b_w d \sqrt{f'_c}$$

269.4 < 499.5 kips **[Satisfactory]**

where ϕ = 0.75

Determine concrete capacity (ACI 22.5.5.1)

$$V_c = 2 b_w d \sqrt{f'_c} = \quad 133.20 \text{ kips}$$

$$V_c = \left(1.9A + 2500\rho_w B\right) b_w d = \quad 138.39 \text{ kips}$$,<== applicable

where $A = MIN\left(\sqrt{f'_c} \;\; , \;\; 100\right) =$ 63.25

$B = MIN\left(\frac{V_u d}{M_u} \;\; , \;\; 1.0\right) =$ 1.000

Check shear reinforcement (ACI 22.5)

$$\left(\frac{A_v}{s}\right)_{ReqD} = \begin{cases} 0 \;\; , \;\; for \;\; V_u < \frac{\phi V_c}{2} \\ MAX\left(\frac{50 b_w}{f_y} \;\; , \;\; \frac{0.75\sqrt{f'_c} b_w}{f_y}\right) \;\; , \;\; for \;\; \frac{\phi V_c}{2} \le V_u \le \phi V_c \\ \frac{V_u - \phi V_c}{\phi d f_y} \;\; , \;\; for \;\; \phi V_c \le V_u \end{cases}$$

= 1.007 in^2 / ft < $\left(\frac{A_v}{s}\right)_{ProvD}$ = 1.488 in^2 / ft **[Satisfactory]**

Check spacing limits for shear reinforcement (ACI 22.6.9.5)

$$V_s = \frac{V_u - \phi V_c}{\phi} = \quad 0.00$$ kips, (ACI 22.5.1.1)

$$S_{\max,shear} = \begin{cases} MIN(\frac{d}{2} \;\; , \;\; 24) \;\; for \;\; V_s \le 4 b_w d \sqrt{f'_c} \\ MIN(\frac{d}{4} \;\; , \;\; 12) \;\; for \;\; V_s > 4 b_w d \sqrt{f'_c} \end{cases}$$

= 21 > S = 10 in

[Satisfactory]

CHECK TORSION CAPACITY

Check section limitation (ACI 22.7.7.1)

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$$

0.283 < 0.474 **[Satisfactory]**

where ϕ = 0.75 (ACI 21.2)

P_h = 130 in, (perimeter of centerline of outermost closed transverse torsional reinforcement.)

A_{12} = 904 in^2 (area enclosed by centerline of the outermost closed transverse torsional reinforcement.)

Check if torsional reinforcement required (ACI 9.5.4.1)

$$T_u \le \phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

108.8 > 36.4 ft-k

Torsional reinforcement reqD.

where b_e = MIN(h-h_f , 4h_f) = 0 in, (one side, ACI 9.2.4.4)

P_{cp} = 144 in, (outside perimeter of the concrete cross section.)

A_{cp} = 1,152 in^2 (area enclosed by outside perimeter of concrete cross section.)

Check the max factored torque causing cracking (ACI 22.7.3.2)

$$T_u \le 4\phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

108.8 < 145.7

Reduction of the torsional moment can occur.

Determine the area of one leg of a closed stirrup (ACI 22.7.6.1)

$$\frac{A_t}{s} = \frac{T_u}{2\phi A_0 f_{yv}} = \frac{T_u}{1.7\phi A_{0h} f_{yv}} =$$ 0.23 in^2 / ft < actual = 0.372 **[Satisfactory]**

(cont'd)

Determine the corresponding area of longitudinal reinforcement (derived from ACI 22.7.6.1 & 9.6.4.3)

$$A_L = MAX\left[\frac{A_t}{s}P_h\frac{f_{yv}}{f_{yL}}\ ,\ \frac{5A_{cp}\sqrt{f'_c}}{f_{yL}} - P_h\frac{f_{yv}}{f_{yL}}MAX\left(\frac{A_t}{s}\ ,\ \frac{25b_w}{f_{yv}}\right)\right] = \quad 3.63 \quad in^2$$

Determine minimum combined area of longitudinal reinforcement

$A_{L,\ top} = A_s' + 0.5A_L$ =	1.81	in^2	**<**	actual	**[Satisfactory]**
$A_{L,\ bot} = A_s + 0.5A_L$ =	4.38	in^2	**<**	actual	**[Satisfactory]**

Determine minimum diameter for longitudinal reinforcement (ACI 25.7.1.2)

d_{bL} = MAX(0.042 S, 3/8) = 0.50 in **<** 1.00 in **[Satisfactory]**

Determine minimum combined area of stirrups (ACI 9.6.4.2 & 9.7.6.3.3)

$(A_v + 2A_t)$ / S = 0.74 in^2 / ft **>** MAX [$0.75(f_c')^{0.5}b_w/f_{yv}$, $50b_w/f_{yv}$] = 0.24 in^2 / ft

$S_{max,\ tor}$ = MIN[(P_h/8, 12) = 12 in **[Satisfactory]**

S_{reqD} = MIN($S_{max,shear}$, $S_{max,tor}$) = 12 in **>** actual **[Satisfactory]**

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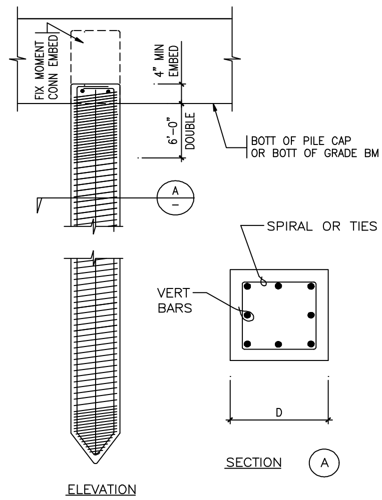
Driven Precast Concrete Pile Design Based on 2018 IBC & ACI 318-14

DESIGN CRITERIA

1. The 2018 IBC & ACI 318-14 do not have slenderness ratio limits. But for solid compression steel member, the AISC 360-16 E2 limits the effective slenderness ratio not exceed 200. Since concrete strength less than steel and concrete cracking, the effective slenderness ratio for compression concrete member should be less than 200. (80 adequate ?) After pile installed, there are no buckling issues, so ACI 318-14 6.6 not apply.
2. There are at least three load cases that have to be checked: the maximum tension section at erection, the maximum bending section at shipping, and the maximum P_u - M_u section at service.
3. Both soil capacity and concrete capacity may control the pile design. Check soil report if soil capacity adequate or not. And the driving top load should be less than vertical soil capacity.



INPUT DATA & DESIGN SUMMARY

REBAR YIELD STRESS	f_y =	60	ksi, (414 MPa)
CONCRETE STRENGTH	f_c' =	5	ksi, (34 MPa)
SECTION DIMENSION	D =	12	in, (305 mm)
PILE LENGTH	L =	42	ft, (12.80 m)

VERT. REINF. (4, 8, 12) 4 # 7

LATERAL REINF. (0=Spirals, 1=Ties) 0 Spirals

3 @ 4 in. (102 mm), o.c.

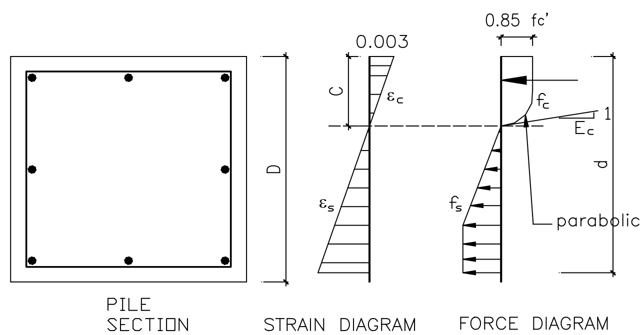
THE PILE DESIGN IS ADEQUATE.

SD LEVEL MAXIMUM SECTION LOADS

P_u = 200 kips, (890 kN)

M_u = 40 ft-kips, (54 kN-m)

V_u = 35 kips, (156 kN)



ANALYSIS

CHECK PILE CAPACITY IN SHIPPING & ERECTION

$T_{u,max}$ = 9 kips

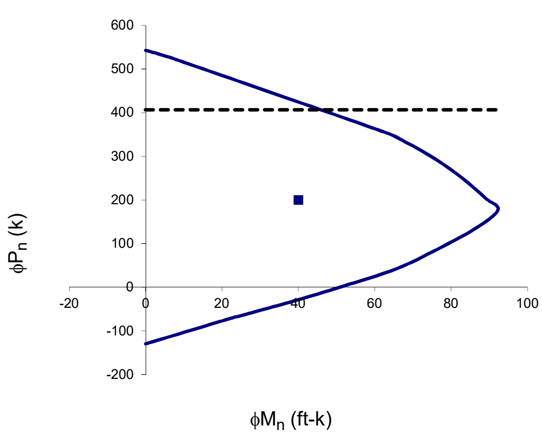
< ϕT_n = 130 kips

$M_{u,pure}$ = 46 ft-kips [Satisfactory]

< ϕM_n = 51 ft-kips

$k L / r = 12^{0.5}\ 1.0 L / D$ = 145 < 200

CHECK FLEXURAL & AXIAL CAPACITY



ε_c = 0.003 , (ACI 318-14 22.2.2)

ϕ = 0.753 , (for P_u & M_u, ACI 318-14 21.2)

d = 10.6 in

Cover = 1.00 in

c_b = 6.3 in, (balance point between Tension Controlled and Compression Controlled.)

P_u = 200 kips

< ϕP_n = 407 kips, (ACI 318-14 22.4.2)

M_u = 40 ft-kips

< ϕM_n = 90 ft-kips, at P_u level.

[Satisfactory]

CHECK SHEAR CAPACITY

V_u = 35 kips < $\phi V_n = \phi (V_s + V_c)$ = 38 kips, (ACI 318-14 22.5.1) [Satisfactory]

where ϕ = 0.75 (ACI 318-14 21.2)

V_c = $2 (f_c')^{0.5} A_0$ = 10.2 kips, (ACI 318-14 22.5.5)

V_s = MIN ($d f_y A_v / s$, $4V_c$) = 40.7 kips, (ACI 318-14 22.5.10.5)

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Foundation Design for Dynamic Equipment Based on ACI 351.3 & ACI 318-14

DESIGN CRITERIA

The basic concept of design is load less than capacity:

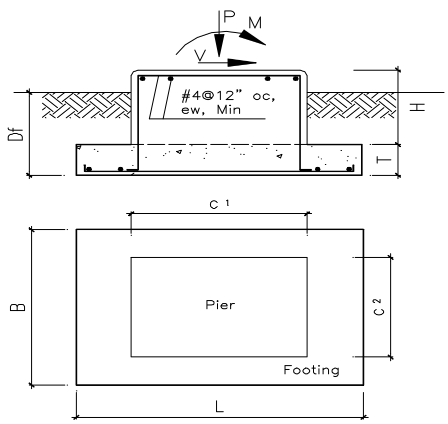
1. For capacity, there are two issues of foundation design: soil capacity and reinforced concrete capacity. No matter the equipment dynamic or static, based on 2018 IBC, ACI 351.3 & ACI 318-14 the capacities of allowable sliding and overturning are the same values.
2. For load, no matter the manufacturer provides equipment loads are dynamic or static, the maximum equipment loads should be used in load combinations. The combination factor of maximum equipment loads is input by this software user, so more design cases can be covered.

INPUT DATA

MAX DYNAMIC (operating) LOADS	P_{Equ}	=	28	*kips, ASD*
(from manufacturer, including	M_{Equ}	=	120	*ft-kips, ASD*
impact increasing.)	V_{Equ}	=	28	*kips, ASD*
LOAD COMBINATION FACTOR	f_{Equ}	=	2.5	*, to ASD level*
AXIAL DEAD LOAD (weight)	P_{DL}	=	23	k
AXIAL LIVE LOAD	P_{LL}	=	50	k
CONCRETE STRENGTH (3 ksi Min)	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SURCHARGE	q_s	=	0.1	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	2.5	ft
FOOTING THICKNESS	T	=	18	in
ALLOW SOIL PRESSURE	Q_a	=	2	ksf
FOOTING LENGTH	L	=	30	ft
FOOTING WIDTH	B	=	15	ft
PIER DIMENSIONS	c_1	=	10	ft
	c_2	=	10	ft
	H	=	6	ft
REINFORCING SIZE		#	7	

DESIGN SUMMARY

FOOTING WIDTH	B	=	15.00	ft
FOOTING LENGTH	L	=	30.00	ft
FOOTING THICKNESS	T	=	18	in
LONGITUDINAL REINF., TOP	6 # 7			
LONGITUDINAL REINF., BOT.	29 # 7 @ 6 in o.c.			
TRANSVERSE REINF., BOT.	21 # 7 @ 17 in o.c.			



THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS AT TOP OF FOOTING (IBC 1605.3.2 & ACI 318 5.3)

CASE 1:	DL + LL	P	=	163	kips	1.2 DL + 1.6 LL	P_u	=	216	kips
		M	=	0	ft-kips		M_u	=	0	ft-kips
		e	=	0.0	ft, fr cl ftg		e_u	=	0.0	ft, fr cl ftg
CASE 2:	DL + LL + $f_{Equ}P_{Equ}$	P	=	233	kips	1.2 DL + 1.0 LL + 1.4 $f_{Equ}P_{Equ}$	P_u	=	284	kips
		M	=	300	ft-kips		M_u	=	420	ft-kips
		V	=	70	kips		V_u	=	98	kips
		e	=	1.3	ft, fr cl ftg		e_u	=	1.5	ft, fr cl ftg
CASE 3:	0.9 DL + $f_{Equ}P_{Equ}$	P	=	172	kips	0.9 DL + 1.4 $f_{Equ}P_{Equ}$	P_u	=	200	kips
		M	=	300	ft-kips		M_u	=	420	ft-kips
		V	=	70	kips		V_u	=	98	kips
		e	=	1.7	ft, fr cl ftg		e_u	=	2.1	ft, fr cl ftg

CHECK OVERTURNING FACTOR (2018 IBC 1605.2.1, 1808.3.1)

M_R / M_O = 4E+11 **>** F = 1.50 **[Satisfactory]**

Where M_O = $f_{Equ}[M_{Equ} + V_{Equ}(T + H) - P_{Equ}0.5\,L]$ = 0 k-ft

P_{ftg} = (0.15 kcf) T B L = 101.25 k, footing weight

P_{soil} = $w_s(D_f - T)$ B L = 49.50 k, soil weight

M_R = $P_{DL}\,0.5\,L + 0.5\,(P_{ftg} + P_{soil})\,L$ = 3956 k-ft

FOR REVERSED LATERAL LOADS,

M_R / M_O = 2.11 **>** F **[Satisfactory]**

Where M_O = $f_{Equ}[M_{Equ} + V_{Equ}(T + H) + P_{Equ}0.5\,L]$ = 1875 k-ft

CHECK SLIDING (2018 IBC 1807.2.3)

F $f_{Equ}(V_{Equ,\ ASD})$ = 105 kips **<** $\mu\,\Sigma W + F_P$ = 85.70 **+** 19.97 **=** 105.67 kips

Where ` μ = 0.4 P_p = 450 psf / ft, *(passive pressure)* **[Satisfactory]**

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318 13.3.1.1)

Service Loads	CASE 1	CASE 2	CASE 3	
P	163.0	233.0	171.7	k
e	0.0	1.7	2.4	ft (from center of footing)
q_s B L	45.0	45	0.0	k, (surcharge load)
$(0.15\text{-}w_s)$T B L	27.0	27.0	24.3	k, (footing increased)
Σ P	235.0	305.0	196.0	k
e_L	0.0 < L/6	1.3 < L/6	2.1 < L/6	ft
e_B	0.0 < B/6	0.0 < B/6	0.0 < B/6	ft
q_L	7.8	12.9	9.2	k / ft
q_{max}	0.5	0.9	0.6	ksf
q_{allow}	2.0	2.7	2.7	ksf

Where

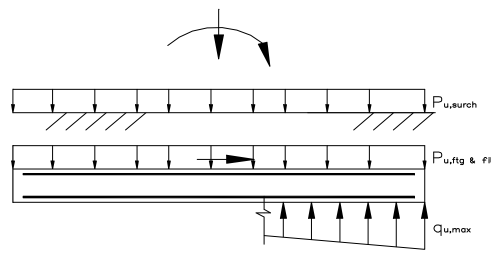
$$q_L = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e_L}{L}\right)}{L}, & for \quad e_L \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3(0.5L-e_L)}, & for \quad e_L > \dfrac{L}{6} \end{cases} \qquad q_{MAX} = \begin{cases} \dfrac{q_L\left(1+\dfrac{6e_B}{B}\right)}{B}, & for \quad e_B \le \dfrac{B}{6} \\ \dfrac{2q_L}{3(0.5B-e_B)}, & for \quad e_B > \dfrac{B}{6} \end{cases}$$

[Satisfactory]

DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318 13, 21, & 22)

$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} \qquad \rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} \qquad \rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	215.6	283.6	199.7	k
e_u	0.0	2.0	2.8	ft
γ q_s B L	72.0	45.0	0.0	k, (factored surcharge load)
γ[0.15T + $w_s(D_f$ - T)]BL	180.9	180.9	135.7	k, (factored footing & backfill loads)
Σ P_u	468.5	509.5	335.4	k
e_u	0.0 < L/6	1.1 < L/6	1.7 < L/6	ft
$q_{u,\ max}$	1.041	1.384	0.997	ksf

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **1**

Section	0	0.125 L	0.25 L	0.375 L	Cen_L	Cen_R	0.625 L	0.75 L	0.875 L	L
X_u (ft, dist. from left of footing)	0	3.75	7.50	11.25	15.00	15.00	18.75	22.50	26.25	30.00
$M_{u,machine}$ (ft-k)	0	-50.531	-202.13	-454.78	-808.5	-808.5	-1263.3	-1819.1	-2476	-3234
$V_{u,machine}$ (k)	0	27.0	53.9	80.9	107.8	107.8	134.8	161.7	188.7	215.6
$P_{u,surch}$ (klf)	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40
$M_{u,surch}$ (ft-k)	0	-16.9	-67.5	-151.9	-270.0	-270.0	-421.9	-607.5	-826.9	-1080.0
$V_{u,surch}$ (k)	0	9.0	18.0	27.0	36.0	36.0	45.0	54.0	63.0	72.0
$P_{u,ftg\ \&\ fill}$ (klf)	6.03	6.03	6.03	6.03	6.03	6.03	6.03	6.03	6.03	6.03
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-42.4	-169.6	-381.6	-678.4	-678.4	-1060.0	-1526.3	-2077.5	-2713.5
$V_{u,ftg\ \&\ fill}$ (k)	0	22.6	45.2	67.8	90.5	90.5	113.1	135.7	158.3	180.9
$q_{u,soil}$ (ksf)	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04
$M_{u,soil}$ (ft-k)	0	109.8	439.2	988.2	1756.9	1756.9	2745.1	3953.0	5380.4	7027.5
$V_{u,soil}$ (k)	0	-58.6	-117.1	-175.7	-234.3	-234.3	-292.8	-351.4	-409.9	-468.5
Σ M_u (ft-k)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
Σ V_u (kips)	**0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0**

(cont'd)

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **2**

Section	0	0.125 L	0.25 L	0.375 L	Cen_L	Cen_R	0.625 L	0.75 L	0.875 L	L
X_u (ft, dist. from left of footing)	0	3.75	7.50	11.25	15.00	15.00	18.75	22.50	26.25	30.00
$M_{u,machine}$ (ft-k)	0	-57.609	-230.44	-518.48	-921.75	-921.75	-1440.2	-2073.9	-2822.9	-3687
$V_{u,machine}$ (k)	0	35.5	70.9	106.4	141.8	141.8	177.3	212.7	248.2	283.6
$P_{u,surch}$ (klf)	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
$M_{u,surch}$ (ft-k)	0	-10.5	-42.2	-94.9	-168.8	-168.8	-263.7	-379.7	-516.8	-675.0
$V_{u,surch}$ (k)	0	5.6	11.3	16.9	22.5	22.5	28.1	33.8	39.4	45.0
$P_{u,ftg\ \&\ fill}$ (klf)	6.03	6.03	6.03	6.03	6.03	6.03	6.03	6.03	6.03	6.03
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-42.4	-169.6	-381.6	-678.4	-678.4	-1060.0	-1526.3	-2077.5	-2713.5
$V_{u,ftg\ \&\ fill}$ (k)	0	22.6	45.2	67.8	90.5	90.5	113.1	135.7	158.3	180.9
$q_{u,soil}$ (ksf)	0.88	0.94	1.01	1.07	1.13	1.13	1.20	1.26	1.32	1.38
$M_{u,soil}$ (ft-k)	0	95.1	389.1	895.3	1627.1	1627.1	2597.8	3820.5	5308.7	7075.5
$V_{u,soil}$ (k)	0	-51.3	-106.1	-164.5	-226.4	-226.4	-291.9	-360.9	-433.4	-509.5
ΣM_u (ft-k)	**0**	**-15.5**	**-53.2**	**-99.7**	**-141.8**	**-141.8**	**-166.1**	**-159.5**	**-108.5**	**0**
ΣV_u (kips)	**0**	**12.4**	**21.3**	**26.6**	**28.4**	**28.4**	**26.6**	**21.3**	**12.4**	**0**

FOOTING MOMENT & SHEAR AT LONGITUDINAL SECTIONS FOR CASE **3**

Section	0	0.125 L	0.25 L	0.375 L	Cen_L	Cen_R	0.625 L	0.75 L	0.875 L	L
X_u (ft, dist. from left of footing)	0	3.75	7.50	11.25	15.00	15.00	18.75	22.50	26.25	30.00
$M_{u,machine}$ (ft-k)	0	-37.945	-151.78	-341.51	-607.13	-607.13	-948.63	-1366	-1859.3	-2428.5
$V_{u,machine}$ (k)	0	25.0	49.9	74.9	99.9	99.9	124.8	149.8	174.7	199.7
$P_{u,surch}$ (klf)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{u,surch}$ (ft-k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$V_{u,surch}$ (k)	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{u,ftg\ \&\ fill}$ (klf)	4.52	4.52	4.52	4.52	4.52	4.52	4.52	4.52	4.52	4.52
$M_{u,ftg\ \&\ fill}$ (ft-k)	0	-31.8	-127.2	-286.2	-508.8	-508.8	-795.0	-1144.8	-1558.1	-2035.1
$V_{u,ftg\ \&\ fill}$ (k)	0	17.0	33.9	50.9	67.8	67.8	84.8	101.8	118.7	135.7
$q_{u,soil}$ (ksf)	0.49	0.56	0.62	0.68	0.75	0.75	0.81	0.87	0.93	1.00
$M_{u,soil}$ (ft-k)	0	54.2	225.8	528.0	974.2	974.2	1577.5	2351.3	3308.9	4463.6
$V_{u,soil}$ (k)	0	-29.5	-62.6	-99.2	-139.3	-139.3	-183.0	-230.3	-281.1	-335.4
ΣM_u (ft-k)	**0**	**-15.5**	**-53.2**	**-99.7**	**-141.8**	**-141.8**	**-166.1**	**-159.5**	**-108.5**	**0**
ΣV_u (kips)	**0**	**12.4**	**21.3**	**26.6**	**28.4**	**28.4**	**26.6**	**21.3**	**12.4**	**0**

DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	use	ρ_{provD}
Top Longitudinal	166	ft-k	15.56	0.0011	0.0009	0.0155	no limit	6 # 7	0.0013
Bottom Longitudinal	1038.2	ft-k	14.56	0.0022	0.0065	0.0155	18	29 # 7 @ 6 in o.c.	0.0066
Bottom Transverse	4	ft-k / ft	14.13	0.0005	0.0004	0.0155	18	21 # 7 @ 17 in o.c.	0.0025

[Satisfactory]

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$		$\phi V_c = 2\phi b d (f_c')^{0.5}$		check $V_u < \phi V_c$
Longitudinal	182.4	k	215	k	**[Satisfactory]**
Transverse	1.8	k / ft	14	k / ft	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$$v_u(psi) = \frac{P_u - R}{A_P} + \frac{0.5\gamma_v M_u b_1}{J}$$

$$A_P = 2\left(b_1 + b_2\right)d$$

$$\phi v_c(psi) = \phi\left(2 + y\right)\sqrt{f_c'}$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right]$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right)$$

$$R = \frac{P_u b_1 b_2}{A_f}$$

$$A_f = BL$$

$$b_0 = \frac{A_P}{d}, b_1 = \left(0.5c_1 + 0.5b_1 + d\right), b_2 = \left(0.5c_2 + 0.5b_2 + d\right)$$

Case	P_u	M_u	b_1	b_2	b_0	γ_v	β_c	y	A_f	A_p	R	J	$v_{u\ (psi)}$	$\phi\ v_c$
1	215.6	0.0	134.1	134.1	3.7	0.4	1.0	2.0	450.0	52.6	59.9	1098.8	20.6	164.3
2	283.6	420.0	134.1	134.1	3.7	0.4	1.0	2.0	450.0	52.6	78.7	1098.8	27.1	164.3
3	199.7	420.0	134.1	134.1	3.7	0.4	1.0	2.0	450.0	52.6	55.4	1098.8	19.1	164.3

[Satisfactory]

where ϕ = 0.75 , (ACI 318 21.2)

Purchaser's LOGO
(Please email us your LOGO at purchase.)

PROJECT :
CLIENT :
JOB NO. :
DATE :
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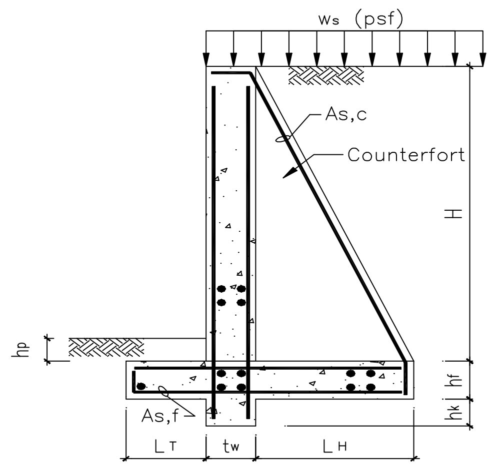
DESIGN BY :
REVIEW BY :

Counterfort Retaining Wall Design Based on 2018 IBC & ACI 318-14

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
LATERAL SOIL PRESSURE	P_a	=	45	pcf (equivalent fluid pressure)
PASSIVE PRESSURE	P_p	=	450	psf / ft
SEISMIC GROUND SHAKING	P_E	=	48	psf / ft
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf
SURCHARGE WEIGHT	w_s	=	50	psf
FRICTION COEFFICIENT	μ	=	0.4	
ALLOW SOIL PRESSURE	Q_a	=	3	ksf

THICKNESS OF COUNTERFORT t_c = 12 in

COUNTERFORT SPACING S_c = 18 ft

EDGE REINF.OF COUNTERFORT ($A_{s,c}$) 4 # 8 *, per counterfort*

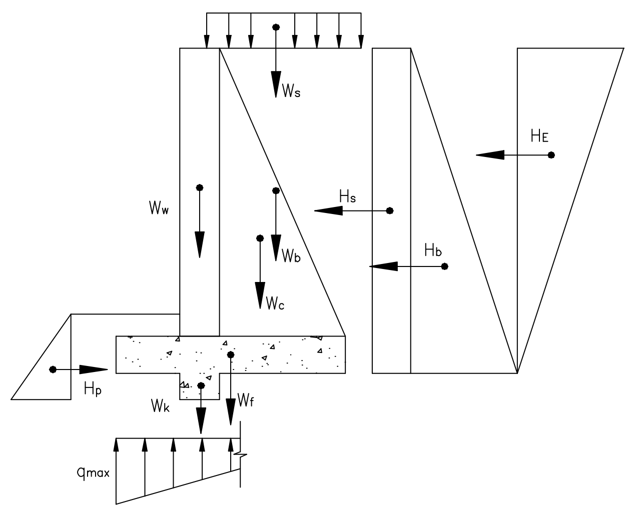
FOOTING THICKNESS	h_f	=	21	in
KEY DEPTH	h_k	=	26	in
SOIL OVER TOE	h_p	=	6	in

THICKNESS OF WALL	t_w	=	14	in
TOE WIDTH	L_T	=	5	ft
HEEL WIDTH	L_H	=	13	ft
HEIGHT OF WALL	H	=	12	ft

[THE WALL DESIGN IS ADEQUATE.]

BOT. REINF.OF FOOTING ($A_{s,f}$) # 8 @ 24 in

ANALYSIS



SERVICE LOADS

$H_b = 0.5\,P_a\,(H + h_f)^2$	=	4.25	kips/ft
$H_s = w_s\,P_a\,(H + h_f)\,/\,\gamma_b$	=	0.28	kips/ft
$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$	=	4.39	kips/ft
$H_E = 0.5\,P_E\,(H + h_f)^2$	=	4.54	kips/ft
$W_s = w_s\,(L_H + t_b - t_t)$	=	0.65	kips/ft
$W_b = (H\,L_H)\,\gamma_b$	=	17.16	kips/ft
$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$	=	5.03	kips/ft
$W_k = h_k\ t_b\ \gamma_c$	=	0.38	kips/ft
$W_w = t_w\ H\ \gamma_c$	=	2.10	kips/ft
$W_c = 0.5\,(H\,L_H)\ (\gamma_c - \gamma_b)\,t_c\,/\,S_c$	=	0.17	kips/ft

FACTORED LOADS

$\gamma H_b = 1.6\,H_b$	=	6.81	kips/ft
$\gamma H_s = 1.6\,H_s$	=	0.45	kips/ft
$\gamma H_E = 1.6\,H_E$	=	7.26	kips/ft
$\gamma W_s = 1.6\,W_s$	=	1.04	kips/ft
$\gamma W_b = 1.2\,W_b$	=	20.59	kips/ft
$\gamma W_f = 1.2\,W_f$	=	6.04	kips/ft
$\gamma W_k = 1.2\,W_k$	=	0.46	kips/ft
$\gamma W_w = 1.2\,W_w$	=	2.52	kips/ft
$\gamma W_c = 1.2\,W_w$	=	0.21	kips/ft

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	4.25	6.81	4.58	19.50	31.20
H_E	4.54	7.26	9.17	41.59	66.55
H_s	0.28	0.45	6.88	1.93	3.09
Σ	9.07	14.52		**63.03**	100.84

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.65	1.04	12.67	8.23	13.17
W_b	17.16	20.59	12.67	217.36	260.83
W_f	5.03	6.04	9.58	48.22	57.86
W_k	0.38	0.46	5.58	2.12	2.54
W_w	2.10	2.52	5.58	11.73	14.07
W_c	0.17	0.21	10.50	1.82	2.18
Σ	25.49	30.85		**289.47**	350.66

M_{HP} = 0.85 ft-kips/ft

OVERTURNING FACTOR OF SAFETY (1807.2.3)

$$SF = \frac{\Sigma Wx + M_{HP}}{\Sigma Hy} = 4.606 > 1.5$$

[Satisfactory]

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318 13.3.1.1)

$L = L_T + t_b + L_H$ = 19.17 ft

$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy + M_{HP}}{\Sigma W}$ = 0.67 ft < L / 3 **[Satisfactory]**

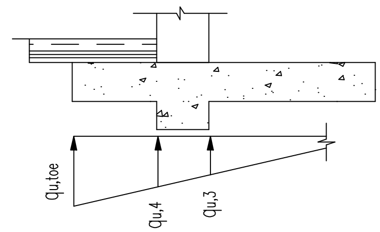
$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$ = 1.61 ksf < Q_a **[Satisfactory]**

CHECK COUNTERFORT FLEXURE CAPACITY, $A_{S,c}$ (ACI 318 13.2.7.1, 21, & 22)

$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$ = 0.015

$\rho_{MIN} = 0.0018\frac{D}{d}$ = 0.0018

$M_{u,c} = \begin{cases} \frac{L_H \gamma W_c}{3} + \frac{L_H}{2}\left(\gamma W_s + \gamma W_b + \frac{L_H}{L}\gamma W_f\right) - \frac{\left(q_{u,3} + 2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H \gamma W_c}{3} + \frac{L_H}{2}\left(\gamma W_s + \gamma W_b + \frac{L_H}{L}\gamma W_f\right) - \frac{q_{u,3} b s^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases}$ = 1261 ft-kips



$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,c}}{0.383 b d^2 f'_c}}\right)}{f_y}$ = 0.002

where	D	=	105.8 in	d	=	103.31	in			
	e_u	=	1.49 ft	$q_{u,\ heel}$	=	0.86	ksf			
	s	=	n/a	$q_{u,\ 3}$	=	1.76	ksf	$q_{u,\ toe}$	=	2.36 ksf

$(A_{S,\ c})_{required}$ = 2.78 in^2 < $A_{S,\ c}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, $A_{S,f}$, FOR FOOTING (ACI 318 13.2.7.1, 21, & 22)

$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$ = 0.015

$\rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2}\frac{h_f}{d}\right)$ = 0.001

$M_{u,4} = \frac{\left(q_{u,4} + 2q_{u,toe}\right)bL_T^2}{6} - \frac{L_T^2}{2L}\gamma W_f$ = 23.52 ft-kips

where d = 17.50 in
$q_{u,\ 4}$ = 1.873 ksf

$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y}$ = 0.001

$(A_{S,\ f})_{required}$ = 0.30 in^2 / ft < $A_{S,\ f}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (CBC 1807A.2.3)

1.5 (H_b + H_s + H_E) = 13.6 kips < $H_p + \mu\,\Sigma W$ = 14.59 kips
[Satisfactory]

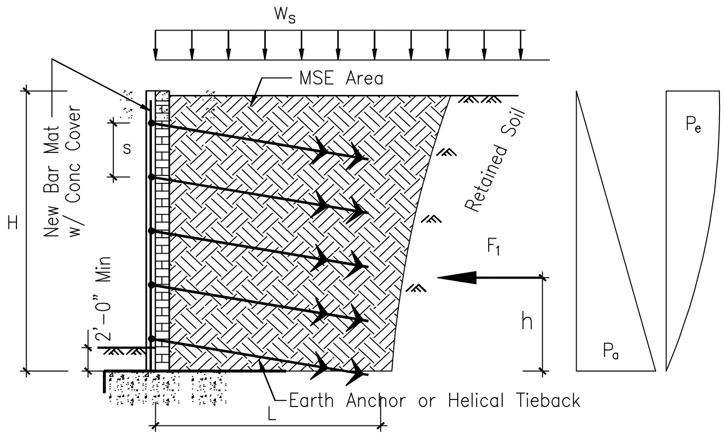
Retaining Wall Repair Design Based on AASHTO/2018 IBC & TMS 402-16

DESIGN CRITERIA

1. The existing ***Retaining Wall*** can be masonry wall, reinforcing concrete wall, or rockery gravity wall. But after this enhanced/repaired, the wall changed to ***Mechanically Stabilized Earth Wall (MSE)***. The MSE area (anchor length, ***L***) should be ensured with each anchor capacity not less than 3.52 kips/anchor (#3 bar service level capacity).
2. To enhanced existing wall for new seismic load (2018 IBC 1807.2.2), the new face bar mat #4 minimum, and total vertical bar area (old plus new) not less than 0.002 times wall section, are suggested (ACI 318-14 14.1.4).

INPUT DATA & DESIGN SUMMARY

WALL HEIGHT	H =	20	*ft, (6.10 m)*
EXISTING WALL THICKNESS	t =	22	*in, (559 mm)*
EXISTING WALL STRENGTH	f_m' =	0.93	*ksi, (6 MPa)*
(modular concrete facing blocks - MBW)			
NEW BAR MAT (w/ 2" conc cover)	f_y =	60	*ksi, (414 MPa)*
# 5	@	18	in o.c. at each way
(connected to Anchors, 457 mm o.c. at side face)			
ANCHOR/HELICAL TIEBACK	L =	18	*ft, (5.49 m)*
s = 36			*in, (914 mm), o.c., vertical & horizontal*
SURCHARGE WEIGHT	w_s =	500	*psf, (24 kPa)*
SOIL SPECIFIC WEIGHT	γ_b =	110	*pcf, (1762 kg/m³)*
SOIL INTERNAL FRICTION ANGLE	ϕ =	29	deg
HORIZONTAL ACCELERATION (in g)	k_h =	0.35	g, (0.5 S_{DS})
ALLOW SOIL PRESSURE	Q_a =	4.2	*ksf, (201 kPa)*



THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE ACTIVE EARTH PRESSURE

$P_e = 0.75\, k_h\, \gamma_b =$ 29 psf / ft, (FEMA P-750 Page 356)

$P_a = \gamma_b\, K_a =$ 38 psf / ft

where $$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2} = 0.347$$

β = 0 deg, slope of backfill
δ = 0 deg, external friction angle
θ = 90 deg, rack angle of wall face

(Coulomb, AASHTO Figure 5.5.2A)

$F_1 = 0.5\, H^2 (P_a + P_e)$ = 13 kips / ft, *Total*

$h = (1/3\, P_a + 0.6\, P_e)\, H / (P_a + P_e) =$ 8.96 ft

CHECK SOIL BEARING CAPACITY (AASHTO Figure 5.8.3A)

$V_1 = H\, L\, \gamma_b =$ 39.6 kips / ft, *Vertical*

$F_2 = K_a\, H\, w_s =$ 3.47 kips / ft, *from surcharge weight*

$e = (F_1\, h + F_2\, 0.5\, H) / (V_1 + L\, w_s) =$ 3.187 ft

$\sigma_v = (V_1 + L\, w_s) / (L - 2e) =$ 4.18 ksf < Q_a [Satisfactory]

CHECK SLIDING CAPACITY (2018 IBC 1807.2.3)

$1.1\,(F_1 + F_2) =$ 18.57 kips / ft < $\mathrm{Tan}(\phi)\,(V_1 + L\, w_s) =$ 26.94 kips / ft [Satisfactory]

$1.5\,(F_1 + F_2 - 0.5\, H^2\, P_e) =$ 16.65 kips / ft < $\mathrm{Tan}(\phi)\,(V_1 + L\, w_s) =$ 26.94 kips / ft [Satisfactory]

CHECK OVERTURNING CAPACITY (2018 IBC 1807.2.3)

$1.5\,(h\, F_1 + 0.5\, H\, F_2) =$ 232.3 ft-kips / ft < $0.5\, L\,(V_1 + L\, w_s) =$ 437.40 ft-kips / ft [Satisfactory]

(All forces with safety factor 1.5 conservatively.)

CHECK FLEXURE CAPACITY OF WALL (TMS 402 8.3.3)

$$M_{allowable} = MIN\left[\frac{1}{2} b_w kd\, F_b\left(d-\frac{kd}{3}\right) - P\left(d-\frac{t_e}{2}\right)\ ,\ A_s F_s\left(d-\frac{kd}{3}\right) + P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right] =$$ 11.19 ft-kips

> $s^2\,(F_1 + F_2) / H =$ 7.6 ft-kips

[Satisfactory]

(cont'd)

where	t_e	=	21.63	in, *conservative value*			
	d	=	19.31	in	E_m	=	651 ksi, 700 f_m' conservative value
	b_w	=	12	in	E_s	=	29000 ksi
	F_b	=	0.307	ksi	n	=	44.55
	F_s	=	32	ksi	k	=	0.24
	A_s	=	0.207	in^2	P	=	2.383333 kips, axial force at wall middle
	ρ	=	0.001	**>** 0.0007 [Satisfactory]			

CHECK EARTH ANCHOR CAPACITY (AASHTO 5.8.4.1)

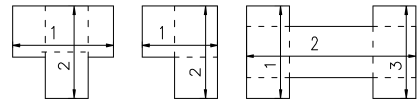
$T_{max} = \sigma_h A_{trib}$ = 2.16 kips / Anchor **<** $F_s A_s$ = 3.52 kips / Anchor [Satisfactory]

where	σ_h	=	$F_2 / H + P_a + P_e$ =	0.24 ksf
	A_{trib}	=	1296	in^2

Purchaser's LOGO (Please email us your LOGO at purchase.)	PROJECT :		PAGE :	
	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

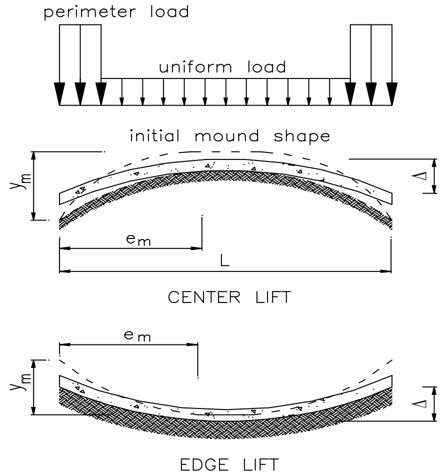
RC Mat Slab Design Based on 2018 IBC, ACI 318-14, AASHTO 17th Edition & ACI 360

DESIGN CRITERIA



1. Divide an irregular foundation plan into overlapping rectangles and using this software design each rectangular section separately.
2. To check soil capacity is based on 2018 IBC 1808.6.2 & ACI 360 9, using the Post-Tension Institute (PTI) method.
3. To check reinforced concrete (RC) slab capacity is based on ACI 318-14, using plate/shell element method.

INPUT DATA & DESIGN SUMMARY



SLAB LENGTH	L =	168	ft, (51.21 m)
SLAB WIDTH	B =	126	ft, (38.40 m)
SLAB THICKNESS	t =	8	in, (203 mm)
CONCRETE STRENGTH	f'_c =	3	ksi, (21 MPa)
SLAB REINF.	2 layers # 4 @	18	in. o.c. (457 mm o.c.)
REBAR YIELD STRESS	f_y =	60	ksi, (414 MPa)
ALLOWABLE SOIL-BEARING PRESSURE	q_{allow} =	2000	psf, (96 kPa)
EDGE MOISTURE VARIATION DIST.	e_m =	4	ft, for center lift, (1.22 m)
		4.5	ft, for edge lift, (1.37 m)
DIFFERENTIAL SOIL MOVEMENT	y_m =	2.68	in, for center lift, (68 mm)
		0.3	in, for edge lift, (8 mm)
PERIMETER LOADING	P =	400	plf, (5.8 kN/m)
MAX BEARING LOADING ON THE SLAB	P_b =	800	plf, (11.7 kN/m)
ADDED DEAD LOAD	DL =	80	plf, (1.2 kN/m)
LIVE LOAD	LL =	125	plf, (1.8 kN/m)

THE DESIGN IS ADEQUATE.

ANALYSIS

1. ASSUME A TRIAL SECTION (ACI 360 & PTI)

AVERAGE STIFFENING BEAM SPACING, L DIRECTION	S_L =	14.00	ft, (4.27 m), L/12 suggested
AVERAGE STIFFENING BEAM SPACING, B DIRECTION	S_B =	10.50	ft, (3.20 m), B/12 suggested
STIFFENING BEAM DEPTH	h =	8	in, (203 mm), flat slab
STIFFENING BEAM WIDTH	b =	20	in, (508 mm), regular beam width
BOTTOM OF STIFFENING BEAM REINF.		2	# 4

2. ASSUME BEAM DEPTH AND SPACING (ACI 360 & PTI)

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR CENTER LIFT, AT L DIRECTION

Δ_{allow} = 12 MIN(L, 6β) / C_Δ = 1.60 in

Where β = 8 ft

C_Δ = 360

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR CENTER LIFT, AT B DIRECTION

Δ_{allow} = 12 MIN(B, 6β) / C_Δ = 1.60 in

Where β = 8 ft

C_Δ = 360

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR EDGE LIFT, AT L DIRECTION

Δ_{allow} = 12 MIN(L, 6β) / C_Δ = 0.80 in

Where β = 8 ft

C_Δ = 720

ALLOWABLE DIFFERENTIAL DEFLECTION, FOR EDGE LIFT, AT B DIRECTION

Δ_{allow} = 12 MIN(B, 6β) / C_Δ = 0.80 in

Where β = 8 ft

C_Δ = 720

BEAM DEPTH, FOR CENTER LIFT, AT L DIRECTION

$h = [(y_m L)^{0.205} S_B^{1.059} P^{0.523} e_m^{1.296} / 380\ \Delta_{allow}]^{0.824}$ = 6.45 in

BEAM DEPTH, FOR CENTER LIFT, AT B DIRECTION

$h = [(y_m B)^{0.205} S_L^{1.059} P^{0.523} e_m^{1.296} / 380\ \Delta_{allow}]^{0.824}$ = 7.90 in

BEAM DEPTH, FOR EDGE LIFT, AT L DIRECTION

$h = [L^{0.35} S_B^{0.88} e_m^{0.74} y_m^{0.76} / 15.9\ \Delta_{allow} P^{0.01}]^{1.176}$ = 2.74 in

BEAM DEPTH, FOR EDGE LIFT, AT B DIRECTION

$h = [B^{0.35} S_L^{0.88} e_m^{0.74} y_m^{0.76} / 15.9\ \Delta_{allow} P^{0.01}]^{1.176}$ = 3.28 in

GOVERNING h = 7.90 in < ACTUAL h = 8.00 in [Satisfactory]

3. DETERMINE SECTION PROPERTIES (ACI 360 & PTI)

L DIRECTION

A_s =	17	in^2	n =	13	beams
E_s / E_c =	9.29		y_b =	4.02	in
CGS =	5.75	in	S_t =	16337	in^3
A =	12252	in^2	S_b =	16156	in^3
I =	64984	in^4	Cover =	1.5	in clear from top of slab, each way.

B DIRECTION

A_s =	22	in^2	n =	13	beams
E_s / E_c =	9.29		y_b =	4.03	in
CGS =	6.25	in	S_t =	21921	in^3
A =	16336	in^2	S_b =	21609	in^3
			I =	87056	in^4

4. CALCULATE MAXIMUM APPLIED SERVICE MOMENTS (ACI 360 & PTI)

CENTER LIFT MOMENT AT L DIRECTION

$M_L = A_0 (B\ e_m^{1.238} + C)$ = 2.08 ft-kips / ft

Where $A_0 = (L^{0.013} S_B^{0.306} h^{0.688} P^{0.534} y_m^{0.193}) / 727$ = 0.374

B = 1, for e_m < 5

B = MIN[(y_m - 1) / 3, 1], for e_m > 5 = 1.00

CENTER LIFT MOMENT AT B DIRECTION

$M_B = (58 + e_m) M_L / 60$, for L /B > 1.1

$M_B = M_L$, for L /B < 1.1 = 2.15 ft-kips / ft

C = 0, for e_m < 5

C = MAX{[8 - (P - 613) / 255] (4 - y_m) / 3], 0}, for e_m > 5 = 0.00

EDGE LIFT MOMENT AT L DIRECTION

$M_L = S_B^{0.10} (h\ e_m)^{0.78} y_m^{0.66} / (7.2\ L^{0.0065} P^{0.04})$ = 0.99 ft-kips / ft

EDGE LIFT MOMENT AT B DIRECTION

$M_B = h^{0.35} (19 + e_m) M_L / 57.75$, for L /B > 1.1

$M_B = M_L$, for L /B < 1.1 = 0.83 ft-kips / ft

APPLIED SERVICE LOAD SHEAR AT L DIRECTION

FOR CENTER LIFT

$V_L = L^{0.09} S_B^{0.71} h^{0.43} P^{0.44} y_m^{0.16} e_m^{0.93} / 1940$ = 0.630 kips/ft

FOR EDGE LIFT

$V_L = L^{0.07} h^{0.4} P^{0.03} y_m^{0.67} e_m^{0.16} / (3.0\ S_B^{0.015})$ = 0.719 kips/ft

APPLIED SERVICE LOAD SHEAR AT B DIRECTION

FOR CENTER LIFT

$V_B = B^{0.19} S_L^{0.45} h^{0.20} P^{0.54} y_m^{0.04} e_m^{0.97} / 1350$ = 0.936 kips/ft

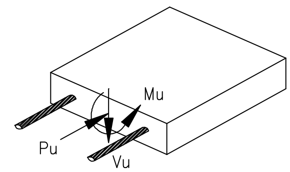
FOR EDGE LIFT

$V_B = B^{0.07} h^{0.4} P^{0.03} y_m^{0.67} e_m^{0.16} / (3.0\ S_L^{0.015})$ = 0.702 kips/ft

(cont'd)

5. CHECK REINFORCED CONCRETE SLAB CAPACITY (ACI 318-14 & AASHTO 17th Edition)

MAX FACTORED AXIAL LOAD	P_u	=	0.07	*kips/ft, (1.0 kN/m), 5% shear load for seismic & slading*
MAX FACTORED MOMENT	M_u	=	3.23	*ft-kips / ft, (14.4 kN-m / m)*
MAX FACTORED SHEAR LOAD	V_u	=	1.40	*kips/ft, (20.5 kN/m)*

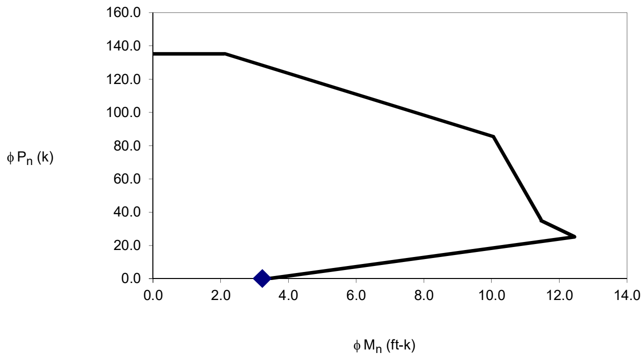


CHECK AXIAL & FLEXURE CAPACITY

ρ_{ProvD} = 0.001932367 < ρ_{MAX} = 0.0400 (tension face only, ACI 318-14 22.2.3 or 10.9.1)

\> ρ_{MIN} = 0.0006 (tension face only, ACI 318-14 6.6.4.3, 9.6.1 or 11.6.1)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	135.3	0.0
AT MAXIMUM LOAD	135.3	2.1
AT MIDDLE	85.5	10.1
AT ε_t = 0.002	35.7	11.5
AT BALANCED	34.8	11.5
AT ε_t = 0.005	25.2	12.5
AT FLEXURE ONLY	0.0	3.4

(Note: For middle reinforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-14 13.2.7.2, 7.4.3, & 22.5)

$\phi V_n = 2\phi b d\sqrt{f_c'} =$ 5.67 kips / ft > V_u **[Satisfactory]**

6. CHECK DIFFERENTIAL DEFLECTIONS (ACI 360 & PTI)

RELATIVE STIFFNESS LENGTH AT L DIRECTION

$\beta = (E_c\, I / E_s)^{1/4} / 12$ = 8.363 ft

Where E_c = (0.5) 57000 $(f_c')^{0.5}$ = 1561009 psi

E_s = 1000 psi, soil

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$\beta = (E_c\, I / E_s)^{1/4} / 12$ = 8.997 ft

Where E_c = (0.5) 57000 $(f_c')^{0.5}$ = 1561009 psi

E_s = 1000 psi, soil

ALLOWABLE DIFFERENTIAL DEFLECTION AT L DIRECTION

FOR CENTER LIFT

Δ_{allow} = 12 MIN(L, 6β) / C_Δ = 1.80 in

Where C_Δ = 360

FOR EDGE LIFT

Δ_{allow} = 12 MIN(L, 6β) / C_Δ = 0.90 in

Where C_Δ = 720

ALLOWABLE DIFFERENTIAL DEFLECTION AT B DIRECTION

FOR CENTER LIFT

Δ_{allow} = 12 MIN(B, 6β) / C_Δ = 1.67 in

Where C_Δ = 360

FOR EDGE LIFT

Δ_{allow} = 12 MIN(B, 6β) / C_Δ = 0.84 in

Where C_Δ = 720

EXPECTED DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

FOR CENTER LIFT, AT L DIRECTION

$\Delta_0 = (y_m\, L)^{0.205}\, S_B^{1.059}\, P^{0.523}\, e_m^{1.296} / (380\, h^{1.214})$ = 1.23 in

< Δ_{allow}

[Satisfactory]

FOR EDGE LIFT, AT L DIRECTION

$\Delta_0 = L^{0.35}\, y_m^{0.76}\, S_B^{0.88}\, e_m^{0.74} / (15.9\, h^{0.85}\, P^{0.01})$ = 0.59 in

< Δ_{allow}

[Satisfactory]

EXPECTED DIFFERENTIAL DEFLECTION WITHOUT PRESTRESSING

FOR CENTER LIFT, AT B DIRECTION

$\Delta_0 = (y_m\, B)^{0.205}\, S_L^{1.059}\, P^{0.523}\, e_m^{1.296} / (380\, h^{1.214})$ = 1.57 in

< Δ_{allow}

[Satisfactory]

FOR EDGE LIFT, AT B DIRECTION

$\Delta_0 = B^{0.35}\, y_m^{0.76}\, S_L^{0.88}\, e_m^{0.74} / (15.9\, h^{0.85}\, P^{0.01})$ = 0.68 in

< Δ_{allow}

[Satisfactory]

7. CHECK SOIL BEARING (ACI 360 & PTI)

APPLIED LOADING

SLAB WEIGHT	150 L B t	=	2116800	lbs
ADDED DL	DL L B	=	1693440	lbs
LIVE LOAD	LL L B	=	2646000	lbs
BEAM WEIGHT	150 (h-t) b (Total Length)	=	0	lbs
PERIMETER LOAD	P (2L + 2B)	=	235200	lbs

BEAM BEARING AREA (b)(Total Length) = 6088.333333 ft^2

SOIL PRESSURE q = Total Load / THE AREA = 1099 psf

< q_{allow}

[Satisfactory]

8. CHECK SLAB STRESS DUE TO LOAD-BEARING PARTITIONS (ACI 360 & PTI)

RELATIVE STIFFNESS LENGTH AT L DIRECTION

$M_{max} = P_b\, \beta / 4$ = 2.10 ft-kips / ft

Where β = MIN[$(E_c\, t^3 / 3\, k_s)^{0.25}$, S_B] = 11 ft

k_s = 4 lb / in^3

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$M_{max} = P_b\, \beta / 4$ = 2.80 ft-kips / ft

Where β = MIN[$(E_c\, t^3 / 3\, k_s)^{0.25}$, S_L] = 14 ft

k_s = 4 lb / in^3

TENSILE STRESS AT L DIRECTION

f = - $M_{max} / 2\, t^2$ = -0.016 ksi

\> $f_{t,allow}$ **[Satisfactory]**

TENSILE STRESS AT B DIRECTION

f = - $M_{max} / 2\, t^2$ = -0.022 ksi

\> $f_{t,allow}$ **[Satisfactory]**

Technical References:

1. "Design of Slabs on Grade, ACI Committee 360R-06", American Concrete Institute, 2006.
2. "Design and Construction of Post-Tensioned Slab-on-Ground, 2nd & 3rd Editions", The Post-Tensioning Institute, 2004, 2007, & 2008.
3. "1997 Uniform Building Code, Volume 2", International Conference of Building Officials, 1997.

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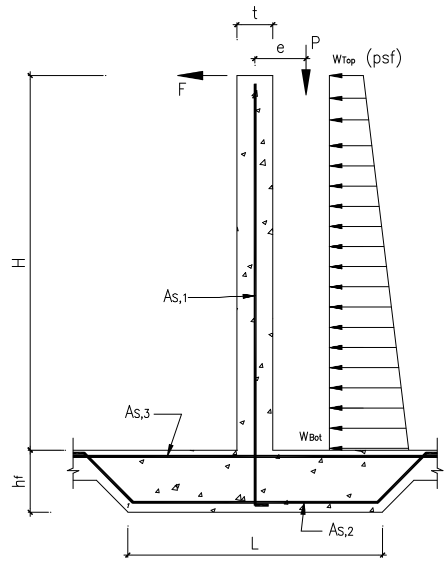
	JOB NO. :	DATE :	REVIEW BY :	

Trapezoidal Loads Retaining Wall Design Based on ACI 318-19

INPUT DATA & DESIGN SUMMARY

STRENGTH	f_y =	60	ksi ,	f_c' =	4	ksi	
PASSIVE SOIL PRESSURE	P_p	=	350	pcf (equivalent fluid pressure)			
ALLOW SOIL PRESSURE	Qa	=	2	ksf			
FRICTION COEFFICIENT	μ	=	0.35				
SOIL SPECIFIC WEIGHT	γ_s	=	110	pcf			
VERTICAL FORCE, ASD	P =	0.2	kips / ft ,	e =	2	ft	
LATERAL POINT FORCE, ASD		F =	0.32	kips / ft			
WALL LATERAL FORCE, ASD	w_{Top} =	30	psf ,	w_{Bot} =	45	psf	
HEIGHT OF STEM	H	=	21	ft			
THICKNESS OF WALL	t	=	12	in			
WALL VERT. REINF. ($A_{s,1}$)		#	6	@	12	in o.c.	
$A_{s,1}$ LOCATION (1=at middle, 2=at each face)			2	at each face			
FOOTING WIDTH	L	=	9	ft			
FOOTING THICKNESS	hf	=	18	in			
BOT. REINF.OF FOOTING ($A_{s,2}$)		#	6	@	18	in o.c.	
TOP. REINF.OF FOOTING ($A_{s,3}$)		#	6	@	18	in o.c.	



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS

$F + H_{Lat} = F + (w_{Top} + w_{Bot})\,H / 2$ = 1.11 kips / ft

$H_p = 0.5\,P_p\,(h_f)^2$ = 0.39 kips / ft

$W_w = t\,H\,\gamma_c$ = 3.15 kips / ft

$W_f = h_f\,L\,\gamma_c$ = 2.03 kips / ft

P = = 0.20 kips / ft

FACTORED LOADS

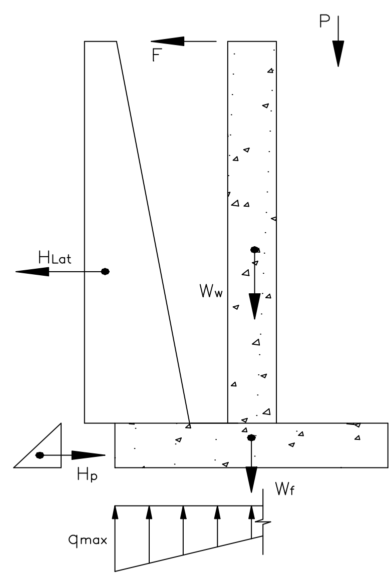
$\gamma(F + H_{Lat}) = 1.6\,(F + H_{lat})$ = 1.77 kips / ft

$\gamma H_p = 0.0\,H_p$ = 0.00 kips / ft

$\gamma W_w = 1.2\,W_w$ = 3.78 kips / ft

$\gamma W_f = 1.2\,W_f$ = 2.43 kips / ft

$\gamma P = 1.2\,P$ = 0.24 kips / ft



OVERTURNING MOMENT

	H	γH	y	H y	γH y
$F + H_{Lat}$	1.11	1.77	10.49	11.62	18.59
Σ	1.11	1.77		**11.62**	18.59

RESISTING MOMENT

	W	γW	x	W x	γW x
P	0.20	0.24	6.5	1.30	1.56
W_f	2.03	2.43	4.5	9.11	10.94
W_w	3.15	3.78	4.5	14.18	17.01
Σ	5.38	6.45		**24.59**	29.51

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 2.116 > 1.5$$

[Satisfactory]

CHECK SOIL BEARING CAPACITY (ACI 318 13.3.1.1)

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 2.09 \text{ ft} < L/4$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1 + \dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L - e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 1.49 \text{ ksf} < Q_a$$

[Satisfactory]

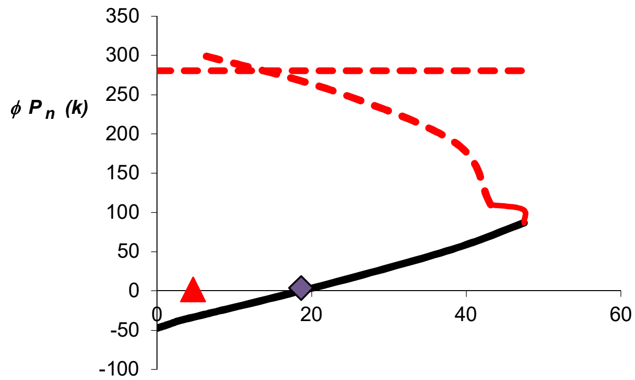
(cont'd)

CHECK FLEXURE CAPACITY, $A_{S,1}$, FOR STEM (ACI 318-19 13, 21, & 22)

ρ_{ProvD} = 0.00381 **<** ρ_{MAX} = 0.04 (tension face only, ACI 318-19 22.2.3 or 10.9.1)

> ρ_{MIN} = 0.00075 (tension face only, ACI 318-19 6.6.4.3, 9.6.1 or 11.6.1)

[Satisfactory]



$$f_c=\begin{cases}\beta_e f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right)-\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right], & for\ \ 0<\varepsilon_c<\varepsilon_o\\ \beta_e f'_c\ , & for\ \ \varepsilon_c\ge\varepsilon_o\end{cases}\ ,\quad f_s=\begin{cases}\varepsilon_s E_s\ , & for\ \ \varepsilon_s\le\varepsilon_{ty}\\ f_y\ , & for\ \ \varepsilon_s>\varepsilon_{ty}\end{cases}$$

$$\varepsilon_o=\frac{2\left(\beta_e f'_c\right)}{E_c}\ ,\quad \varepsilon_{ty}=\frac{f_y}{E_s}\ ,\quad \varepsilon_t=\begin{cases}\varepsilon_{ty}\ , & for\ \ compression\ \ controlled\\ \varepsilon_{ty}+0.003\ , & for\ \ tension\ \ controlled\end{cases}$$

$$E_c=57\sqrt{f'_c}\ ,\quad E_s=29000ksi\ ,\quad \beta_e=0.85\ ,\quad \varepsilon_{cu}=0.003$$

	at bottom	at middle
P_u	3.78	1.89
M_u	18.61	4.65

Solid Black Line - **Tension Controlled** ***ϕ Mn (ft-k)***

Solid Red Line - **Transition**

Dash Line - **Compression Controlled**

ϕM_n = 19.23 ft-kips @ P_u = 1.89 kips **>** M_u = 18.61 ft-kips **[Satisfactory]**

CHECK STEM SHEAR CAPACITY (ACI 318 13.2.7.2 & 22.5)

$V_u = Max.\ \ Horiz.\ \ Shear$ = 1.77 kips / ft, at bottom

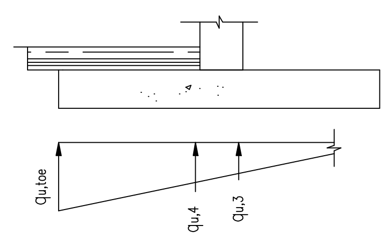
$\phi V_n = 2\phi bd\sqrt{f'_c}$ = 10.96 kips / ft **>** V_u **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,3}$, FOR FOOTING (ACI 318-19 13, 21, & 22)

$$\rho_{MAX}=\frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = 0.021 \qquad \rho_{MIN}=\frac{0.0018}{2}\frac{h_f}{d} = 0.001$$

$$M_{u,3}=\begin{cases}\frac{(L-t)}{4}\left(0.5\gamma W_s+\frac{(L-t)}{2L}\gamma W_f\right)-\frac{\left(q_{u,3}+2q_{u,heel}\right)b}{6}\left(\frac{L-t}{2}\right)^2, & for\quad e_u\le\frac{L}{6}\\ \frac{(L-t)}{4}\left(0.5\gamma W_s+\frac{(L-t)}{2L}\gamma W_f\right)-\frac{q_{u,3}bS^2}{6}, & for\quad e_u>\frac{L}{6}\end{cases} = 2.4\ \text{ft-kips}$$

$$\rho=\frac{0.85f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2f'_c}}\right)}{f_y} = 0.000182$$



where							
	d	=	15.63 in	$q_{u,\ toe}$	=	2.54	ksf
	e_u	=	2.81 ft	$q_{u,\ heel}$	=	n/a	ksf
	S	=	0.08 ft	$q_{u,\ 3}$	=	0.04	ksf

$(A_{S,3})_{required}$ = 0.19 in^2 / ft **<** $A_{S,3}$ = 0.29 in^2 / ft **[Satisfactory]**

CHECK FLEXURE CAPACITY, $A_{S,2}$, FOR FOOTING (ACI 318-19 13, 21, & 22)

$$\rho_{MAX}=\frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = 0.021 \qquad \rho_{MIN}=MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.001$$

(cont'd)

$$M_{u,2} = \frac{\left(q_{u,4} + 2q_{u,toe}\right)b}{6}\left(\frac{L-t}{2}\right)^2 - \frac{\gamma W_f}{2L}\left(\frac{L-t}{2}\right)^2 \quad = \quad 12.83 \text{ ft-kips}$$

where d = 14.63 in

$q_{u,4}$ = 0.539 ksf

$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,2}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.00112$$

$(A_{S,2})_{required}$ = 0.20 in^2 / ft < $A_{S,2}$ = 0.29 in^2 / ft **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 1807.2.3)

1.5 (H_{Lat}) = 1.66 kips / ft < $H_p + \mu \Sigma W$ = 2.28 kips / ft

[Satisfactory]

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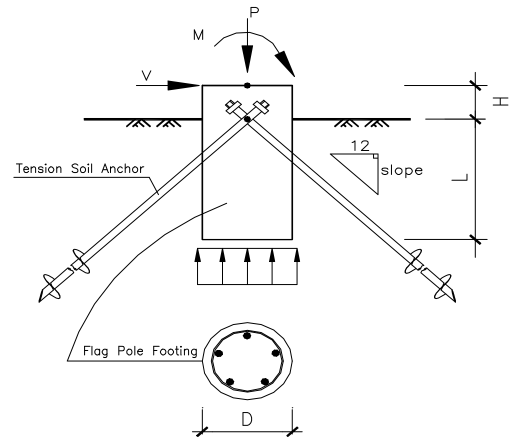
Tree Root Foundation Design Based on AASHTO (HB-17), 2018 IBC & 2019 CBC



DESIGN CRITERIA

1. This foundation system has to include at least 4 soil anchors, which central to ground level of flagpole, with 90 degree each around.
2. Each soil anchor only has tension capacity, which from soil report and/or the site test.
3. If tension soil anchor can balance all horizontal load, V, the flagpole footing is constrained. Otherwise, nonconstrained apply. (IBC/CBC 1807.3.2)

INPUT DATA & DESIGN SUMMARY

DIMENTION L = **10** *ft, (3.05 m)*

H = **3** *ft, (0.91 m)*

D = **72** *in, (1829 mm)*

SOIL ANCHOR Slope = **10** **:12,** (θ = *39.806* ^{0})

$T_{allowable}$ = **100** kips/anchor, ASD, *from Soil Report or AASHTO Tab 5.7.6.2*

MAXIMUM LOADS *(ASD level)* M = **120** *ft-kips, (163 kN-m)*

P = **80** *kips, (356 kN), Downward*

V = **120** *kips, (534 kN)*

THE DESIGN IS ADEQUATE.

ALLOW SOIL PRESSURE IN BOTTOM END FLAGPOLE Q_a = **5.35** *ksf, (256.16* kN/m^2)

ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT P_p = **0.25** *ksf / ft, (39.272* $[kN/m^2]/m$)

ANALYSIS

CHECK FLAGPOLE CAPACITY WITHOUT ANCHOR (IBC/CBC 1807.3)

By trials, use pole depth, $d = L$ = 10.00 *ft, (3.05 m)*

Lateral bearing @ bottom, $S_3 = 2 P_p \, Min(d, 12')$ = 6.00 *ksf, (287.28* kN/m^2)

Lateral bearing @ $d/3$, $S_1 = 2 P_p \, Min(d/3, 12')$ = 3.10 *ksf, (148.57* kN/m^2)

Require Depth is given by

$$d = \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad =$$ 18.61 *ft, (5.67 m)* **>** *L* **[Unsatisfactory]**

Where $p = 1.0\,V$ = 120 *kips, (534 kN)* $A = 2.34\,p/(D\,S_1)$ = 15.082

$h = M/V + H$ = 4.00 *ft, (1.22 m)*

$1.0\,P + 0.25\,\pi\,D^2\,(\gamma_{pole} - \gamma_{soil})\,(L + H)$ = 85.405 *kips, (380 kN)*

< $0.25\,\pi\,D^2\,Q_a$ = 151.27 *kips, (673 kN), <== check bearing (ACI 318-19 13.3.1.1)* **[Satisfactory]**

- P **<** $0.25\,\pi\,D^2\,(\gamma_{pole})\,(L + H)$ = 55.13 *kips, (245 kN), <== check uplift* **[Satisfactory]**

Where γ_{pole} = 0.15 *kcf, one cubic foot of flagpole footing weight.* γ_{soil} = 0.07 *kcf, soil/water weight*

CHECK FLAGPOLE CAPACITY WITH ANCHORS (IBC/CBC 1807.3)

T_{anch} = $Min\,(T_{allowable}\,,\,V/Cos\,\theta)$ = 100 *kips, (445 kN)*

By trials, use pole depth, $d = L$ = 10.00 *ft, (3.05 m)*

Lateral bearing @ bottom, $S_3 = 2 P_p \, Min(d, 12')$ = 6.00 *ksf, (287.28* kN/m^2)

Lateral bearing @ $d/3$, $S_1 = 2 P_p \, Min(d/3, 12')$ = 2.06 *ksf, (98.565* kN/m^2)

Require Depth is given by

$$d = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & for \quad nonconstrained \\ \sqrt{\frac{4.25ph}{bS_3}} & for \quad constrained \end{cases} =$$ 7.06 *ft, (2.15 m)* **<** *L* **[Satisfactory]**

Where $p = V - T_{anch}\,Cos\,\theta$ = 43.178 *kips, (192 kN)* $A = 2.34\,p/(D\,S_1)$ = 0

$h = M/p + H$ = 5.7792 *ft, (1.76 m)*

$1.0\,P + 0.25\,\pi\,D^2\,(\gamma_{pole} - \gamma_{soil})\,(L + H) + T_{anch}\,Sin\,\theta$ = 149.4237 *kips, (665 kN)*

< $0.25\,\pi\,D^2\,Q_a$ = 151.27 *kips, (673 kN), <== check bearing (ACI 318-19 13.3.1.1)* **[Satisfactory]**

- P **<** $0.25\,\pi\,D^2\,(\gamma_{pole})\,(L + H) + 2\,T_{allowable}\,Sin\,\theta$ = 183.17 *kips, (815 kN), <== check uplift*

[Satisfactory]

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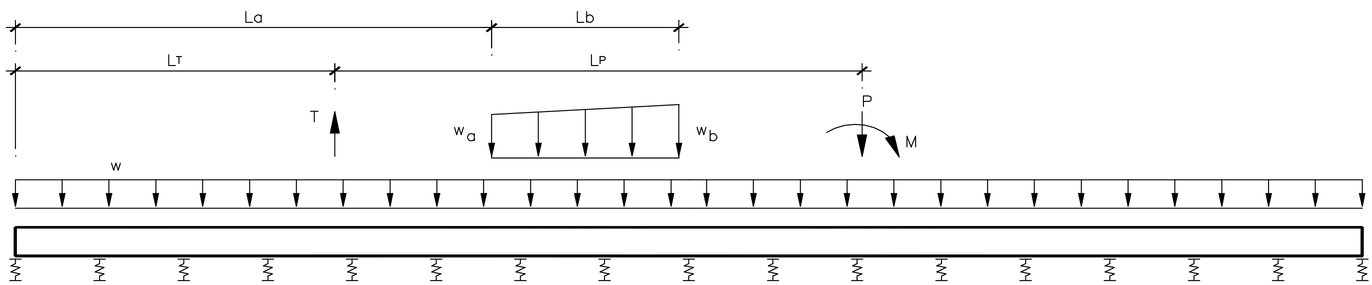
Elastic Strip Foundation Analysis using Finite Element Method Based on 2018 IBC

DESIGN CRITERIA

1. The strip foundation can be reinforced concrete, plain concrete, masonry, steel, or even wood grade beam. But it is assumed elastic at soil–structure interaction.
2. The soil boundary springs are compression only without tension capacity. But this strip is inadequate for overturning controlled foundation.
3. To check soil capacity, the uniform load, ***w***, should be input net value at the foundation bottom face. But to design strip, the ***w*** has to be total gravity value.

INPUT DATA



MODULUS OF SUBGRADE K_1 = 100 *lb / in³ (27144.6 kN / m³), from soil report*

DIMENSION L = 38 *ft, (11.58 m), total strip length*

B = 18 *in, (45.72 cm), strip bottom width*

A = 432 *in² (2787.09 cm²), strip section area*

I = 20736 *in⁴ (863097 cm⁴), moment of inertia*

MODULUS OF ELASTICITY E = 4030.509 *ksi, (27789 MPa)*

LOADS w = 1.2 *kips/ft, (17.5 kN/m)*

w_a = 0.4 *kips/ft, (5.8 kN/m)* w_b = 1.1 *kips/ft, (16.0 kN/m)*

L_a = 9.5 *ft, (2.90 m)* L_b = 12 *ft, (3.66 m)*

T = 18 *kips, (80.1 kN)* P = 9 *kips, (40.0 kN), compression*

(uplift, downward negative) M = 12 *ft-kips, (16.3 kN-m)*

L_T = 4.75 *ft, (1.45 m)* L_P = 24 *ft, (7.32 m)*

ANALYSIS & DESIGN SUMMARY

SPRING VALUE = ***324.583*** *kips / inch, at grid tributary area of* ***3.56*** *ft²*

SETTLEMENTS *Left End =* ***-0.1338*** *in*

Maximum Value = ***0.0175*** *in, (from left end* ***23.75*** *ft)*

Right End = ***0.0063*** *in*

TOTAL VERTICAL SPRING REACTION = ***45.60*** *kips, upward positive*

TOTAL MOMENT SPRING REACTION AT CENTER OF STRIP = ***379.35*** *ft-kips, counterclockwise positive*

SOIL PRESSURE *Left End =* ***0.00*** *psf*

Maximum Value = ***1590.40*** *psf, (from left end* ***23.75*** *ft)*

Right End = ***570.42*** *psf*

DESIGN VALUES OF STRIP SECTION *N =* ***0*** *kips, axial compression*

M = ***48.21*** *ft-kips, (from left end* ***14.25*** *ft)*

V = ***10.87*** *kips, (vertical shear from left end* ***4.75*** *ft)*

L = ***38.00*** *ft*

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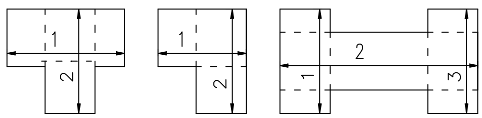
Design of PT Slabs with Rebar Stiffening Beam Based on ACI 318-19, PTI DC10.5-12 & PTI 3rd Edition

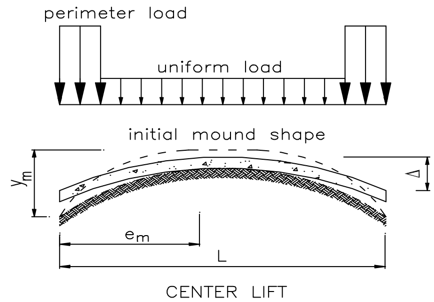
1. DESIGN METHODS

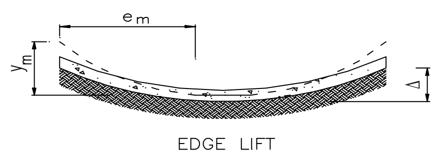
1.1 MAKE SURE THAT THE FOUNDATION PLAN SATISFIES THE SHAPE FACTOR REQUIREMENT:
SF = (ENTIRE FOUNDATION PERIMETER) / (ENTIRE FOUNDATION AREA) ≤ 24 , (PTI Sec. 6.1.3)

1.2 DIVIDE AN IRREGULAR FOUNDATION PLAN INTO OVERLAPPING RECTANGLES AND USING THIS SPREADSHEET DESIGN EACH RECTANGULAR SECTION SEPARATELY.

1.3 If the stiffening beam with only regular rebars, the cracked section capacity for edge lift is checked by mentally moved slab PT forces as out side loads and assumed the secondary section forces zero.



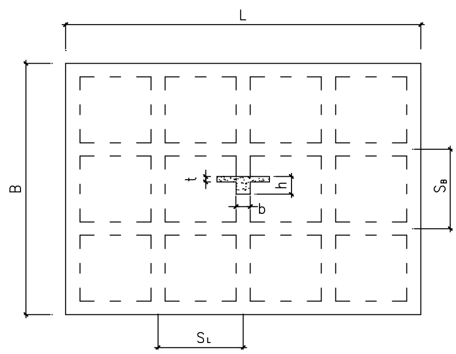




2. INPUT DATA & DESIGN SUMMARY

2.1 SOILS PROPERTIES (FROM SOIL REPORT / GEOTECHNICAL INVESTIGATION)

ALLOWABLE SOIL-BEARING PRESSURE	q_{allow}	=	2000	psf
EDGE MOISTURE VARIATION DISTANCE	e_m	=	9	ft, for center lift
		=	5.2	ft, for edge lift
DIFFERENTIAL SOIL MOVEMENT	y_m	=	0.07	in, for center lift
		=	0.46	in, for edge lift
SLAB-SUBGRADE FRICTION COEFFICIENT	μ	=	0.75	



2.2 STRUCTURAL DATA AND MATERIALS PROPERTIES

SLAB LENGTH		L	=	42	ft
SLAB WIDTH		B	=	24	ft
SLAB THICKNESS		t	=	4	in
PERIMETER LOADING		P	=	695	plf
MAX BEARING LOADING ON THE SLAB		P_b	=	2700	plf
ADDED DEAD LOAD		DL	=	15	psf
LIVE LOAD		LL	=	40	psf
AVERAGE STIFFENING BEAM SPACING, L DIRECTION		S_L	=	12	ft
AVERAGE STIFFENING BEAM SPACING, B DIRECTION		S_B	=	12	ft
STIFFENING BEAM DEPTH		h	=	24	in
STIFFENING BEAM WIDTH		b	=	12	in
CONCRETE STRENGTH		f'_c	=	3	ksi
EFFECTIVE PRESTRESS AFTER ALL LOSSES EXCEPT SG		f_e	=	174	ksi
SLAB PRESTRESSING TENDONS, L DIRECTION	6	tendons w/		0.153	in^2 at each tendon.
SLAB PRESTRESSING TENDONS, B DIRECTION	9	tendons w/		0.153	in^2 at each tendon.
REBAR or TENDON IN THE BOTTOM OF EACH BEAM	0	tendons w/		0.153	in^2 (only for edge lift governing required)
	2	#		5	

THE DESIGN IS ADEQUATE.

3. DETERMINE SECTION PROPERTIES

L DIRECTION						B DIRECTION					
n =	3		y_b =	17.38	in	n =	5		y_b =	17.52	in
A =	1872	in^2	S_t =	13505	in^3	A =	3216	in^2	S_t =	23313	in^3
I =	89339	in^4	S_b =	5139	in^3	I =	151010	in^4	S_b =	8618	in^3
CGS =	22.00	in	e =	4.62	in	CGS =	22.00	in	e =	4.48	in

4. CALCULATE MAXIMUM APPLIED SERVICE MOMENTS (PTI Sec. 7.0)

4.1 CENTER LIFT MOMENT AT L DIRECTION

For e_m = 9 ft

$M_L = A_0 (B\, e_m^{1.238} + C)$ = 2.90 ft-kips / ft

Where $A_0 = (L^{0.013} S_B^{0.306} h^{0.688} P^{0.534} y_m^{0.193}) / 727$ = 0.542

B = 1, for $e_m < 5$
B = MIN[$(y_m - 1) / 3$, 1], for $e_m > 5$ = -0.31

C = 0, for $e_m < 5$
C = MAX{[8 - (P - 613) / 255] $(4 - y_m) / 3$], 0}, for $e_m > 5$ = 10.06

For e_m = 5 ft

$M_L = A_0 (B\, e_m^{1.238} + C)$ = 3.98 ft-kips / ft

Where $A_0 = (L^{0.013} S_B^{0.306} h^{0.688} P^{0.534} y_m^{0.193}) / 727$ = 0.542

B = 1.00

C = 0.00

USE M_L = 3.98 ft-kips / ft

CENTER LIFT MOMENT AT B DIRECTION

For e_m = 9 ft

$M_B = (58 + e_m) M_L / 60$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1 = 3.24 ft-kips / ft

For e_m = 5 ft

$M_B = (58 + e_m) M_L / 60$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1 = 4.17 ft-kips / ft

USE M_B = 4.17 ft-kips / ft

4.2 EDGE LIFT MOMENT AT L DIRECTION

$M_L = S_B^{0.10} (h\, e_m)^{0.78} y_m^{0.66} / (7.2\, L^{0.0065} P^{0.04})$ = 3.46 ft-kips / ft

EDGE LIFT MOMENT AT B DIRECTION

$M_B = h^{0.35} (19 + e_m) M_L / 57.75$, for L /B > 1.1
$M_B = M_L$, for L /B < 1.1 = 4.41 ft-kips / ft

5. CHECK FLEXURAL CONCRETE STRESSES (PTI Sec. 7.3)

5.1 ALLOWABLE CONCRETE STRESSES

FLEXURAL TENSILE STRESS	$f_{t,allow} = - 6 (f_c')^{0.5}$ =	-0.329	ksi
FLEXURAL COMPRESSIVE STRESS	$f_{c,allow} = - 0.45 f_c'$ =	1.350	ksi

(cont'd)

5.2 TOP STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = P_r / A - M_L / S_t + P_r\, e / S_t =$ 0.020 ksi

Where $P_r = P_e - SG =$ 119.18 kips

$P_e = f_e A_{ps} =$ 159.73 kips

$SG = W_{slab}\, \mu / 2000 =$ 40.56 kips

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

TOP STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = P_r / A - M_B / S_t + P_r\, e / S_t =$ 0.010 ksi

Where $P_r = P_e - SG =$ 199.04 kips

$P_e = f_e A_{ps} =$ 239.60 kips / ft

$SG = W_{slab}\, \mu / 2000 =$ 40.56 kips

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

5.3 BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT L DIRECTION

$f = P_r / A + M_L / S_b - P_r\, e / S_b =$ 0.179 ksi

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

BOTTOM STRESS, FOR CENTER LIFT MOMENT, AT B DIRECTION

$f = P_r / A + M_B / S_b - P_r\, e / S_b =$ 0.203 ksi

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

5.4 TOP STRESS, FOR EDGE LIFT MOMENT, AT L DIRECTION

$f = P_r / A - M_L / S_b - P_r\, e / S_b =$ -0.237 ksi

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

TOP STRESS, FOR EDGE LIFT MOMENT, AT B DIRECTION

$f = P_r / A - M_B / S_b - P_r\, e / S_b =$ -0.299 ksi

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

5.5 BOTTOM STRESS, FOR EDGE LIFT MOMENT, AT L DIRECTION

$f = P_r / A + M_L / S_t + P_r\, e / S_t =$ 0.178 ksi

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

BOTTOM STRESS, FOR EDGE LIFT MOMENT, AT B DIRECTION

$f = P_r / A + M_B / S_t + P_r\, e / S_t =$ 0.195 ksi

Then f $>$ $f_{t,allow}$ **[Satisfactory]**

$<$ $f_{c,allow}$ **[Satisfactory]**

6. CHECK MINIMUM FOUNDATION STIFFNESS (PTI Sec. 9.0)

6.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$\beta = (E_{cr}\, I / E_s)^{1/4} / 12 =$ 9.197 ft

Where $E_{cr} = (0.5)\, 33\, w^{1.5}\, (f_c')^{0.5} =$ 1660280 psi

$E_s =$ 1000 psi

$w =$ 150 pcf

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$\beta = (E_{cr}\, I / E_s)^{1/4} / 12 =$ 10.486 ft

Where $E_{cr} = (0.5)\, 33\, w^{1.5}\, (f_c')^{0.5} =$ 1660280 psi

$E_s =$ 1000 psi

6.2 CHECK MINIMUM FOUNDATION STIFFNESS AT L DIRECTION

FOR CENTER LIFT

$12000\, M_L\, B\, C_\Delta\, z_L / E_{cr} =$ 10428 in^4 $<$ I_L

Where C_Δ = 360 **[Satisfactory]**

$z_L = \min(L\,,\, 6\beta) =$ 42.00 ft

$I_L =$ 89339 in^4

FOR EDGE LIFT

$12000\, M_L\, B\, C_\Delta\, z_L / E_{cr} =$ 18139 in^4 $<$ I_L

Where C_Δ = 720 **[Satisfactory]**

CHECK MINIMUM FOUNDATION STIFFNESS AT B DIRECTION

FOR CENTER LIFT

$12000\, M_B\, L\, C_\Delta\, z_B / E_{cr} =$ 10949 in^4 $<$ I_B

Where C_Δ = 360 **[Satisfactory]**

$z_B = \min(B\,,\, 6\beta) =$ 24.00 ft

$I_B =$ 151010 in^4

FOR EDGE LIFT

$12000\, M_B\, L\, C_\Delta\, z_B / E_{cr} =$ 23118 in^4 $<$ I_B

Where C_Δ = 720 **[Satisfactory]**

7. CHECK SHEAR CAPACITY (PTI Sec. 8.0)

7.1 APPLIED SERVICE LOAD SHEAR AT L DIRECTION

FOR CENTER LIFT

$V_L = L^{0.09}\, S_B^{0.71}\, h^{0.45}\, P^{0.44}\, y_m^{0.16}\, e_m^{0.93} / 1940 =$ 1.580 kips/ft

FOR EDGE LIFT

$V_L = L^{0.07}\, h^{0.4}\, P^{0.03}\, y_m^{0.67}\, e_m^{0.16} / (3.0\, S_B^{0.015}) =$ 1.400 kips/ft

APPLIED SERVICE LOAD SHEAR AT B DIRECTION

FOR CENTER LIFT

$V_B = B^{0.19}\, S_L^{0.45}\, h^{0.20}\, P^{0.54}\, y_m^{0.04}\, e_m^{0.97} / 1350 =$ 2.031 kips/ft

FOR EDGE LIFT

$V_B = B^{0.07}\, h^{0.4}\, P^{0.03}\, y_m^{0.67}\, e_m^{0.16} / (3.0\, S_L^{0.015}) =$ 1.347 kips/ft

7.2 ALLOWABLE CONCRETE SHEAR STRESS, AT L DIRECTION

$v_c = 2.4\, (f_c')^{0.5} + 0.2\, f_p =$ 0.144 ksi

Where $f_p =$ 0.064 ksi $>$ 50 psi

[Satisfactory]

ALLOWABLE CONCRETE SHEAR STRESS, AT B DIRECTION

$v_c = 2.4\, (f_c')^{0.5} + 0.2\, f_p =$ 0.144 ksi

Where $f_p =$ 0.062 ksi $>$ 50 psi

[Satisfactory]

7.3 SHEAR STRESS OF RIBBED FOUNDATION, AT L DIRECTION

FOR CENTER LIFT

$v = V\, B / (n\, h\, b) =$ 0.044 ksi $<$ v_c

[Satisfactory]

FOR EDGE LIFT

$v = V\, B / (n\, h\, b) =$ 0.039 ksi $<$ v_c

[Satisfactory]

SHEAR STRESS OF RIBBED FOUNDATION, AT B DIRECTION

FOR CENTER LIFT

$v = V\, L / (n\, h\, b) =$ 0.059 ksi $<$ v_c

[Satisfactory]

FOR EDGE LIFT

$v = V\, L / (n\, h\, b) =$ 0.039 ksi $<$ v_c

[Satisfactory]

8. CHECK SOIL BEARING (PTI Sec. 6.2.4 & 6.3.3)

8.1 APPLIED LOADING

SLAB WEIGHT	150 L B t	=	50400	lbs
ADDED DL	DL L B	=	15120	lbs
LIVE LOAD	LL L B	=	40320	lbs
BEAM WEIGHT	150 (h-t) b (Total Length)	=	57750	lbs
PERIMETER LOAD	P (2L + 2B)	=	91740	lbs

RIB BEARING AREA = 811.44 ft^2

SOIL PRESSURE q = Total Load / THE AREA = 315 psf

$<$ q_{allow}

[Satisfactory]

9. CHECK SLAB STRESS DUE TO LOAD-BEARING PARTITIONS (PTI Sec. 3.0 & 7.0)

9.1 RELATIVE STIFFNESS LENGTH AT L DIRECTION

$M_{max} = P_b\, \beta / 4 =$ 8.10 ft-kips / ft

Where $\beta = \mathrm{MIN}[(E_c\, t^3 / 3\, k_s)^{0.25},\, S_B] =$ 12 ft

$k_s =$ 4 lb / in^3

RELATIVE STIFFNESS LENGTH AT B DIRECTION

$M_{max} = P_b\, \beta / 4 =$ 8.10 ft-kips / ft

Where $\beta = \mathrm{MIN}[(E_c\, t^3 / 3\, k_s)^{0.25},\, S_L] =$ 12 ft

$k_s =$ 4 lb / in^3

9.2 TENSILE STRESS AT L DIRECTION

$f = P_r / A - M_{max} / 2\, t^2 =$ -0.189 ksi

$>$ $f_{t,allow}$ **[Satisfactory]**

TENSILE STRESS AT B DIRECTION

$f = P_r / A - M_{max} / 2\, t^2 =$ -0.191 ksi

$>$ $f_{t,allow}$ **[Satisfactory]**

(cont'd)

10. CHECK CRACKED SECTION CAPACITY FOR CENTER LIFT (PTI Sec. 7.4)

10.1 CHECK CRACKED SECTION CAPACITY AT L DIRECTION

M_{cr} = F (h - 2" - 0.5a) =		281.3	ft-kips	**>**	0.9 M_L
Where	F =	159.73	kips	**[Satisfactory]**	
	a = F / 0.85 f_c' b =	1.74	in		
	0.9 M_L =	85.9	ft-kips, total		

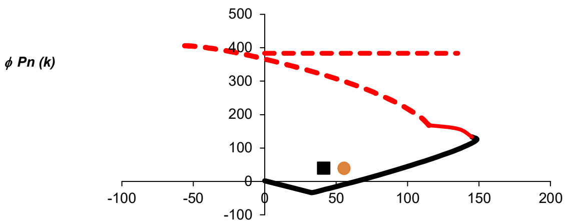
CHECK CRACKED SECTION CAPACITY AT B DIRECTION

M_{cr} = F (h - 2" - 0.5a) =		423.6	ft-kips	**>**	0.9 M_B
Where	F =	239.60	kips	**[Satisfactory]**	
	a = F / 0.85 f_c' b =	1.57	in		
	0.9 M_B =	157.8	ft-kips, total		

11. CHECK CRACKED SECTION CAPACITY FOR EDGE LIFT (ACI 318-19 & PTI Sec. 7.4)

M_{cr} = F (h - 3" - 0.5a) =		0.0	ft-kips	<	0.9 M_L
Where	F =	0.00	kips		
	a = F / 0.85 f_c' b =	0.00	in		
	0.9 M_L =	74.7	ft-kips, total		

M_{cr} = F (h - 3" - 0.5a) =		0.0	ft-kips	<	0.9 M_B
Where	F =	0.00	kips		
	a = F / 0.85 f_c' b =	0.00	in		
	0.9 M_B =	166.6	ft-kips, total		



Solid Black Line - **Tension Controlled**
Solid Red Line - **Transition**
Dash Line - **Compression Controlled**

$$f_c = \begin{cases} \beta_e f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] , & for \ 0 < \varepsilon_c < \varepsilon_o \\ \beta_e f'_c , & for \ \varepsilon_c \ge \varepsilon_o \end{cases} , \quad f_s = \begin{cases} \varepsilon_s E_s , & for \ \varepsilon_s \le \varepsilon_{ty} \\ f_y , & for \ \varepsilon_s > \varepsilon_{ty} \end{cases}$$

$$\varepsilon_o = \frac{2\left(\beta_e f'_c\right)}{E_c} , \quad \varepsilon_{ty} = \frac{f_y}{E_s} , \quad \varepsilon_t = \begin{cases} \varepsilon_{ty} , & for \ compression \ controlled \\ \varepsilon_{ty} + 0.003 , & for \ tension \ controlled \end{cases}$$

$$E_c = 57\sqrt{f'_c} , \quad E_s = 29000 ksi , \quad \beta_e = 0.85 , \quad \varepsilon_{cu} = 0.003$$

Where	$M_{u,L}$ = 1.5 M_L / n =	41.5	ft-kips / beam, SD level
	$P_{u,L}$ = 1.0 P_r / n =	39.7	kips / beam, SD level

$M_{u,B}$ = 1.5 M_B / n =	55.5	ft-kips / beam, SD level
$P_{u,B}$ = 1.0 P_r / n =	39.8	kips / beam, SD level

[Satisfactory]

[Satisfactory]

12. SUGGEST RATIO OF EXPECTED ELONGATION

r = f_e / 0.8 E_{ps} = 7.77E-03

Where E_{ps} = 28000 ksi

Technical References:

1. "Design of Post-Tensioned Slab-on-Ground, Third Edition", Post-Tensioning Institute, 2004.
2. "Addendum No.1 to The 3RD Edition of Design of Post-Tensioned Slab-on-Ground", Post-Tensioning Institute, May 2007.
3. "Addendum No.2 to The 3RD Edition of Design of Post-Tensioned Slab-on-Ground", Post-Tensioning Institute, May 2008.

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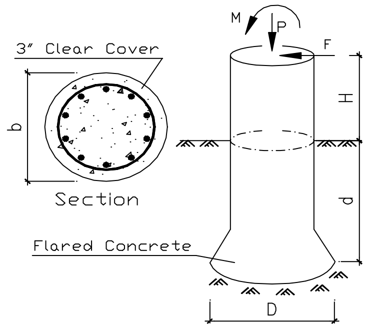
Sonotube Footing Design Based on 2021 IBC, ASCE 7-22, & ACI 318-19

DESIGN CRITERIA

1. Since the same reinforcement from flared bottom to footing top, the 3 in clear concrete cover designed in both soil (3 in required) and exposed to weather (2 in required), (ACI 318-19 Table 20.5.1.3.1). So the capacity is less than regular circular concrete column.
2. If group footings supporting seismic loads, all footings should be tied together by 25% of the smaller footing design gravity load, (2021 IBC 1809.13).
3. If single footing supporting seismic loads, the footings seismic loads have to be based on the response modification coefficient, R, 1.0, (ASCE 7-22 Table 12.2-1).

INPUT DATA & DESIGN SUMMARY

FLARED BOTTOM DIAMETER	D =	30	in, (762 mm)		
FOOTING SECTION DIAMETER	b =	20	in, (508 mm)		
VERTICAL REINFORCEMENT		6	#	6	
LATERAL REINFORCEMENT OPTION (0=Spirals, 1=Ties)				1	Ties
LATERAL REINFORCEMENT	#	4	@	12	in. (305 mm), o.c.
REBAR YIELD STRESS	f_y =	60	ksi, (414 MPa)		
CONCRETE STRENGTH	f_c' =	5	ksi, (34 MPa)		
THE MAX CONCRETE STRESS	β =	0.85	f_c' (0.85 on ACI, 1.0 on AISC)		
SERVICE AXIAL LOAD	P =	18	kips, (80 kN), ASD		
SERVICE MOMENT	M =	3	ft-kips, (4 kN-m), ASD		
SERVICE SHEAR LOAD	F =	2	kips, (9 kN), ASD		
LATERAL SOIL CAPACITY	P_P =	0.35	ksf / ft, (55 kPa / m)		
RESTRAINED @ GRADE ?(1=yes,0=no)		0	No		
ALLOW SOIL PRESSURE	Q_a =	4.5	ksf, (215 kPa)		
HEIGHT ABOVE GRADE	H =	3	ft, (0.91 m)		
DEPTH IN SOIL EMBEDMENT	d =	4.90	ft, (1.49 m)		



THE DESIGN IS ADEQUATE.

ANALYSIS

DESIGN COLUMN FOOTING (2021 IBC 1807.3)

Lateral bearing @ bottom, $S_3 = 2\,P_P\,Min(\,d\,,\,12'\,)$ = 3.43 ksf

Lateral bearing @ $d/3$, $S_1 = 2\,P_P\,Min(\,d/3\,,\,12'\,)$ = 1.14 ksf

Require Depth is given by

$$d=\begin{cases}\dfrac{A}{2}\left[1+\sqrt{1+\dfrac{4.36h}{A}}\right] & for\quad nonconstrained\\ \sqrt{\dfrac{4.25Ph}{bS_3}} & for\quad constrained\end{cases} = \quad 4.904 \text{ ft}$$

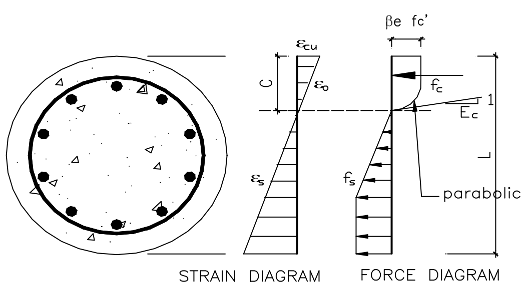
[Satisfactory]

Where $P = F =$ 2.00 kips

$A = 2.34\,P/(b\,S_1) =$ 2.45

$h = H =$ 3.00 ft

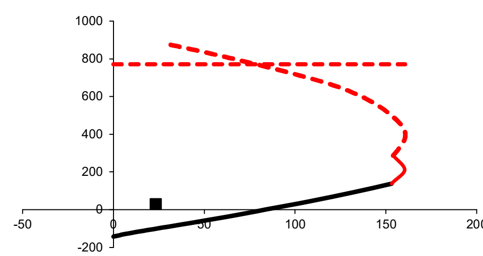
CHECK COLUMN FLEXURAL & AXIAL CAPACITY



$$f_c=\begin{cases}\beta_e f_c'\left[2\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right)-\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right)^2\right] & ,\ for\ \ 0<\varepsilon_c<\varepsilon_o\\ \beta_e f_c' & ,\ for\ \ \varepsilon_c\ge\varepsilon_o\end{cases}\quad ,\quad f_s=\begin{cases}\varepsilon_s E_s & ,\ for\ \ \varepsilon_s\le\varepsilon_{ty}\\ f_y & ,\ for\ \ \varepsilon_s>\varepsilon_{ty}\end{cases}$$

$$\varepsilon_o=\frac{2\left(\beta_e f_c'\right)}{E_c}\ ,\ \varepsilon_{ty}=\frac{f_y}{E_s}\ ,\ \varepsilon_t=\begin{cases}\varepsilon_{ty} & ,\ for\ \ compression\ \ controlled\\ \varepsilon_{ty}+0.003 & ,\ for\ \ tension\ \ controlled\end{cases}$$

$$E_c=57\sqrt{f_c'}\ ,\ E_s=29000ksi\ ,\ \beta_e=1.0\ ,\ \varepsilon_{cu}=0.003$$



Solid Black Line - **Tension Controlled**

Solid Red Line - **Transition**

Dash Line - **Compression Controlled**

P_u = 30.1 kips, (134 kN)

ϕM_{nc} @ P_u = 100.2 ft-kips, (136 kN-m)

> M_u = 23.3 ft-kips, (32 kN-m)

[Satisfactory]

where ϕ = 0.9 , (AISC 341-16 B3.2)

CHECK SHEAR CAPACITY (ACI 318-19 10 & 22.5)

$\phi\,V_n = \phi\,(V_s + V_c)$ (ACI 318-19 22.5)

> V_u = 3.0 kips, (13 kN) **[Satisfactory]**

where ϕ = 0.75 (ACI 318-19 21.2)

	d	A_0	A_v	$V_c = 2\,(f_c')^{0.5} A_0$	V_s = MIN (d f_y A_v / s , $4V_c$)	$\phi\,V_n$
x	16.1	143.14	0.40	20.2	32.3	39

s_{max} = 12 (ACI 318-19 25.7) s_{provd} = 12 in

s_{min} = 1 **[Satisfactory]**

CHECK SOIL BEARING CAPACITY (ACI 318-19 13.3.1.1)

$$q_{max}=\frac{4P}{\pi D^2}+0.15(H+d)-0.11d = \quad 4.31 \text{ ksf} \quad < \quad Q_a$$

[Satisfactory]

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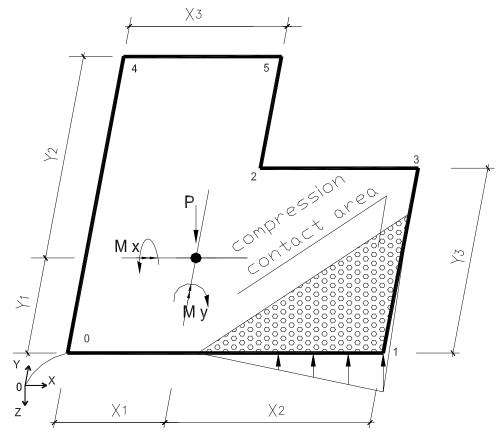
Rigid Footing Moment Capacity Design Based on ASCE 41-17 & ACI 318-19

DESIGN CRITERIA

1. Soil resists compression only, without tension capacity. Soil bearing pressure distribution is a plane, and a triangular distribution block when un-full area compression under the footing.

*2. The input **P**, M_x, & M_y are net loads on the footing bottom face, which the location (X_1 & Y_1) is directly from top building analysis and not has to be at the centroid of footing bottom section.*

*3. This general methodology cannot apply to Dynamic Analysis (ASCE 7-22 12.9) and higher level seismic design, because all CQC results are probability results without **P**, M_x, & M_y load directions.*



INPUT DATA & DESIGN SUMMARY

ALLOWABLE SOIL PRESSURE Q_a = **5** *ksf, (239 kPa)*

DIMENSIONS
X_1 = **3** *ft, (0.9m)*
X_2 = **12** *ft, (3.7m)*
X_3 = **9** *ft, (2.7m)*
Y_1 = **3.5** *ft, (1.1m)*
Y_2 = **9** *ft, (2.7m)*
Y_3 = **7** *ft, (2.1m)*

LOADS
P = **8** *kips, (36 kN), ASD*
My = **88** *ft-kips, (119 kN-m), rotation about the Y axis*
Mx = **20** *ft-kips, (27 kN-m), rotation about the X axis*

THE FOOTING DESIGN IS ADEQUATE.

The Maximum Soil Pressure:
Pmax = ***4.523*** *ksf, (217 kPa)*
at Point ***1*** *Corner.*

ANALYSIS

CHECK THE CENTER OF GRAVITY WITHIN FOOTING

XCG = X1 + My / P =	*14.00*	*ft*	YCG = Y1 - Mx / P =	*1.00*	*ft*	A =	*154.50*	*ft2*
>	*0*		>	*0*		(full footing area)		
>	*9.00*	[Satisfactory]	<	*7.00*		AC =	*3.78*	*ft2*
<	*15.00*		<	*12.50*		(compression contact area)		

DETERMINE EQUATION OF SOIL PRESSURE DISTRIBUTION PLANE

$$\begin{vmatrix} X\ (ft) & Y\ (ft) & Z\ (ksf) & 1 \\ 0.45 & 0.35 & -31.367 & 1 \\ 14.25 & 0.35 & 4.523 & 1 \\ 0.45 & 11.875 & -49.043 & 1 \end{vmatrix} = 0$$

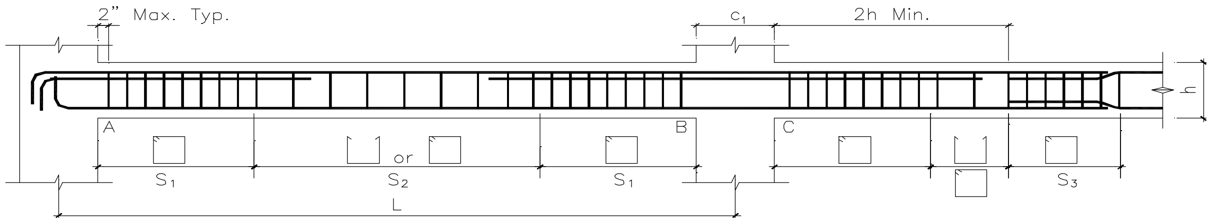
CHECK SOIL BEARING CAPACITY

Corner	0	1	2	3	4	5
P (ksf)	**0.00**	**4.52**	**0.00**	**0.00**	**0.00**	**0.00**

< Qa [Satisfactory]

Seismic Design for Special Moment Resisting Frame Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c' =	3	ksi	DISTRIBUTED UNFACTORED LOADS	D =	4.1	kips / ft
REBAR YIELD STRESS	f_y =	60	ksi		L =	0.6	kips / ft
BEAM LENGTH BET. COL. CENTERS	L =	25	ft	SECTION MOMENTS & SHEARS AT FACE OF COL. (ft-kips, kips)			
BEAM SIZE	b =	24	in				
	h =	36	in				
COLUMN SIZE	c_1 =	36	in				
	c_2 =	24	in				
SEISMIC PARAMETER	S_{DS} =	0.44					
REDUNDANCY FACTOR	ρ =	1.15					

	M_A	V_A	M_B	V_B
D	-130	45	-180	-45
L	-25	7	-25	-7
Q_E	665	61	-665	-61

LONGITUDINAL REINFORCING

SECTION	A	MID SPAN	B
TOP	9 # 9	3 # 9	9 # 9
	(d = 33.31 in)	(d = 33.31 in)	(d = 33.31 in)
	(1 Layer)	(1 Layer)	(1 Layer)
BOTTOM	5 # 9	5 # 9	5 # 9
	(d = 33.31 in)	(d = 33.31 in)	(d = 33.31 in)
	(1 Layer)	(1 Layer)	(1 Layer)

HOOP & STIRRUP LOCATIONS (ACI 21.5.3)

LOCATION	AT END, S_1	AT MID, S_2	AT SPLICE, S_3
LENGTH	72 in	120 in	48 in
	(2h)	(L-4h-c_1)	1.3 MAX{$0.075f_y\psi_t\psi_b\psi_s d_b/[(f_c')^{0.5}(c+K_{tr})/d_b]$, 12}
TYPE	Hoops	Stirrups	Hoops
BAR	5 Legs # 5	3 Legs # 5	5 Legs # 5
	(Legs to alternate long bars supported, ACI 7.10.5.3)		
SPACING	@ 8 in o.c.	@ 16 in o.c.	@ 4 in o.c.
	MIN(d/4, 8d_b, 24d_t, 12)	(d/2)	MIN(d/4, 4)

THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS

$U = (1.2+0.2S_{DS})D + \rho Q_E + 1.0L$ (ACI 9-5)

AT SECTION **A**, FACE OF COLUMN

V_u = 135.1 kips or -5.2 kips

M_u = 572.3 ft-kips or -957.2 ft-kips

AT SECTION **B**, FACE OF COLUMN

V_u = -135.1 kips or 5.2 kips

M_u = -1021.6 ft-kips or 507.9 ft-kips

AT MIDDLE OF THE SPAN

V_u = 0.0 kips or 0.0 kips

M_u = 131.1 ft-kips or 131.1 ft-kips

$U = (0.9-0.2S_{DS})D + \rho Q_E$ (ACI 9-7)

AT SECTION **A**, FACE OF COLUMN

V_u = 106.7 kips or -33.6 kips

M_u = 659.2 ft-kips or -870.3 ft-kips

AT SECTION **B**, FACE OF COLUMN

V_u = -106.7 kips or 33.6 kips

M_u = -910.9 ft-kips or 618.6 ft-kips

AT MIDDLE OF THE SPAN

V_u = 0.0 kips or 0.0 kips

M_u = 75.6 ft-kips or 75.6 ft-kips

$U = 1.2D + 1.6L$ (ACI 9-2)

AT SECTION **A**, FACE OF COLUMN

V_u = 65.2 kips

M_u = -196.0 ft-kips

AT MIDDLE OF THE SPAN

V_u = 0.0 kips

M_u = 129.8 ft-kips

AT SECTION **B**, FACE OF COLUMN

V_u = -65.2 kips

M_u = -256.0 ft-kips

cont'd

CHECK SECTION REQUIREMENTS (ACI 21.5.1)

$P_u < 0.1A_g f_c'$ **[Satisfactory]**

$L_u = L - c_1 =$ 22.00 ft **>** 4 d = 11.10 ft **[Satisfactory]**

b / h = 0.67 **>** 0.3 **[Satisfactory]**

b = 24 in **>** 10 in **[Satisfactory]**

< $c_2 + 1.5h =$ 78 in **[Satisfactory]**

CHECK FLEXURAL REQUIREMENTS

AT SECTION **A**, FACE OF COLUMN

(ACI 21.5.2.1) $\rho_{top} =$ 0.011 **>** $\rho_{min} = MIN[3(f_c')^{0.5}/f_y,\ 200/f_y] =$ 0.003 **[Satisfactory]**

< $\rho_{max} =$ 0.025 **[Satisfactory]**

$\rho_{bot} =$ 0.006 **>** $\rho_{min} =$ 0.003 **[Satisfactory]**

< $\rho_{max} =$ 0.025 **[Satisfactory]**

(ACI 21.5.2.2) $M_{n,bot}$ **>** $(1/2)M_{n,top}$ **[Satisfactory]**

where $M_{n,bot} = \rho_{bot}\, bd^2 f_y\, (1 - 0.588\rho_{bot}\, f_y/f_c') =$ 772 ft-kips **>** M_u / ϕ **[Satisfactory]**

$M_{n,top} = \rho_{top}\, bd^2 f_y\, (1 - 0.588\rho_{top}\, f_y/f_c') =$ 1301 ft-kips **>** M_u / ϕ **[Satisfactory]**

ϕ = 0.9

AT SECTION **B**, FACE OF COLUMN

(ACI 21.5.2.1) $\rho_{top} =$ 0.011 **>** $\rho_{min} =$ 0.003 **[Satisfactory]**

< $\rho_{max} =$ 0.025 **[Satisfactory]**

$\rho_{bot} =$ 0.006 **>** $\rho_{min} =$ 0.003 **[Satisfactory]**

< $\rho_{max} =$ 0.025 **[Satisfactory]**

(ACI 21.5.2.2) $M_{n,bot}$ **>** $(1/2)M_{n,top}$ **[Satisfactory]**

where $M_{n,bot} = \rho_{bot}\, bd^2 f_y\, (1 - 0.588\rho_{bot}\, f_y/f_c') =$ 772 ft-kips **>** M_u / ϕ **[Satisfactory]**

$M_{n,top} = \rho_{top}\, bd^2 f_y\, (1 - 0.588\rho_{top}\, f_y/f_c') =$ 1301 ft-kips **>** M_u / ϕ **[Satisfactory]**

AT MIDDLE OF THE SPAN

(ACI 21.5.2.1) $\rho_{top} =$ 0.004 **>** $\rho_{min} =$ 0.003 **[Satisfactory]**

< $\rho_{max} =$ 0.025 **[Satisfactory]**

$\rho_{bot} =$ 0.006 **>** $\rho_{min} =$ 0.003 **[Satisfactory]**

< $\rho_{max} =$ 0.025 **[Satisfactory]**

(ACI 21.5.2.2) $M_{n,top}$ **>** $(1/4)M_{n,max}$ **[Satisfactory]**

$M_{n,bot}$ **>** $(1/4)M_{n,max}$ **[Satisfactory]**

where $M_{n,top} = \rho_{top}\, bd^2 f_y\, (1 - 0.588\rho_{top}\, f_y/f_c') =$ 478 ft-kips **>** M_u / ϕ **[Satisfactory]**

$M_{n,bot} = \rho_{bot}\, bd^2 f_y\, (1 - 0.588\rho_{bot}\, f_y/f_c') =$ 772 ft-kips **>** M_u / ϕ **[Satisfactory]**

$M_{n,max} =$ 1301 ft-kips

CHECK SHEAR STRENGTH (ACI 21.5.4)

FOR SEISMIC LOAD ACTING TO THE LEFT

$V_e = (M_{pr,\,A,\,top} + M_{pr,\,B,bot}) / L_n + V_{gL} =$ 178.7 kips **<** $8\phi(f_c')^{0.5}bd =$ 262.7 kips **[Satisfactory]**

$V_e - dw_u =$ 162.4 kips **<** $\phi[V_c + A_v f_y d/s_1] =$ 290.4 kips **[Satisfactory]**

$V_e - (2h + d)w_u =$ 127.1 kips **<** $\phi[2(f_c')^{0.5}bd + A_v f_y d/s_2] =$ 152.8 kips **[Satisfactory]**

where $V_c = 2(f_c')^{0.5}bd =$ 0.0 kips, (Per ACI 21.5.4.2, $V_c = 0$, if $(V_e - V_{gL}) \geq 50\%\ V_e$ AND $P_u < A_g f_c' / 20$)

$M_{pr,\,A,\,top} = \rho_{top}\, bd^2 f_y\, (1.25 - 0.919\rho_{top}\, f_y/f_c') =$ 1564 ft-kips

$M_{pr,\,B\,,bot} = \rho_{bot}\, bd^2 f_y\, (1.25 - 0.919\rho_{bot}\, f_y/f_c') =$ 945 ft-kips

$L_n = L - c_1 =$ 22.0 ft

$w_u = (1.2+0.2S_{SD})D + 1.0L =$ 5.9 kips / ft, (for CBC, only D + L, without factor)

$V_{gL} = w_u L_n / 2 =$ 64.7 kips

ϕ = 0.75 (ACI 9.3.2.3)

$A_v =$ 1.55 in^2 @ end , 0.93 in^2 @ mid of beam

FOR SEISMIC LOAD ACTING TO THE RIGHT

$V_e = (M_{pr,\,A,\,bot} + M_{pr,\,B,top}) / L_n + V_{gL} =$ 178.7 kips **<** $8\phi(f_c')^{0.5}bd =$ 262.7 kips **[Satisfactory]**

$V_e - dw_u =$ 162.4 kips **<** $\phi[V_c + A_v f_y d/s_1] =$ 290.4 kips **[Satisfactory]**

$V_e - (2h - d)w_u =$ 110.8 kips **<** $\phi[2(f_c')^{0.5}bd + A_v f_y d/s_2] =$ 152.8 kips **[Satisfactory]**

where $M_{pr,\,A\,,bot} = \rho_{bot}\, bd^2 f_y\, (1.25 - 0.919\rho_{bot}\, f_y/f_c') =$ 945 ft-kips

$M_{pr,\,B,\,top} = \rho_{top}\, bd^2 f_y\, (1.25 - 0.919\rho_{top}\, f_y/f_c') =$ 1564 ft-kips

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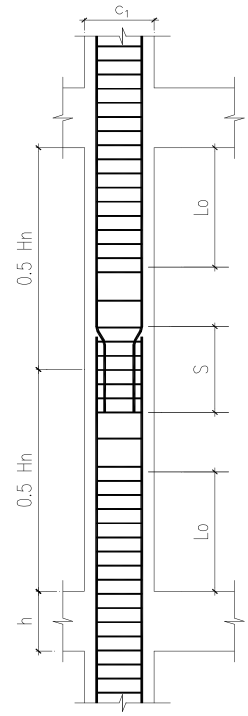
Seismic Design for Special Moment Resisting Frame Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	3	ksi
REBAR YIELD STRESS	f_y =	60	ksi
COLUMN CLEAR HEIGHT	H_n =	10.17	ft
COLUMN SIZE	c_1 =	36	in
	c_2 =	36	in
SEISMIC PARAMETER	S_{DS} =	0.44	
REDUNDANCY FACTOR	ρ =	1.15	



SECTION MOMENTS & SHEARS AT END OF COL. (ft-kips, kips)

	P	M_{top}	V	M_{bot}
D	692	-4	-20	-2
L	87	-1	-1	-1
Q_E	2	900	93	500

LONGITUDINAL REINFORCING

SECTION	TOP	BOTTOM
LEFT	6 # 8 (d = 33.38 in) (1 Layer)	6 # 8 (d = 33.38 in) (1 Layer)
RIGHT	6 # 8 (d = 33.38 in) (1 Layer)	6 # 8 (d = 33.38 in) (1 Layer)
TOTAL BARS	20 # 8 (8 # 8 at sides)	20 # 8 (8 # 8 at sides)

TRANSVERSE REINFORCEMENT FOR CONFINEMENT (ACI 21.6.4 & 21.6.3)

LOCATION	AT END, Lo	AT MID	AT SPLICE, S
LENGTH	36 in MAX(c_1, H_n/16, 18)	4 in + 4 in	43 in 1.3Max{(0.075)$f_y d_b$/[$(f_c')^{0.5}$(c+Ktr)/d_b],12}
TYPE	Hoops	Hoops	Hoops
BAR SIZE	4 Legs # 5	4 Legs # 5	4 Legs # 5
	(Legs to alternate long bars supported, ACI 7.10.5.3)		
SPACING	@ 4 in o.c. MIN{c_1/4, 6d_b, MAX[MIN(4+(14-h_x)/3, 6), 4]}	@ 6 in o.c. MIN(6d_b, 6)	@ 4 in o.c. Same as END Lo

THE COLUMN DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS

$U = (1.2+0.2S_{DS})D + \rho Q_E + 1.0L$ (ACI 9-5)

P_u = 980.6 kips $M_{u,top}$ = 1028.8 ft-kips
V_u = 80.2 kips $M_{u,bot}$ = 571.4 ft-kips

$U = (0.9-0.2S_{DS})D + \rho Q_E$ (ACI 9-7)

P_u = 564.2 kips $M_{u,top}$ = 1031.8 ft-kips
V_u = 90.7 kips $M_{u,bot}$ = 573.4 ft-kips

U = 1.2D + 1.6L (ACI 9-2)

P_u = 969.6 kips $M_{u,top}$ = -6.4 ft-kips
V_u = -25.6 kips $M_{u,bot}$ = -4.0 ft-kips

CHECK SECTION REQUIREMENTS (ACI 21.6.1)

P_u = 564.2 kips > $0.1A_g f_c'$ = 388.8 kips **[Satisfactory]**
c_{min} =MIN(c_1, c_2) = 36 in > 12 in **[Satisfactory]**
c_{min} / c_{max} = 1.00 > 0.4 **[Satisfactory]**

CHECK TRANSVERSE REINFORCING AT END OF COLUMN (ACI 21.6.4)

A_{sh} = 1.24 in^2 > MAX[$0.09sh_c f_c' / f_{yh}$, $0.3sh_c(A_g/A_{ch}-1)f_c' / f_{yh}$] = 0.73 in^2
[Satisfactory] where s = MAX[MIN(c_1/4, 6d_b, 4+(14-h_x)/3, 6), 4] = 5 in
h_c = c_1 - 2Cover - d_t = 32.4 in
A_{ch} = (c_1-3)(c_2-3) = 1089.0 in^2

CHECK FLEXURAL REINFORCING (ACI 21.6.3.1)

AT TOP SECTION ρ_{total} = 0.013 > ρ_{min} = 0.010 **[Satisfactory]**

(cont'd)

AT BOTTOM SECTION	ρ_{total} = 0.013	>	ρ_{min} =	0.010	**[Satisfactory]**
AT SPLICE SECTION	ρ_{total} = 0.026	<	ρ_{max} =	0.060	(0.030 should be used for lap splice existed.)
				[Satisfactory]	

CHECK CAPACITY SUBJECTED TO BENDING AND AXIAL LOAD

APPLIED LOADING (ACI 10.12.3)

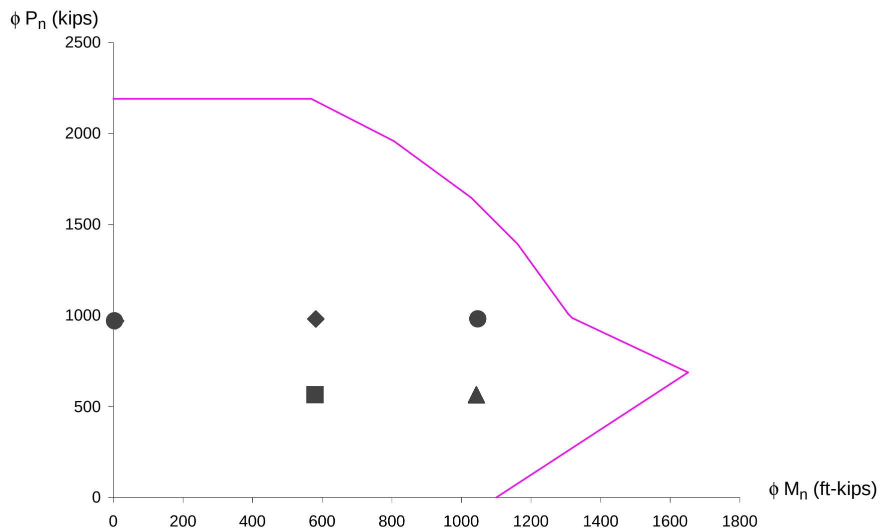
LOADS	1	2	3	4	5	6
P_u (kips)	980.6	980.6	564.2	564.2	969.6	969.6
M_u (ft-kips)	1028.8	571.4	1031.8	573.4	6.4	4.0
$\delta_{ns} = C_m/[1-P_u/(0.75P_c)]$	1.018	1.018	1.011	1.011	1.018	1.018
$\delta_{ns}M_u$ (ft-kips)	1047.8	581.9	1042.6	579.4	6.5	4.1
ϕM_n (ft-kips) @ P_u	1325.3	1566.8	1293.9	1564.8	1104.9	1103.0

where $EI = 0.4E_cI_g / (1+\beta_d) = 0.25\ E_cI_g$

$P_c = \pi^2 EI / (kL_u)^2$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES (ACI 10.2 & 10.3)

CAPACITY	**ϕP_n (kips)**	**ϕM_n (ft-kips)**
AT AXIAL LOAD ONLY	2191	0
AT MAXIMUM LOAD	2191	568
AT 0 % TENSION	1959	805
AT 25 % TENSION	1647	1028
AT 50 % TENSION	1392	1162
AT ε_t = 0.002	1008	1307
AT BALANCED CONDITION	987	1318
AT ε_t = 0.005	687	1651
AT FLEXURE ONLY	0	1100



All load points to be within capacity diagram. **[Satisfactory]**

CHECK SHEAR STRENGTH (ACI 21.6.5)

$V_e = MAX[\ (M_{pr,\ left,\ top} + M_{pr,\ right,bot}) / H_n\ ,\ V_{u,max}] =$ 246.7 kips

< $8\phi(f_c')^{0.5}c_2d$ = 394.9 kips **[Satisfactory]**

< $\phi[2(f_c')^{0.5}c_2d + A_vf_yd/s_{mid}\ ]$ = 409.1 kips **[Satisfactory]**

where $\rho_{top,left}$ = 0.004 > ρ_{min}=MIN[$3(f_c')^{0.5}/f_y$, $200/f_y$]= 0.003 **[Satisfactory]**

$\rho_{bot,right}$ = 0.004 > ρ_{min} = 0.003 **[Satisfactory]**

$M_{pr,\ left,\ top} = MIN\ [1.25M_{n,col,max}\ ,\ 0.5\ (M_{pr,top\ beam,\ left} + M_{pr,top\ beam,\ right})\ ] =$ 1254 ft-kips

$M_{pr,\ right,\ bot} = MIN\ [1.25M_{n,col,max}\ ,\ 0.5\ (M_{pr,bot\ beam,\ left} + M_{pr,bot\ beam,\ right})\ ] =$ 1254 ft-kips

ϕ = 0.75 (ACI 9.3.2.3)

A_v = 1.24 in^2

Seismic Design for Special Moment Resisting Frame Based on ACI 318-08

CHECK STRONG COLUMN - WEAK BEAM (ACI 21.6.2.2)

$\Sigma M_c = M_{n,top} @ P_{u,\,top} + M_{n,bot} @ P_{u,\,bot} =$ 4077.7 ft-kips

> $1.2\Sigma M_g = 1.2(M_{n,top} + M_{n,bot}) =$ 2486.5 ft-kips **[Satisfactory]**

where $M_{n,top} @ P_{u,\,top} =$ 2038.9 ft-kips

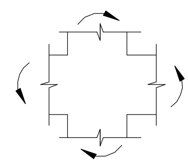
$M_{n,bot} @ P_{u,\,bot} =$ 2038.9 ft-kips

$M_{n,top} =$ 1300.5 ft-kips, (slab bars included, ACI 318-08)

$M_{n,bot} =$ 771.5 ft-kips

Note: For UBC 97, M_c & M_g shall be at the center of the joint with ϕ factors,

which means $\Sigma M_c > (0.9/0.7)1.2\Sigma M_g$.



CHECK JOINT CAPACITY (ACI 21.7)

$V_e = 1.25 f_y (A_{s,top} + A_{s,bot}) - (M_{pr,top} + M_{pr,bot})/H_n =$ 803.3 kips

where $A_{s,top} =$ 9.00 in^2

$A_{s,bot} =$ 5.00 in^2

$M_{pr,top} =$ 1563.6 ft-kips

$M_{pr,bot} =$ 945.2 ft-kips

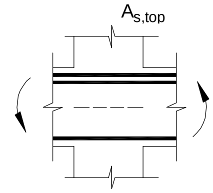
$H_n =$ 10.17 ft

$\phi V_n = k\, A_j\, (f_c')^{0.5} =$ 804.5 kips **>** V_e **[Satisfactory]**

where $A_j = c_1\, MIN(b + c_1\,,\, c_2) =$ 864 in^2

$k =$ 20 (20 for four faces, 15 for three faces, & 12 for others)

$\phi =$ 0.85 (ACI 9.3.4 c)



THEJOINT DESIGN IS ADEQUATE.

Technical References:

1. Alan Williams: "Seismic and Wind Forces, Structural Design Examples", International Code Council, 2003.
2. SEAOC: "2000 IBC Structural/Seismic Design Manual - Volume 3", International Code Council, 2003.
3. David A. Fanella: "Design of Concrete Buildings for Earthquake and Wind Forces", Portland Cement Association, 1998.

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Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 / IBC 09

INPUT DATA

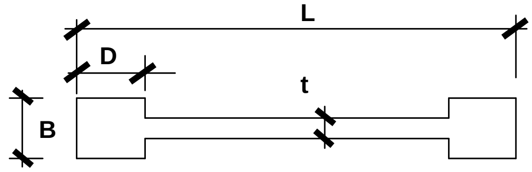
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
FACTORED AXIAL LOAD	P_u	=	8060	k
FACTORED MOMENT LOAD	M_u	=	87822	ft-k
FACTORED SHEAR LOAD	V_u	=	1404	k
LENGTH OF SHEAR WALL	L	=	28.83	ft
THICKNESS OF WALL	t	=	16	in
DEPTH AT FLANGE	D	=	34	in
WIDTH AT FLANGE	B	=	34	in
TOTAL WALL HEIGHT TO TOP	h_w	=	237.5	ft

REINF. BARS AT BULB		28	#	11		
WALL DIST. HORIZ. REINF.		2	#	7	@ 12	in. o.c.
WALL DIST. VERT. REINF.		2	#	7	@ 12	in. o.c.
HOOP REINF - WIDTH, B, DIR.	5	legs of	#	5		
HOOP REINF - LENGTH DIR.	22	legs of	#	5		
WALL EFFECTIVELY CONTINUOUS ?				Yes	(ACI 21.9.6.2 apply)	

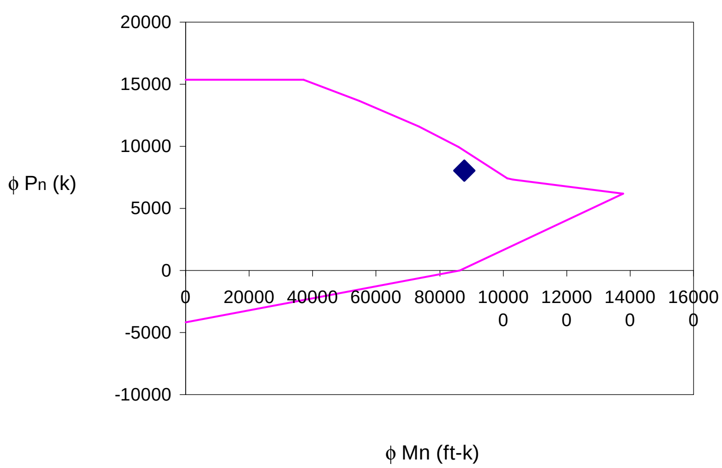
DESIGN SUMMARY

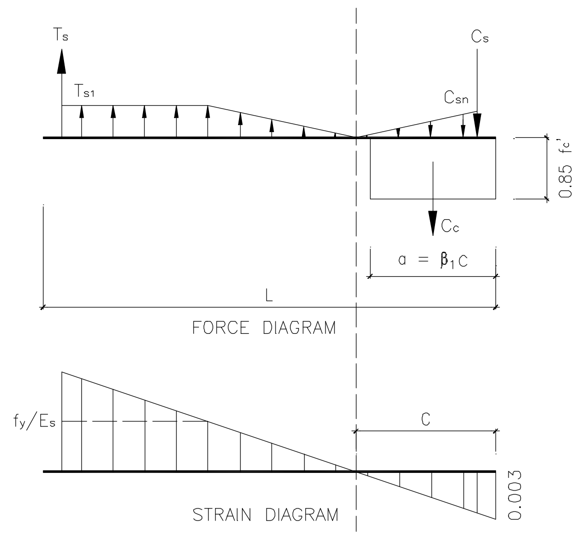
SHEAR WALL LENGTH	L	=	28.83	ft			
SHEAR WALL THICKNESS	t	=	16.00	in			
BULB END WIDTH	B	=	34.00	in			
BULB END DEPTH	D + Web	=	173.71	in			
BULB REINFORCING	28	#	11				
WALL HORIZ. REINF	2	#	7	@	12	in o.c.	
WALL VERT. REINF	2	#	7	@	12	in o.c.	
HOOP REINF - WIDTH, B, D	5	#	5	@	6	in o.c.	
HOOP REINF - LENGTH DII	22	#	5	@	6	in o.c.	

THE WALL DESIGN IS ADEQUATE.



ANALYSIS





CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 21.9.2.1 & 14.3)

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv}(f_c')^{0.5}$ = 350.09 kips < Vu , and bar size # 7 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}(f_c')^{0.5}$ = 350.09 kips < Vu , and bar size # 7 vertical]

$(\rho_t)_{provd.}$ = 0.0063 **>** $(\rho_t)_{min.}$ [Satisfactory]

$(\rho_l)_{provd.}$ = 0.0063 **>** $(\rho_l)_{min.}$ [Satisfactory]

where A_{cv} = 5535 in^2 (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall V_u **>** $2\,A_{cv}(f_c')^{0.5}$, two curtains reinforcement required. (ACI 318-08 21.9.2.2)

CHECK SHEAR CAPACITY (ACI 318-08 21.9.4.1 & 21.9.4.4)

ϕV_n =MIN [$\phi\, A_{cv}(\alpha_c(f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8\, A_{cv}(f_c')^{0.5}$] = 1665.56 kips **>** V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-08 9.3.4 a)

α_c = 2.0 (for h_w / L = 8.24 > 2)

ρ_l **>** ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-08 21.9.4.3)

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-08 21.9.5.1 & Eq.10-2)

$\phi P_{max} = 0.8\ \phi [\ 0.85\ f_c'\ (A_g - A_{st}) + f_y A_{st}]$ = 15375 kips. **>** P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-08 9.3.2.2)

A_g = 6759 in^2.

A_{st} = 116.36 in^2.

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 37085 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-08 10.3.2

AS ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 195 in $a = C_b\ \beta_1$ = 165 in β_1 = 0.85 (ACI 318-08 10.2.7.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-08 Fig. R9.3.2) $d = (L - 0.5D)$ = 329 in

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 7332 kips AND 103102 ft-kips.

IN ACCORDANCE WITH ACI 318-08 9.3.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$\phi M_n = 0.9\ M_n$ = 86262 kips.

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI SEC. 11.9.6, THE PURE AXIAL TENSION CAPACITY IS

$-\phi P_n = -\ 0.9\ \mathrm{MIN}(A_{st} F_y\ ,\ 3.3\ f_c'^{0.5}\ 4\ L\ t\ )$ = -4159 kips.

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

		ϕP_n (kips)	**ϕM_n (ft-kips)**
AT AXIAL LOAD ONLY	=	15375	0
AT MAXIMUM LOAD	=	15375	37085
AT 0 % TENSION	=	13650	54654
AT 25 % TENSION	=	11613	73516
AT 50 % TENSION	=	9949	85897
AT ε_t = 0.002	=	7428	101295
AT BALANCED CONDITION	=	7332	103102
AT ε_t = 0.005	=	6172	137743
AT FLEXURE ONLY	=	0	86262
AT TENSION ONLY	=	-4159	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ϕM_n = 97675 kips. **>** M_u [Satisfactory]

where $\phi = \mathrm{Min}\{0.9,\ \mathrm{Max}[0.65 + (\varepsilon_t - 0.002)(250/3)\ ,\ 0.65]\}$ = 0.650 (ACI 318-08 Fig. R9.3.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply   or $f_c < 0.2\ f_c'$ for ACI 21.9.6.3 apply   [Unsatisfactory]

where c = 208 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 20.0 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-08 21.9.6.2a.)

$f_c = (P_u / A) + (M_u\ y / I)$ = 2.978 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 173 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 7696 in^2. (area of transformed section.)

I = 94443188 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.038 **>** $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = MAX(c - 0.1L , 0.5 c , D + 12) 173.71 in. (ACI 318-08 21.9.6.4)

The maximum hoop spacing = MIN[B/3 , $6d_b$, 6 , $4+(14-h_x)/3$] = 6 in.o.c. (ACI 318-08 21.6.4.2 & 21.9.6.5a)

The required hoop reinforcement $A_{sh,\ B\ DIR} = (0.09\ s\ h_c\ f_c')\ /\ f_{yh}$ = 0.273 in^2. **<** # 5 provided [Satisfactory]

(ACI 318-08, Eq.21-4) $A_{sh,\ L\ DIR} = (0.09\ s\ h_c\ f_c')\ /\ f_{yh}$ = 0.294 in^2. **<** # 5 provided [Satisfactory]

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Ordinary Reinforced Concrete Shear Wall Design Based on ACI 318-08

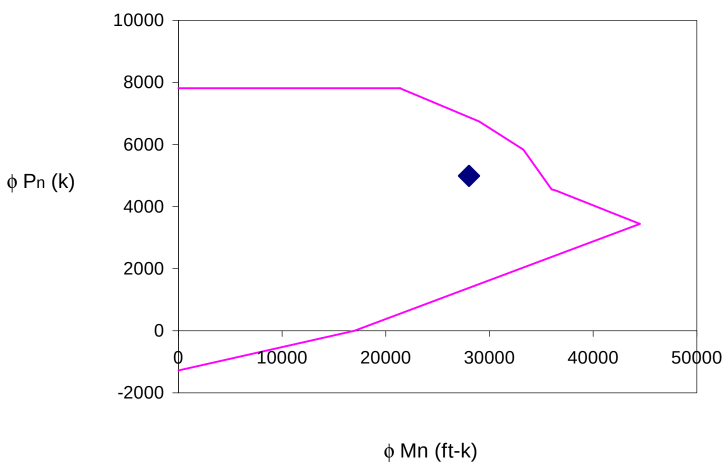
INPUT DATA & DESIGN SUMMARY

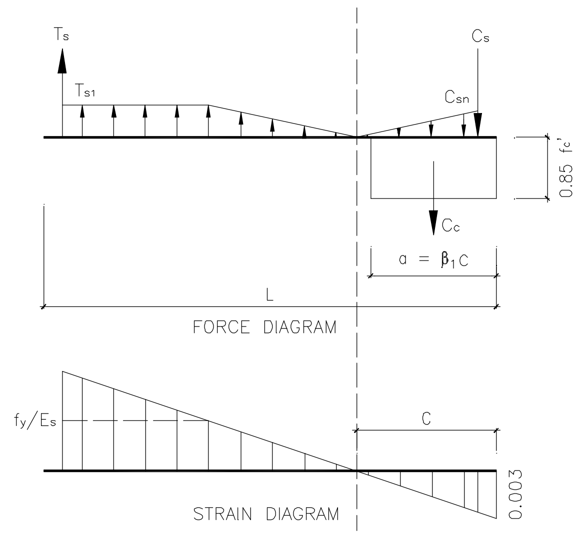
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi	FACTORED AXIAL LOAD	P_u	=	5000	k
REBAR YIELD STRESS	f_y	=	60	ksi	FACTORED MOMENT LOAD	M_u	=	28026	ft-k
					FACTORED SHEAR LOAD	V_u	=	1100	k
LENGTH OF SHEAR WALL	L	=	28	ft					
THICKNESS OF WALL	t	=	12	in					

THE WALL DESIGN IS ADEQUATE.

WALL DIST. HORIZ. REINF.	2	#	7	@	18	in. o.c.
WALL DIST. VERT. REINF.	2	#	7	@	18	in. o.c.
VERT. REINF. BARS AT END	2	#	7			

ANALYSIS





CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3)

$(\rho_t)_{provd.}$ = 0.0056 **>** $(\rho_t)_{min.}$ = 0.0025 [Satisfactory]

$(\rho_l)_{provd.}$ = 0.0056 **>** $(\rho_l)_{min.}$ = 0.0015 [Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

ϕV_n =MIN [$\phi A_{cv} (2 (f_c')^{0.5} + \rho_t f_y), \phi\, 8 A_{cv} (f_c')^{0.5}$] = 1112.4 kips **>** V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-08 9.3.4 a)

A_{cv} = 4032 in^2

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ϕM_n = 35406 kips. **>** M_u [Satisfactory]

where ϕ = Min{0.9, Max[0.65 + (ε_t - 0.002)(250/3) , 0.65]} = 0.650 (ACI 318-08 Fig. R9.3.2)

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Special Reinforced Concrete Shear Wall Design Based on ACI 318-05 / 2010 CBC Chapter A

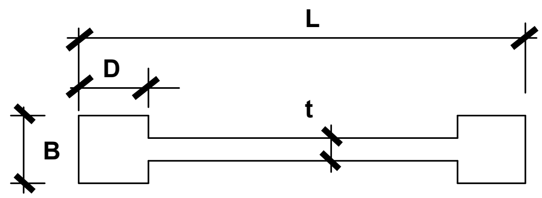
INPUT DATA

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
FACTORED AXIAL LOAD	P_u	=	8060	k		
FACTORED MOMENT LOAD	M_u	=	87822	ft-k		
FACTORED SHEAR LOAD	V_u	=	1404	k		
LENGTH OF SHEAR WALL	L	=	28.83	ft		
THICKNESS OF WALL	t	=	16	in		
DEPTH AT FLANGE	D	=	34	in		
WIDTH AT FLANGE	B	=	34	in		
TOTAL WALL HEIGHT TO TOP	h_w	=	237.5	ft		
REINF. BARS AT BULB	28	#	11			
WALL DIST. HORIZ. REINF.	2	#	7	@	12	in. o.c.
WALL DIST. VERT. REINF.	2	#	7	@	12	in. o.c.
HOOP REINF - WIDTH, B, DIR.	5	legs of #	5			
HOOP REINF - LENGTH DIR.	22	legs of #	5			
WALL EFFECTIVELY CONTINUOUS ?			Yes	(ACI 21.7.6.2 apply)		

DESIGN SUMMARY

SHEAR WALL LENGTH		L	=	28.83	ft		
SHEAR WALL THICKNESS		t	=	16.00	in		
BULB END WIDTH		B	=	34.00	in		
BULB END DEPTH		D + Web	=	173.71	in		
BULB REINFORCING	28	#	11				
WALL HORIZ. REINF	2	#	7	@	12	in o.c.	
WALL VERT. REINF	2	#	7	@	12	in o.c.	
HOOP REINF - WIDTH, B, D	5	#	5	@	6	in o.c.	
HOOP REINF - LENGTH DIR	22	#	5	@	6	in o.c.	

THE WALL DESIGN IS ADEQUATE.

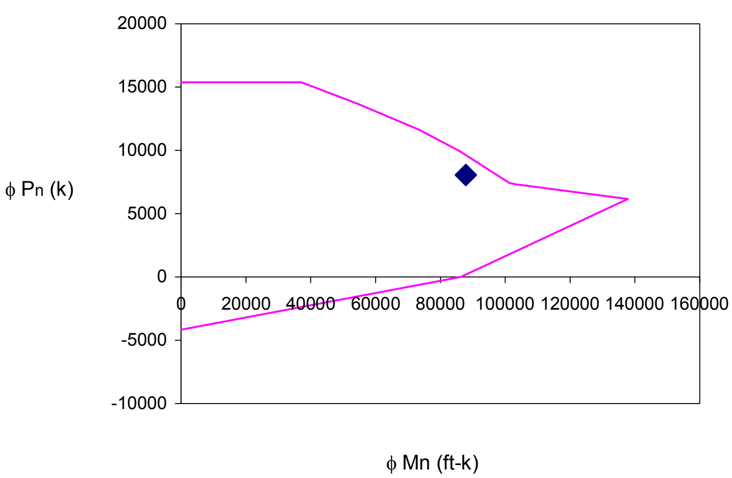


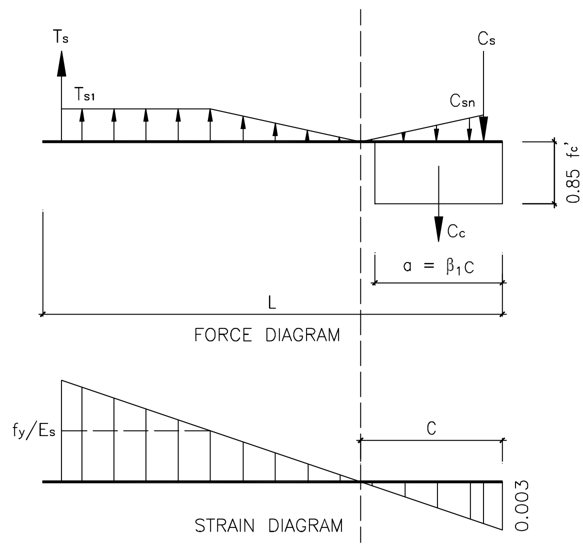
ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (CBC 10 1908A.1.25)

Pu = 8060 k **<** $0.35\ A_g\ f_c'$ = 9463 k [Satisfactory]

where A_g = 6759 in^2.





CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-05 21.7.2.1 & 14.3)

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv}\ (f_c')^{0.5}$ = 350.09 kips < Vu , and bar size # 7 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}\ (f_c')^{0.5}$ = 350.09 kips < Vu , and bar size # 7 vertical]

$(\rho_t)_{provd.}$ = 0.0063 **>** $(\rho_t)_{min.}$ [Satisfactory]

$(\rho_l)_{provd.}$ = 0.0063 **>** $(\rho_l)_{min.}$ [Satisfactory]

where A_{cv} = 5535 in^2 (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall V_u **>** $2\ A_{cv}\ (f_c')^{0.5}$, two curtains reinforcement required. (ACI 318-05 21.7.2.2)

CHECK SHEAR CAPACITY (ACI 318-05 21.7.4.1 & 21.7.4.4)

ϕV_n =MIN [$\phi\ A_{cv}\ (\alpha_c\ (f_c')^{0.5} + \rho_t f_y),\ \phi\ 8\ A_{cv}\ (f_c')^{0.5}$] = 1665.56 kips **>** V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-05 9.3.4 a)

α_c = 2.0 (for h_w / L = 8.24 > 2)

ρ_l **>** ρ_t [Satisfactory] (only for hw / L > 2.0, ACI 318-05 21.7.4.3)

(cont'd)

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-05 21.7.5.1 & Eq.10-2)

$\phi P_{max} = 0.8\ \phi\ [\ 0.85\ f_c'\ (A_g - A_{st}) + f_y\ A_{st}]$ = 15375 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-05 9.3.2.2)

A_{st} = 116.36 in^2.

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 37085 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-05 10.3.2

AS ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 195 in $a = C_b\ \beta_1$ = 165 in β_1 = 0.85 (ACI 318-05 10.2.7.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-05 Fig. R9.3.2) $d = (L - 0.5D)$ = 329 in

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 7332 kips AND 103102 ft-kips.

IN ACCORDANCE WITH ACI 318-05 9.3.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$\phi M_n = 0.9\ M_n$ = 86262 kips.

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI SEC. 11.10.6, THE PURE AXIAL TENSION CAPACITY IS

$-\phi P_n = -0.9\ \text{MIN}(A_{st}\ F_y\ ,\ 3.3\ f_c'^{0.5}\ 4\ L\ t\ )$ = -4159 kips.

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

		ϕP_n (kips)	**ϕM_n (ft-kips)**
AT AXIAL LOAD ONLY	=	15375	0
AT MAXIMUM LOAD	=	15375	37085
AT 0 % TENSION	=	13650	54654
AT 25 % TENSION	=	11613	73516
AT 50 % TENSION	=	9949	85897
AT ε_t = 0.002	=	7428	101295
AT BALANCED CONDITION	=	7332	103102
AT ε_t = 0.005	=	6172	137743
AT FLEXURE ONLY	=	0	86262
AT TENSION ONLY	=	-4159	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ϕM_n = 97675 kips. > M_u [Satisfactory]

where $\phi = \text{Min}\{0.9,\ \text{Max}[0.65 + (\varepsilon_t - 0.002)(250/3)\ ,\ 0.65]\}$ = 0.650 (ACI 318-05 Fig. R9.3.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.7.6.2, 21.7.6.3, and 21.7.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.7.6.2 apply or $f_c < 0.2\ f_c'$ for ACI 21.7.6.3 apply [Unsatisfactory]

where c = 208 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 20.0 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-05 21.7.6.2a.)

$f_c = (P_u / A) + (M_u\ y / I)$ = 2.978 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 173 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 7696 in^2. (area of transformed section.)

I = 94443188 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.038 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = MAX(c - 0.1L , 0.5 c , D + 12) 173.71 in. (ACI 318-05 21.7.6.4)

The maximum hoop spacing = MIN[B/4 , $6d_b$, 6 , $4+(14-h_x)/3$] = 6 in.o.c. (ACI 318-05 21.4.4.2 & 21.7.6.5a)

The required hoop reinforcement $A_{sh,\ B\ DIR} = (0.09\ s\ h_c\ f_c'\ ) / f_{yh}$ = 0.273 in^2. < # 5 provided [Satisfactory]

(ACI 318-05, Eq.21-4) $A_{sh,\ L\ DIR} = (0.09\ s\ h_c\ f_c'\ ) / f_{yh}$ = 0.294 in^2. < # 5 provided [Satisfactory]

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PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Verify Existing Reinforced Concrete Shear Wall Capacity Based on ASCE 41-06 / CBC 10 / IBC 09

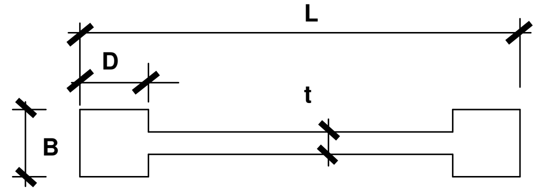
INPUT DATA

CONCRETE STRENGTH (ACI 318 5.1.1)		f_c'	=	4	ksi		
REBAR YIELD STRESS		f_y	=	60	ksi		
FACTORED AXIAL LOAD		P_u	=	8060	k		
FACTORED MOMENT LOAD		M_u	=	87822	ft-k		
FACTORED SHEAR LOAD		V_u	=	1404	k		
LENGTH OF SHEAR WALL		L	=	28.83	ft		
THICKNESS OF WALL		t	=	16	in		
DEPTH AT FLANGE		D	=	34	in		
WIDTH AT FLANGE		B	=	34	in		
TOTAL WALL HEIGHT TO TOP		h_w	=	237.5	ft		
REINF. BARS AT BULB		28	#	11			
WALL DIST. HORIZ. REINF.		2	#	7	@	12	in. o.c.
WALL DIST. VERT. REINF.		2	#	7	@	12	in. o.c.
HOOP REINF - WIDTH, B, DIR.	5	legs of	#	5			
HOOP REINF - LENGTH DIR.	22	legs of	#	5			
WALL EFFECTIVELY CONTINUOUS ?				Yes	(ACI 21.9.6.2 apply)		
MAX CONCRETE COMPRESSION STRAINS				0.002	(ASCE 6.3.3.1)		
MAX REINFORCEMENT TENSILE STRAINS				0.05	(ASCE 41-06 Sec. 6.3.3.1)		

OUTPUT SUMMARY

SHEAR WALL LENGTH	L	=	28.83	ft			
SHEAR WALL THICKNESS	t	=	16.00	in			
BULB END WIDTH	B	=	34.00	in			
BULB END DEPTH	D + Web	=	169.80	in			
BULB REINFORCING	28	#	11				
WALL HORIZ. REINF	2	#	7	@	12	in o.c.	
WALL VERT. REINF	2	#	7	@	12	in o.c.	
HOOP REINF - WIDTH, B, D	5	#	5	@	6	in o.c.	
HOOP REINF - LENGTH DIF	22	#	5	@	6	in o.c.	

THE WALL DESIGN IS ADEQUATE.

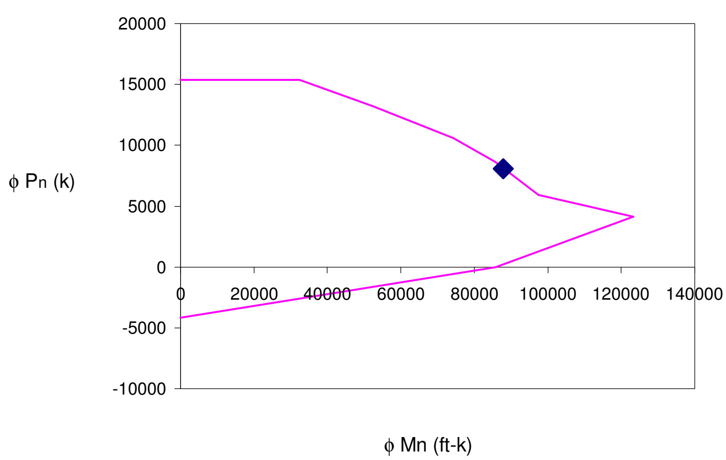


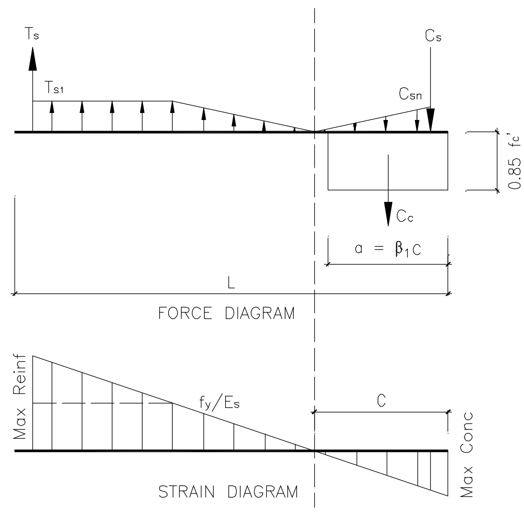
ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (CBC 2010 1908A.1.38)

Pu = 8060 k < $0.35\ A_g\ f_c'$ = 9463 k [Satisfactory]

where A_g = 6759 in^2.





CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 21.9.2.1 & 14.3)

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv}\ (f_c')^{0.5}$ = 350.09 kips < Vu , and bar size # 7 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}\ (f_c')^{0.5}$ = 350.09 kips < Vu , and bar size # 7 vertical]

$(\rho_t)_{provd.}$ = 0.0063 > $(\rho_t)_{min.}$ [Satisfactory]

$(\rho_l)_{provd.}$ = 0.0063 > $(\rho_l)_{min.}$ [Satisfactory]

where A_{cv} = 5535 in^2 (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall V_u > $2\ A_{cv}\ (f_c')^{0.5}$, two curtains reinforcement required. (ACI 318-08 21.9.2.2)

CHECK SHEAR CAPACITY (ACI 318-08 21.9.4.1 & 21.9.4.4)

ϕV_n =MIN [$\phi\ A_{cv}\ (\alpha_c\ (f_c')^{0.5} + \rho_t\ f_y)$, $\phi\ 8\ A_{cv}\ (f_c')^{0.5}$] = 1665.56 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-08 9.3.4 a)

α_c = 2.0 (for h_w / L = 8.24 > 2)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-08 21.9.4.3)

(cont'd)

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-08 21.9.5.1 & Eq.10-2)

$\phi P_{max} = 0.8\ \phi [\ 0.85 f_c' (A_g - A_{st}) + f_y A_{st}]$ = 15375 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-08 9.3.2.2)

A_{st} = 116.4 in^2.

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 32514 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-08 10.3.2 AS ε_c AND ε_t INPUT . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 162 in $a = C_b\ \beta_1$ = 137 in β_1 = 0.85 (ACI 318-08 10.2.7.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-08 Fig. R9.3.2) $d = (L - 0.5D)$ = 329 in

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 5807 kips AND 98948 ft-kips.

IN ACCORDANCE WITH ACI 318-08 9.3.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$\phi M_n = 0.9\ M_n$ = 85864 kips.

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI SEC. 11.10.6, THE PURE AXIAL TENSION CAPACITY IS

$-\phi P_n = -0.9\ \mathrm{MIN}(Ast\ Fy\ ,\ A_{st} E_s\ \varepsilon_s,\ 3.3\ f_c'^{0.5}\ 4\ L\ t)$ = -4159 kips.

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

		ϕP_n (kips)	**ϕM_n (ft-kips)**
AT AXIAL LOAD ONLY	=	15375	0
AT MAXIMUM LOAD	=	15375	32514
AT 0 % TENSION	=	13247	52220
AT 25 % TENSION	=	10605	74301
AT 50 % TENSION	=	8649	85560
AT $\varepsilon_t = 0.002$	=	5924	97415
AT BALANCED CONDITION	=	5807	98948
AT $\varepsilon_t = 0.005$	=	4121	123235
AT FLEXURE ONLY	=	0	85864
AT TENSION ONLY	=	-4159	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ϕM_n = 88382 kips. > M_u [Satisfactory]

where $\phi = \mathrm{Min}\{0.9,\ \mathrm{Max}[0.65 + (\varepsilon_t - 0.002)(250/3)\ ,\ 0.65]\}$ = 0.650 (ACI 318-08 Fig. R9.3.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2\ f_c'$ for ACI 21.96.3 apply [Unsatisfactory]

where c = 204 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 20.0 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-08 21.9.6.2a.)

$f_c = (P_u / A) + (M_u\ y / I)$ = 2.978 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 173 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 7696 in^2. (area of transformed section.)

I = 94443188 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.038 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = MAX(c - 0.1L , 0.5 c , D + 12) = 169.80 in. (ACI 318-08 21.9.6.4)

The maximum hoop spacing = MIN[B/3 , $6d_b$, 6 , $4+(14-h_x)/3$] = 6 in.o.c. (ACI 318-08 21.6.4.2 & 21.9.6.5a)

The required hoop reinforcement $A_{sh,\ B\ DIR} = (0.09\ s\ h_c\ f_c') / f_{yh}$ = 0.273 in^2. < # 5 provided [Satisfactory]

(ACI 318-08, Eq.21-4) $A_{sh,\ L\ DIR} = (0.09\ s\ h_c\ f_c') / f_{yh}$ = 0.287 in^2. < # 5 provided [Satisfactory]

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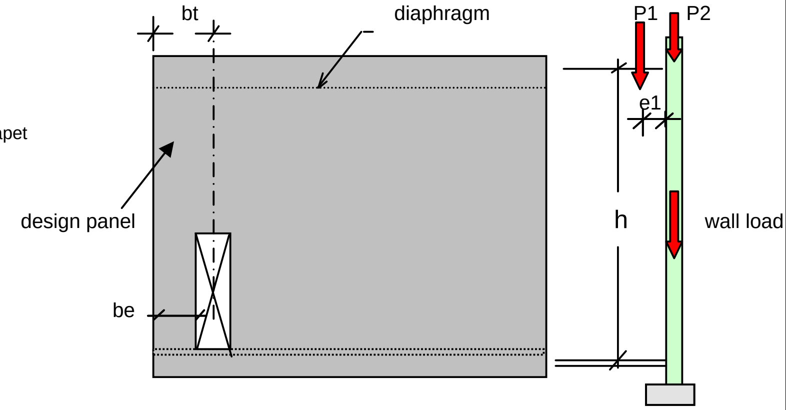
PROJECT :
CLIENT :
JOB NO. :
DATE :
PAGE :
DESIGN BY :
REVIEW BY :

Tilt-up Panel Design based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f 'c =	4	ksi	
REBAR STRENGTH	fy =	60	ksi	
LATERAL SERVICE LOAD	w_1 =	27	psf, for wall	
(ASD level, 0.7 E)	w_2 =	86	psf, for parapet	
LATERAL LOAD TYPE (0 or 1)		1	seismic	
LEDGER DL	$P1_{DL}$ =	0.8	k/ft	
LEDGER LL	$P1_{LL}$ =	0.8	k/ft	
DIST. FROM FACE	e1 =	6	in	
PARAPET DL	$P2_{DL}$ =	0.18	k/ft	
PARAPET LIVE LOAD	$P2_{LL}$ =	0	k/ft	
ELEMENT WIDTH	be =	1	ft	
TRIBUTARY WIDTH	bt =	1	ft	
WALL THICKNESS	t =	8	in	
WALL VERT. SPAN	h =	22	ft	
PARAPET HEIGHT	h_p =	3	ft	REVEAL THICKNESS = 0.75 in
VERT. REINF.- MIDDLE		1	LAYER # 7 @ 12 in, o.c.	
HORIZ. REINF. - MIDDLE		1	LAYER # 7 @ 12 in, o.c.	



ANALYSIS

CHECK VERTICAL FACTORED LOAD LESS THAN 0.06fc'Ag (ACI 318-08 Sec. 14.8.2.6)

$$P_u = (1.2P1_{DL} + 1.6P1_{LL} + 1.2P2_{DL} + 1.6P2_{LL} + 1.2w_{wt}0.5h)b_t$$ = 3.78 k < 0.06fc'Ag = 20.88 k

[SATISFACTORY]

CHECK VERTICAL REINFORCEMENT LESS THAN 0.6ρ_b (ACI 318-08 Sec. 14.8.2.3 & R9.3.2.2)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+0.005}$$ = 0.018 > ρ_{actual} = 0.014

[SATISFACTORY]

CHECK M_{cr} LESS THAN ϕM_n (ACI 318-08 Sec. 14.8.2.4)

$$\phi = MAX\left[0.9 - \frac{0.25P_u}{MIN(0.1f'_c A_g\ ,\ \phi P_b)}\ ,\ 0.65\right]$$ = 0.8736 (ACI 318-08 Sec. 9.3.2.2)

$$M_{cr} = \frac{7.5\sqrt{f'_c}I_g}{y_t} =$$ 49.87 k-in < $\phi M_n = \phi A_{se} f_y\left(d - \frac{a}{2}\right)$ = 117.19 k-in

where d = 3.63 in

(ACI 318-08 Sec. 9.5.2.3) **[SATISFACTORY]** $A_{se} = A_s + P_u\,t / (2\,f_y\,d)$ = 0.72 in^2

(ACI 318 R14.8.3)

CHECK WALL STRENGTH

$$I_{cr} = nA_{se}(d-c)^2 + \frac{b_e c^3}{3}$$ = 41 in^4 (ACI 318-08 Sec. 14.8.3)

$$M_{ua} = \frac{\gamma w_1 b_t h^2}{8} + P1_u\frac{e+0.5t}{2}$$ = 43.4 k-in, (γ = 1 / 0.7 for seismic)

$$M_u = \frac{M_{ua}}{1 - \frac{5P_u h^2}{(0.75)48E_c I_{cr}}}$$ = 55.5 k-in < ϕM_n = 117.19 k-in

(ACI 318-08 Sec. 14.8.3, Eq14-6) **[SATISFACTORY]**

CHECK SERVICE LOAD OUT-OF-PLANE DEFLECTION (ACI 318-08 Sec. 14.8.4

$$M_{sa} = \frac{w_1 b_t h^2}{8} + (P1_{DL} + 0.5P1_{LL})\frac{e+0.5t}{2}$$ = 25.6 k-in

$$\Delta_{cr} = \frac{5M_{cr}h^2}{48E_c I_g} =$$ 0.26 in $$\Delta_n = \frac{5M_n h^2}{48E_c I_{cr}} =$$ 6.65 in

$$\Delta_s = \begin{cases} \frac{2}{3}\Delta_{cr} + \frac{M_{sa} - 2/3M_{cr}}{M_n - 2/3M_{cr}}(\Delta_n - 2/3\Delta_{cr})\ ,\ for\ M_{sa} \ge 2/3M_{cr} \\ \left(\frac{M_{sa}}{M_{cr}}\right)\Delta_{cr}\ ,\ for\ M_{sa} < 2/3M_{cr} \end{cases}$$ = 0.14 in < $\frac{h}{150}$ = 1.76 in

[SATISFACTORY]

CHECK PARAPET STRENGTH

$$M_{u,parapet} = \frac{\gamma w_2 b_t h_p^2}{2}$$ = 7 k-in < $\phi M_n = 0.9A_{se}f_y\left(d - \frac{a}{2}\right)$ = 117 k-in **[SATISFACTORY]**

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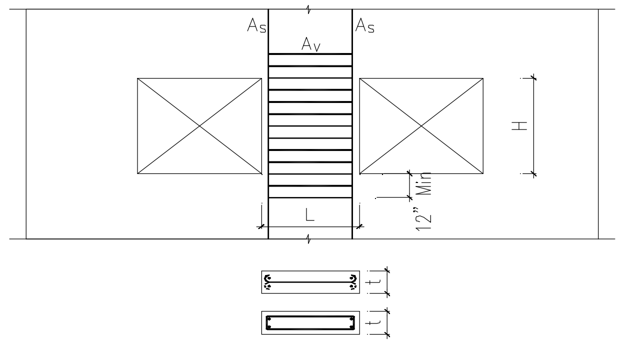
PROJECT :
CLIENT :
JOB NO. : DATE :

PAGE :
DESIGN BY :
REVIEW BY :

Wall Pier Design Based on CBC 2007 / IBC 2009

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH
f_c' = 3 ksi

REBAR YIELD STRESS
f_y = 60 ksi

WALL PIER LENGTH
L = 5 ft

WALL PIER HEIGHT
H = 12 ft

WALL PIER THICKNESS
t = 10 in

VERTICAL EDGE BARS, A_s
2 # 9

TRANSVERSE REINFORCEMENT, A_v
2 # 5 @ 6 in, o.c. (at each face.)

THE DESIGN IS ADEQUATE.

FACTORED AXIAL LOAD P_u = 60 kips

FACTORED SHEAR FORCE V_u = 35 kips, (in plane)

ANALYSIS

CHECK WALL PIER DEFINITION (CBC 2007 1908.1.3 / IBC 2009 1908.1.4)

L / t = 6.00 within [2.5 , 6] & H / L = 2.40 > 2
[Satisfactory]

CHECK SHEAR STRENGTH (CBC 2007 1908.1.8 / IBC 2009 1908.1.4 / ACI 318-08 21.6.5.1)

$V_e = (M_{pr,\ left,\ top} + M_{pr,\ right,bot})\ /\ H + V_u$ = 149.8 kips

< $8\phi(f_c')^{0.5}bd$ = 152.6 kips [Satisfactory]

< $\phi[V_c + A_v f_y d/s]$ = 216.0 kips [Satisfactory]

where d = 58.06 in, (ACI 318 3.3.2, 7.6.1, & 7.7.1)

ρ_{left} = 0.003 > ρ_{min}=MIN[$3(f_c')^{0.5}/f_y$, $200/f_y$]= 0.003 [Satisfactory]

ρ_{right} = 0.003 > ρ_{min} = 0.003 [Satisfactory]

$M_{pr,\ left,\ top} = \rho_{left}\ bd^2 f_y\ (1.25 - 0.919\rho_{left}\ f_y/f_c')$ = 689 ft-kips

$M_{pr,\ right\ ,bot} = \rho_{right}\ bd^2 f_y\ (1.25 - 0.919\rho_{right}\ f_y/f_c')$ = 689 ft-kips

ϕ = 0.6 (ACI 318 9.3.4)

A_v = 0.62 in^2

$V_c = 2(f_c')^{0.5}bd$ = 0.0 kips, (Per ACI 318-08 21.6.5.2, V_c = 0, if $(V_e - V_u) \geq 50\%\ V_e$ AND $P_u < A_g\ f_c'\ /\ 20$)

Concrete Slab Capacity Based on ACI 318-08

f'c = 4.5 ksi   t = 7.25 in
fy = 60 ksi c_C = 1 in

REBAR					A_s, in^2/ft	d, in	a, in	T, k/ft	ϕM_n, ft-k/ft	ϕV_n, k/ft
#	6	@	24	" O.C.	0.220	5.88	0.288	13.20	5.67	7.09
#	6	@	22	" O.C.	0.240	5.88	0.314	14.40	6.18	7.09
#	6	@	20	" O.C.	0.264	5.88	0.345	15.84	6.77	7.09
#	6	@	18	" O.C.	0.293	5.88	0.383	17.60	7.50	7.09
#	6	@	16	" O.C.	0.330	5.88	0.431	19.80	8.40	7.09
#	6	@	14	" O.C.	0.377	5.88	0.493	22.63	9.55	7.09
#	6	@	12	" O.C.	0.440	5.88	0.575	26.40	11.06	7.09
#	6	@	10	" O.C.	0.528	5.88	0.690	31.68	13.14	7.09
#	6	@	8	" O.C.	0.660	5.88	0.863	39.60	16.17	7.09
#	6	@	6	" O.C.	0.880	5.88	1.150	52.80	20.99	7.09
#	6	@	4	" O.C.	1.320	5.88	1.725	79.20	29.77	7.09
#	6	@	2	" O.C.	2.640	5.88	3.451	158.40	49.30	7.09
#	5	@	24	" O.C.	0.155	5.94	0.203	9.30	4.07	7.17
#	5	@	22	" O.C.	0.169	5.94	0.221	10.15	4.43	7.17
#	5	@	20	" O.C.	0.186	5.94	0.243	11.16	4.87	7.17
#	5	@	18	" O.C.	0.207	5.94	0.270	12.40	5.40	7.17
#	5	@	16	" O.C.	0.233	5.94	0.304	13.95	6.05	7.17
#	5	@	14	" O.C.	0.266	5.94	0.347	15.94	6.89	7.17
#	5	@	12	" O.C.	0.310	5.94	0.405	18.60	8.00	7.17
#	5	@	10	" O.C.	0.372	5.94	0.486	22.32	9.53	7.17
#	5	@	8	" O.C.	0.465	5.94	0.608	27.90	11.79	7.17
#	5	@	6	" O.C.	0.620	5.94	0.810	37.20	15.44	7.17
#	5	@	4	" O.C.	0.930	5.94	1.216	55.80	22.30	7.17
#	5	@	2	" O.C.	1.860	5.94	2.431	111.60	39.52	7.17

f'c = 5 ksi   t = 7.25 in
fy = 60 ksi c_C = 1.5 in

REBAR					A_s, in^2/ft	d, in	a, in	T, k/ft	ϕM_n, ft-k/ft	ϕV_n, k/ft
#	5	@	24	" O.C.	0.155	5.44	0.182	9.30	3.73	6.92
#	5	@	22	" O.C.	0.169	5.44	0.199	10.15	4.06	6.92
#	5	@	20	" O.C.	0.186	5.44	0.219	11.16	4.46	6.92
#	5	@	18	" O.C.	0.207	5.44	0.243	12.40	4.94	6.92
#	5	@	16	" O.C.	0.233	5.44	0.274	13.95	5.55	6.92
#	5	@	14	" O.C.	0.266	5.44	0.313	15.94	6.31	6.92
#	5	@	12	" O.C.	0.310	5.44	0.365	18.60	7.33	6.92
#	5	@	10	" O.C.	0.372	5.44	0.438	22.32	8.74	6.92
#	5	@	8	" O.C.	0.465	5.44	0.547	27.90	10.81	6.92
#	5	@	6	" O.C.	0.620	5.44	0.729	37.20	14.15	6.92
#	5	@	4	" O.C.	0.930	5.44	1.094	55.80	20.47	6.92
#	5	@	2	" O.C.	1.860	5.44	2.188	111.60	36.35	6.92
#	4	@	24	" O.C.	0.100	5.50	0.118	6.00	2.45	7.00
#	4	@	22	" O.C.	0.109	5.50	0.128	6.55	2.67	7.00
#	4	@	20	" O.C.	0.120	5.50	0.141	7.20	2.93	7.00
#	4	@	18	" O.C.	0.133	5.50	0.157	8.00	3.25	7.00
#	4	@	16	" O.C.	0.150	5.50	0.176	9.00	3.65	7.00
#	4	@	14	" O.C.	0.171	5.50	0.202	10.29	4.17	7.00
#	4	@	12	" O.C.	0.200	5.50	0.235	12.00	4.84	7.00
#	4	@	10	" O.C.	0.240	5.50	0.282	14.40	5.79	7.00
#	4	@	8	" O.C.	0.300	5.50	0.353	18.00	7.19	7.00
#	4	@	6	" O.C.	0.400	5.50	0.471	24.00	9.48	7.00
#	4	@	4	" O.C.	0.600	5.50	0.706	36.00	13.90	7.00
#	4	@	2	" O.C.	1.200	5.50	1.412	72.00	25.89	7.00

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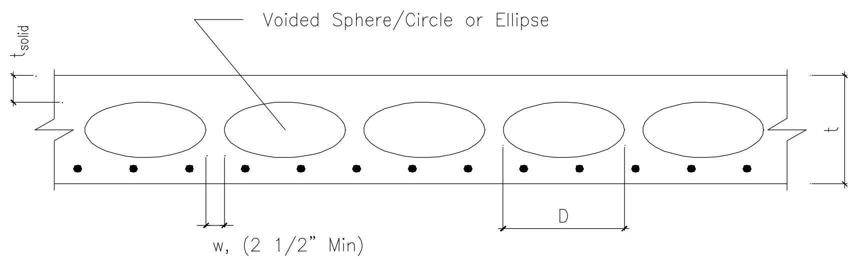
Voided Section Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH
f_c' = 4 ksi

REBAR YIELD STRESS
f_y = 60 ksi

TOTAL SLAB THICKNESS
t = 18 in

TOP & BOTTOM SOLID THICKNESS
t_{solid} = 4 in

VOIDED HORIZONTAL DIAMETER
D = 20 in, (input 10 for Sphere/Circle)

VERTICAL WEB THICKNESS
w = 2.5 in

ϕM_n = 32.7 ft-kips / ft
ϕV_n = 11.0 kips / ft

THE DESIGN IS ADEQUATE.

SECTION BARS # 6 @ 12 o.c.
with 0.75 in. bottom concrete cover

ANALYSIS

CHECK SECTION LIMITATIONS

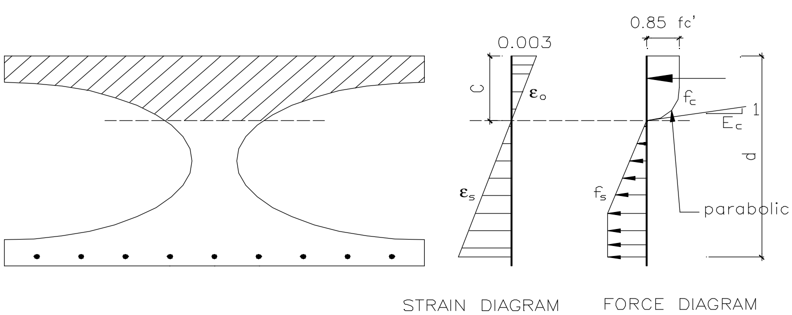
t_{solid} = 4 in > 0.75 (inside cover) + 1.50 (2 rebar thick) + 0.75 (top & bot cover) = 3.00 in, solid min thk [Satisfactory]

D = 20 in > 10 in, voided height [Satisfactory]

w = 2.5 in > 2.5 in [Satisfactory]

DETERMINE FLEXURE CAPACITY (ACI 318-08 7.12.2.1, 10.2, 10.5.1)

w_c = 150 pcf, (ACI 318-08 8.5.1) $E_c = w_c^{1.5}\, 33\, f'^{0.5}_c$ = 3834 ksi, (ACI 318-08 8.5.1)



$$\varepsilon_O = \frac{2\left(0.85 f_c'\right)}{E_C} \quad , \quad E_S = 29000 ksi$$

$$f_C = \begin{cases} 0.85 f_c' \left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2 \right] \quad , \quad for \quad 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f_c' \quad , \quad for \quad \varepsilon_C \ge \varepsilon_O \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S \quad , \quad for \quad \varepsilon_S \le \varepsilon_t \\ f_y \quad , \quad for \quad \varepsilon_S > \varepsilon_t \end{cases}$$

d = 16.88 in   c = 0.84 in

A_s = 0.44 in^2 / ft > A_{min} = 0.39 in^2 / ft [Satisfactory]

ϕM_n = 32.7 ft-kips / ft, (by pure math method) ϕ = 0.9 ,(ACI 318-08 Fig R9.3.2)

DETERMINE ONE WAY SHEAR CAPACITY (ACI 318-08 11.1.3.1, & 11.2)

$\phi V_n = \phi\, 2 A_c\, (f_c')^{0.5}$ = 11.0 kips / ft ϕ = 0.75 ,(ACI 318-08 9.3.2.3)

A_c = 116 in^2 / ft

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Concrete Diaphragm in-plane Shear Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH (ACI 318, 5.1.1)	f_c'	=	4	ksi
LIGHTWEIGHT CONCRETE ? (ACI 318, 8.6)			Yes	(Lightweight Concrete)
REBAR YIELD STRESS	f_y	=	60	ksi
THICKNESS OF DIAPHRAGM	t	=	3.5	in
DIAPHRAGM REINFORCING	1	#	4	@ 12 in. o.c. (at Middle, Each Way)
FACTORED IN-PLANE SHEAR LOAD	V_u	=	10	kips/ft

THE DIAPHRAGM DESIGN IS ADEQUATE.

ANALYSIS

CHECK MINIMUM REINFORCEMENT RATIOS (ACI 318, 21.11.7 & 7.12)

$(\rho_t)_{provd.}$ = 0.0048 **>** $(\rho_t)_{min.}$ = 0.0018 [Satisfactory]

CHECK SHEAR CAPACITY (ACI 318, 21.11.9)

ϕV_n =MIN [$\phi A_{cv} (2\lambda (f_c')^{0.5} + \rho_t f_y), \phi\, 8 A_{cv} (f_c')^{0.5}$] = 12.0 kips/ft **>** V_u [Satisfactory]

where ϕ = 0.75 (ACI 318, 9.3.2.3)

A_{cv} = 42 in^2

λ = 0.75 (ACI 318, 8.6.1)

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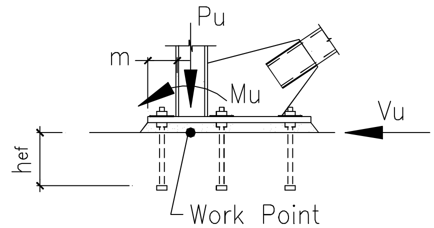
Base Plate and Group Anchors Design Based on ACI 318-08 & AISC 360-05

INPUT DATA & DESIGN SUMMARY



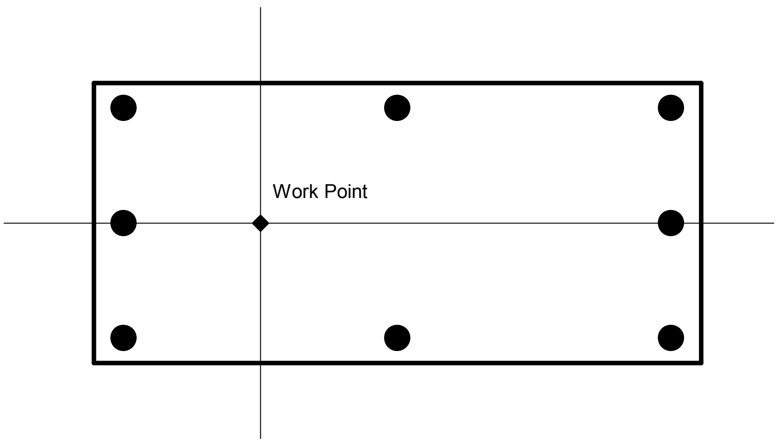
CONCRETE STRENGTH	f_c' =	4	ksi
SPECIFIED STRENGTH OF ANCHOR	f_{uta} =	58	ksi
BASE PLATE YIELD STRESS	f_y =	36	ksi
CRITICAL BASE PLATE CANTILEVER	m =	3.3	in
FACTORED AXIAL LOAD AT WORK POINT	P_u =	150	kips
FACTORED SHEAR LOAD AT WORK POINT	V_u =	16	kips, (0 if shear lug used.)
FACTORED MOMENT AT WORK POINT	M_u =	260	ft-kips
EFFECTIVE EMBEDMENT DEPTH	h_{ef} =	22	in
ANCHOR DIAMETER	d =	1	in
ANCHOR HEAD TYPE		4	Heavy Hex

(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)

BASE PLATE THICKNESS	t =	1.75	in
SEISMIC LOAD ? (ACI 318 D3.3)	== >	Yes	

ANCHOR LOCATION TO WORK POINT

Anchor	X (in)	Y (in)
1	-8	8
2	-8	-8
3	8	8
4	8	-8
5	24	8
6	24	-8
7	-8	0
8	24	0



BASE PLATE SIZE - 36 in x 20 in x 1.75 in (thk.)

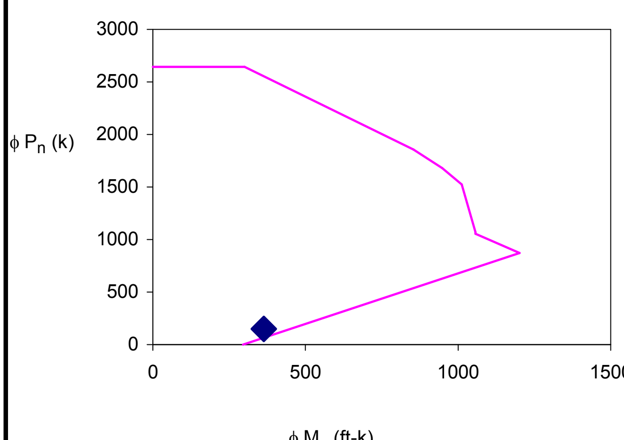
Corner	X (in)	Y (in)
1	-9.75	-9.75
2	-9.75	9.75
3	25.75	9.75
4	25.75	-9.75

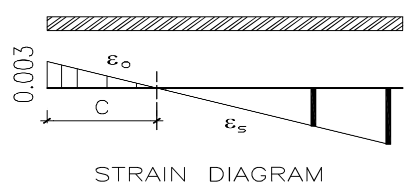
Edge = 1.75 in
(AISC 360-05, Tab J3.4 & J3.5)

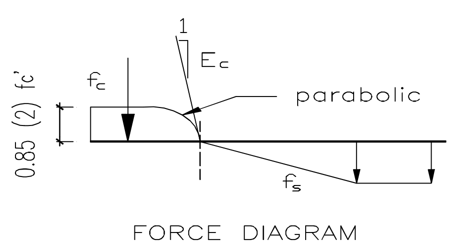
THE BASE PLATE & GROUP ANCHORS DESIGN IS ADEQUATE.

ANALYSIS

CHECK FLEXURAL & AXIAL COMPRESSION CAPACITY OF ANCHORED CONCRETE (ACI 318-08 Sec 9 & Sec 10)







(cont'd)

ϕM_n = 451 ft-kips @ P_u = 150 kips

> $M_{u,c}$ = 363 ft-kips to center of base plate [Satisfactory]

ϕP_{max} = 2643 kips

> P_u = 150 ft-kips [Satisfactory]

$N_{ua,1}$ = 3.13 kips

(The 1/0.4 times max tension, ACI 318 D3.3.6)

f_p = 4.42 ksi, the max possible concrete compression stress.

(ACI 318-08, 10.17.1 & 9.3.2.4, or AISC Guide - 1)

$$\varepsilon_O = \frac{2f'_C}{E_C} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right)\ ,\ E_C = 57\sqrt{f'_C}\ ,\ E_S = 29000ksi$$

$$f_C = \begin{cases} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f'_C \left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right]\ ,\ for\ \ 0 < \varepsilon_C < \varepsilon_O \\ 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f'_C\ ,\ for\ \ \varepsilon_C \ge \varepsilon_O \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S\ ,\ for\ \ \varepsilon_S \le \varepsilon_y \\ f_y\ ,\ for\ \ \varepsilon_S > \varepsilon_y \end{cases}$$

DETERMINE EACH ANCHOR FORCES WITHOUT CONCTERE COMPRESSION CAPCAITY CONSIDERED

Anchor	X (in)	Y (in)	X_{CG}^2	Y_{CG}^2	$N_{ua,2}$	V_{ua}
1	-8	8	256	64	1.25	2.00
2	-8	-8	256	64	1.25	2.00
3	8	8	0	64	-18.75	2.00
4	8	-8	0	64	-18.75	2.00
5	24	8	256	64	-38.75	2.00
6	24	-8	256	64	-38.75	2.00
7	-8	0	256	0	1.25	2.00
8	24	0	256	0	-38.75	2.00
Σ			1536	384	-150	16

X_{CG} = 8 in

Y_{CG} = 0 in

$P_{u,CG}$ = 150.0 kips

$V_{u,CG}$ = 16.0 kips

$M_{u,CG,X}$ = 160.0 ft-kips

$M_{u,CG,Y}$ = 0.0 ft-kips

CHECK GOVERNING ANCHOR TENSILE STRENGTH (ACI 318, D.5.1.2)

$\phi N_s = \phi A_{se}(f_{uta})$ = 19.77 kips > N_{ua} = Max($N_{ua,1}$, $N_{ua,2}$) = 3.13 kips [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625 , (ACI 318-08 D.4.4 & D.3.3.3)

A_{se} = 0.606 in^2

CHECK CONCRETE BREAKOUT STRENGTH OF GOVERNING ANCHOR (ACI 318, D.5.2.1)

$$\phi N_{cb} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{\left(9h_{ef}^2\right)} \psi_{ec,N} 1.0 \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5}\right)$$

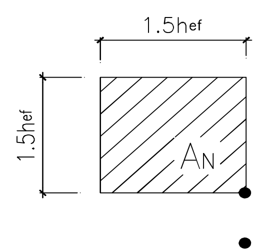
= 22.026 k > N_{ua} = 3.13 kips [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{ec,N}$ = 1.0, for no eccentricity in the connection.

$\psi_{c,N}$ = 1.0, for location where concrete cracking is likely to occur.

A_N = 1089 in^2, (conservative value)



CHECK CONCRETE BREAKOUT STRENGTH OF ALL ANCHORS (ACI 318, D.5.2.1)

$$\phi N_{cbg} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{\left(9h_{ef}^2\right)} \psi_{ec,N} 1.0 \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5}\right)$$

= 136.134 k > - P_u = -150 kips [Satisfactory]

where :	ϕ	=	0.75	x	0.75	=	0.5625		(cont'd)
	$\psi_{ec,N}$	=	0.84	, (ACI 318-08 Fig. RD.5.2.4)					
	A_N	=	98	x	82	=	8036	in^2	

CHECK PULLOUT STRENGTH OF GOVERNING ANCHOR (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi \psi_{cp,N}\left(A_b 8 f'_c\right)$$ = 27.018 k > N_{ua} = 3.13 kips [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

A_b = 1.501 in^2, (or determined from manufacture's catalogs.)

$\psi_{cp,N}$ = 1.0, for location where concrete cracking is likely to occur.

CHECK SIDE-FACE BLOWOUT STRENGTH (ACI 318, D.5.4.1)

cmin > 0.4 hef [Satisfactory]

Since this fastener is located far from a free edge of concrete (c > 0.4 hef) this type of failure mode is not applicable.

DETERMINE DESIGN TENSILE STRENGTH OF GOVERNING ANCHOR

$$\phi N_n = \min\left(\phi N_s \ , \phi N_{cb} \ , \phi N_{pn}\right)$$ = 19.771 K

CHECK GOVERNING ANCHOR SHEAR STRENGTH (ACI 318, D.6.1.2b & D.3.3.6)

$$\phi V_s = \phi 0.6 A_{se} f_{ut}$$ = 10.28 k > V_{ua} = 5.00 kips [Satisfactory]

where : ϕ = 0.65 x 0.75 = 0.4875

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D.6.1.3)

CHECK CONCRETE BREAKOUT STRENGTH OF GOVERNING ANCHOR FOR SHEAR LOAD (ACI 318, D.6.2.1b)

$$\phi V_{cb} = \phi \frac{A_V}{A_{Vo}} \psi_{ec,N} \psi_{cd,V} \psi_{c,V} V_b = \phi 1.0 \psi_{ec,V} 1.0 \psi_{c,V} \left(7\left(\frac{l}{d}\right)^{0.2} \sqrt{d} \sqrt{f'_c} \left(1.5 h_{ef}\right)^{1.5}\right)$$

= 71.555 k > V_{ua} = 5.00 kips [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{ec,V}$ = 1.0, for no eccentricity in the connection.

$\psi_{c,V}$ = 1.0, for location where concrete cracking is likely to occur.

l term is load bearing length of the anchor for shear, not to exceed 8d.

CHECK CONCRETE BREAKOUT STRENGTH OF ALL ANCHORS FOR SHEAR LOAD : (ACI 318, D.6.2.1b)

$$\phi V_{cbg} = \phi \frac{A_V}{A_{Vo}} \psi_{ec,N} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{A_V}{4.5\left(1.5 h_{ef}\right)^2} \psi_{ec,V} 1.0 \psi_{c,V} \left(7\left(\frac{l}{d}\right)^{0.2} \sqrt{d} \sqrt{f'_c} \left(1.5 h_{ef}\right)^{1.5}\right)$$

= 77.826 k > V_u [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

CHECK PRYOUT STRENGTH FOR SHEAR LOAD ON GOVERNING ANCHOR (ACI 318, D.6.3.1)

$$\phi V_{cp} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{\left(9 h_{ef}^2\right)} 1.0 \psi_{c,N} \left(24 \sqrt{f'_c} h_{ef}^{1.5}\right)$$

= 44.052 k > V_{ua} = 5.00 kips [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

k_{cp} = 2.0 for $h_{ef} > 2.5$ in.

CHECK PRYOUT STRENGTH FOR SHEAR LOAD ON ALL ANCHOR (ACI 318, D.6.3.1)

$$\phi V_{cpg} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{\left(9 h_{ef}^2\right)} 1.0 \psi_{c,N} \left(24 \sqrt{f'_c} h_{ef}^{1.5}\right)$$

= 325.072 k > V_u [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

DETERMINE DESIGN SHEAR STRENGTH OF GOVERNING ANCHOR

$$\phi V_n = \min\left(\phi V_s \ , \phi V_{cb} \ , \phi V_{cp}\right)$$ = 10.281 K

REQUIRED EDGE DISTANCES AND SPACING TO PRECLUDE SPLITTING FAILURE :

Since headed cast-in-place anchors are not like to be highly torqued, the minimum cover requirements of ACI 318 Sec. 7.7 apply.

$Cover_{Provd}$ > $Cover_{Reqd}$ [Satisfactory]

(cont'd)

CHECK TENSION AND SHEAR INTERACTION OF GOVERNING ANCHORS : (ACI 318, D.7)

Since $N_{ua,2}$ $0.2\,\phi\,N_n$ and

$V_{ua,2}$ $0.2\,\phi\,V_n$ the full tension design strength is permitted.

The interaction e uation may be used

$$\frac{N_{ua,2}}{\phi N_n}+\frac{V_{ua,2}}{\phi V_n}=$$ 0.26 1.2 [Satisfactory]

S ar o D en ona Pro er e o Anchor

Anchor Diameter (in)		Gross Area of Anchor (in^2)	Effective Area of Threaded Anchor (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				S uare	Heavy S uare	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

CHECK BASE PLATE THICKNESS (AISC Guide - 1, E . 3.3.14a)

$$t_{reqD}=1.5m\sqrt{\frac{f_p}{F_y}}=$$ 1.73 in 1.75 in

[Satisfactory]

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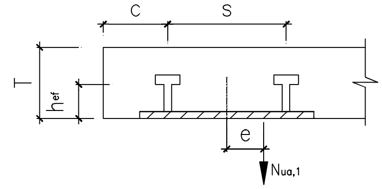
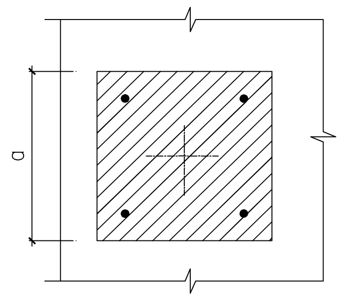
Group of Tension Fasteners Near an Edge with Eccentricity Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	4	ksi
SPECIFIED STRENGTH OF FASTENER	f_{uta}	=	60	ksi

(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)



FACTORED DESIGN LOAD	$N_{ua,1}$	=	11.7	k
EFFECTIVE EMBEDMENT DEPTH	h_{ef}	=	4.5	in
FASTENER DIAMETER	d	=	0.5	in
FASTENER HEAD TYPE			2	Heavy Square

(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)

ECCENTRICITY	e	=	2	in
FASTENER CENTER-TO-CENTER SPACI	s	=	6	in
DIST. FR. THE OUTER FASTENERS TO E	c	=	3	in
SEISMIC LOAD ? (ACI 318 D3.3)			No	

ANALYSIS

TOTAL NUMBER OF FASTENERS	n	=	4	
EFFECTIVE AREA OF FASTENER	A_{se}	=	0.142	in^2
BEARING AREA OF HEAD	A_b	=	0.569	in^2, (or determined from manufacture's catalogs.)

[THE FASTENER DESIGN IS ADEQUATE.]

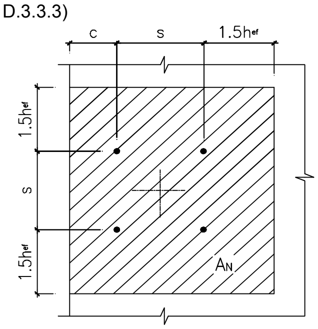
CHECK HIGHTEST TENSILE STRENGTH : (ACI 318, D.5.1.2)

$$\phi N_{s,1stud} = \phi A_{se}\left(f_{uta}\right) = 6.390 \text{ k} > N_{ua,\max,1stud} = \frac{N_u\left(s+2e\right)}{ns} = 4.875 \text{ k}$$

[Satisfactory]

where : ϕ = 0.75 x 1 = 0.75 , (ACI 318-08 D.4.4 & D.3.3.3)

CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318, D.5.2.1)



$$\phi N_{cbg} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{\left(9h_{ef}^2\right)} \left(\frac{1}{1+\frac{2e}{3h_{ef}}}\right) \left(0.7+\frac{0.3c}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f_c'}\, h_{ef}^{1.5}\right)$$

= 11.773 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK PULLOUT STRENGTH OF SINGLE STUD : (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi \psi_{cp,N}\left(A_b 8 f_c'\right) = 13.656 \text{ k} > N_{ua} = 4.875 \text{ k, (ACI 318, D.3.3.6)}$$

[Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur.

EVALUATE SIDE-FACE BLOWOUT : (ACI 318, D.5.4.1)

c > 0.4 h_{ef} [Satisfactory]

Since this fastener is located far from a free edge of concrete (c > 0.4 hef) this type of failure mode is not applicable.

REQUIRED EDGE DISTANCES AND SPACING TO PRECLUDE SPLITTING FAILURE :

Since a welded, headed fastener is not torqued, the minimum cover requirements of ACI 318 Sec. 7.7 apply.

$Cover_{Provd}$ > $Cover_{Reqd}$ [Satisfactory]

Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in^2)	Effective Area of Threaded Fastener (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:
1. Ronald Cook, "Strength Design of Anchorage to Concrete," PCA, 1999.

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Single Fastener in Tension and Shear Near an Edge Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

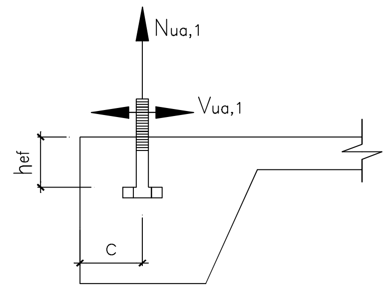
CONCRETE STRENGTH	f_c'	=	4	ksi
SPECIFIED STRENGTH OF FASTENER	f_{uta}	=	60	ksi

(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)

FACTORED DESIGN TENSION LOAD	$N_{ua,1}$	=	2.47	k
FACTORED DESIGN SHEAR LOAD	$V_{ua,1}$	=	1	k
EFFECTIVE EMBEDMENT DEPTH	h_{ef}	=	7	in
FASTENER DIAMETER	d	=	0.5	in
FASTENER HEAD TYPE			3	Hex

(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)

DIST. BETWEEN THE FASTENER AND EI	c	=	6	in
SEISMIC LOAD ? (ACI 318 D3.3)			Yes	



[THE FASTENER DESIGN IS ADEQUATE.]

ANALYSIS

EFFECTIVE AREA OF FASTENER A_{se} = 0.142 in^2

BEARING AREA OF HEAD A_b = 0.291 in^2, (or determined from manufacture's catalogs.)

CHECK FASTENER TENSILE STRENGTH (ACI 318, D.3.3.6):

$$\phi N_s = \phi n A_{se}\left(f_{uta}\right)$$ = 4.793 k = N_{ua} = 4.793 k [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625 , (ACI 318-08 D.4.4 & D.3.3.3)

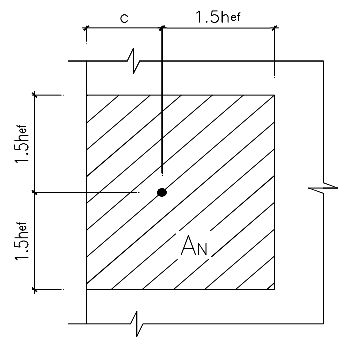
CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318, D.5.1.2)

$$\phi N_{cb} = \phi \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{\left(9h_{ef}^2\right)} \left(0.7 + \frac{0.3c}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f_c'}\, h_{ef}^{1.5}\right)$$

= 10.827 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.



CHECK PULLOUT STRENGTH : (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi \psi_{cp,N} \left(A_b 8 f_c'\right)$$ = 5.238 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK SIDE-FACE BLOWOUT STRENGTH : (ACI 318, D.5.4.1)

$$\phi N_{sb} = \phi\left(160c\sqrt{A_b}\sqrt{f_c'}\right)$$ = 18.423 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

DETERMINE DESIGN TENSILE STRENGTH :

$$\phi N_n = \min\left(\phi N_s \;, \phi N_{cb} \;, \phi N_{pn} \;, \phi N_{sb}\right)$$ = 4.793 K

CHECK FASTENER SHEAR STRENGTH : (ACI 318, D.6.1.2b & D.3.3.6)

$$\phi V_s = \phi n 0.6 A_{se} f_{ut}$$ = 2.492 k = V_{ua} = 2.492 k [Satisfactory]

where : ϕ = 0.65 x 0.75 = 0.4875

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D6.1.3)

CHECK CONCRETE BREAKOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.2.1a)

$$\phi V_{cb} = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} \left(7\left(\frac{l}{d}\right)^{0.2} \sqrt{d}\sqrt{f_c'}\, c^{1.5}\right)$$ = 3.923 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

A_V / A_{Vo} and $\psi_{cd,V}$ terms are 1.0 for single shear fastener not influenced by more than one free edge.

l term is load bearing length of the anchor for shear, not to exceed 8d.

(Cont'd)

CHECK PRYOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.3.1)

$$\phi V_{cp} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{\left(9h_{ef}^2\right)} \left(0.7 + \frac{0.3c}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f'_c}\, h_{ef}^{1.5}\right)$$

= 21.654 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

k_{cp} = 2.0 for $h_{ef} > 2.5$ in.

DETERMINE DESIGN SHEAR STRENGTH :

$$\phi V_n = \min\left(\phi V_s\ , \phi V_{cb}\ , \phi V_{cp}\right)$$ = 2.492 K

CHECK TENSION AND SHEAR INTERACTION : (ACI 318, D.7)

Since $N_{ua,1}$ > $0.2\,\phi\, N_n$ and

$V_{ua,1}$ > $0.2\,\phi\, V_n$ the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua,1}}{\phi N_n} + \frac{V_{ua,1}}{\phi V_n} =$$ 0.92 < 1.2 [Satisfactory]

Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in^2)	Effective Area of Threaded Fastener (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:

1. Ronald Cook, "Strength Design of Anchorage to Concrete," PCA, 1999.

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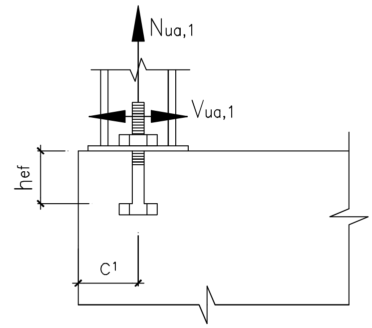
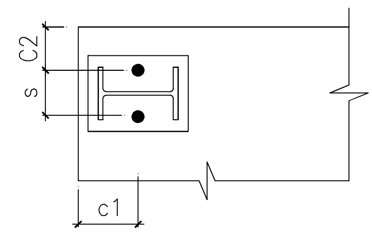
Group of Tension and Shear Fasteners Near Two Edges Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY





CONCRETE STRENGTH	f_c'	=	4	ksi
SPECIFIED STRENGTH OF FASTENER	f_{uta}	=	58	ksi

(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)

FACTORED DESIGN TENSION LOAD	$N_{ua,1}$	=	10	k
FACTORED DESIGN SHEAR LOAD	$V_{ua,1}$	=	3	k
EFFECTIVE EMBEDMENT DEPTH	h_{ef}	=	20	in
FASTENER DIAMETER	d	=	1	in
FASTENER HEAD TYPE			4	Heavy Hex

(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)

FASTENER CENTER-TO-CENTER SPACI	s	=	7	in
DIST. BETWEEN THE FASTENER AND EI	c1	=	14	in
DIST. BETWEEN THE FASTENER AND EI	c2	=	9	in
SEISMIC LOAD ? (ACI 318 D3.3)			Yes	

[THE FASTENER DESIGN IS ADEQUATE.]

ANALYSIS

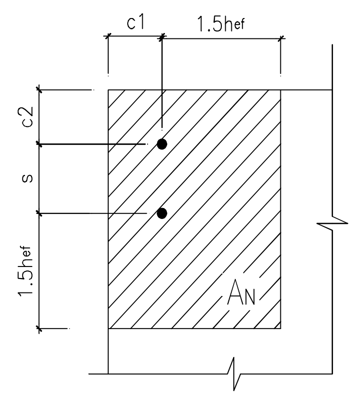
NUMBER OF FASTENERS	n	=	2	
EFFECTIVE AREA OF FASTENER	A_{se}	=	0.606	in^2
BEARING AREA OF HEAD	A_b	=	1.501	in^2, (or determined from manufacture's catalogs.)

CHECK THE FASTENERS TENSILE STRENGTH : (ACI 318, D.5.1.2 & D.3.3.6)

$$\phi N_s = \phi n A_{se}\left(f_{uta}\right) \quad = \quad 39.542 \text{ k} \quad > \quad N_{ua} = 25.000 \text{ k}$$ [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625 , (ACI 318-08 D.4.4 & D.3.3.3)

CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318, D.5.2.1)



$$\phi N_{cbg} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{\left(9h_{ef}^2\right)} \psi_{ec,N} \left(0.7 + \frac{0.3c_{\min}}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f_c'}\, h_{ef}^{1.5}\right)$$

= 33.919 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{ec,N}$ term is 1.0 for no eccentricity in the connection.

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK PULLOUT STRENGTH : (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi n \psi_{cp,N}\left(A_b 8 f_c'\right) \quad = \quad 54.036 \text{ k} \quad > \quad N_{ua}$$ [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK SIDE-FACE BLOWOUT STRENGTH : (ACI 318, D.5.4.1)

c_{min} > 0.4 h_{ef} [Satisfactory]

Since this fastener is located far from a free edge of concrete (c > 0.4 hef) this type of failure mode is not applicable.

DETERMINE DESIGN TENSILE STRENGTH :

$$\phi N_n = \min\left(\phi N_s \;, \phi N_{cb} \;, \phi N_{pn}\right) \quad = \quad 33.919 \text{ K}$$

CHECK Fasteners SHEAR STRENGTH : (ACI 318, D.6.1.2b & D.3.3.6)

$$\phi V_s = \phi n 0.6 A_{se} f_{ut} \quad = \quad 20.562 \text{ k} \quad > \quad V_{ua} = 7.500 \text{ k}$$ [Satisfactory]

where : ϕ = 0.65 x 0.75 = 0.4875

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D6.1.3)

CHECK CONCRETE BREAKOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.2.1b) (Cont'd)

$$\phi V_{cbg} = \phi \frac{A_V}{A_{Vo}} \psi_{ec,V} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{(1.5c1)(1.5c1 + s + c2)}{4.5c1^2} \psi_{ec,V} \left(0.7 + 0.3 \frac{c2}{1.5c1}\right) \psi_{c,V} \left(7\left(\frac{l}{d}\right)^{0.2} \sqrt{d} \sqrt{f'_c} c1^{1.5}\right)$$

= 14.433 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{cp,N}$ term is 1.0 for no eccentricity in the connection.

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

l term is load bearing length of the anchor for shear, not to exceed 8d.

CHECK PRYOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.3.1)

$$\phi V_{cpg} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{(9h_{ef}^2)} \left(0.7 + \frac{0.3c_{min}}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5}\right)$$

= 67.838 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

k_{cp} = 2.0 for $h_{ef} > 2.5$ in.

DETERMINE DESIGN SHEAR STRENGTH :

$$\phi V_n = \min\left(\phi V_s \ , \phi V_{cb} \ , \phi V_{cp}\right)$$ = 14.433 K

REQUIRED EDGE DISTANCES AND SPACING TO PRECLUDE SPLITTING FAILURE :

Since headed cast-in-place fasteners are not like to be highly torqued, the minimum cover requirements of ACI 318 Sec. 7.7 apply.

$Cover_{Provd}$ > $Cover_{Reqd}$ [Satisfactory]

CHECK TENSION AND SHEAR INTERACTION : (ACI 318, D.7)

Since $N_{ua,1}$ > $0.2\,\phi\,N_n$ and

$V_{ua,1}$ > $0.2\,\phi\,V_n$ the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua,1}}{\phi N_n} + \frac{V_{ua,1}}{\phi V_n} =$$ 0.50 < 1.2 [Satisfactory]

Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in^2)	Effective Area of Threaded Fastener (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:

1. Ronald Cook, "Strength Design of Anchorage to Concrete," PCA, 1999.

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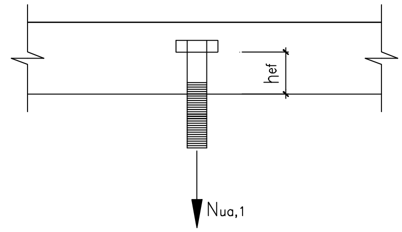
Single Tension Fastener Away from Edges Based on ACI 318-08



INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH f_c' = 4 ksi

SPECIFIED STRENGTH OF FASTENER f_{uta} = 60 ksi

(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)

FACTORED DESIGN LOAD $N_{ua,1}$ = 4 k

EFFECTIVE EMBEDMENT DEPTH h_{ef} = 5 in

FASTENER DIAMETER d = 0.5 in

FASTENER HEAD TYPE 2 Heavy Square

(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)

SEISMIC LOAD ? (ACI 318 D3.3) Yes

[THE FASTENER DESIGN IS ADEQUATE.]

ANALYSIS

EFFECTIVE AREA OF FASTENER A_{se} = 0.142 in^2

BEARING AREA OF HEAD A_b = 0.569 in^2

CHECK FASTENER TENSILE STRENGTH : (ACI 318, D.5.1.2 & D.3.3.6)

$$\phi N_s = \phi n A_{se}\left(f_{uta}\right)$$ = 4.793 k = N_{ua} = 4.793 k [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625 , (ACI 318-08 D.4.4 & D.3.3.3)

CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318, D.5.2.1)

$$\phi N_{cb} = \phi \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N}\left(24\sqrt{f_c'}\, h_{ef}^{1.5}\right)$$ = 9.546 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

A_N/A_{no} and $\psi_{ed,N}$ terms are 1.0 for single fasteners away form edges.

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur (i.e., bottom of the slab)

CHECK PULLOUT STRENGTH : (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi \psi_{cp,N}\left(A_b 8 f_c'\right)$$ = 10.242 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur.

EVALUATE SIDE-FACE BLOWOUT :

Since this fastener is located far from a free edge of concrete (c > 0.4 h_{ef}) this type of failure mode is not applicable.

REQUIRED EDGE DISTANCES AND SPACING TO PRECLUDE SPLITTING FAILURE :

Since this fastener is located far from a free edge of concrete (c>0.4h_{ef}) this type of failure mode is not applicable.

Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in^2)	Effective Area of Threaded Fastener (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:

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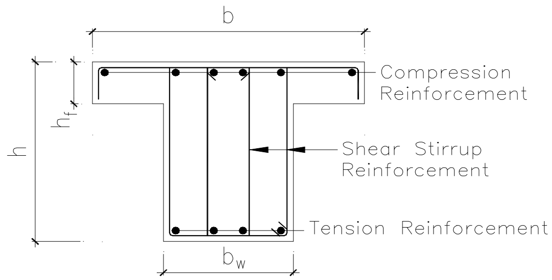
Concrete Beam Design, for New or Existing, Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH		f'_c =	6	ksi				
REBAR STRENGTH	MAIN	f_y =	60	ksi				
	STIRRUP	f_y =	60	ksi				
FACTORED BENDING MOMENT		M_u =	1000	ft-k				
FACTORED SHEAR FORCE		V_u =	230	kips				
FACTORED TORSIONAL MOMENT		T_u =	36.5	ft-k				
SECTION DIMENSIONS	b_w =	24	in					
	h =	48	in					
	h_f =	8	in					
	b =	88	in, (ACI 318-08 8.12.2, 11.5.1.1, & 13.2.4)					
COMPRESSION REINFORCEMENT		4	#	7				
TENSION REINFORCEMENT		6	#	9				
SHEAR REINFORCEMENT		4	legs #	4	@	12	in o.c.	



THE DESIGN IS ADEQUATE.

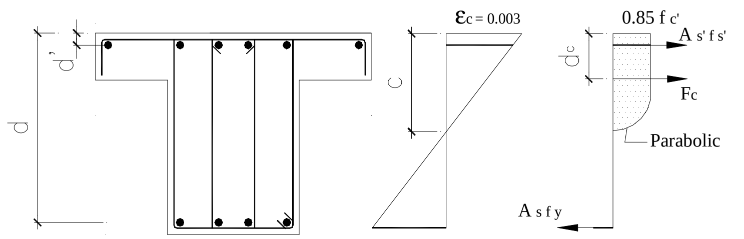
ANALYSIS

CHECK FLEXURAL CAPACITY

$$\varepsilon_o = \frac{2(0.85 f'_c)}{E_c}\ ,\ E_c = 57\sqrt{f'_c}\ ,\ E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] & ,\ for\ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c\ ,\ for\ \ \varepsilon_c \geq \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \leq \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$



Cover =	1.5	in, (ACI 318-08 7.7.1)	$\rho_{prov'd}$ =	0.0055	<	ρ_{max} =	0.0188	, (ACI 318 R10.3.5)
d =	45.44	in			>	ρ_{min} =	0.0039	, (ACI 318 10.5)
d' =	2.44	in			**[Satisfactory]**			
ϕ =	0.90	, (ACI 318-08 R9.3.2)	c =	3.99	in, by pure math method			
$\varepsilon_{c,max}$ =	0.0004		F_c =	346.99	kips			
$\varepsilon_{s,max}$ =	0.0050	, (ACI 318-08 10.3.4)	d_c =	1.38	in			

ϕM_n = 1193.23 ft-k > M_u **[Satisfactory]**

CHECK SHEAR CAPACITY

Check section limitation (ACI, 11.4.7.9)

$$V_u \leq 10\phi b_w d\sqrt{f'_c}$$

230.0 < 633.5 kips **[Satisfactory]**

where ϕ = 0.75

Determine concrete capacity (ACI, 11.2.1.1 or 11.2.2.1)

$$V_C = 2 b_w d\sqrt{f'_c} = \quad 168.93 \text{ kips}$$

$$V_C = (1.9A + 2500\rho_w B) b_w d = \quad 173.55 \text{ kips}$$,<== applicable

where $A = MIN\left(\sqrt{f'_c}\ ,\ 100\right) =$ 77.46

$B = MIN\left(\frac{V_u d}{M_u}\ ,\ 1.0\right) =$ 0.871

Check shear reinforcement (ACI, 11.4)

$$\left(\frac{A_V}{s}\right)_{ReqD} = \begin{cases} 0\ ,\ for\ \ V_u < \frac{\phi V_c}{2} \\ MAX\left(\frac{50 b_w}{f_y}\ ,\ \frac{0.75\sqrt{f'_c} b_w}{f_y}\right)\ ,\ for\ \ \frac{\phi V_c}{2} \leq V_u \leq \phi V_c \\ \frac{V_u - \phi V_c}{\phi d f_y}\ ,\ for\ \ \phi V_c \leq V_u \end{cases}$$

= 0.586 in^2 / ft < $\left(\frac{A_V}{s}\right)_{ProvD}$ = 0.800 in^2 / ft **[Satisfactory]**

Check spacing limits for shear reinforcement (ACI 11.4.5)

$$V_s = \frac{V_u - \phi V_c}{\phi} = \quad 0.00$$ kips, (ACI 11.1.1)

$$S_{max,shear} = \begin{cases} MIN(\frac{d}{2}\ ,\ 24)\ \ for\ \ V_s \leq 4 b_w d\sqrt{f'_c} \\ MIN(\frac{d}{4}\ ,\ 12)\ \ for\ \ V_s > 4 b_w d\sqrt{f'_c} \end{cases} = \quad 22$$

> S = 12 in

[Satisfactory]

(cont'd)

CHECK TORSION CAPACITY

Check section limitation (ACI, 11.5.3.1)

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$$

where ϕ = 0.75 (ACI, 9.3.2.3)

P_h = 258 in, (perimeter of centerline of outermost closed transverse torsional reinforcement.)

A_{oh} = 1,287 in^2 (area enclosed by centerline of the outermost closed transverse torsional reinforcement.)

0.215 < 0.581 **[Satisfactory]**

Check if torsional reinforcement required (ACI, 11.5.1)

$$T_u \le \phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

where b_e = MIN(h-h_f , 4h_f) = 32 in, (one side, ACI, 11.5.1.1)

P_{cp} = 160 in, (outside perimeter of the concrete cross section.)

A_{cp} = 1,216 in^2 (area enclosed by outside perimeter of concrete cross section.)

36.5 < 44.7 ft-k

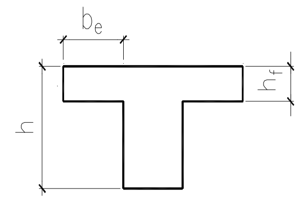
Torsional reinforcement NOT reqD.

Check the max factored torque causing cracking (ACI, 11.5.2.2)

$$T_u \le 4\phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

36.5 < 179.0

Reduction of the torsional moment can occur.



Determine the area of one leg of a closed stirrup (ACI, 11.5.3.6)

$$\frac{A_t}{s} = \frac{T_u}{2\phi A_0 f_{yv}} = \frac{T_u}{1.7\phi A_{0h} f_{yv}} = \quad 0.00 \quad in^2 / ft$$

Determine the corresponding area of longitudinal reinforcement (derived from ACI, 11.5.3.7 & 11.5.5.3)

$$A_L = MAX\left[\frac{A_t}{s} P_h \frac{f_{yv}}{f_{yL}} \ , \ \frac{5A_{cp}\sqrt{f'_c}}{f_{yL}} - P_h \frac{f_{yv}}{f_{yL}} MAX\left(\frac{A_t}{s} \ , \ \frac{25 b_w}{f_{yv}}\right)\right] = \quad 0.00 \quad in^2$$

Determine minimum combined area of longitudinal reinforcement

$A_{L,\ top} = A_s' + 0.5A_L$ = 0.00 in^2 < actual **[Satisfactory]**

$A_{L,\ bot} = A_s + 0.5A_L$ = 5.03 in^2 < actual **[Satisfactory]**

Determine minimum diameter for longitudinal reinforcement (ACI, 11.5.6.2)

d_{bL} = MAX(0.042 S, 3/8) = 0.50 in < 0.88 in **[Satisfactory]**

Determine minimum combined area of stirrups (ACI, 11.5.5.2 & 11.5.6.1)

(A_v+2A_t) / S = 0.40 in^2 / ft > MAX [$0.75(f_c')^{0.5} b_w/f_{yv}$, $50 b_w/f_{yv}$] = 0.24 in^2 / ft

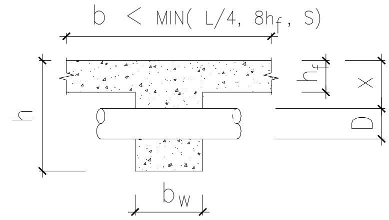
$S_{max,\ tor}$ = MIN[(P_h/8, 12) = 12 in **[Satisfactory]**

S_{reqD} = MIN($S_{max,shear}$, $S_{max,tor}$) = 12 in > actual **[Satisfactory]**

Design for Concrete Beam with Penetration Based on ACI 318-08

INPUT DATA

f'_c	=	5	ksi	M_u =	305.9	ft-k
Main Bar f_y	=	60	ksi	V_u =	40	k
Stirrup f_y	=	40	ksi	b =	30	in, (ACI 8.10.2)
				h =	26	in
Top bars 2	#	5		b_w =	20	in
Bot bars 5	#	7		h_f =	5	in
				x =	10	in
				D =	10	in



Stirrup size ==> # 3 @ 8 in o.c.
No. of legs = 2 d (optional) = in **The design is adequate.**

DESIGN SUMMARY

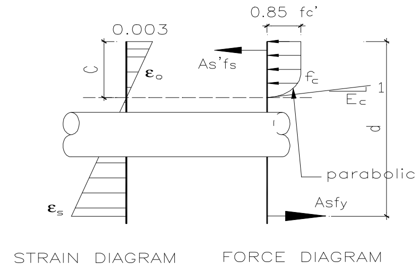
Main bar top (Compressive Reinf.) : **Use 2 # 5** Stirrups : **Use # 3 @ 8 in. o.c. (2 legs)**
Main bar bottom (Tensile Reinf.) : **Use 5 # 7 (1 layer)**

FLEXURAL ANALYSIS

$$\varepsilon_O = \frac{2\left(0.85 f'_c\right)}{E_C} \quad , \quad E_C = 57\sqrt{f'_C} \quad , \quad E_S = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2 \right] & , \quad for \quad 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f'_c & , \quad for \quad \varepsilon_C \ge \varepsilon_O \end{cases}$$

$$f_s = \begin{cases} \varepsilon_S E_S & , \quad for \quad \varepsilon_S \le \varepsilon_y \\ f_y & , \quad for \quad \varepsilon_S > \varepsilon_y \end{cases}$$



Check flexural capacity (ACI 318, 10)

A_s =	3.0	in^2		A'_s =	0.6	in^2
ρ_{min} =	0.0035		<	ρ_{proD} =	0.0063	
ρ_{max} =	0.0243		>	**[Satisfactory]**		
d =	23.7	in		β_1 =	0.80	
d' =	2.2	in		ϕ =	0.90	
ϕM_n =	306.0	ft-kips	>	M_u =	305.9	ft-kips
				[Satisfactory]		

SHEAR ANALYSIS

Check section limitation (ACI 11.4.6.8)

$$V_u \le 10\phi b_w (d - D)\sqrt{f'_c}$$

40.0 < 145.2 k **[Satisfactory]**
where ϕ = 0.75

Determine concrete capacity (ACI 11.2.1.1 or 11.2.2.1)

$$V_C = 2 b_w (d - D)\sqrt{f'_c} = \quad 38.71 \text{ k}$$

$$V_C = \left(1.9A + 2500\rho_w B\right) b_w (d - D) = \quad 41.17 \text{ k ,<== applicable}$$

where $A = MIN\left(\sqrt{f'_c} \ , \ 100\right) =$ 77.00

$B = MIN\left(\frac{V_u d}{M_u} \ , \ 1.0\right) =$ 0.258

Check shear reinforcement (ACI 11.4)

$$\left(\frac{A_V}{s}\right)_{ReqD} = \begin{cases} 0 & , \quad for \quad V_u < \frac{\phi V_c}{2} \\ \frac{50 b_w}{f_y} & , \quad for \quad \frac{\phi V_c}{2} \le V_u \le \phi V_c \\ \frac{V_u - \phi V_c}{\phi (d - D) f_y} & , \quad for \quad \phi V_c \le V_u \end{cases}$$

= 0.267 in^2 / ft < $\left(\frac{A_V}{s}\right)_{ProvD}$ = 0.330 in^2 / ft **[Satisfactory]**

Check spacing limits for shear reinforcement (ACI 11.4.4)

$$V_s = \frac{V_u - \phi V_C}{\phi} = \quad 12.17 \text{ k}$$

$$S_{max, shear} = \begin{cases} MIN(\frac{d}{2} \ , \ 24) & for \quad V_s \le 4 b_w (d - D)\sqrt{f'_c} \\ MIN(\frac{d}{4} \ , \ 12) & for \quad V_s > 4 b_w (d - D)\sqrt{f'_c} \end{cases} =$$

11 > S = 10 in
[Satisfactory] (penetration diameter control)

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Deep Beam Design Based on ACI 318-08

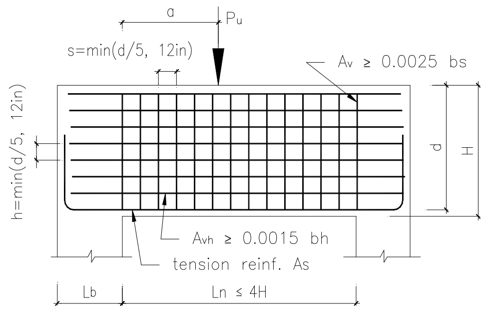
INPUT DATA

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
FACTORED TOP LOAD	P_u	=	145	k

(Input total load for non-point load condition.)

TOP LOAD LOCATION	a	=	5	ft
DEEP BEAM WIDTH	b	=	12	in
OVERALL BEAM DEPTH	H	=	42	in
CLEAR SPAN	L_n	=	12	ft
SUPPORT WIDTH	L_b	=	8	in

VERTICAL REINF.	2	#	4	@	6	in o.c.
HORIZONTAL REINF.	2	#	4	@	6	in o.c.
TENSION REINFORCEMENT	1	Layer	3	#	10	



THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

CHECK SECTION LIMITATION (ACI 318-08 11.7.3)

$V_u = P_u \, Max(a\,,\, L_n - a) / L_n =$ 84.5833 k

$V_u / \phi =$ 112.778 kips < $10\sqrt{f'_c}\,b_w d =$ 259 kips [Satisfactory]

where $d =$ 39.4 in

$\phi =$ 0.75 , (ACI 318-08 9.3.2.3)

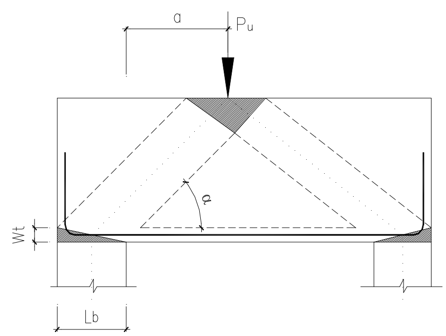
CHECK MINIMUM FLEXURAL REINFORCEMENT (ACI 318-08 10.5.1)

$A_s =$ 3.81 in^2 > $MIN\left(\frac{3b_w d\sqrt{f'_c}}{f_y}\,,\,\frac{200 b_w d}{f_y}\right) =$ 1.29 kips [Satisfactory]

CHECK ANGLE LIMITATION OF STRUT-AND TIE MODEL (ACI 318-08 A.2.5)

$\alpha_{left} =$ 33.27 ^{o} > 25 ^{o} [Satisfactory]

$\alpha_{right} =$ 25.51 ^{o} > 25 ^{o} [Satisfactory]



CHECK STRUT CAPACITY (ACI 318-08 A.2)

$R_{left} =$ 83.95 kips

$R_{right} =$ 61.05 kips

$F_{us,left} =$ 100.41 kips

$F_{us,right} =$ 67.65 kips

$\beta_s =$ 1.00 , (ACI 318-08 A.3.2.1)

$f_{ce} = 0.85\,\beta_s\,f_c' =$ 2.55 ksi $\qquad \phi =$ 0.75 , (ACI 318-08 9.3.2.6)

$A_{cs,left} =$ 52.67 in^2 $\qquad A_{cs,right} =$ 41.35 in^2

$\phi F_{ns,left} = \phi f_{ce} A_{cs,left} =$ 100.73 kips > $F_{us,left}$ [Satisfactory]

$\phi F_{ns,right} = \phi f_{ce} A_{cs,right} =$ 79.08 kips > $F_{us,right}$ [Satisfactory]

CHECK TIE CAPACITY (ACI 318-08 A.4)

$F_{ut,left} =$ 153.01 kips < $\phi F_{nt} = \phi f_y A_s =$ 171.45 kips [Satisfactory]

$F_{ut,right} =$ 141.74 kips < $\phi F_{nt} = \phi f_y A_s =$ 171.45 kips [Satisfactory]

CHECK NODAL ZONES CAPACITY (ACI 318-08 A.5)

$\beta_n =$ 0.80 , (ACI 318-08 A.3.5.2) $\qquad A_{nz} = b\,L_b =$ 96 in^2

$f_{ce} = 0.85\,\beta_n\,f_c' =$ 2.04 ksi $\qquad \phi F_{nn} = \phi f_{ce} A_{nz} =$ 146.88 kips

$F_{un,left} = R_{left} =$ 83.9 kips < ϕF_{nn} [Satisfactory]

$F_{un,right} = R_{right} =$ 61.1 kips < ϕF_{nn} [Satisfactory]

Purchaser's LOGO

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Non Deep Beam Design Based on ACI 318-11

DESIGN CRITERIA

This design method is, based on linear distribution of strain, for Non Deep Beam which cannot satisfy the requirements of ACI 318-11 10.7.1.

INPUT DATA & DESIGN SUMMARY

REBAR YIELD STRESS f_y = 60 ksi

CONCRETE STRENGTH f_c' = 3 ksi

OVERALL BEAM DEPTH H = 48 in

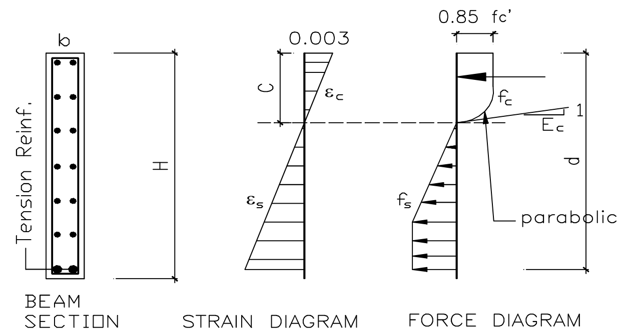
BEAM WIDTH b = 12 in

SD LEVEL SECTION LOADS

P_u = 280 kips, (horizontal axial force)

M_u = 300 ft-kips

V_u = 117 kips



VERTICAL REINF.

2 # 4 @ 6 in o.c.

HORIZONTAL REINF.

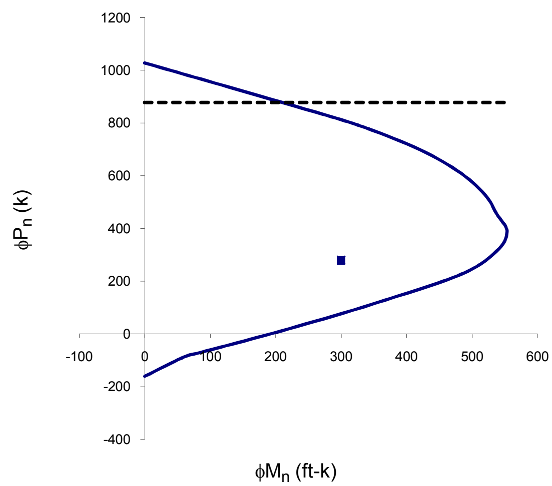
2 # 4 @ 6 in o.c.

TENSION REINFORCEMENT 1 Layer 3 # 6

THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY



ε_c = 0.003 , (ACI 318-11 10.2.3)

ϕ = 0.836 , (for Pu & Mu, ACI 318-11 9.3.2)

d = 46.9 in

c_b = 27.7 in, (balance point between Tension Controlled and Compression Controlled.)

P_u = 280 kips < ϕP_n = 879 kips, (ACI 318-11 10.3.6.1)

M_u = 300 ft-kips < ϕM_n = 517 ft-kips, at P_u level.

[Satisfactory]

CHECK SHEAR CAPACITY

V_u = 117 kips < $\phi V_n = \phi (V_s + V_c)$ = 118 kips, (ACI 318-11 11.1.1) [Satisfactory]

where ϕ = 0.75 (ACI 318-11 9.3.2.3)

V_c = $2 (f_c')^{0.5} A_0$ = 31.5 kips, (ACI 318-11 11.2.1)

V_s = MIN ($d f_y A_v / s$, $4V_c$) = 126.2 kips, (ACI 318-11 11.4.7.2)

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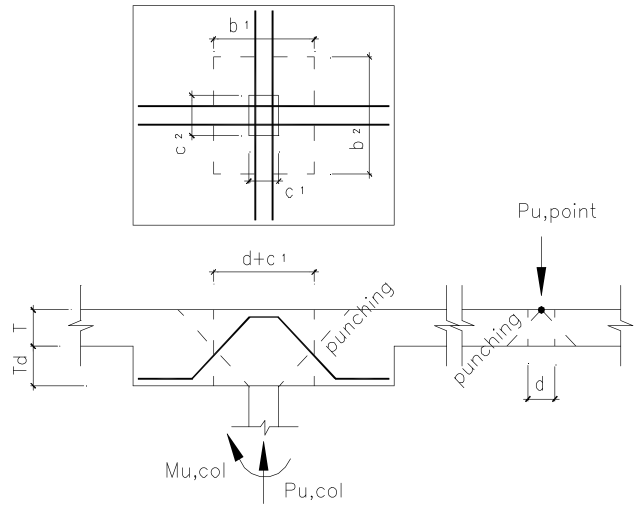
Slab Punching Design Based on ACI 318-08

INPUT DATA

COLUMN WIDTH	c_1	=	24	in
COLUMN DEPTH	c_2	=	24	in
SLAB CONCRETE STRENGTH	f_c'	=	4	ksi
SLAB THICKNESS	T	=	14	in
DROP CAP / PANEL THICKNESS	T_d	=	10	in
BAR SIZE AT TOP SLAB		#	10	
TRUSS BARS	2	#	6	each way
FACTORED AXIAL LOAD	$P_{u,\ col}$	=	400	kips
FACTORED BENDING LOAD	$M_{u,\ col}$	=	77.2	ft-kips
FACTORED POINT LOAD	$P_{u,\ point}$	=	50	kips



THE PUNCHING DESIGN IS ADEQUATE.

ANALYSIS

CHECK PUNCHING CAPACITY FOR COLUMN (ACI 318-08 SEC.11.4.7.4, 11.11.1.2, 11.11.6 & 13.5.3.2)

$$\phi v_c(psi) = \phi(2+y)\sqrt{f_c'} + \phi\left(A_v f_y \sin\alpha\right)/A_p = \quad 125 \text{ ksi} \quad > \quad v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} = \quad 114 \text{ ksi}$$

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3) **[Satisfactory]**

β_c = ratio of long side to short side of concentrated load = 1.00

d = T + T_c - 2 (0.5 d_b) - 2 " cover = 20.73 in

b_1 = (c_1 + d) = 44.73 in

b_2 = (c_2 + d) = 44.73 in

b_0 = $2b_1 + 2b_2 = 2(c_1 + d) + 2(c_2 + d)$ = 178.9 in

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = \quad 0.4$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1+\left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] = \quad 62.85 \text{ ft}^4$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right) = \quad 0.0$$

, (if truss bars used, y must be zero per ACI 11.11.3.1)

A_p = b_0 d = 3709 in^2

A_v = 3.52 in^2, total 4 sides

α = 45 ^{0}

f_y = 60 ksi

CHECK PUNCHING CAPACITY FOR POINT LOAD (ACI 318-08 SEC.11.11.1.2, & 11.11.6)

$$\phi P_{n,\ point} = 4\phi\sqrt{f_c'} A_p = \quad 68.6 \text{ kips} \quad > \quad P_{u,\ point} = \quad 50 \text{ kips}$$

[Satisfactory]

where ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)

d = T - 2" cover - 2 (0.5 d_b) = 10.73 in

b_0 = d π = 33.71 in

A_p = b_0 d = 362 in^2

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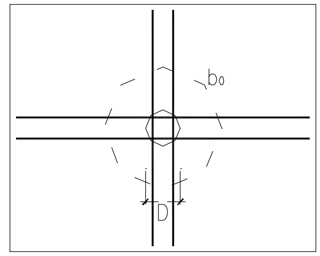
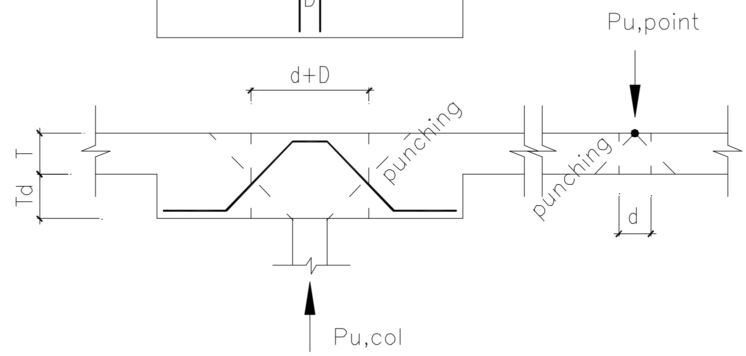
Slab Punching Design Based on ACI 318-08



INPUT DATA

COLUMN DIAMETER	D	=	22	in
SLAB CONCRETE STRENGTH	f_c'	=	4	ksi
SLAB THICKNESS	T	=	16	in
DROP CAP / PANEL THICKNESS	T_d	=	8	in
BAR SIZE AT TOP SLAB		#	10	
TRUSS BARS	3	#	6	each way
FACTORED AXIAL LOAD	$P_{u,\ col}$	=	400	kips
FACTORED POINT LOAD	$P_{u,\ point}$	=	80	kips



THE PUNCHING DESIGN IS ADEQUATE.

ANALYSIS

CHECK PUNCHING CAPACITY FOR COLUMN (ACI 318-08 SEC.11.4.7.4, 11.11.1.2, & 11.11.6)

$$\phi P_{n,col} = (2 \quad or \quad 4)\phi\sqrt{f'_c}A_p + \phi MIN\left(A_v f_y \sin\alpha \quad , \quad 3 b_0 d\sqrt{f'_c}\right) =$$ 432.0 kips, (if truss bars used, factor 2 apply per ACI 11.11.3.1)

> $P_{u,\ col}$ = 400 kips **[Satisfactory]**

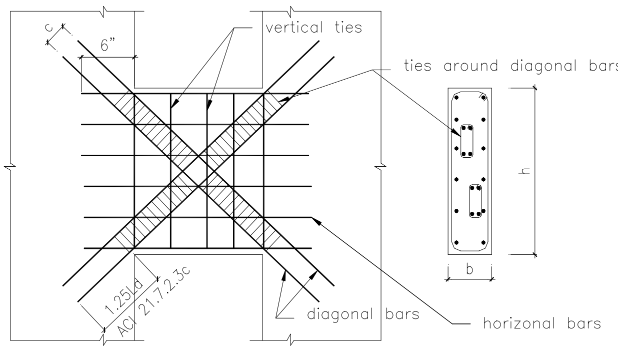
where
ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)
d = T + T_c - 2 (0.5 d_b) - 2 " cover = 20.73 in
b_0 = (D + d) π = 134.2 in
A_p = b_0 d = 2783 in^2
A_v = 5.28 in^2
α = 45 ^{0}
f_y = 60 ksi

CHECK PUNCHING CAPACITY FOR POINT LOAD (ACI 318-08 SEC.11.11.1.2, & 11.11.6)

$$\phi P_{n,\ point} = 4\phi\sqrt{f'_c}A_p =$$ 96.6 kips > $P_{u,\ point}$ = 80 kips **[Satisfactory]**

where
ϕ = 0.75 (ACI 318-08, Section 9.3.2.3)
d = T - 2" cover - 2 (0.5 d_b) = 12.73 in
b_0 = d π = 39.99 in
A_p = b_0 d = 509 in^2

Coupling Beam Design Based on ACI 318-08 / IBC 2009



INPUT DATA

CONCRETE STRENGTH		f_c'	=	4	ksi
REBAR YIELD STRESS		f_y	=	60	ksi
FACTORED SHEAR LOAD		V_u	=	239	k
WIDTH		b	=	16	in
OVERALL DEPTH		h	=	72	in
COUPLING BEAM DEPTH		c	=	16	in
CLEAR SPAN		L	=	6	ft
DIAGONAL BARS, A_d		4	#	9	
DIAGONAL TIES, A_{sh}	#	4	@	4	
HORIZONTAL BARS, A_{vh}		16	#	4	(2 layers @ 10 in o.c.)
VERTICAL TIES, A_v	2	legs #	3	@	6 in o.c.

ANALYSIS

CHECK DIAGONAL BARS REQUIREMENT (ACI 318-08, 21.9.7.2 & 21.9.7.3)

L / h = 1.00 < 4
< 2

V_u < $\phi V_n = 4bh\sqrt{f_c'} =$ 291 k [Coupling Beam Permitted]

CHECK DIAGONAL BARS (ACI 318-08, 21.9.7.4)

V_u < $\phi V_n = MIN(2\phi f_y A_d \sin\alpha \;,\; 10bh\sqrt{f_c'}) =$ 240 k [Satisfactory]

CHECK TIES AROUND DIAGONAL BARS (ACI 318-08, 21.4.4)

$$A_{sh} = MAX\left[\frac{0.3sh_c f_c'}{f_y}\left(\frac{A_g}{A_{ch}}-1\right) \;,\; \frac{0.09sh_c f_c'}{f_y}\right] =$$ 0.384 in² < $A_{sh,\,provd}$

[Satisfactory]

CHECK VERTICAL TIES (ACI 318-08, 11.7.5)

$A_v = 0.0015bs =$ 0.144 in² < $A_{v,\,provd}$

[Satisfactory]

CHECK HORIZONTAL BARS (ACI 318-08, 11.7.4)

$A_{vh} = 0.0025bh =$ 0.383 in² < $A_{v,\,provd}$

[Satisfactory]

DETERMINE LONGITUDINAL BARS (ACI 318-08, 10.5.1)

$$A_{s,\min} = MAX\left(\frac{200bd}{f_y} \;,\; \frac{3bd\sqrt{f_c'}}{f_y}\right) =$$ 3.07 in² ==> (4 # 8 longitudinal bars both top & bottom)

(Note: These bars are not recommended by SEAOC to be used, and are not shown in figure above.)

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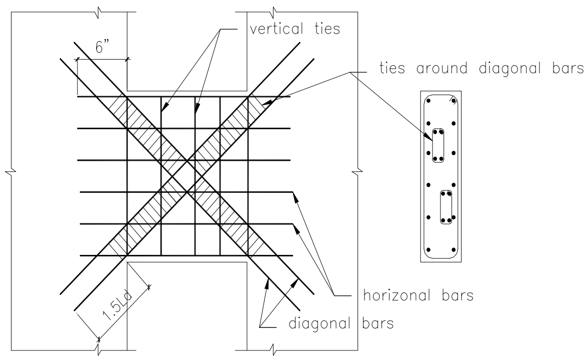
Coupling Beam Design Based on CBC 2001 / UBC 1997



INPUT DATA

CONCRETE STRENGTH		f_c'	=	4	ksi	
REBAR YIELD STRESS		f_y	=	60	ksi	
FACTORED SHEAR LOAD		V_u	=	254	k	
WIDTH		b	=	16	in	
OVERALL DEPTH		h	=	72	in	
CLEAR SPAN		L	=	6	ft	
DIAGONAL BARS, A_d		4	#	9		
DIAGONAL TIES, A_{sh}	#	4	@	4		
HORIZONTAL BARS, A_{vh}		16	#	4	(2 layers @ 10 in o.c.)	
VERTICAL TIES, A_v	2	legs #	3	@	6	in o.c.

ANALYSIS

CHECK DIAGONAL BARS REQUIREMENT (SEC.1921.6.10.2)

L / d = 1.02 **<** 4
where d = 70.25 in

V_u **<** $\phi V_n = 4bd\sqrt{f_c'} =$ 284 k [Coupling Beam Permitted]

CHECK DIAGONAL BARS (SEC.1921.6.10.2)

V_u **<** $\phi V_n = MIN(2\phi f_y A_d \sin\alpha \;,\; 10bd\sqrt{f_c'}) =$ 255 k [Satisfactory]

CHECK TIES AROUND DIAGONAL BARS (SEC. 1921.4.4)

$$A_{sh} = MAX\left[\frac{0.3sh_c f_c'}{f_y}\left(\frac{A_g}{A_{ch}}-1\right) \;,\; \frac{0.09sh_c f_c'}{f_y}\right] =$$ 0.384 in^2 **<** $A_{sh,\,provd}$

[Satisfactory]

CHECK VERTICAL TIES (SEC. 1911.8.9)

$A_v = 0.0015bs =$ 0.144 in^2 **<** $A_{v,\,provd}$

[Satisfactory]

CHECK HORIZONTAL BARS (SEC. 1911.8.10)

$A_{vh} = 0.0025bh =$ 0.383 in^2 **<** $A_{v,\,provd}$

[Satisfactory]

DETERMINE LONGITUDINAL BARS (SEC. 1910.5.1)

$$A_{s,\min} = MAX\left(\frac{200bd}{f_y} \;,\; \frac{3bd\sqrt{f_c'}}{f_y}\right) =$$ 3.07 in^2 ==> (4 # 8 longitudinal bars both top & bottom)

(Note: These bars are not recommended by SEAOC to be used, and are not shown in figure above.)

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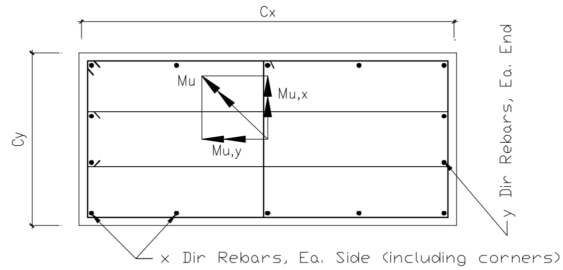
Concrete Column Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

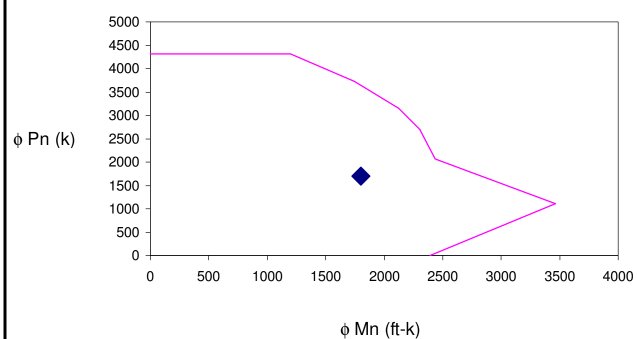
CONCRETE STRENGTH	f_c'	=	6	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SECTION SIZE	C_x	=	40	in
	C_y	=	36	in
FACTORED AXIAL LOAD	P_u	=	1700	k
FACTORED MAGNIFIED MOMENT	$M_{u,x}$	=	1800	ft-k
	$M_{u,y}$	=	100	ft-k
FACTORED SHEAR LOAD	$V_{u,x}$	=	130	k
	$V_{u,y}$	=	150	k

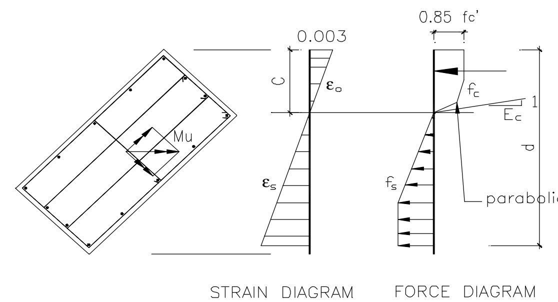
COLUMN VERT. REINFORCEMENT 8 # 8 at x dir. (Total 22 # 8)
3 # 8 at y dir.
LATERAL REINF. OPTION (0=Spirals, 1=Ties) 1 Ties
LATERAL REINFORCEMENT 4 legs,# 4 @ 12 in o.c., at x dir.
3 legs,# 4 @ 12 in o.c., at y dir.



THE COLUMN DESIGN IS ADEQUATE.

ANALYSIS





	φ Pn (k)	φ Mn (ft-k)
AT AXIAL LOAD ONLY	4315	0
AT MAXIMUM LOAD	4315	1196
AT 0 % TENSION	3724	1744
AT 25 % TENSION	3149	2121
AT 50 % TENSION	2700	2305
AT ε_t = 0.002	2062	2435
AT BALANCED CONDITION	2041	2461
AT ε_t = 0.005	1104	3463
AT FLEXURE ONLY	0	2390

$$\varepsilon_o = \frac{2(0.85 f'_c)}{E_c}\ ,\quad E_c = 57\sqrt{f'_c}\ ,\quad E_s = 29000ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] & , \ for \quad 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c & , \ for \quad \varepsilon_c \geq \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s & , \ for \quad \varepsilon_s \leq \varepsilon_t \\ f_y & , \ for \quad \varepsilon_s > \varepsilon_t \end{cases}$$

CHECK FLEXURAL & AXIAL CAPACITY

$\phi P_{max} = F\ \phi\ [0.85 f_c' (A_g - A_{st}) + f_y A_{st}]$ = 4315 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.2)

where ϕ = 0.65 (ACI 318-08, Sec.9.3.2.2) > Pu **[Satisfactory]**

F = 0.8 A_g = 1440 in^2 A_{st} = 17.38 in^2

ϕ = 0.75 + (ε_t - 0.002) (50), for Spiral ; 0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-08, Fig. R9.3.2)

where C_b = d ε_c / (ε_c + ε_s) = 22.2 in ε_t = 0.0021 ε_c = 0.003

d = 37.5 in, (ACI 7.7.1) D = 40.0 in Cover = 1.5 in, (ACI 318 7.7.1)

ϕM_n = 2826 ft-kips @ P_u = 1700 kips > M_u = 1803 ft-kips **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.012

ρ_{min} = 0.01 (ACI 318-08, Section 10.9) **[Satisfactory]**

CHECK SHEAR CAPACITY (ACI 318-08 Sec. 11.1.1, 11.2.1, & 11.4.6.2)

$\phi V_n = \phi (V_s + V_c)$ (ACI 318-08 Sec. 11.1.1)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3) f_y = 60 ksi

	d	A_0	A_v	$V_c = 2 (f_c')^{0.5} A_0$	V_s = MIN (d f_y A_v / s , 4V_c)	ϕV_n
x	37.5	1240	0.80	192.1	150.0	257
y	33.5	1240	0.60	192.1	100.5	219

s_{max} = 16 (ACI 318-08, Section 7.10.5.2) s_{provd} = 12 in

s_{min} = 1 **[Satisfactory]**

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Magnified Moment Calculation for Concrete Column Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

EFFECTIVE LENGTH FACTOR	k	=	1.6	, (ACI 10.10.6.3 or 10.10.7.1)
COLUMN UNSUPPORTED LENGTH	L_u	=	12	ft
LARGER FACTORED MOMENT	M_2	=	200	ft-k
SMALLER FACTORED END MOMENT	M_1	=	100	ft-k, (positive if single curvature.)
CONCRETE STRENGTH	f_c'	=	4	ksi
COLUMN DIMENSIONS	h	=	20	in
	b	=	20	in
FACTORED AXIAL LOAD	P_u	=	400	k
SUMMATION FOR ALL VERTICAL LOADS IN THE STORY	ΣP_u	=	1200	k
SUMMATION FOR ALL CRITICAL LOADS IN THE STORY	ΣP_c	=	13600	k, (ACI Eq. 10-21)

THE MAGNIFIED MOMENT: **M_u = 236.7 ft-k , Sway**

ANALYSIS

MAGNIFIED MOMENT - NONSWAY

$r = 0.3\,h =$ 6.0 in, ACI 10.11.2

$k\,L_u / r =$ 38.4 > $34 - 12(M_1 / M_2) =$ 28 < = = Slenderness effect must be considered. (ACI Eq 10-7)

$E_c = 57000\,(f_c')^{0.5} =$ 3605.0 ksi, ACI 8.5.1

$I_g = b\,h^3 / 12 =$ 13333 in^4

$$EI = \frac{0.4 E_c I_g}{1+\beta_d} = \frac{0.4 E_c I_g}{1+0.6} = 0.25 E_c I_g =$$ 1E+07 $k\text{-}in^2$, ACI 10.10.6.1

$$P_c = \frac{\pi^2 EI}{(k L_u)^2} =$$ 2234.2 k, ACI Eq (10-14)

$M_{2,min} = MAX[\,M_2\ ,\ P_u\,(0.6+0.03\,h)\,] =$ 200 ft-k, ACI 10.10.6.5

$C_m = MAX[\,0.6 + 0.4\,(M_1 / M_{2,\,min})\ ,\ 0.4\,] =$ 0.8 , ACI Eq(10-16)

$$\delta_{ns} = MAX\left(\frac{C_m}{1-\frac{P_u}{0.75 P_c}}\ ,\ 1.0\right) =$$ 1.05 , ACI Eq (10-12)

$M_{u,\,ns} = \delta_{ns}\,M_{2,\,min} =$ 210.2 ft-k, ACI Eq (10-11) > $1.05\,M_2 =$ 210.0 ft-k **[Unsatisfactory]** ,(ACI 10.10.5.1)

The column is sway. See calculation as follows.

MAGNIFIED MOMENT - SWAY

$k\,L_u / r =$ 38.4 > 22 < = = Slenderness effect must be considered. ACI Eq (10-6)

$$\delta_s = MIN\left[MAX\left(\frac{1}{1-\frac{\Sigma P_u}{0.75 \Sigma P_c}}\ ,\ 1.0\right)\ ,\ 2.5\right] =$$ 1.13 , ACI Eq (10-21)

$A_g = b\,h =$ 400 in^2

$L_u / r =$ 24.00 < $35 / [P_u / (f_c'\,A_g)]^{0.5} =$ 70.00 [Satisfactory]

$M_{2s} = M_2 =$ 200.0 ft-k, as given

$M_{2ns} =$ 5% $M_{2s} =$ 10.0 ft-k, assumed conservatively

$M_{u,\,s} = M_{2ns} + \delta_s\,M_{2s} =$ 236.7 ft-k, ACI Eq (10-19)

Note: For column subject to bending about both principal axis, the moment about each axis shall be magnified separately based on the conditions corresponding to that axis.

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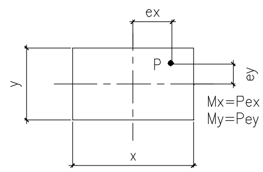
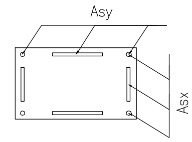
Rectangular Concrete Column Design

INPUT DATA

f'_c	=		4	ksi				
f_y	=		60	ksi				
x	=		48	in				
y	=		32	in				
Bar Size	==>	#	11					
No. of A_{sx}	=		8	**8 # 11**				
No. of A_{sy}	=		9	**9 # 11**				
Total Bars	==>	#	30 # 11					
ρ	=		3.0%					
P_u	=		1700	k				
M_{ux}	=		2900	ft-k	e_x	=	20.5	in
M_{uy}	=		1200	ft-k	e_y	=	8.5	in





CHECK COLUMN CAPACITY BY THE BRESLER METHOD

$$P_u \le (0.80)(0.70)P_o \qquad P_n \le \frac{1}{\frac{1}{P_{ox}}+\frac{1}{P_{oy}}-\frac{1}{P_o}} \qquad \frac{M_{nx}}{M_{nox}}+\frac{M_{ny}}{M_{noy}} \le 1.0$$

1700.0 < 4407.9 **ok** 2428.6 < 3145.4 **ok** 1.199 > 1.0 **NG**

ANALYSIS

ε_u = 0.003

ϕ = 0.70

β_1 = 0.85

		x DIRECTION	y DIRECTION
d	=	45.295 in	29.295 in
d'	=	2.705 in	2.705 in
A_s	=	12.48 in^2	14.04 in^2
b	=	32 in	48 in
M_n	=	4142.9 ft-k	1714.3 ft-k
M_{no}	=	5485.1 ft-k	3864.7 ft-k
P_o	=	3820.9 k	5457.1 k

MOMENT STRENGTH (Pn=0)

$$c=\frac{A_s f_y - A'_s\varepsilon_u E_s + \sqrt{\left(A_s f_y - A'_s\varepsilon_u E_s\right)^2 + 3.4\beta_1 f'_c b A'_s \varepsilon_u E_s d'}}{1.7\beta_1 f'_c b} =$$
4.10078 in | 3.70173 in

$$f'_s = E_s\varepsilon_u\left(\frac{c-d'}{c}\right)=$$
29612.2 psi | 23425.7 psi

$$M_n = 0.85 f'_c \beta_1 c b\left(d-\frac{\beta_1 c}{2}\right)+A'_s f'_s\left(d-d'\right)=$$
4034.01 ft-k | 3052.89 ft-k

AXIAL LOAD STRENGTH (Mn=0)

$$P_o = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y =$$
7871.28 k = 7871.28 k

BALANCED CONDITION

$$a_b = \frac{\varepsilon_u E_s \beta_1 d}{\varepsilon_u E_s + f_y} =$$
22.7862 in | 14.7372 in

$$f'_{sb} = \varepsilon_u E_s\left[1-\frac{d'}{d}\left(\frac{\varepsilon_u E_s + f_y}{\varepsilon_u E_s}\right)\right]=$$
60000 psi | 60000 psi

$$P_b = 0.85 f'_c b a_b + A'_s f'_{sb} - A_s f_y =$$
2479.13 k | 2405.11 k

$$M_b = 0.85 f'_c b a_b\left(\frac{h}{2}-\frac{a_b}{2}\right)+A'_s f'_{sb}\left(\frac{h}{2}-d'\right)-A_s f_y\left(\frac{d}{2}-h\right)=$$
5515.32 ft-k | 3881.41 ft-k

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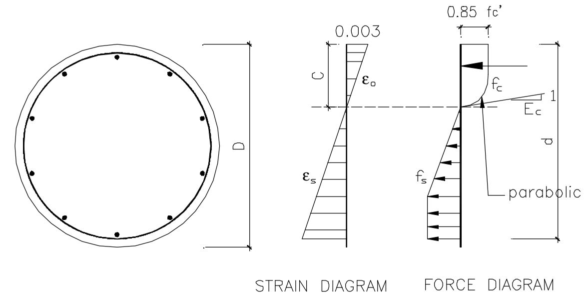
Circular Column Design Based on ACI 318-08

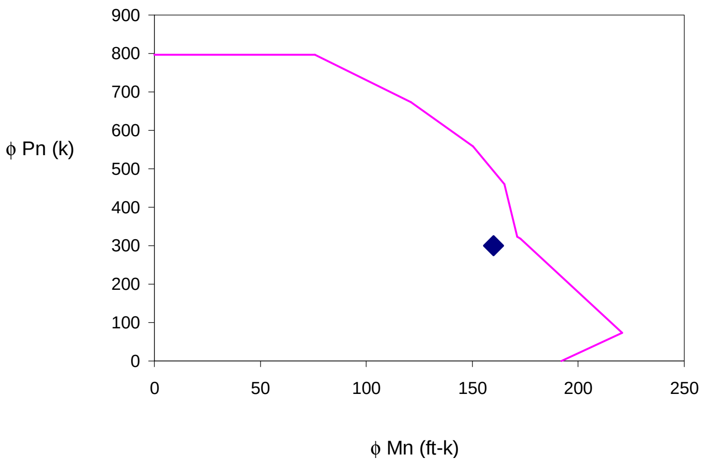
INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SECTION SIZE	D	=	20	in
FACTORED AXIAL LOAD	P_u	=	300	k
FACTORED MAGNIFIED MOMENT	M_u	=	160	ft-k
FACTORED SHEAR LOAD	V_u	=	20	k
COLUMN VERT. REINFORCEMENT	8	#	6	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT #	4	@	12	in o.c.



THE COLUMN DESIGN IS ADEQUATE.

ANALYSIS



$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c} \ , \ E_c = 57\sqrt{f'_c} \ , \ E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] \ , \ for \ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c \ , \ for \ \ \varepsilon_c \geq \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s \ , \ for \ \ \varepsilon_s \leq \varepsilon_t \\ f_y \ , \ for \ \ \varepsilon_s > \varepsilon_t \end{cases}$$

	ϕ Pn (k)	ϕ Mn (ft-k)
AT AXIAL LOAD ONLY	796	0
AT MAXIMUM LOAD	796	76
AT 0 % TENSION	673	121
AT 25 % TENSION	558	150
AT 50 % TENSION	460	165
AT ε_t = 0.002	323	171
AT BALANCED CONDITION	318	173
AT ε_t = 0.005	73	221
AT FLEXURE ONLY	0	192

CHECK FLEXURAL & AXIAL CAPACITY

ϕP_{max} =F ϕ [0.85 f_c' (A_g - A_{st}) + f_y A_{st}] = 796.34 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.2)

where ϕ = 0.65 (ACI 318-08, Sec.9.3.2.2) > Pu **[Satisfactory]**

F = 0.8 A_g = 314 in^2 A_{st} = 3.52 in^2

ϕ = 0.75 + (ε_t - 0.002) (50), for Spiral / 0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-08, Fig. R9.3.2)

where C_b = d ε_c / (ε_c + ε_s) = 10.4 in ε_t = 0.0021 ε_c = 0.003

d = 17.6 in, (ACI 7.7.1) D = 20.0 in Cover = 1.5 in, (ACI 318 7.7.1)

ϕM_n = 176 ft-kips @ P_u = 300 kips > M_u = 160 ft-kips **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.011

ρ_{min} = 0.01 (ACI 318-08, Section 10.9) **[Satisfactory]**

CHECK SHEAR CAPACITY (ACI 318-08 Sec. 11.1.1, 11.2.1, & 11.4.6.2)

$\phi V_n = \phi (V_s + V_c)$ (ACI 318-08 Sec. 11.1.1)

\> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3) f_y = 60 ksi

	d	A_0	A_v	V_c = 2 $(f_c)^{0.5} A_0$	V_s = MIN (d f_y A_v / s , 4V_c)	ϕ V_n
x	17.6	183	0.40	25.8	35.3	46

s_{max} = 12 (ACI 318-08, Section 7.10.5.2) s_{provd} = 12 in

s_{min} = 1 **[Satisfactory]**

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Magnified Moment Calculation for Circular Column Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

EFFECTIVE LENGTH FACTOR	k	=	1	, (ACI 10.10.6.3 or 10.10.7.1)
COLUMN UNSUPPORTED LENGTH	L_u	=	12	ft
LARGER FACTORED MOMENT	M_2	=	200	ft-k
SMALLER FACTORED END MOMENT	M_1	=	12	ft-k, (positive if single curvature.)
CONCRETE STRENGTH	f_c'	=	4	ksi
COLUMN DIAMETER	D	=	20	in
FACTORED AXIAL LOAD	P_u	=	400	k
SUMMATION FOR ALL VERTICAL LOADS IN THE STORY	ΣP_u	=	1200	k
SUMMATION FOR ALL CRITICAL LOADS IN THE STORY	ΣP_c	=	13600	k, (ACI Eq. 10-21)

THE MAGNIFIED MOMENT: **M_u = 200.0 ft-k , Nonsway**

ANALYSIS

MAGNIFIED MOMENT - NONSWAY

$r = 0.25\ D =$ 5.0 in, ACI 10.11.2

$k\ L_u / r =$ 28.8 **<** $34 - 12(M_1 / M_2) =$ 33.28 < = = Slenderness effect may be ignored. (ACI Eq 10-7)

$E_c = 57000\ (f_c')^{0.5} =$ 3605.0 ksi, ACI 8.5.1

$I_g = \pi\ D^4 / 64 =$ 7854 in^4

$$EI = \frac{0.4 E_c I_g}{1+\beta_d} = \frac{0.4 E_c I_g}{1+0.6} = 0.25 E_c I_g =$$ 7E+06 $k\text{-}in^2$, ACI 10.10.6.1

$$P_c = \frac{\pi^2 EI}{(k L_u)^2} =$$ 3369.1 k, ACI Eq (10-14)

$M_{2,min} = MAX[\ M_2\ ,\ P_u\ (0.6+0.03\ D)\ ] =$ 200 ft-k, ACI 10.10.6.5

$C_m = MAX[\ 0.6 + 0.4\ (M_1 / M_{2,\ min})\ ,\ 0.4\ ] =$ 0.624 , ACI Eq(10-16)

$$\delta_{ns} = MAX\left(\frac{C_m}{1-\frac{P_u}{0.75 P_c}}\ ,\ 1.0\right) =$$ 1.00 , ACI Eq (10-12)

$M_{u,\ ns} = \delta_{ns} M_{2,\ min} =$ 200.0 ft-k, ACI Eq (10-11) **<** $1.05\ M_2 =$ 210.0 ft-k **[Satisfactory]** ,(ACI 10.10.5.1)

The column is nonsway. Ignore following calculations.

MAGNIFIED MOMENT - SWAY < = = Not apply

$k\ L_u / r =$ 28.8 **>** 22 < = = Slenderness effect must be considered. ACI Eq (10-6)

$$\delta_s = MIN\left[MAX\left(\frac{1}{1-\frac{\Sigma P_u}{0.75 \Sigma P_c}}\ ,\ 1.0\right)\ ,\ 2.5\right] =$$ 1.13 , ACI Eq (10-21)

$A_g = \pi\ D^2 / 4 =$ 314 in^2

$L_u / r =$ 28.80 **<** $35 / [P_u / (f_c'\ A_g)]^{0.5} =$ 62.04 **[Satisfactory]**

$M_{2s} = M_2 =$ 200.0 ft-k, as given

$M_{2ns} =$ 5% $M_{2s} =$ 10.0 ft-k, assumed conservatively

$M_{u,\ s} = M_{2ns} + \delta_s M_{2s} =$ 236.7 ft-k, ACI Eq (10-19)

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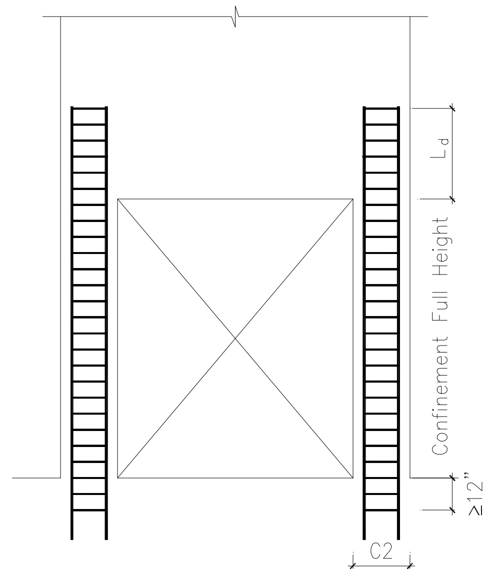
Design of Column Supporting Discontinuous System Based on ASCE 7-05 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	3	ksi
REBAR YIELD STRESS	f_y =	60	ksi
COLUMN CLEAR HEIGHT	h =	16	ft
COLUMN SIZE	c_1 =	24	in
	c_2 =	24	in
SEISMIC COEFFICIENT (Tab 12.2-1)	C_d =	5	
AMPLIFICATION FACTOR (Tab 12.2-1)	Ω_0 =	2.5	
DESIGN STORY DRIFT	δ_{xe} =	0.1	in



COLUMN LOADS, ASD (ft-kips, kips)

	P	M_{top}	V	M_{bot}
DL	40	0	0	0
LL	20	0	0	0
E/1.4	100	80	20	80

LONGITUDINAL REINFORCING

SECTION	TOP	BOTTOM
LEFT	4 # 8 (d = 21.50 in) (1 Layer)	4 # 8 (d = 21.50 in) (1 Layer)
RIGHT	4 # 8 (d = 21.50 in) (1 Layer)	4 # 8 (d = 21.50 in) (1 Layer)

THE COLUMN DESIGN IS ADEQUATE.

TRANSVERSE REINFORCEMENT FOR CONFINEMENT
3 Legs # 4 @ 5 in, o.c., full height (ACI 318-08 21.4.4.5)

ANALYSIS

DESIGN CRITERIA

1. since the column supported reaction from discontinued stiff member, ASCE 7-05 12.3.3.3 apply.
2. since the column is not part of the lateral force resisting system, ACI 318-08 21.13 apply.
3. since the column Ld required into top & 12" at least into footing per ACI 318-08 21.6.4.6, a fixed-fixed condition should be used.

DESIGN LOADS

$U_1 = 1.2\,D + f_1\,L + 1.0\,\Omega_0\,E_h$, (ACI 318-08 21.13)

P_u = 417.6 kips $M_{u,top}$ = 280.0 ft-kips
V_u = 70.0 kips $M_{u,bot}$ = 280.0 ft-kips
f_1 = 0.5 S_{DS} = 1.2

$U_2 = (0.9\pm0.2S_{DS})\,D \pm 1.0\,\Omega_0\,E_h$

P_u = 395.6 kips $M_{u,top}$ = 280.0 ft-kips
V_u = 70.0 kips $M_{u,bot}$ = 280.0 ft-kips

$U_3 = 1.2\,D + 1.6\,L$, (ACI 318-08 9.2.1)

P_u = 80.0 kips $M_{u,top}$ = 0.0 ft-kips
V_u = 0.0 kips $M_{u,bot}$ = 0.0 ft-kips

CHECK CAPACITY SUBJECTED TO BENDING AND AXIAL LOAD

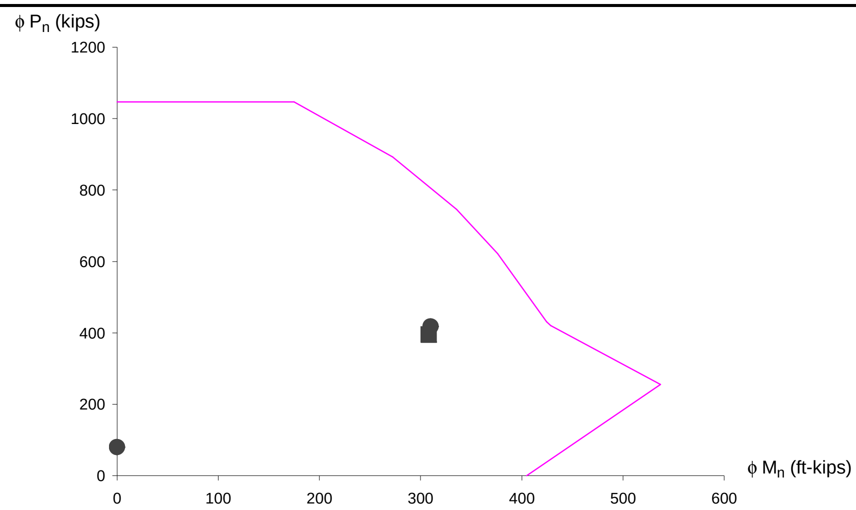
LOADING	U_1,top	U_1,bot	U_2,top	U_2,bot	U_3,top	U_3,bot
P_u (kips)	417.6	417.6	395.6	395.6	80.0	80.0
M_u (ft-kips)	280.0	280.0	280.0	280.0	0.0	0.0
$\delta_{ns} = C_m/[1-P_u/(0.75P_c)]$	1.107	1.107	1.100	1.100	1.019	1.019
$\delta_{ns}M_u$ (ft-kips)	309.9	309.9	308.1	308.1	0.0	0.0
ϕM_n (ft-kips) @ P_u	430.1	501.4	502.5	502.5	404.7	404.7

where $EI = 0.4E_cI_g / (1+\beta_d) = 0.25\,E_cI_g$
$P_c = \pi^2 EI / (kL_u)^2$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES (for ACI 318-08 10.2 & 10.3 only)

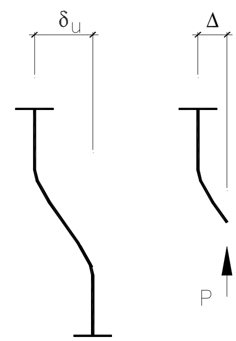
CAPACITY	**ϕ P_n (kips)**	**ϕ M_n (ft-kips)**
AT AXIAL LOAD ONLY	1047	0
AT MAXIMUM LOAD	1047	175
AT 0 % TENSION	893	272
AT 25 % TENSION	745	336
AT 50 % TENSION	622	376
AT ε_t = 0.002	431	425
AT BALANCED CONDITION	420	428
AT ε_t = 0.005	256	537
AT FLEXURE ONLY	0	405

(cont'd)



All load points to be within capacity diagram. [Satisfactory]

DETERMINE INDUCED MOMENT IN THE COLUMN



$$M_{col} = \frac{6E_C I_C \delta_u}{h^2} + P\Delta =$$ 273.6766 ft-kips, at top & bottom of column

where $E_c = 57000\,(f_c')^{0.5} =$ 3122 ksi, ACI 318-08 8.5.1

$I_g = c_1\, c_2^3 / 12 =$ 27648 in^4

$I_c =$ 0.7 $I_g =$ 19354 in^4, ACI 318-08 9.5.2.3 & 10.11.1

$\delta_u = C_d\, \delta_{xe} / I =$ 0.33 $I =$ 1.5, ASCE 7-05 (12.8-15)

$P = 0.9\, P_{DL} =$ 36 kips, ACI 318-08 21.13.3

$\Delta = 0.5\, \delta_u =$ 0.17 in, ACI 318-08 21.13.3

CHECK REQUIREMENTS OF NOT PART OF THE LATERAL RESISTING SYSTEM

$M_u = 1.2\, M_{DL} + 1.0\, M_{LL} + M_{col} =$ 273.6766 ft-kips

$<$ $\phi\, M_n =$ 404.7 kips [Satisfactory]

$P_{u,\,max} =$ 417.6 kips $>$ $0.1A_g f_c' =$ 172.8 kips [Satisfactory]

Per ACI 318-08 21.13.4.3, the column shall satisfy ACI 318-08 21.6.3, 21.6.4, 21.6.5, and 21.7.3.1.

CHECK SECTION REQUIREMENTS (ACI 318-08 21.6.1)

$c_{min} = MIN(c_1, c_2) =$ 24 in $>$ 12 in [Satisfactory]

$c_{min} / c_{max} =$ 1.00 $>$ 0.4 [Satisfactory]

CHECK TRANSVERSE REINFORCING AT END OF COLUMN (ACI 318-08 21.6.4)

$A_{sh} =$ 0.60 in^2 $>$ $MAX[\,0.09 s h_c f_c' / f_{yh}\,,\ 0.3 s h_c (A_g/A_{ch}-1) f_c' / f_{yh}\,] =$ 0.47 in^2

[Satisfactory] where $s = MAX[MIN(c_1/4,\ 6d_b,\ 4+(14-h_x)/3,\ 6),\ 4] =$ 5 in

$h_c = c_1 - 2Cover - d_t =$ 20.5 in

$A_{ch} = (c_1-3)(c_2-3) =$ 441.0 in^2

CHECK FLEXURAL REINFORCING (ACI 318-08 21.6.1.1)

$\rho_{total} =$ 0.018 $>$ $\rho_{min} =$ 0.010 [Satisfactory]

$<$ $\rho_{max} =$ 0.060 [Satisfactory]

CHECK SHEAR STRENGTH (ACI 318-08 21.6.4.6)

$V_e = MAX[\,(M_{pr,\,left,\,top} + M_{pr,\,right,bot}) / h\,,\ V_{u,max}] =$ 120.8 kips

$<$ $8\phi(f_c')^{0.5} c_2 d =$ 169.6 kips [Satisfactory]

$<$ $\phi[2(f_c')^{0.5} c_2 d + A_v f_y d / s] =$ 158.5 kips [Satisfactory]

where $\rho_{top,left} =$ 0.006 $>$ $\rho_{min} = MIN[3(f_c')^{0.5}/f_y,\ 200/f_y] =$ 0.003 [Satisfactory]

$\rho_{bot,left} =$ 0.006 $>$ $\rho_{min} =$ 0.003 [Satisfactory]

$M_{pr,\,left,\,top} = 1.25 M_{n,col,max} =$ 966 ft-kips $\phi =$ 0.75, ACI 318-08 9.3.2.3

$M_{pr,\,right,\,bot} = 1.25 M_{n,col,max} =$ 966 ft-kips $A_v =$ 0.6 in^2

DETERMINE SEISMIC TENSION DEVELOPMENT, Ld, INTO THE TOP PER ACI 318-08 21.6.4.6

$$L_{dh} = MAX\left(\frac{d_b f_y}{65\sqrt{f_c'}}\ ,\ 8d_b\ ,\ 6\ \ in\right) =$$ 17 d_b = 17 in, (ACI 318-08 21.7.5.1)

$$L_d = MAX\,(3.5 L_{dh}\beta\ ,\ 12\ \ in) =$$ 59 d_b = 59 in, (ACI 318-08 21.7.5.2)

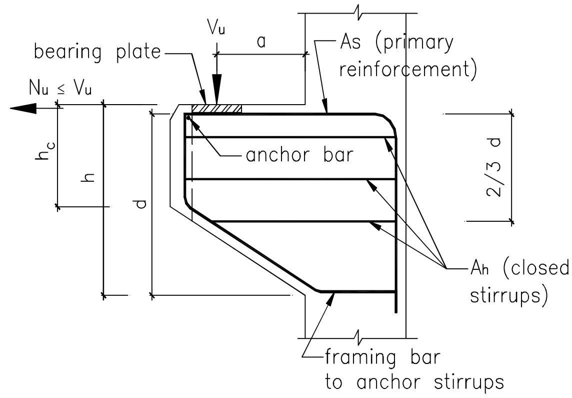
where d_b = 1 in

β = 1.0, (1.2 for epoxy-coated, ACI 318-08 21.7.5.4 & 12.2.4)

Corbel Design Based on IBC 09 / ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
FACTORED SHEAR LOAD	V_u	=	80	k
FACTORED TENSILE LOAD	N_{uc}	=	40	k
WIDTH	b	=	15	in
EFFECTIVE DEPTH	d	=	20	in
OVERALL DEPTH	h	=	22	in
SHEAR SPAN	a	=	4	in
EDGE DEPTH	h_c	=	12	in
PRIMARY REINFORCEMENT	3	#	7	
CLOSED STIRRUPS	3	#	3	(spacing 4 in o.c.)



ANALYSIS

CHECK DIMENSIONAL REQUIREMENTS (ACI Sec. 11.8.1 & 11.8.2)

a / d	=	0.20	<	1	[SATISFACTORY]
Nuc / Vu	=	0.50	<	1	[SATISFACTORY]
hc	=	12.00	>	0.5 d	[SATISFACTORY]

CHECK SECTION (ACI Sec. 11.8.3.2.1)

$$V_u \quad < \quad MIN(0.2\phi f_c' bd \ , \ 0.8\phi bd) \quad = 180 \text{ k}$$ **[SATISFACTORY]**

where ϕ = 0.75

CHECK REINFORCEMENT

$$A_n = MAX\left(\frac{N_{uc}}{\phi f_y}, \ \frac{0.2V_u}{\phi f_y}\right) = 0.889 \text{ in}^2$$ (ACI Sec. 11.8.3.4)

$$A_{vf} = \frac{V_u}{\phi \mu f_y} = 1.270 \text{ in}^2$$ (ACI Sec. 11.7.4.3)

where ϕ = 0.75 μ = 1.4

$$A_f = \frac{0.85bd f_c'\left(1-\sqrt{1-\frac{M_u}{0.383bd^2 f_c'}}\right)}{f_y} = 0.373 \text{ in}^2$$ (ACI Sec. 10.2)

$$M_u = V_u a + N_{uc}(h-d) = 400 \text{ in-k}$$

$$A_{sc} = MAX\left(A_f + A_n \ , \ \frac{2A_{vf}}{3} + A_n \ , \ \frac{0.04 f_c'}{f_y} bd\right) = 1.735 \text{ in}^2 \quad < \quad A_{s,provd} = 1.800 \text{ in}^2$$

(ACI Sec. 11.8.3.5 & 11.8.5) **[SATISFACTORY]**

$$A_h = \frac{A_{sc} - A_n}{2} = 0.423 \text{ in}^2 \quad < \quad A_{h,provd} = 0.660 \text{ in}^2$$

(ACI Sec. 11.8.4) **[SATISFACTORY]**

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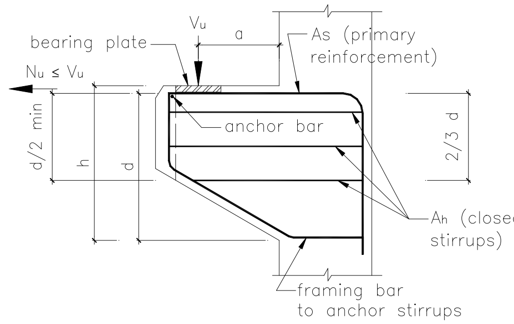
Corbel Design Based on CBC 01 / ACI 318-95

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	fc'	=	5	ksi
REBAR YIELD STRESS	fy	=	60	ksi
FACTORED SHEAR LOAD	Vu	=	100	k
FACTORED TENSILE LOAD	Nuc	=	40	k
WIDTH	b	=	15	in
EFFECTIVE DEPTH	d	=	20	in
OVERALL DEPTH	h	=	22	in
SHEAR SPAN	a	=	4	in
PRIMARY REINFORCEMENT	3	#	7	
CLOSED STIRRUPS	3	#	3	(spacing 4 in o.c.)



ANALYSIS

CHECK DIMENSIONAL REQUIREMENTS (ACI Sec. 11.9.1 & 11.9.2)

a / d	=	0.20	<	1	**[SATISFACTORY]**
Nuc / Vu	=	0.40	<	1	**[SATISFACTORY]**
hc	=	12.00	>	0.5 d	**[SATISFACTORY]**

CHECK SECTION (ACI Sec. 11.9.3.2)

$$V_u < MIN(0.2\phi f'_c bd \;,\; 0.8\phi bd) = 204 \text{ k}$$ **[SATISFACTORY]**

CHECK REINFORCEMENT

$$A_n = MAX\left(\frac{N_{uc}}{\phi f_y}, \frac{0.2V_u}{\phi f_y}\right) = 0.784 \text{ in}^2$$

(ACI Sec. 11.9.3.4)

$$A_{vf} = \frac{V_u}{\phi u f_y} = 1.401 \text{ in}^2$$

(ACI Sec. 11.7.4)

$$A_f = \frac{0.85bd f'_c\left(1-\sqrt{1-\frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} = 0.449 \text{ in}^2$$

$$M_u = V_u a + N_{uc}(h-d) = 480 \text{ in-k}$$

(ACI Sec. 10.2)

$$A_s = MAX\left(A_f + A_n \;,\; \frac{2A_{vf}}{3} + A_n \;,\; \frac{0.04 f'_c}{f_y}bd\right) = 1.718 \text{ in}^2 < A_{s,provd} = 1.800 \text{ in}^2$$

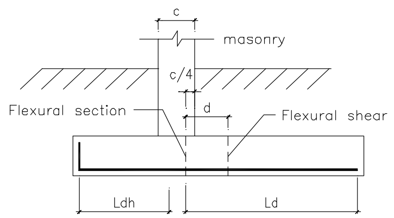
(ACI Sec. 11.9.3.5 & 11.9.5) **[SATISFACTORY]**

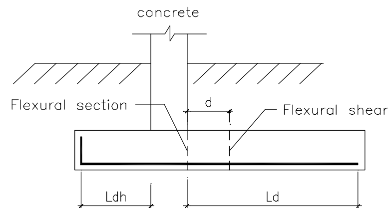
$$A_h = \frac{A_s - A_n}{2} = 0.467 \text{ in}^2 < A_{h,provd} = 0.660 \text{ in}^2$$

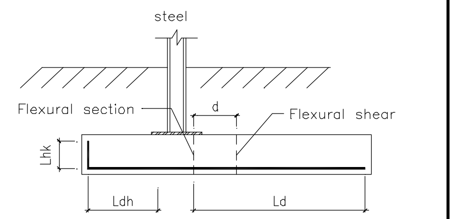
(ACI Sec. 11.9.4) **[SATISFACTORY]**

Development of Reinforcement Based on ACI 318-08

NON-SEISMIC TENSION DEVELOPMENTS



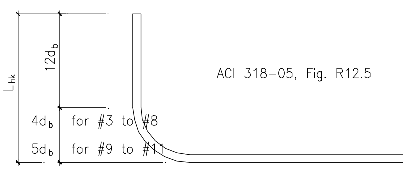




$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f'_c}}\ ,\ 8d_b\ ,\ 6\ \ in\right) =$$ 15 d_b = 12 in, (ACI 318-08 12.5.2)

$$L_d = MAX\left(\frac{\rho_{requird}}{\rho_{provided}} \frac{0.075\psi_t\psi_e\psi_s d_b f_y}{\lambda\sqrt{f'_c}\left(\frac{c+K_{tr}}{d_b}\right)}\ ,\ 12\ \ in\right) =$$ 26 d_b = 20 in, (ACI 318-08 12.2.3)

L_{hk} = 12 in, (ACI 318-08, Fig. R12.5)

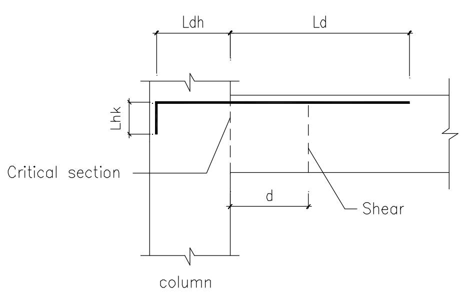


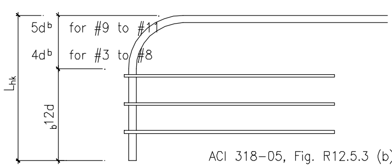
ACI 318−05, Fig. R12.5

where

Bar size	#	6		
d_b	=	0.75	in	
$\rho_{required} / \rho_{provided}$	=	1	($A_{s,reqd} / A_{s,provd}$, ACI 318-08, 12.2.5)	
f_y	=	60	ksi	
f'_c	=	3	ksi	
ψ_t	=	1.0	(1.3 for horizontal bar over 12" concrete, ACI 318-08 12.2.4)	
ψ_e	=	1.0	(1.2 for epoxy-coated, ACI 318-08 12.2.4)	
ψ_s	=	0.8	(0.8 for # 6 or smaller, 1.0 for other)	
λ	=	1.0	(0.75 for light weight, ACI 318-08, 12.2.4)	
c	=	3.4	in, min(d' , 0.5s), (ACI 318-08, 12.2.4)	
K_{tr}	=	(A_{tr} f_{yt} / 1500 s n) =	0	(ACI 318-08, 12.2.3)
$(c + K_{tr}) / d_b$ =		2.5	< 2.5 , (ACI 318-08, 12.2.3)	
η	=	0.7	(#11 or smaller, cover > 2.5" & side >2.0", ACI 318-08 12.5.3)	

SEISMIC TENSION DEVELOPMENTS





ACI 318−05, Fig. R12.5.3 (b)

$$L_{dh} = MAX\left(\frac{d_b f_y}{65\sqrt{f'_c}}\psi_e\ ,\ 8d_b\ ,\ 6\ \ in\right) =$$ 17 d_b = 13 in
(ACI 318-08 21.5.4.1)

$$L_d = MAX\left(3.5 L_{dh}\psi_e\ ,\ 12\ \ in\right) =$$ 59 d_b = 44 in, (ACI 318-08 21.5.4.2)

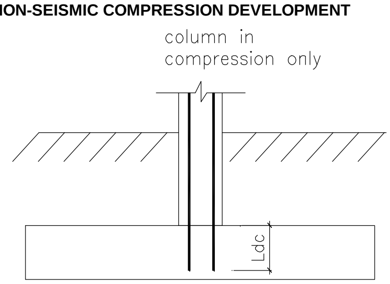
L_{hk} = 12 in, (ACI 318-08, Fig. R12.5)

where

Bar size	#	6	
d_b	=	0.75	in
f_y	=	60	ksi
f'_c	=	3	ksi
ψ_e	=	1.0	

(1.2 for epoxy-coated, ACI 318-08 21.5.4.4 & 12.2.4)

NON-SEISMIC COMPRESSION DEVELOPMENT



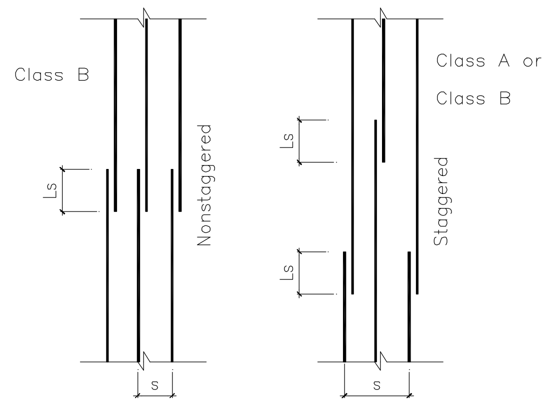
$$L_{dc} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02 d_b f_y}{\sqrt{f'_c}}\ ,\ \eta \frac{\rho_{requird}}{\rho_{provided}} 0.3 d_b f_y\ ,\ 8\ \ in\right) =$$ 21 d_b = 16 in
(ACI 318-08 12.3)

where

Bar size	#	6	
d_b	=	0.75	in
$\rho_{required} / \rho_{provided}$	=	0.95	(ACI 318-08 12.3.3 a)
f_y	=	60	ksi
f'_c	=	3	ksi
η	=	1.0	(for spiral, 0.75, ACI 318-08 12.3.3 b)

Splice of Reinforcement Based on ACI 318-08

NON-SEISMIC TENSION SPLICE



Vertical Bars in Shear Wall Web

ACI 318-08 Tab. R12.15.2

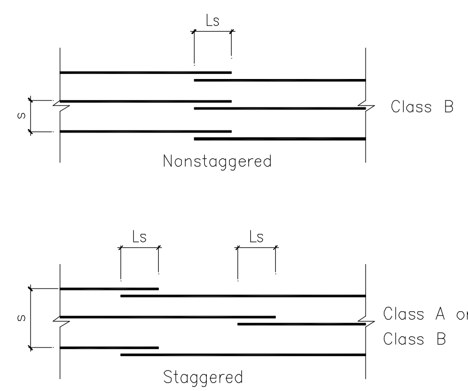
$A_{s,provd}$ / $A_{s,reqd}$	50% , Staggered	100% Lap
≥ 2	Class A	Class B
< 2	Class B	Class B

$\boldsymbol{L_s}$ = **1.3 $\boldsymbol{L_d}$** = 30 d_b = 18 in
(for Class A, ***1.0 L_d***, ACI 318-08, 12.15.1)

$$L_d = MAX\left(\frac{0.075\psi_t\psi_e\psi_s d_b f_y}{\lambda\sqrt{f'_c}\left(\frac{c+K_{tr}}{d_b}\right)}\ ,\ 12\ \ in\right) =$$ 23 d_b = 14 in
(ACI 318-08 12.15.1 & 12.2.3)

where Bar size		#	5	
d_b	=		0.625	in
f_y	=		60	ksi
f'_c	=		4	ksi
ψ_t	=		1.0	(1.3 for horizontal bar over 12" concrete, ACI 318-08 12.2.4)
ψ_e	=		1.0	(1.2 for epoxy-coated, ACI 318-08 12.2.4)
ψ_s	=		0.8	(0.8 for # 6 or smaller, 1.0 for other)
λ	=		1.0	(0.75 for light weight, ACI 318-08, 12.2.4)
c	=		1.7	in, min(d' , 0.5s), s = see fig above, (ACI 318-08, 12.2.4)
K_{tr}	=	(A_{tr} f_{yt} / 1500 s n) =	0	(ACI 318-08, 12.2.3)
(c + K_{tr}) / d_b =			2.5	< 2.5 , (ACI 318-08, 12.2.3)

SEISMIC TENSION SPLICE



Horizontal Bars in Shear Wall Web

$\boldsymbol{L_s}$ = **1.3 $\boldsymbol{L_d}$** = 66 d_b = 42 in
(for Class A, ***1.0 L_d***, ACI 318-08, 12.15.1)

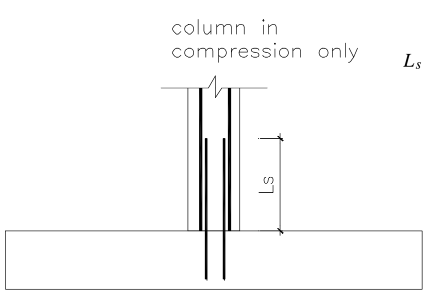
$$L_{dh} = MAX\left(\frac{d_b f_y}{65\sqrt{f'_c}}\psi_e\ ,\ 8d_b\ ,\ 6\ \ in\right) =$$ 15 d_b = 9 in
(ACI 318-08 21.5.4.1)

$$L_d = MAX\left(3.5L_{dh}\psi_e\ ,\ 12\ \ in\right) =$$ 51 d_b = 32 in, (ACI 318-08 21.5.4.2)

where Bar size		#	5	
d_b	=		0.625	in
f_y	=		60	ksi
f'_c	=		4	ksi
ψ_e	=		1.0	

(1.2 for epoxy-coated, ACI 318-08 21.5.4.4 & 12.2.4)

NON-SEISMIC COMPRESSION SPLICE



$$L_s = \begin{cases}\eta MAX\left(0.0005 d_b f_y\ ,\ 12\ \ in\right)\ ,\ for\ \ f_y \le 60\ \ ksi\\ \eta MAX\left(0.0009 d_b f_y - 24 d_b\ ,\ 12\ \ in\right)\ ,\ for\ \ f_y > 60\ \ ksi\end{cases} =$$ 30 d_b = 23 in, (ACI 318-08 12.16.1)

where Bar size		#	6	
d_b	=		0.75	in
f_y	=		60	ksi
f'_c	=		3	ksi
η	=		1.0	(for f_c' < 3 ksi, 4/3, ACI 318-08 12.16.1)

Tables for Development & Splice of Reinforcement Based on ACI 318-08

NON-SEISMIC TENSION DEVELOPMENTS

(FOR ALL DEVELOPMENTS EXCEPT LATERAL MOMENT FRAME, BEARING WALL & GRAVITY COLUMN)

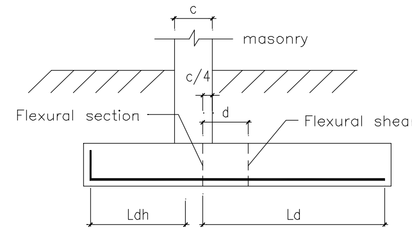
Table 1: L_d Values (inch)

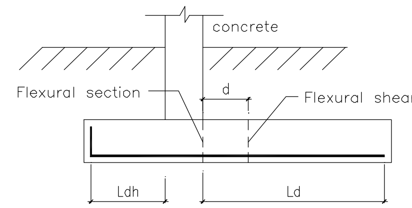
Bar Size	f_c' (psi) 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	12	12	12	12	12	12	12	12	12
# 4	20	16	14	13	12	12	12	12	12
# 5	30	24	21	19	17	16	15	14	13
# 6	40	33	28	25	23	22	20	19	18
# 7	65	53	46	41	37	35	32	31	29
# 8	80	66	57	51	46			38	36
# 9	97	80	69	62	56			46	44
# 10	117	96	83	74	68			55	52
# 11	137	112	97	87	79			65	61
# 14	181	148	128	114	104	97	90	85	81
# 18	273	223	193	173	158	146	136	129	122

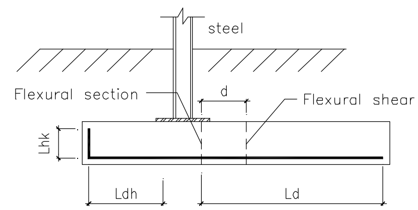
Table 2: L_{dh} Values (inch)

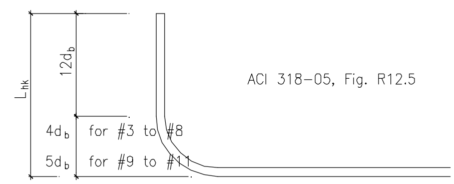
Bar Size	f_c' (psi) 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	7	6	6	6	6	6	6	6	6
# 4	9	8	7	6	6	6	6	6	6
# 5	12	10	8	7	7	6	6	6	6
# 6	14	12	10	9	8	8	7	7	6
# 7	16	13	12	10	9	9	8	8	7
# 8	19	15	13	12	11	10	9	9	8
# 9	21	17	15	13	12	11	11	10	9
# 10	24	19	17	15	14	13	12	11	11
# 11	26	22	19	17	15	14	13	12	12
# 14	32	26	22	20	18	17	16	15	14
# 18	42	35	30	27	24	23	21	20	19

Note: Where horizontal bar is pleased such that more than 12 in of fresh concrete is cast below the table values above must be increased 30%.









SEISMIC TENSION DEVELOPMENTS

(FOR LATERAL MOMENT FRAME ONLY)

Table 3: L_d Values (inch)

Bar Size	f_c' (psi) 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	27	22	21	21	21	21	21	21	21
# 4	36	29	26	23	21	21	21	21	21
# 5	45	37	32	29	26	24	23	21	21
# 6	54	44	38	34	31	29	27	26	24
# 7	63	52	45	40	36	34	32	30	28
# 8	72	59	51	46	42	39	36	34	32
# 9	81	67	58	52	47	44	41	38	36
# 10	92	75	65	58	53	49	46	43	41
# 11	102	83	72	64	59	54	51	48	46
# 14	122	100	86	77	71	65	61	58	55
# 18	163	133	115	103	94	87	82	77	73

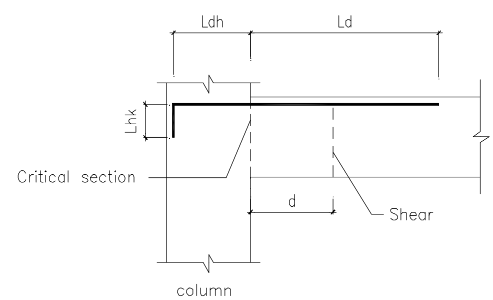
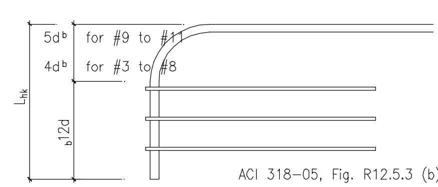


Table 4: L_{dh} Values (inch)

Bar Size	f_c' (psi) 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	8	6	6	6	6	6	6	6	6
# 4	10	8	7	7	6	6	6	6	6
# 5	13	11	9	8	7	7	6	6	6
# 6	15	13	11	10	9	8	8	7	7
# 7	18	15	13	11	10	10	9	9	8
# 8	21	17	15	13	12	11	10	10	9
# 9	23	19	16	15	13	12	12	11	10
# 10	26	21	19	17	15	14	13	12	12
# 11	29	24	21	18	17	16	15	14	13
# 14	35	29	25	22	20	19	17	16	16
# 18	47	38	33	29	27	25	23	22	21



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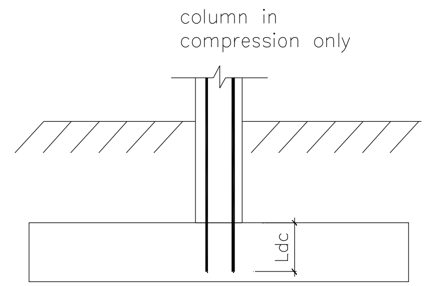
NON-SEISMIC COMPRESSION DEVELOPMENT

(FOR BEARING WALL & GRAVITY COLUMN)

Table 5: L_{dc} Values (inch)

Bar Size	**f_c' (psi)** 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	10	8	8	8	8	8	8	8	8
# 4	13	11	9	9	9	9	9	9	9
# 5	17	14	12	11	11	11	11	11	11
# 6	20	16	14	14	14	14	14	14	14
# 7	23	19	17	16	16	16	16	16	16
# 8	27	22	19	18	18	18	18	18	18
# 9	30	25	21	20	20	20	20	20	20
# 10	34	28	24	23	23	23	23	23	23
# 11	38	31	27	25	25	25	25	25	25
# 14	45	37	32	30	30	30	30	30	30
# 18	61	49	43	41	41	41	41	41	41



NON-SEISMIC TENSION SPLICE

(FOR DOUBLE BARS USED WITH 50% STAGGERED IN CLASS B, TABLE 7)

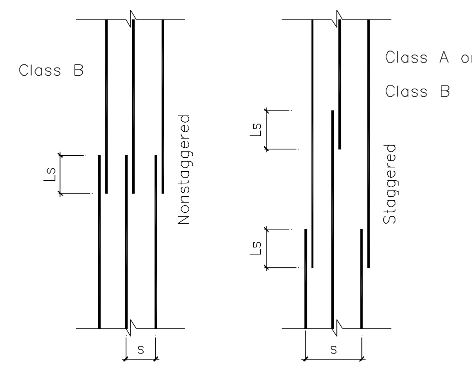
Table 6: L_S Values (inch), CLASE A

Bar Size	**f_c' (psi)** 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	12	12	12	12	12	12	12	12	12
# 4	16	13	12	12	12	12	12	12	12
# 5	20	16	14	13	12	12	12	12	12
# 6	24	20	17	15	14	13	12	12	12
# 7	37	30	26	24	22	20	19	18	17
# 8	45	37	32	28	26	24	22	21	20
# 9	52	43	37	33	30	28	26	25	23
# 10	61	50	43	39	35	33	31	29	27
# 11	70	57	49	44	40	37	35	33	31
# 14	88	72	62	55	51	47	44	41	39
# 18	124	101	88	78	72	66	62	58	55

(FOR ALL SPLICE EXCEPT LATERAL FRAME, SHEAR WALL HORIZONTAL BARS & GRAVITY COLUMN)

Table 7: L_S Values (inch), CLASE B

Bar Size	**f_c' (psi)** 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	16	16	16	16	16	16	16	16	16
# 4	21	17	16	16	16	16	16	16	16
# 5	26	21	18	17	16	16	16	16	16
# 6	31	26	22	20	18	17	16	16	16
# 7	49	40	34	31	28	26	24	23	22
# 8	58	47	41	37	34	31	29	27	26
# 9	68	56	48	43	39	36	34	32	30
# 10	79	65	56	50	46	42	40	37	36
# 11	91	74	64	57	52	49	45	43	41
# 14	114	93	81	72	66	61	57	54	51
# 18	161	132	114	102	93	86	81	76	72



Vertical Bars in Shear Wall Web

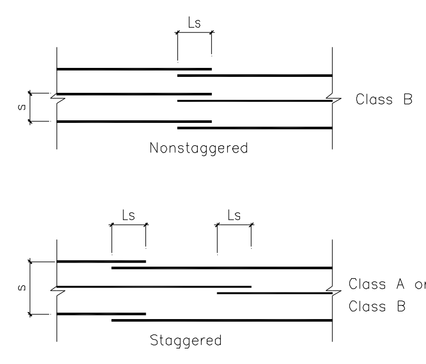
Note: Where horizontal bar is pleased such that more than 12 in of fresh concrete is cast below the table values above must be increased 30%.

SEISMIC TENSION SPLICE

(FOR DOUBLE BARS USED WITH 50% STAGGERED IN CLASS B, TABLE 9)

Table 8: L_S Values (inch), CLASE A

Bar Size	**f_c' (psi)** 2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	27	22	21	21	21	21	21	21	21
# 4	36	29	26	23	21	21	21	21	21
# 5	45	37	32	29	26	24	23	21	21
# 6	54	44	38	34	31	29	27	26	24
# 7	63	52	45	40	36	34	32	30	28
# 8	72	59	51	46	42	39	36	34	32
# 9	81	67	58	52	47	44	41	38	36
# 10	92	75	65	58	53	49	46	43	41
# 11	102	83	72	64	59	54	51	48	46
# 14	122	100	86	77	71	65	61	58	55
# 18	163	133	115	103	94	87	82	77	73



Horizontal Bars in Shear Wall Web

(Cont'd)

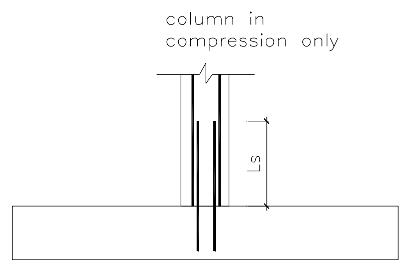
(FOR LATERAL FRAME & SHEAR WALL HORIZONTAL BARS)

Table 9: L_s Values (inch), CLASE B									
Bar	**f_c' (psi)**								
Size	2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	35	29	27	27	27	27	27	27	27
# 4	47	38	33	30	27	27	27	27	27
# 5	59	48	42	37	34	31	29	28	27
# 6	70	58	50	45	41	38	35	33	32
# 7	82	67	58	52	47	44	41	39	37
# 8	94	77	66	59	54	50	47	44	42
# 9	106	86	75	67	61	57	53	50	47
# 10	119	97	84	75	69	64	60	56	53
# 11	132	108	94	84	76	71	66	62	59
# 14	159	130	112	101	92	85	79	75	71
# 18	212	173	150	134	122	113	106	100	95

NON-SEISMIC COMPRESSION SPLICE

(FOR GRAVITY COLUMN ONLY)

Table 10: L_s Values (inch)									
Bar	**f_c' (psi)**								
Size	2000	3000	4000	5000	6000	7000	8000	9000	10000
# 3	16	12	12	12	12	12	12	12	12
# 4	20	15	15	15	15	15	15	15	15
# 5	25	19	19	19	19	19	19	19	19
# 6	30	23	23	23	23	23	23	23	23
# 7	35	26	26	26	26	26	26	26	26
# 8	40	30	30	30	30	30	30	30	30
# 9	45	34	34	34	34	34	34	34	34
# 10	51	38	38	38	38	38	38	38	38
# 11	56	42	42	42	42	42	42	42	42
# 14	68	51	51	51	51	51	51	51	51
# 18	90	68	68	68	68	68	68	68	68



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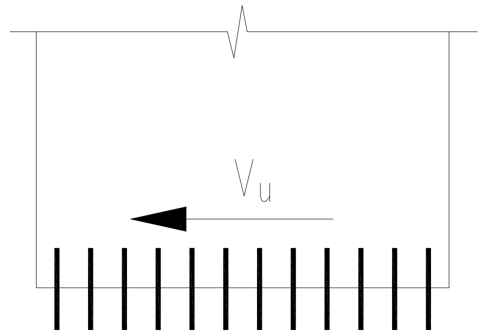
JOB NO. :	DATE :	REVIEW BY :	

Shear Friction Reinforcing Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

FACTORED FRICTION FORCE	V_u	=	500	kips
CONCRETE STRENGTH	f_c'	=	4	ksi
REINFORCEMENT STRENGTH	f_y	=	60	ksi
FRICTION COEFFICIENT [See Table belov	μ	=	0.6	
SHEAR PLANE THICKNESS	t	=	8	in
SHEAR PLANE LENGTH	L	=	20	ft
DOWEL SIZE		#	5	



Use # 5 Dowels @ 4.0 in o.c.

ANALYSIS

CHECK SHEAR STRENGTH LIMITATION (Sec. 11.6.5)

$\phi V_n = \phi\, \mathrm{MIN}(\, 0.2 f_c' \,,\, 480 + 0.08 f_c' \,,\, 1600\,)\ A_c =$ 1152 kips > V_u

Where ϕ = 0.75 [Section 9.3.2.3] **[SATISFACTORY]**

A_c = 1920 in^2

THE REQUIRED AREA OF SHEAR-TRANSFER REINFORCEMENT IS GIVEN BY SECTION 11.6.4.1 AS

$A_{vf} = V_u \,/\, (\phi\, f_y\, \mu) =$ 18.5 in^2

COEFFICIENT OF FRICTION FOR NORMAL WEIGHT CONCRETE [Sec.11.6.4.3]	μ	
Concrete place monolithically	1.40	
Concrete placed against hardened concrete with surface intentionally roughened	1.00	
Concrete placed against hardened concrete NOT intentionally roughened	0.60	<=
Concrete anchored to as-rolled structural steel by headed stud or by reinforcing bars	0.70	

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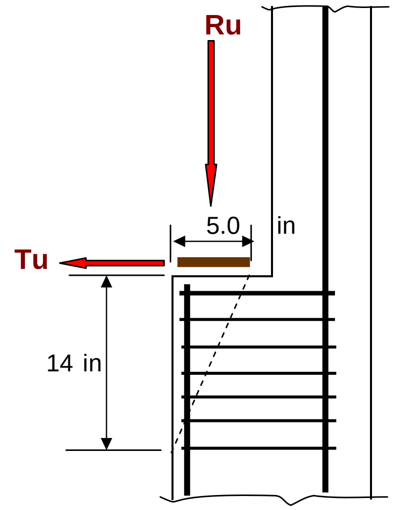
Shear Friction Reinforcing Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

FACTORED FRICTION FORCE VERTICAL	R_u	=	**86**	kips
FACTORED FRICTION FORCE HORIZONTAL	T_u	=	**34**	kips
CONCRTET STRENGTH	f_c'	=	**4**	ksi
REINFORCEMENT STRENGTH	f_y	=	**60**	ksi
BEARING WIDTH	a	=	**5**	in
BEARING THICKNESS	b	=	**18**	in
REINFORCEMENT	**2** - leg ties	#	**3**	
ASSUME A POTENTIAL CRACK PLANE WITH AN ANGLE OF	α_f	=	**70**	degrees



Use 6 # 3 with (2) - leg closed ties
Distribute over a vert. distance of 14 in

ANALYSIS

FRICTION COEFFICIENT [Sec.11.6.4.3] μ = 1.4

FORCES ON INCLINED PLANE:

$V_u = R_u \sin\alpha_f + T_u \cos\alpha_f$ = 92.4 kips **<** $\phi V_n = \phi$ MIN($0.2f_c'$, $480+0.08f_c'$, 1600) A_c = 160.6 kips

Where ϕ = 0.75 [Section 9.3.2.3] **[SATISFACTORY]** (Sec. 11.6.5)

A_c = 268 in^2

$N_u = T_u \sin\alpha_f - R_u \cos\alpha_f$ = 2.5 kips (net tension)

SHEAR FRICTION REINFORCEMENT IS GIVEN BY EQ(11-26) AS

$A_{vf} = V_u / [\phi f_y (\mu \sin\alpha_f + \cos\alpha_f)]$ = 1.24 in^2

REINFORCEMENT TO RESIST TENSION IS GIVEN BY SEC. 11.6.7 AS

$A_n = N_u / (\phi f_y \sin\alpha_f)$ = 0.06 in^2

TOTAL REINFORCEMENT: $A_s = A_{vf} + A_n$ = = 1.30 in^2

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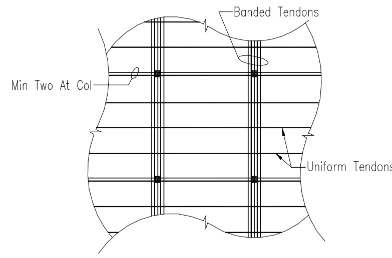
REVIEW BY :

Design of Post-Tensioned Concrete Floor Based on ACI 318-08



1. DESIGN METHODS

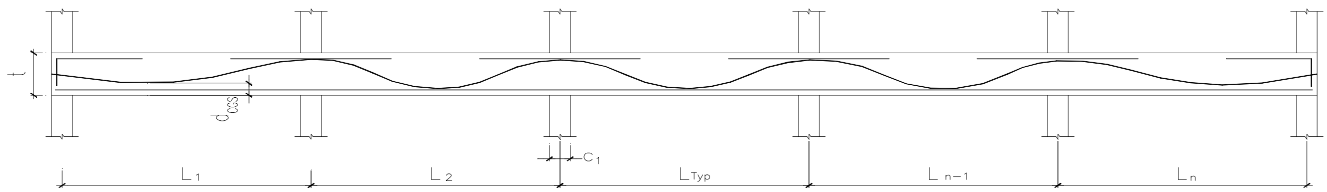
1.1 BREAKDOWN TWO WAYS FLOOR INTO DESIGN STRIPS IN ONE DIRECTION AND ONE WAY SLABS IN OTHER DIRECTION. DESIGN STRIPS WORK AS CONTINUOUS BEAMS BY BANDED ALL TENDONS AT COLUMN. THE PERPENDICULAR DIRECTION LIKE MULTI-SPAN ONE WAY SLABS, USING DISTRIBUTED TENDONS.
1.2 SPECIFY TOTAL REQUIRED EFFECTIVE POST-TENSIONING FORCES AT BANDED TENDONS, ON STRUCTURAL DRAWINGS, AND UNIFORM FORCES IN DISTRIBUTED TENDONS.
1.3 WHEN TENDON LESS THAN 140 FT, STRESS AT ONE END. OTHERWISE STRESS AT BOTH ENDS.
1.4 THE VARIOUS LIVE LOADING CONDITIONS SHOULD BE CONSIDER BY INPUT LL ZERO AT SOME SPANS.

2. INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c	=	4.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
TENDON PROPERTIES	f_{pu}	=	270	ksi
	f_{py}	=	243	ksi
	f_{se}	=	175	ksi
	dia	=	1 / 2	in
	A_{ps}	=	0.153	in^2
SLAB THICKNESS	t	=	8	in
TRIBUTARY WIDTH IF BANDED AS DESIGN STRIPS	W	=	30	ft
COLUMN WIDTH	c_1	=	22	in
COLUMN DEPTH	c_2	=	22	in
TOP BAR SIZE AT COLUMN	#		5	
BOTTOM CONTINUOUS BAR SIZE	#		4	

THE DESIGN IS ADEQUATE.

CONCRETE COST = 100 lb / ft^2
TENDONS COST = 0.476 lb / ft^2
REBARS COST = 0.349 lb / ft^2



Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Support	Mid L_n	Right
Span (ft)		20		30		30		30		20	
DL (psf)		120		120		120		120		120	
LL (psf)		75		75		75		75		75	
Balanced DL (60%-80% suggested)		75%		75%		80%		75%		75%	
d_{CGS} (in, from bottom)	4	3.33	7.5	2.5	7.5	2.5	7.5	2.5	7.5	3.33	4
REQD EFFECTIVE PT (k / ft) Total if banded (kips)		22.31 669.4		24.30 729.0		25.92 777.6		24.30 729.0		22.31 669.4	
Tendons Total Number if banded		1 / 2 in Dia @ 14 in o.c. 26		1 / 2 in Dia @ 13 in o.c. 28		1 / 2 in Dia @ 12 in o.c. 30		1 / 2 in Dia @ 13 in o.c. 28		1 / 2 in Dia @ 14 in o.c. 26	
Top Bars at Column	7 # 5 , L = 3.03 ft		10 # 5 , L = 9.39 ft		22 # 5 , L = 9.39 ft		22 # 5 , L = 9.39 ft		10 # 5 , L = 9.39 ft		7 # 5 , L = 3.03 ft
Bot Bars, Cont., E. Way		Not ReqD		# 4 @ 45in. o.c.		Not ReqD		# 4 @ 45in. o.c.		Not ReqD	
Required column cap thk, (in)	Not ReqD		Not ReqD		Not ReqD		Not ReqD		Not ReqD		Not ReqD

3. DESIGN LOADS & SECTION FORCES

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Support	Mid L_n	Right
M_{DL} (ft-k / ft)	0.00	2.14	-7.72	5.01	-9.26	4.24	-9.26	5.01	-7.72	2.14	0.00
M_{LL} (ft-k / ft)	0.00	1.34	-4.83	3.13	-5.78	2.65	-5.78	3.13	-4.83	1.34	0.00
Balanced Load (psf, uplift)		-90		-90		-96		-90		-90	
Balanced M_{Bal} (ft-k / ft)	0.00	-1.65	5.71	-3.66	7.23	-3.57	7.23	-3.66	5.71	-1.65	0.00
Required Effective PT (k / ft) Tendon Spacing (in)		22.31 14		24.30 13		25.92 12		24.30 13		22.31 14	
Primary M_{Fe} (ft-k / ft)	0.00	-1.25	6.51 7.09	-3.04	7.56 7.09	-3.24	7.56 7.09	-3.04	6.51 7.09	-1.25	0.00
Secondary M_{Sec} (ft-k / ft)	0.00	0.40	0.80 1.38	0.62	0.33 -0.14	0.33	0.33 -0.14	0.62	0.80 1.38	0.40	0.00

4. CHECK SERVICE LOAD STRESSES

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Support	Mid L_n	Right
A (in^2 / ft)	96	96	96	96	96	96	96	96	96	96	96
S (in^3 / ft)	128	128	128	128	128	128	128	128	128	128	128
F / A (ksi)	0.232	0.232	0.232 0.253	0.253	0.270 0.253	0.270	0.270 0.253	0.253	0.232 0.253	0.232	0.232
Check 125 psi < F /A < 275 psi			**[Satisfactory]**		(ADAPT suggestion)						
M / S + F / A , (ksi) for load combination (DL + PT)	0.232	0.279	0.043 0.064	0.380	0.080 0.063	0.333	0.080 0.063	0.380	0.043 0.064	0.279	0.232
- M / S + F / A , (ksi) for load combination (DL + PT)	0.232	0.186	0.421 0.442	0.126	0.460 0.443	0.207	0.460 0.443	0.126	0.421 0.442	0.186	0.232
Check $f_t < 7.5 (f_c')^{0.5}$ and $f_c <$ 0.45 f_c'			**[Satisfactory]**		(ACI 318-08, 18.3.3 & 18.4.2a),		where	$7.5 (f_c')^{0.5}$ =	0.503	0.45 f_c' =	2.025
M / S + F / A , (ksi) for load (DL + LL + PT)	0.232	0.404	-0.409 -0.388	0.674	-0.462 -0.479	0.582	-0.462 -0.479	0.674	-0.409 -0.388	0.404	0.232
- M / S + F / A , (ksi) for load (DL + LL + PT)	0.232	0.061	0.874 0.895	-0.167	1.002 0.985	-0.042	1.002 0.985	-0.167	0.874 0.895	0.061	0.232
Check $f_t < 7.5 (f_c')^{0.5}$ and $f_c <$ 0.6 f_c'			**[Satisfactory]**		(ACI 318-08, 18.3.3 & 18.4.2b),		where	$7.5 (f_c')^{0.5}$ =	0.503	0.6 f_c' =	2.700

5. CALCULATE NON-PRESTRESSED REINFORCEMENT

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Support	Mid L_n	Right
Max. N_c (k / ft), (ACI 318, 18.0)		0.000		1.599		0.134		1.599		0.000	
A_s (in^2 / ft), (ACI 318, 18.9.3.2)		0.000		0.053		0.000		0.053		0.000	
Bottom Bars, Each Way		Not ReqD		# 4 @ 45in. o.c.		Not ReqD		# 4 @ 45in. o.c.		Not ReqD	
Max. A_{cf} (in^2), (ACI 318, 18.0)	2880		2880		2880		2880		2880		2880
A_s' (in^2), (ACI 318, 18.9.3.2)	2.160		2.160		2.160		2.160		2.160		2.160
Top Bars at Column	7 # 5		7 # 5		7 # 5		7 # 5		7 # 5		7 # 5
L (ft), (ACI 318, 18.9.4.1)	3.03		9.39		9.39		9.39		9.39		3.03

6. CHECK FLEXURAL CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Support	Mid L_n	Right
Factored M_u (ft-k / ft)	0.00	5.11	-16.19		-20.03	9.67	-20.03		-16.19	5.11	0.00
$1.2\ M_{DL} + 1.6\ M_{LL} + 1.0\ M_{Sec}$			-15.61	11.64	-20.50		-20.50	11.64	-15.61		
d_p (in)	4.00	4.67	7.50	5.50	7.50	5.50	7.50	5.50	7.50	4.67	4.00
ρ_p	0.00273	0.00234	0.00146		0.00170	0.00232	0.00170		0.00146	0.00234	0.00273
			0.00157	0.00214	0.00157		0.00157	0.00214	0.00157		
L / d_p	60.00	51.39	40.00	65.45	48.00	65.45	48.00	65.45	40.00	51.39	60.00
f_{ps} (ksi)	185.64	186.56	190.44		188.24	185.88	188.24		190.44	186.56	185.64
(ACI 318, 18.7.2, b & c)			191.62	203.09	191.62		191.62	203.09	191.62		
A_{ps} (in^2 / ft)	0.131	0.131	0.131		0.153	0.153	0.153		0.131	0.131	0.131
Actual Area			0.141	0.14	0.14		0.141	0.14	0.14		
d (in)	6.63	6.75	6.63	6.75	6.63	6.75	6.63	6.75	6.63	6.75	6.63
a (in)	0.21	0.40	0.67		0.85	0.55	0.85		0.67	0.40	0.21
			0.64	0.65	0.88		0.88	0.65	0.64		
Required A_s (in^2 / ft)		0.000		0.018		0.000		0.018		0.000	
Bottom Bars, Each Way		Not ReqD		# 4 @ 136in. o.c.		Not ReqD		# 4 @ 136in. o.c.		Not ReqD	
Actual A_s (in^2 / ft)		0.000		0.053		0.000		0.053		0.000	
Required A_s' (in^2)	0.000		2.933		6.655		6.655		2.933		0.000
Top Bars at Column	Not ReqD		10 # 5		22 # 5		22 # 5		10 # 5		Not ReqD
Actual A_s' (in^2)	2.170		3.100		6.820		6.820		3.100		2.170
ϕ M_n (ft-k / ft)	-9.23	8.21	-16.34		-21.62	11.14	-21.62		-16.34	8.21	-9.23
Actual Capacity			-17.51	12.68	-20.66		-20.66	12.68	-17.51		
Check ϕ M_n > M_u			[Satisfactory]								
ε_{pt}, (ACI 318, 18.8.1)	0.0433	0.0262	0.0246		0.0188	0.0216	0.0188		0.0246	0.0262	0.0433
$\varepsilon_c(d_p - a / \beta_1) / (a / \beta_1)$			0.0261	0.0180	0.0181		0.0181	0.0180	0.0261		
Check ε_{pt} > 0.005			[Satisfactory]		(ACI 318, 18.8.1)						

7. CHECK PUNCHING SHEAR CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Support	Mid L_n	Right
R_{DL} (k)	35.99		90.01		108.00		108.00		90.01		35.99
R_{LL} (k)	22.49		56.26		67.50		67.50		56.26		22.49
R_{Sec} (k)	-26.99		-67.51		-83.70		-83.70		-67.51		-26.99
$V_u = 1.2\ R_{DL} + 1.6\ R_{LL} + 1.0$ Rse	52.18		130.52		153.90		153.90		130.52		52.18
Required b_0d, (ACI 318, 11-36)	228.48		565.70		655.06		655.06		565.70		228.48
Required d, (ACI 318, 11.0)	3.16		5.20		5.88		5.88		5.20		3.16
For ϕ V_n < V_u, the required	0.00		0.00		0.00		0.00		0.00		0.00
column cap thickness, t_{cap} (in)	Not ReqD		Not ReqD		Not ReqD		Not ReqD		Not ReqD		Not ReqD

8. CALCULATE COST FOR SLAB & CAP

Location		Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Support	Mid L_n	Right
Concrete	cap dim (ft)	1.67		5.00		5.00		5.00		5.00		1.67
	cap concrete	0.00		0.00		0.00		0.00		0.00		0.00
			Total =	**179395**	T / 30 ft TW			Average =	**100**	lb / ft^2		
Tendons	length (ft)		20.01		30.02		30.02		30.02		20.01	
			Total =	**854.2**	T / 30 ft TW	, (ACI 318 App. E)		Average =	**0.476**	lb / ft^2 , (AISC Manual 2nd page 7-15)		
Rebars		44.71	0.00	198.08	10.89	435.77	0.00	435.77	10.89	198.08	0.00	44.71
			Total =	**625.5**	T / 30 ft TW			Average =	**0.349**	lb / ft^2		

Note:

1. The column moments are negligible for gravity punching design. Lateral loads, seismic and wind, should be supported by shear walls. Using equivalent frames to support lateral loads is not suggested.
2. By inspection, the deflections of slab do not govern PT concrete floor design. Otherwise, using PT concrete floor is inadequate for larger live load. (ACI 318, 9.5.4.1)
3. The secondary moments are very important concept of PT floor design. Based on this concept, PT floor design are always continuous beams design and one way slabs design. So using two ways finite element analysis to design PT floor is inadequate.

Technical References:

1. "Design of Post-Tensioned Slabs Using Unbounded Tendons, Third Edition", The Post-Tensioning Institute, 2004.
2. "Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures, First Edition", The Post-Tensioning Institute, 2001.
3. Bijan O. Aalami & Allan Bommer, "Design Fundamentals of Post-Tensioned Concrete Floors, First Edition", The Post-Tensioning Institute, 1999.

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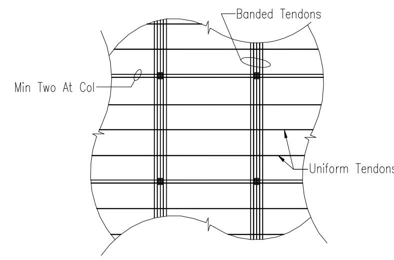
Design of Post-Tensioned Concrete Floor Based on ACI 318-08



1. DESIGN METHODS

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1.2 SPECIFY TOTAL REQUIRED EFFECTIVE POST-TENSIONING FORCES AT BANDED TENDONS, ON STRUCTURAL DRAWINGS, AND UNIFORM FORCES IN DISTRIBUTED TENDONS.

1.3 WHEN TENDON LESS THAN 140 FT, STRESS AT ONE END. OTHERWISE STRESS AT BOTH ENDS.

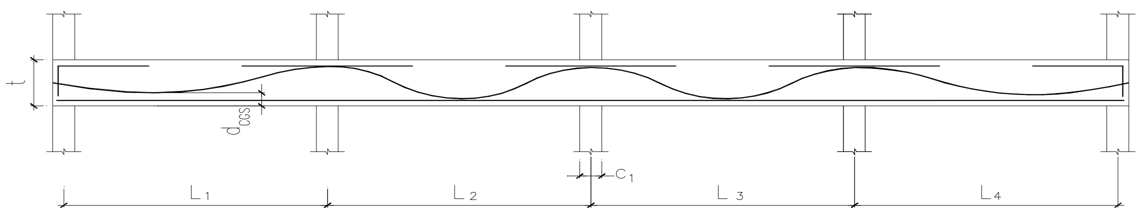
1.4 THE VARIOUS LIVE LOADING CONDITIONS SHOULD BE CONSIDER BY INPUT LL ZERO AT SOME SPANS.

2. INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c	=	5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
TENDON PROPERTIES	f_{pu}	=	270	ksi
	f_{py}	=	243	ksi
	f_{se}	=	175	ksi
	dia	=	1 / 2	in
	A_{ps}	=	0.153	in^2
SLAB THICKNESS	t	=	8	in
TRIBUTARY WIDTH IF BANDED AS DESIGN STRIPS	W	=	30	ft
COLUMN WIDTH	c_1	=	22	in
COLUMN DEPTH	c_2	=	22	in
TOP BAR SIZE AT COLUMN		#	5	
BOTTOM CONTINUOUS BAR SIZE		#	4	

THE DESIGN IS ADEQUATE.

CONCRETE COST =	100	lb / ft^2
TENDONS COST =	0.484	lb / ft^2
REBARS COST =	0.289	lb / ft^2



Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Support	Mid L_4	Right
Span (ft)		20		30		30		20	
DL (psf)		120		120		120		120	
LL (psf)		75		75		75		75	
Balanced DL (60%-80% suggested)		75%		80%		80%		75%	
d_{CGS} (in, from bottom)	4	3.33	7.5	2.5	7.5	2.5	7.5	3.33	4
REQD EFFECTIVE PT (k / ft)		22.31		25.92		25.92		22.31	
Total if banded (kips)		669.4		777.6		777.6		669.4	
Tendons		1 / 2 in Dia @ 14 in o.c.		1 / 2 in Dia @ 12 in o.c.		1 / 2 in Dia @ 12 in o.c.		1 / 2 in Dia @ 14 in o.c.	
Total Number if banded		26		30		30		26	
Top Bars at Column	7 # 5 , L = 3.03 ft		9 # 5 , L = 9.39 ft		22 # 5 , L = 9.39 ft		9 # 5 , L = 9.39 ft		7 # 5 , L = 3.03 ft
Bot Bars, Cont., E. Way		Not ReqD		Not ReqD		# 4 @ 378in. o.c.		Not ReqD	
Required column cap thk, (in)	Not ReqD		Not ReqD		Not ReqD		Not ReqD		Not ReqD

3. DESIGN LOADS & SECTION FORCES

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Support	Mid L_4	Right
M_{DL} (ft-k / ft)	0.00	2.21	-7.59	4.85	-9.71	4.85	-7.59	2.21	0.00
M_{LL} (ft-k / ft)	0.00	1.38	-4.74	3.03	-6.07	3.03	-4.74	1.38	0.00
Balanced Load (psf, uplift)		-90		-96		-96		-90	
Balanced M_{Bal} (ft-k / ft)	0.00	-1.54	5.93	-3.92	7.84	-3.92	5.93	-1.54	0.00
Required Effective PT (k / ft)		22.31		25.92		25.92		22.31	
Tendon Spacing (in)		14		12		12		14	
Primary M_{Fe} (ft-k / ft)	0.00	-1.25	6.51		7.56	-3.24	7.56		
			7.56	-3.24	7.56		6.51	-1.25	0.00
Secondary M_{Sec} (ft-k / ft)	0.00	0.29	0.58		-0.28	0.68	1.63		
			1.63	0.68	-0.28		0.58	0.29	0.00

4. CHECK SERVICE LOAD STRESSES

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Support	Mid L_4	Right		
A (in^2 / ft)	96	96	96	96	96	96	96	96	96		
S (in^3 / ft)	128	128	128	128	128	128	128	128	128		
F / A (ksi)	0.232	0.232	0.232		0.270	0.270	0.270				
			0.270	0.270	0.270		0.232	0.232	0.232		
Check 125 psi < F /A < 275 psi			[Satisfactory]	(ADAPT suggestion)							
M / S + F / A , (ksi)	0.232	0.295	0.077		0.095	0.358	0.114				
for load combination (DL + PT)			0.114	0.358	0.095		0.077	0.295	0.232		
- M / S + F / A , (ksi)	0.232	0.170	0.388		0.445	0.182	0.426				
for load combination (DL + PT)			0.426	0.182	0.445		0.388	0.170	0.232		
Check $f_t < 7.5\ (f_c')^{0.5}$ and $f_c < 0.45\ f_c'$			[Satisfactory]		(ACI 318-08, 18.3.3 & 18.4.2a),		where	$0.45\ f_c'$ =	0.530	$0.45\ f_c'$ =	2.250
M / S + F / A , (ksi)	0.232	0.425	-0.368		-0.474	0.642	-0.330				
for load (DL + LL + PT)			-0.330	0.642	-0.474		-0.368	0.425	0.232		
- M / S + F / A , (ksi)	0.232	0.040	0.833		1.014	-0.102	0.870				
for load (DL + LL + PT)			0.870	-0.102	1.014		0.833	0.040	0.232		
Check $f_t < 7.5\ (f_c')^{0.5}$ and $f_c < 0.6\ f_c'$			[Satisfactory]		(ACI 318-08, 18.3.3 & 18.4.2b),		where	$7.5\ (f_c')^{0.5}$ =	0.530	$0.6\ f_c'$ =	3.000

(cont'd)

5. CALCULATE NON-PRESTRESSED REINFORCEMENT

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Support	Mid L_4	Right
Max. N_c (k / ft), (ACI 318, 18.0)		0.000		0.672		0.672		0.000	
A_S (in^2 / ft), (ACI 318, 18.9.3.2)		0.000		0.000		0.000		0.000	
Bottom Bars, Each Way		Not ReqD		Not ReqD		Not ReqD		Not ReqD	
Max. A_{cf} (in^2), (ACI 318, 18.0)	2880		2880		2880		2880		2880
A_S' (in^2), (ACI 318, 18.9.3.2)	2.160		2.160		2.160		2.160		2.160
Top Bars at Column	7 # 5		7 # 5		7 # 5		7 # 5		7 # 5
L (ft), (ACI 318, 18.9.4.1)	3.03		9.39		9.39		9.39		3.03

6. CHECK FLEXURAL CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Support	Mid L_4	Right
Factored M_u (ft-k / ft)	0.00	5.14	-16.12		-21.63	11.35	-15.06	5.14	0.00
1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{Sec}			-15.06	11.35	-21.63		-16.12		
d_p (in)	4.00	4.67	7.50	5.50	7.50	5.50	7.50	4.67	4.00
ρ_p	0.00273	0.00234	0.00146		0.00170	0.00232	0.00170		
			0.00170	0.00232	0.00170		0.00146	0.00234	0.00273
L / d_p	60.00	51.39	40.00	65.45	48.00	65.45	40.00	51.39	60.00
f_{ps} (ksi)	186.25	187.27	191.59		189.22	186.60	189.22		
(ACI 318, 18.7.2, b & c)			189.22	200.98	189.22		191.59	187.27	186.25
A_{ps} (in^2 / ft)	0.131	0.131	0.131		0.153	0.153	0.153		
Actual Area			0.153	0.15	0.15		0.131	0.131	0.131
d (in)	6.63	6.75	6.63	6.75	6.63	6.75	6.63	6.75	6.63
a (in)	0.19	0.36	0.60		0.83	0.57	0.54		
			0.54	0.58	0.83		0.60	0.36	0.19
Required A_S (in^2 / ft)		0.000		0.000		0.006		0.000	
Bottom Bars, Each Way		Not ReqD		Not ReqD		# 4 @ 378in. o.c.		Not ReqD	
Actual A_S (in^2 / ft)		0.000		0.000		0.006		0.000	
Required A_S' (in^2)	0.000		2.683		6.705		2.683		0.000
Top Bars at Column	Not ReqD		9 # 5		22 # 5		9 # 5		Not ReqD
Actual A_S' (in^2)	2.170		2.790		6.820		2.790		2.170
ϕ M_n (ft-k / ft)	-9.28	8.27	-16.22		-21.74	11.35	-18.36		
Actual Capacity			-18.36	12.02	-21.74		-16.22	8.27	-9.28
Check ϕ M_n > M_u			[Satisfactory]						
ε_{pt}, (ACI 318, 18.8.1)	0.0469	0.0284	0.0271		0.0187	0.0203	0.0302		
$\varepsilon_c(d_p - a / \beta_1) / (a / \beta_1)$			0.0302	0.0199	0.0187		0.0271	0.0284	0.0469
Check ε_{pt} > 0.005			[Satisfactory]		(ACI 318, 18.8.1)				

7. CHECK PUNCHING SHEAR CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Support	Mid L_4	Right
R_{DL} (k)	35.99		90.01		108.00		90.01		35.99
R_{LL} (k)	22.49		56.26		67.50		56.26		22.49
R_{Sec} (k)	-26.99		-70.21		-86.40		-70.21		-26.99
V_u =1.2 R_{DL} + 1.6 R_{LL} + 1.0 Rse	52.18		127.81		151.21		127.81		52.18
Required b_0d, (ACI 318, 11-36)	219.34		527.85		613.74		527.85		219.34
Required d, (ACI 318, 11.0)	3.04		4.90		5.57		4.90		3.04
For ϕ V_n < V_u, the required	0.00		0.00		0.00		0.00		0.00
column cap thickness, t_{cap} (in)	Not ReqD		Not ReqD		Not ReqD		Not ReqD		Not ReqD

8. CALCULATE COST FOR SLAB & CAP

Location		Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Support	Mid of L_{n-1}	Right	
Concrete	cap dim (ft)	1.67		5.00		5.00		5.00		5.00	
	cap concrete	0.00		0.00		0.00		0.00		0.00	
			Total =	**138572**	T / 30 ft TW			Average =	**100**	lb / ft^2	
Tendons	length (ft)		20.01		30.02		30.02		20.01		
			Total =	**670.1**	T / 30 ft TW	, (ACI 318 App. E)		Average =	**0.484**	lb / ft^2 , (AISC Manual 2nd page 7-15)	
Rebars		44.71	0.00	178.27	0.00	435.77	1.30	178.27	0.00	44.71	
			Total =	**400.5**	T / 30 ft TW			Average =	**0.289**	lb / ft^2	

Note:

1. The column moments are negligible for gravity punching design. Lateral loads, seismic and wind, should be supported by shear walls. Using equivalent frames to support lateral loads is not suggested.
2. By inspection, the deflections of slab do not govern PT concrete floor design. Otherwise, using PT concrete floor is inadequate for larger live load. (ACI 318, 9.5.4.1)
3. The secondary moments are very important concept of PT floor design. Based on this concept, PT floor design are always continuous beams design and one way slabs design. So using two ways finite element analysis to design PT floor is inadequate.

Technical References:

1. "Design of Post-Tensioned Slabs Using Unbounded Tendons, Third Edition", The Post-Tensioning Institute, 2004.
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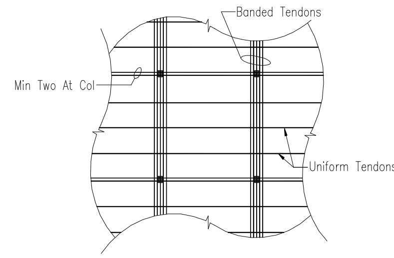
Design of Post-Tensioned Concrete Floor Based on ACI 318-08



1. DESIGN METHODS

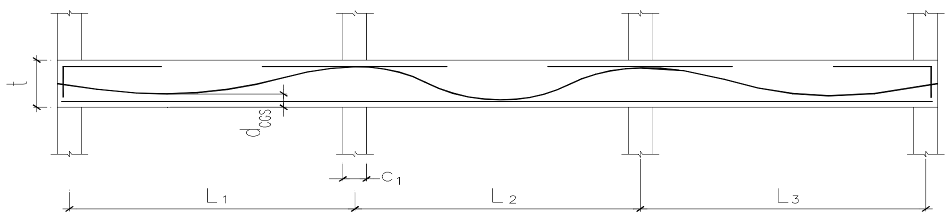
1.1 BREAKDOWN TWO WAYS FLOOR INTO DESIGN STRIPS IN ONE DIRECTION AND ONE WAY SLABS IN OTHER DIRECTION. DESIGN STRIPS WORK AS CONTINUOUS BEAMS BY BANDED ALL TENDONS AT COLUMN. THE PERPENDICULAR DIRECTION LIKE MULTI-SPAN ONE WAY SLABS, USING DISTRIBUTED TENDONS.
1.2 SPECIFY TOTAL REQUIRED EFFECTIVE POST-TENSIONING FORCES AT BANDED TENDONS, ON STRUCTURAL DRAWINGS, AND UNIFORM FORCES IN DISTRIBUTED TENDONS.
1.3 WHEN TENDON LESS THAN 140 FT, STRESS AT ONE END. OTHERWISE STRESS AT BOTH ENDS.
1.4 THE VARIOUS LIVE LOADING CONDITIONS SHOULD BE CONSIDER BY INPUT LL ZERO AT SOME SPANS.

2. INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c	=	5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
TENDON PROPERTIES	f_{pu}	=	270	ksi
	f_{py}	=	243	ksi
	f_{se}	=	175	ksi
	dia	=	1 / 2	in
	A_{ps}	=	0.153	in^2
SLAB THICKNESS	t	=	8	in
TRIBUTARY WIDTH IF BANDED AS DESIGN STRIPS	W	=	30	ft
COLUMN WIDTH	c_1	=	22	in
COLUMN DEPTH	c_2	=	22	in
TOP BAR SIZE AT COLUMN		#	5	
BOTTOM CONTINUOUS BAR SIZE		#	4	

THE DESIGN IS ADEQUATE.

CONCRETE COST =	100	lb / ft^2
TENDONS COST =	0.468	lb / ft^2
REBARS COST =	0.321	lb / ft^2



Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Right
Span (ft)		20		30		20	
DL (psf)		120		120		120	
LL (psf)		75		75		75	
Balanced DL (60%-80% suggested)		75%		80%		75%	
d_{CGS} (in, from bottom)	4	3.33	7.5	2.5	7.5	3.33	4
REQD EFFECTIVE PT (k / ft)		22.31		25.92		22.31	
Total if banded (kips)		669.4		777.6		669.4	
Tendons		1 / 2 in Dia @ 14 in o.c.		1 / 2 in Dia @ 12 in o.c.		1 / 2 in Dia @ 14 in o.c.	
Total Number if banded		26		30		26	
Top Bars at Column	7 # 5 , L = 3.03 ft		15 # 5 , L = 9.39 ft		15 # 5 , L = 9.39 ft		7 # 5 , L = 3.03 ft
Bot Bars, Cont., E. Way		Not ReqD		# 4 @ 58in. o.c.		Not ReqD	
Required column cap thk, (in)	Not ReqD		Not ReqD		Not ReqD		Not ReqD

3. DESIGN LOADS & SECTION FORCES

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Right
M_{DL} (ft-k / ft)	0.00	1.96	-8.08	5.42	-8.08	1.96	0.00
M_{LL} (ft-k / ft)	0.00	1.23	-5.05	3.39	-5.05	1.23	0.00
Balanced Load (psf, uplift)		-90		-96		-90	
Balanced M_{Bal} (ft-k / ft)	0.00	-1.32	6.37	-4.43	6.37	-1.32	0.00
Required Effective PT (k / ft)		22.31		25.92		22.31	
Tendon Spacing (in)		14		12		14	
Primary M_{Fe} (ft-k / ft)	0.00	-1.25	6.51		6.51	-1.25	0.00
			7.56	-3.24	7.56		
Secondary M_{Sec} (ft-k / ft)	0.00	0.07	0.14		0.14	0.07	0.00
			1.19	1.19	1.19		

4. CHECK SERVICE LOAD STRESSES

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Right					
A (in^2 / ft)	96	96	96	96	96	96	96					
S (in^3 / ft)	128	128	128	128	128	128	128					
F / A (ksi)	0.232	0.232	0.232		0.232	0.232	0.232					
			0.270	0.270	0.270							
Check 125 psi < F /A < 275 psi			**[Satisfactory]**	(ADAPT suggestion)								
M / S + F / A , (ksi)	0.232	0.293	0.072		0.072	0.293	0.232					
for load combination (DL + PT)			0.110	0.363	0.110							
- M / S + F / A , (ksi)	0.232	0.172	0.393		0.393	0.172	0.232					
for load combination (DL + PT)			0.430	0.177	0.430							
Check f_t < 7.5 $(f_c')^{0.5}$ and f_c < 0.45 f_c'			**[Satisfactory]**		(ACI 318-08, 18.3.3 & 18.4.2a),			where	0.45 f_c' =	0.530	0.45 f_c' =	2.250
M / S + F / A , (ksi)	0.232	0.408	-0.401		-0.401	0.408	0.232					
for load (DL + LL + PT)			-0.363	0.681	-0.363							
- M / S + F / A , (ksi)	0.232	0.057	0.866		0.866	0.057	0.232					
for load (DL + LL + PT)			0.903	-0.141	0.903							
Check f_t < 7.5 $(f_c')^{0.5}$ and f_c < 0.6 f_c'			**[Satisfactory]**		(ACI 318-08, 18.3.3 & 18.4.2b),			where	7.5 $(f_c')^{0.5}$ =	0.530	0.6 f_c' =	3.000

(cont'd)

5. CALCULATE NON-PRESTRESSED REINFORCEMENT

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Right
Max. N_c (k / ft), (ACI 318, 18.0)		0.000		1.158		0.000	
A_s (in^2 / ft), (ACI 318, 18.9.3.2)		0.000		0.000		0.000	
Bottom Bars, Each Way		Not ReqD		Not ReqD		Not ReqD	
Max. A_{cf} (in^2), (ACI 318, 18.0)	2880		2880		2880		2880
A_s' (in^2), (ACI 318, 18.9.3.2)	2.160		2.160		2.160		2.160
Top Bars at Column	7 # 5		7 # 5		7 # 5		7 # 5
L (ft), (ACI 318, 18.9.4.1)	3.03		9.39		9.39		3.03

6. CHECK FLEXURAL CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Right
Factored M_u (ft-k / ft)	0.00	4.39	-17.63		-17.63	4.39	0.00
1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{Sec}			-16.58	13.12	-16.58		
d_p (in)	4.00	4.67	7.50	5.50	7.50	4.67	4.00
ρ_p	0.00273	0.00234	0.00146		0.00146	0.00234	0.00273
			0.00170	0.00232	0.00170		
L / d_p	60.00	51.39	40.00	65.45	40.00	51.39	60.00
f_{ps} (ksi)	186.25	187.27	191.59		191.59	187.27	186.25
(ACI 318, 18.7.2, b & c)			189.22	200.98	189.22		
A_{ps} (in^2 / ft)	0.131	0.131	0.131		0.131	0.131	0.131
Actual Area			0.153	0.15	0.15		
d (in)	6.63	6.75	6.63	6.75	6.63	6.75	6.63
a (in)	0.19	0.33	0.66		0.66	0.33	0.19
			0.61	0.65	0.61		
Required A_s (in^2 / ft)		0.000		0.041		0.000	
Bottom Bars, Each Way		Not ReqD		# 4 @ 58in. o.c.		Not ReqD	
Actual A_s (in^2 / ft)		0.000		0.041		0.000	
Required A_s' (in^2)	0.000		4.367		4.367		0.000
Top Bars at Column	Not ReqD		15 # 5		15 # 5		Not ReqD
Actual A_s' (in^2)	2.170		4.650		4.650		2.170
ϕ M_n (ft-k / ft)	-9.28	8.30	-17.90		-17.90	8.30	-9.28
Actual Capacity			-20.03	13.13	-20.03		
Check ϕ M_n > M_u			[Satisfactory]				
ε_{pt}, (ACI 318, 18.8.1)	0.0469	0.0314	0.0241		0.0241	0.0314	0.0469
$\varepsilon_c(d_p$ - a / β_1) / (a / β_1)			0.0267	0.0173	0.0267		
Check ε_{pt} > 0.005			[Satisfactory]		(ACI 318, 18.8.1)		

7. CHECK PUNCHING SHEAR CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_3	Right
R_{DL} (k)	35.99		90.01		90.01		35.99
R_{LL} (k)	22.49		56.26		56.26		22.49
R_{Sec} (k)	-26.99		-70.21		-70.21		-26.99
V_u =1.2 R_{DL} + 1.6 R_{LL} + 1.0 Rse	52.18		127.82		127.82		52.18
Required b_0d, (ACI 318, 11-36)	219.34		527.86		527.86		219.34
Required d, (ACI 318, 11.0)	3.04		4.90		4.90		3.04
For ϕ V_n < V_u, the required	0.00		0.00		0.00		0.00
column cap thickness, t_{cap} (in)	Not ReqD		Not ReqD		Not ReqD		Not ReqD

8. CALCULATE COST FOR SLAB & CAP

Location		Left	Mid of L_1	Support	Mid of L_2	Support	Mid of L_{Typ}	Right		
Concrete	cap dim (ft)	1.67		5.00		5.00		5.00		
	cap concrete	0.00		0.00		0.00		0.00		
			Total =	**97749**	T / 30 ft TW			Average =	**100**	lb / ft^2
Tendons	length (ft)		20.01		30.02		20.01			
			Total =	**457.8**	T / 30 ft TW	, (ACI 318 App. E)		Average =	**0.468**	lb / ft^2 , (AISC Manual 2nd page 7-15)
Rebars		44.71	0.00	297.12	8.45	297.12	0.00	44.71		
			Total =	**313.9**	T / 30 ft TW			Average =	**0.321**	lb / ft^2

Note:

1. The column moments are negligible for gravity punching design. Lateral loads, seismic and wind, should be supported by shear walls. Using equivalent frames to support lateral loads is not suggested.
2. By inspection, the deflections of slab do not govern PT concrete floor design. Otherwise, using PT concrete floor is inadequate for larger live load. (ACI 318, 9.5.4.1)
3. The secondary moments are very important concept of PT floor design. Based on this concept, PT floor design are always continuous beams design and one way slabs design. So using two ways finite element analysis to design PT floor is inadequate.

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Daniel T. Li Engineering International	PROJECT : CLIENT : JOB NO. : DATE :	PAGE : DESIGN BY : REVIEW BY :

Design of Post-Tensioned Concrete Floor Based on ACI 318-08

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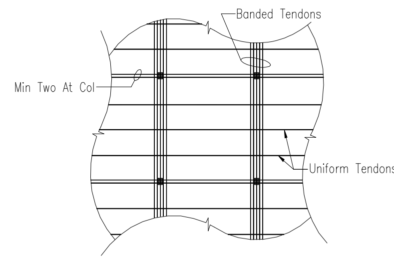
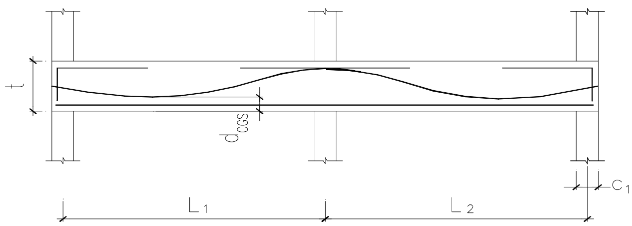


2. INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c	=	5	ski
REBAR YIELD STRESS	f_y	=	60	ski
TENDON PROPERTIES	f_{pu}	=	270	ski
	f_{py}	=	243	ski
	f_{se}	=	175	ski
	dia	=	1 / 2	in
	A_{ps}	=	0.153	in^2
SLAB THICKNESS	t	=	8	in
TRIBUTARY WIDTH IF BANDED AS DESIGN STRIPS	W	=	30	ft
COLUMN WIDTH	c_1	=	22	in
COLUMN DEPTH	c_2	=	22	in
TOP BAR SIZE AT COLUMN		#	5	
BOTTOM CONTINUOUS BAR SIZE		#	4	

THE DESIGN IS ADEQUATE.

CONCRETE COST =	100	lb / ft^2
TENDONS COST =	0.431	lb / ft^2
REARS COST =	0.157	lb / ft^2



Location	Left	Mid of L_1	Support	Mid L_2	Right
Span (ft)		20		20	
DL (psf)		120		120	
LL (psf)		75		75	
Balanced DL (60%-80% suggested)		75%		75%	
d_{CGS} (in, from bottom)	4	3.33	7.5	3.33	4
REQD EFFECTIVE PT (k / ft)		22.31		22.31	
Total if banded (kips)		669.4		669.4	
Tendons		1 / 2 in Dia @ 14 in o.c.		1 / 2 in Dia @ 14 in o.c.	
Total Number if banded		26		26	
Top Bars at Column	7 # 5 , L = 3.03 ft		7 # 5 , L = 6.06 ft		7 # 5 , L = 3.33 ft
Bot Bars, Cont., E. Way		# 4 @ 73in. o.c.		# 4 @ 73in. o.c.	
Required column cap thk, (in)	Not ReqD		Not ReqD		Not ReqD

3. DESIGN LOADS & SECTION FORCES

Location	Left	Mid of L_1	Support	Mid L_2	Right
M_{DL} (ft-k / ft)	0.00	3.50	-5.00	3.50	0.00
M_{LL} (ft-k / ft)	0.00	2.19	-3.13	2.19	0.00
Balanced Load (psf, uplift)		-90		-90	
Balanced M_{Bal} (ft-k / ft)	0.00	-2.63	3.75	-2.63	0.00
Required Effective PT (k / ft)		22.31		22.31	
Tendon Spacing (in)		14		14	
Primary M_{Fe} (ft-k / ft)	0.00	-1.25	6.51		
			6.51	-1.25	0.00
Secondary M_{Sec} (ft-k / ft)	0.00	1.38	2.76		
			2.76	1.38	0.00

4. CHECK SERVICE LOAD STRESSES

Location	Left	Mid of L_1	Support	Mid L_2	Right
A (in^2 / ft)	96	96	96	96	96
S (in^3 / ft)	128	128	128	128	128
F / A (ksi)	0.232	0.232	0.232		
			0.232	0.232	0.232
Check 125 psi < F /A < 275 psi			[Satisfactory]	(ADAPT suggestion)	
M / S + F / A , (ksi)	0.232	0.314	0.115		
for load combination (DL + PT)			0.115	0.314	0.232
- M / S + F / A , (ksi)	0.232	0.150	0.350		
for load combination (DL + PT)			0.350	0.150	0.232
Check f_t < 7.5 $(f_c')^{0.5}$ and f_c < 0.45 f_c'			[Satisfactory]	(ACI 318-08, 18.3.3 & 18.4.2a), where 0.45 f_c' = 0.530 0.45 f_c' = 2.250	
M / S + F / A , (ksi)	0.232	0.520	-0.178		
for load (DL + LL + PT)			-0.178	0.520	0.232
- M / S + F / A , (ksi)	0.232	-0.055	0.643		
for load (DL + LL + PT)			0.643	-0.055	0.232
Check f_t < 7.5 $(f_c')^{0.5}$ and f_c < 0.6 f_c'			[Satisfactory]	(ACI 318-08, 18.3.3 & 18.4.2b), where 7.5 $(f_c')^{0.5}$ = 0.530 0.6 f_c' = 3.000	

(cont'd)

5. CALCULATE NON-PRESTRESSED REINFORCEMENT

Location	Left	Mid of L_1	Support	Mid L_2	Right
Max. N_c (k / ft), (ACI 318, 18.0)		0.250		0.250	
A_s (in^2 / ft), (ACI 318, 18.9.3.2)		0.000		0.000	
Bottom Bars, Each Way		Not ReqD		Not ReqD	
Max. A_{cf} (in^2), (ACI 318, 18.0)	2880		2880		2880
A_s' (in^2), (ACI 318, 18.9.3.2)	2.160		2.160		2.160
Top Bars at Column	7 # 5		7 # 5		7 # 5
L (ft), (ACI 318, 18.9.4.1)	3.03		6.06		3.33

6. CHECK FLEXURAL CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid L_2	Right
Factored M_u (ft-k / ft)	0.00	9.08	-8.24	9.08	0.00
$1.2\ M_{DL} + 1.6\ M_{LL} + 1.0\ M_{Sec}$			-8.24		
d_p (in)	4.00	4.67	7.50	4.67	4.00
ρ_p	0.00273	0.00234	0.00146		
			0.00146	0.00234	0.00273
L / d_p	60.00	51.39	32.00	51.39	60.00
f_{ps} (ksi)	186.25	187.27	214.46		
(ACI 318, 18.7.2, b & c)			191.59	187.27	186.25
A_{ps} (in^2 / ft)	0.131	0.131	0.131		
Actual Area			0.131	0.131	0.131
d (in)	6.63	6.75	6.63	6.75	6.63
a (in)	0.19	0.52	0.26		
			0.27	0.52	0.19
Required A_s (in^2 / ft)		0.033		0.033	
Bottom Bars, Each Way		# 4 @ 73in. o.c.		# 4 @ 73in. o.c.	
Actual A_s (in^2 / ft)		0.033		0.033	
Required A_s' (in^2)	0.000		0.000		0.000
Top Bars at Column	Not ReqD		0 # 5		Not ReqD
Actual A_s' (in^2)	2.170		2.170		2.170
ϕ M_n (ft-k / ft)	-9.28	9.08	-17.66		
Actual Capacity			-16.00	9.08	-9.28
Check ϕ $M_n > M_u$			**[Satisfactory]**		
ε_{pt}, (ACI 318, 18.8.1)	0.0469	0.0186	0.0669		
$\varepsilon_c(d_p - a / \beta_1) / (a / \beta_1)$			0.0648	0.0186	0.0469
Check $\varepsilon_{pt} > 0.005$			**[Satisfactory]**		(ACI 318, 18.8.1)

7. CHECK PUNCHING SHEAR CAPACITY BY STRENGTH DESIGN METHOD

Location	Left	Mid of L_1	Support	Mid L_2	Right
R_{DL} (k)	35.99		72.02		35.99
R_{LL} (k)	22.50		45.01		22.50
R_{Sec} (k)	-26.99		-54.01		-26.99
$V_u = 1.2\ R_{DL} + 1.6\ R_{LL} + 1.0$ Rse	52.19		104.42		52.19
Required b_0d, (ACI 318, 11-36)	219.36		438.91		219.36
Required d, (ACI 318, 11.0)	3.04		4.19		3.04
For ϕ $V_n < V_u$, the required	0.00		0.00		0.00
column cap thickness, t_{cap} (in)	Not ReqD		Not ReqD		Not ReqD

8. CALCULATE COST FOR SLAB & CAP

Location		Left	Mid of L_1	Support	Mid of L_2	Right				
Concrete	cap dim (ft)	1.67		5.00		5.00				
	cap concrete	0.00		0.00		0.00				
			Total =	**56926**	T / 30 ft TW		Average =	**100**	lb / ft^2	
Tendons	length (ft)		20.01		20.01					
			Total =	**245.4**	T / 30 ft TW	, (ACI 318 App. E)	Average =	**0.431**	lb / ft^2 , (AISC Manual 2nd page 7-15)	
Rebars		44.71	6.71	89.43	6.71	49.23				
			Total =	**89.3**	T / 30 ft TW		Average =	**0.157**	lb / ft^2	

Note:

1. The column moments are negligible for gravity punching design. Lateral loads, seismic and wind, should be supported by shear walls. Using equivalent frames to support lateral loads is not suggested.
2. By inspection, the deflections of slab do not govern PT concrete floor design. Otherwise, using PT concrete floor is inadequate for larger live load. (ACI 318, 9.5.4.1)
3. The secondary moments are very important concept of PT floor design. Based on this concept, PT floor design are always continuous beams design and one way slabs design. So using two ways finite element analysis to design PT floor is inadequate.

Technical References:

1. "Design of Post-Tensioned Slabs Using Unbounded Tendons, Third Edition", The Post-Tensioning Institute, 2004.
2. "Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures, First Edition", The Post-Tensioning Institute, 2001.
3. Bijan O. Aalami & Allan Bommer, "Design Fundamentals of Post-Tensioned Concrete Floors, First Edition", The Post-Tensioning Institute, 1999.

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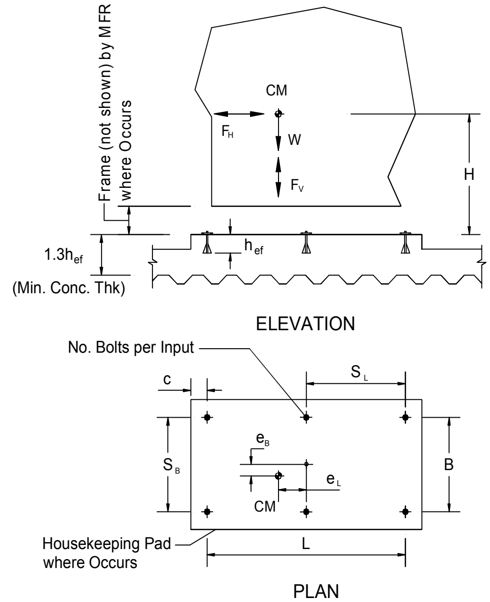
Design for Equipment Anchorage to Bottom Concrete Based on IBC 09 / CBC 10 Chapter A

INPUT DATA & DESIGN SUMMARY



EQUIPMENT WEIGHT	W	=	26	kips
CENTER OF MASS	H	=	4	ft
ECCENTRICITY	e_L	=	0.5	ft
	e_B	=	0.5	ft
KWIK BOLT-TZ DIAMETER	ϕ	=	3/4	in. Per ICC ESR-1917
ANCHOR DEPTH	h_{ef}	=	4 3/4	in. Per ICC ESR-1917
EDGE DISTANCE	c	=	12	in
ANCHORAGE LENGTH	L	=	12.33	ft
BOLTS ALONG ***L*** EDGE	N_L	=	3	per line
ANCHOR SPACING	$S_L = L / (N_L - 1) =$		74	in
ANCHORAGE WIDTH	B	=	5	ft
BOLTS ALONG ***B*** EDGE	N_B	=	2	per line
ANCHOR SPACING	$S_B = B / (N_B - 1) =$		60	in

[THE ANCHORAGE, KWIK BOLT-TZ, DESIGN IS ADEQUATE.]

ANALYSIS

ALLOWABLE TENSION & SHEAR VALUES (ICC ESR-1917, Table 9 or 10)

P_t = 4933 lbs

V_t = 6313 lbs

SPACING & EDGE REQUIREMENTS (ICC ESR-1917, Table 3 or 4)

S_{cr} = 7 3/4 in, (Critical Spacing for $K_{edge\text{-}space}$ based on ACI 318-08 Appendix D)

S_{min} = 4 in

C_{cr} = 8 7/8 in, shear, 8 7/8 in, tension

C_{min} = 4 1/8 in, shear, 4 1/8 in, tension

DESIGN LOADS

$F_H = F_p =$ (K_H) MAX{ $0.3S_{DS}I_pW$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p$ W , $1.6S_{DS}I_pW$] } , (ASCE 7-05, Sec. 13.3.1)

= 1.3 MAX{ 0.43W , MIN[0.81W , 2.30W]}

= 1.05 W , (SD)

= 0.75 W , (ASD) = 19.57 kips

$F_V = K_V$ W = 0.18 W , (ASD) = 4.64 kips, up & down

where

S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
I_p =	1.5	(ASCE Sec. 13.1.3)
a_p =	1	(ASCE Tab. 13.6-1)
R_p =	1.5	(ASCE Tab. 13.6-1)
z =	20	ft
h =	36	ft
K_H =	1.3	(ASCE Sec. 13.4.2a)
$K_V = K_H\ 0.2\ S_{DS} / 1.4$ =	0.18	(vertical seismic factor)

MAXIMUM OVERTURNING MOMENT AT ANCHOR EDGE

$M_{OT} = F_p H + (0.9W - F_v) e_B =$ 87.67 ft-kips

$M_{RES} = (0.9W - F_v)(0.5B) =$ 46.91 ft-kips < M_{OT} ,therefore design tension anchors.

CHECK TENSION CAPACITY

$P_s = [(F_V - 0.9 W) / A + M_{OT}\ y / I] =$ 2717 lbs / bolt < $P_t\ K_{DSA}\ K_{seismic}\ K_{edge\text{-}space}$ [SATISFACTORY]

where $A = 2(N_L + N_B) - 4 =$ 6 (total bolts)

$I = MIN(\Sigma X_i^2 , \Sigma Y_i^2) =$ 5400 in^2-bolts (B direction governs)

$y = 0.5$ B = 30 in

K_{DSA} = 0.8 (DSA/OSHPD adapted ICBO / ICC value)

$K_{seismic}$ = 1 1/3 (allowable increase? CBC 1605A.3.2)

$K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

CHECK SHEAR CAPACITY

$V_s = F_H / A =$ 3262 lbs / bolt < $V_t\ K_{DSA}\ K_{seismic}\ K_{edge\text{-}space}$ [SATISFACTORY]

where $K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

CHECK COMBINED LOADING CAPACITY (ICC ESR-1917 Sec. 4.2.2)

$(P_s / P_t) + (V_s / V_t) =$ 1.001 < 1.20 [SATISFACTORY]

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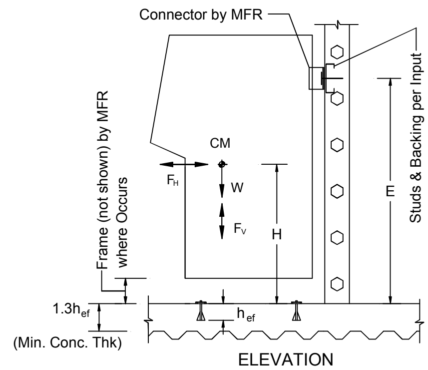
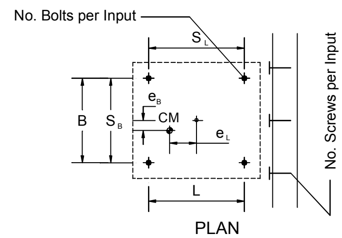
Design for Equipment Anchorage to Bottom Concrete & Mounting on Metal Wall Based on IBC 09 / CBC 10 Chapter A

INPUT DATA & DESIGN SUMMARY

EQUIPMENT WEIGHT	W	=	3.4	kips
CENTER OF MASS	H	=	3.5	ft
ECCENTRICITY	e_L	=	0.2	ft
	e_B	=	0.2	ft
KWIK BOLT-TZ DIAMETER	ϕ	=	1/2	in. Per ICC ESR-1917
ANCHOR DEPTH	h_{ef}	=	2	in. Per ICC ESR-1917
EDGE DISTANCE	c	=	48	in
ANCHORAGE LENGTH	L	=	3	ft
BOLTS ALONG ***L*** EDGE	N_L	=	2	per line
ANCHOR SPACING	$S_L = L / (N_L - 1) =$		36	in
ANCHORAGE WIDTH	B	=	3	ft
BOLTS ALONG ***B*** EDGE	N_B	=	2	per line
ANCHOR SPACING	$S_B = B / (N_B - 1) =$		36	in
SCREW SIZE (#8, #10, #12)		#	10	
WALL METAL GAUGE (18GA, 16GA, 14GA)			16	GA, (54 mils)
TOTAL SCREW NUMBERS	N_E	=	8	along horiz. line
MOUNTING HEIGHT	E	=	4	ft





[THE ANCHORAGE & MOUNTING DESIGN IS ADEQUATE.]

ANALYSIS

BOLT-TZ ALLOWABLE TENSION & SHEAR VALUES (ICC ESR-1917, Table 9 or 10)

P_t = 1167 lbs

V_t = 2239 lbs

BOLT-TZ SPACING & EDGE REQUIREMENTS (ICC ESR-1917, Table 3 or 4)

S_{cr} = 4 1/8 in, (Critical Spacing for $K_{edge\text{-}space}$ based on ACI 318-08 Appendix D)

S_{min} = 2 3/4 in

C_{cr} = 5 1/2 in, shear, 5 1/2 in, tension

C_{min} = 2 3/4 in, shear, 2 3/4 in, tension

DESIGN LOADS

$F_H = F_p =$ (K_H) MAX{ $0.3S_{DS}I_pW$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p$ W , $1.6S_{DS}I_pW$] } , (ASCE 7-05, Sec. 13.3.1)

= 1.3 MAX{ 0.43W , MIN[0.81W , 2.30W]}

= 1.05 W , (SD)

= 0.75 W , (ASD) = 2.56 kips

$F_V = K_V W$ = 0.18 W , (ASD) = 0.61 kips, up & down

$K_V = K_H\ 0.2\ S_{DS} / 1.4 =$ 0.18 (vertical seismic factor)

where

S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
I_p =	1.5	(ASCE Sec. 13.1.3)
a_p =	1	(ASCE Tab. 13.6-1)
R_p =	1.5	(ASCE Tab. 13.6-1)
z =	20	ft
h =	36	ft
K_H =	1.3	(ASCE Sec. 13.4.2a)

OVERTURNING MOMENT AT ANCHOR EDGE AT PARALLEL WALL DIRECTION

$M_{OT} = F_p\ H + (0.9W - F_v)\ e_B =$ 9.45 ft-kips

$M_{RES} = (0.9W - F_v)\ (0.5B) =$ 3.68 ft-kips < M_{OT} ,therefore design tension anchors.

CHECK BOLT-TZ TENSION CAPACITY

$P_s = [(F_V - 0.9\ W) / A + M_{OT}\ y / I\ ] =$ 961 lbs / bolt < $P_t\ K_{DSA}\ K_{seismic}\ K_{edge\text{-}space}$ [SATISFACTORY]

where $A = 2(N_L + N_B) - 4 =$ 4 (total bolts)

$I = MIN(\Sigma X_i^2 , \Sigma Y_i^2) =$ 1296 in^2-bolts (L direction governs)

$y = 0.5\ L =$ 18 in

K_{DSA} = 0.8 (DSA/OSHPD adapted ICBO / ICC value)

$K_{seismic}$ = 1 1/3 (allowable increase? IBC 1605A.3.2)

$K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

CHECK BOLT-TZ SHEAR CAPACITY

$V_s = F_H / A =$ 640 lbs / bolt < $V_t\ K_{DSA}\ K_{seismic}\ K_{edge\text{-}space}$ [SATISFACTORY]

where $K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

CHECK BOLT-TZ COMBINED LOADING CAPACITY (ICC ESR-1917 Sec. 4.2.2)

$(P_s / P_t) + (V_s / V_t) =$ 1.040 < 1.20 [SATISFACTORY]

OVERTURNING MOMENT AT ANCHOR EDGE FOR PERPENDICULAR WALL DIRECTION

$M_{OT} = F_p\ H + (0.9W - F_v)\ e_L =$ 9.45 ft-kips

$M_{RES} = (0.9W - F_v)\ (0.5L) =$ 3.68 ft-kips < M_{OT} ,therefore design tension screws.

CHECK SCREW TENSION CAPACITY(ER-4943P, SSMA page 48)

$T = (M_{OT} - M_{RES}) / E =$ 1442 lbs / total screws < 1 1/3 **x** 8 screws **x** 137 lbs / screw

[SATISFACTORY] = 1461 lbs

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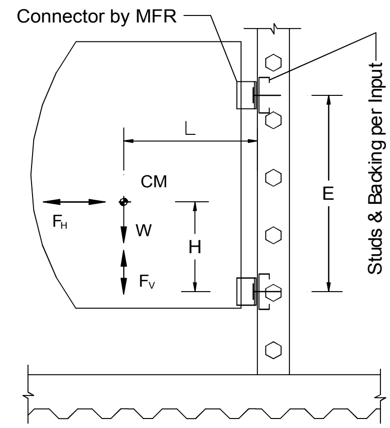
Design for Equipment Mounting on Metal Wall Based on IBC 09 / CBC 10 Chapter A

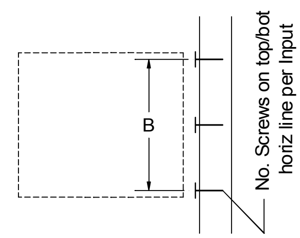
INPUT DATA & DESIGN SUMMARY

EQUIPMENT WEIGHT	W	=	1.3	kips
CENTER OF MASS	H	=	2.7	ft
	L	=	1.5	ft
SCREW SIZE (#8, #10, #12)		#	10	
WALL METAL GAUGE (18GA, 16GA, 14GA)			16	GA, (54 mils)
TOP SCREW NUMBERS	N_T	=	8	along top horiz. line
BOT. SCREW NUMBERS	N_B	=	2	along bottom horiz. line
VERTICAL SPACING	E	=	4	ft
VERT. EDGE SCREW NUMBERS	N_V	=	3	along vertical edge
VERT. EDGE SCREW DIST.	B	=	2.5	ft
SITE ON FLOOR? (Yes, No)		== >	No	, hounted on wall only



ELEVATION

[THE MOUNTING DESIGN IS ADEQUATE.]



PLAN

ANALYSIS

DESIGN LOADS

$F_H = F_p =$ (K_H) MAX{ $0.3S_{DS}I_pW$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p$ W , $1.6S_{DS}I_pW$] } , (ASCE 7-05, Sec. 13.3.1)

= 1.3 MAX{ 0.43W , MIN[0.81W , 2.30W]}

= 1.05 W , (SD)

= 0.75 W , (ASD) = 0.98 kips

$F_V = K_V W$ = 0.18 W , (ASD) = 0.23 kips, up & down

$K_V = K_H\ 0.2\ S_{DS} / 1.4 =$ 0.18 (vertical seismic factor)

where

S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
I_p =	1.5	(ASCE Sec. 13.1.3)
a_p =	1	(ASCE Tab. 13.6-1)
R_p =	1.5	(ASCE Tab. 13.6-1)
z =	20	ft
h =	36	ft
K_H =	1.3	(ASCE Sec. 13.4.2a)

OVERTURNING MOMENT AT BOTTOM HORIZONTAL SCREWS

$M_{OT} = F_p\ H + (W + F_V)\ L =$ 4.94 ft-kips

TORSION AT VERTICAL EDGE SCREWS

$M_T = F_p\ 0.5\ B =$ 1.22 ft-kips

CHECK SCREW TENSION CAPACITY(ER-4943P, SSMA page 48)

$P_{nt,OM} = M_{OT} / E =$ 1235 lbs / top screws < 1 1/3 x 8 screws x 137 lbs / screw
[SATISFACTORY] = 1461 lbs

$P_{nt,T} = M_T / B =$ 489 lbs / vert. edge < 1 1/3 x 3 screws x 137 lbs / screw
[SATISFACTORY] = 548 lbs

CHECK SCREW SHEAR CAPACITY(ER-4943P, SSMA page 48)

$P_{ns,V} = W + F_V =$ 1532 lbs / total screws < 1 1/3 x 12 screws x 370 lbs / screw
[SATISFACTORY] = 5920 lbs

$P_{ns,H} = F_H =$ 979 lbs / total screws < 1 1/3 x 12 screws x 370 lbs / screw
[SATISFACTORY] = 5920 lbs

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Suspended Anchorage to Concrete Based on 2009 IBC / 2010 CBC

INPUT DATA & DESIGN SUMMARY

EQUIPMENT WEIGHT

W = 3.5 kips

CENTER OF MASS

H = 4 ft

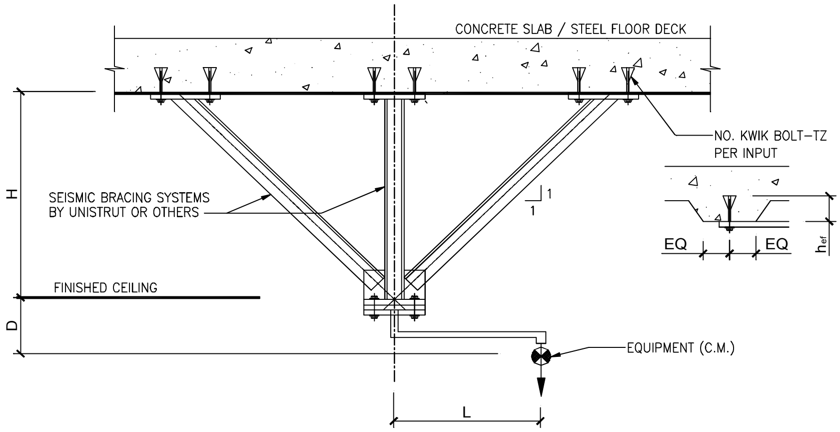
D = 3.5 ft

L = 4.5 ft

NUMBER OF BOLT - TZ

n_{brace} = 4

n_{post} = 4



USE, 5/8 DIA - 4 in EMB KWIK BOLT - TZ, 4 AT EACH BRACE, 4 AT POST (TOTAL 20 BOLTS)

ANALYSIS

KWIK BOLT - TZ DIAMETER ϕ = 5/8 in. Per ICC ESR-1917

ANCHOR DEPTH h_{ef} = 4 in. Per ICC ESR-1917

ALLOWABLE TENSION & SHEAR VALUES (ICC ESR-1917, Table 11)

P_t = 2255 lbs

V_t = 2677 lbs

SPACING & EDGE REQUIREMENTS (ICC ESR-1917, Table 3 or 4)

S_{cr} = 8 3/4 in

S_{min} = 3 1/2 in

C_{cr} = 5 7/8 in, shear, 5 7/8 in, tension

C_{min} = 3 1/4 in, shear, 3 1/4 in, tension

DESIGN LOADS

$F_H = F_p =$ (K_H) MAX{ $0.3S_{DS}I_pW$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p W$, $1.6S_{DS}I_pW$] } , (ASCE 7-05, Sec. 13.3.1)

= 1.3 MAX{ 0.24W , MIN[0.65W , 1.30W]}

= 0.84 W , (SD)

= 0.60 W , (ASD) = 2.11 kips

$F_V = K_V W =$ 0.10 W , (ASD) = 0.35 kips, up & down

where

S_{DS} =	0.54	(ASCE 7-05 Sec 11.4.4)
I_p =	1.5	(ASCE Sec. 13.1.3)
a_p =	1	(ASCE Tab. 13.5-1)
R_p =	1.5	(ASCE Tab. 13.5-1)
z =	h	ft
h =	36	ft
K_H =	1.3	(ASCE Sec. 13.4.2a)
$K_V = K_H\ 0.2\ S_{DS} / 1.4$ =	0.10	(vertical seismic factor)

CHECK TENSION CAPACITY

$P_s = [(F_V + W) / A + M\ y / I] =$ 2263 lbs / bolt < $P_t\ K_{DSA}\ K_{seismic}\ K_{edge\text{-}space}$ [SATISFACTORY]

where A = 20 (total bolts)

$M = F_p\ (H + D) + (W + F_v)\ L =$ 33.12 ft-kips

$I = MIN(\Sigma X_i^2\ , \Sigma Y_i^2) =$ 9216 in^2-bolts

y = H = 48 in

K_{DSA} = 0.8 (DSA/OSHPD adapted ICBO / ICC value)

$K_{seismic}$ = 1 1/3 (allowable increase? IBC 1605A.3.2)

$K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

c = 48 in, min distance from bolt to slab edge

S = 12 in, min bolt spacing

CHECK SHEAR CAPACITY

$V_s = F_H / A =$ 105 lbs / bolt < $V_t\ K_{DSA}\ K_{seismic}\ K_{edge\text{-}space}$ [SATISFACTORY]

where $K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

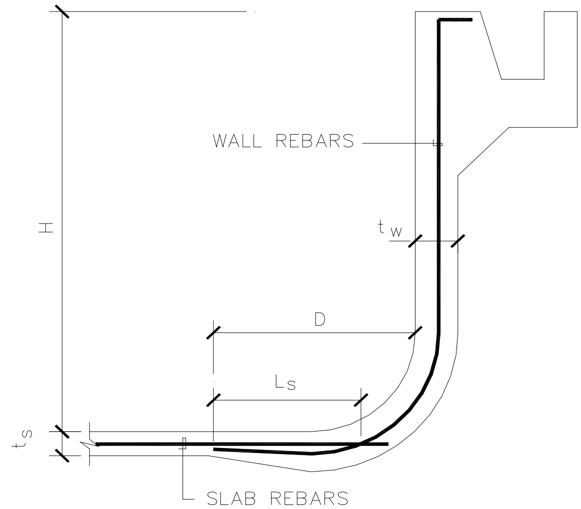
CHECK COMBINED LOADING CAPACITY (ICC ESR-1917 Sec. 4.2.2)

$(P_s / P_t) + (V_s / V_t) =$ 0.978 < 1.20 [SATISFACTORY]

Concrete Pool Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
LATERAL SOIL PRESSURE	P_a	=	45	pcf
(equivalent fluid pressure)				
BACKFILL WEIGHT	γ_b	=	110	pcf
SURCHARGE WEIGHT	w_s	=	50	psf
SEISMIC GROUND SHAKING	P_E	=	20	psf /ft, ASD
(soil pressure, if no report $35S_{DS}$ suggested.)				
POOL DEPTH	H	=	6	ft
THICKNESS OF WALL	t_w	=	8	in
THICKNESS OF SLAB	t_s	=	6	in
SLAB REBARS	# 5	@	10	in o.c. at mid
WALL VERTICAL REBARS	# 5	@	8	in o.c.
WALL BAR LOCATION (1=at middle, 2=at each face)			1	at middle
LAP LENGTH	L_s	=	36	in
SLAB THICKER DISTANCE	D	=	4	ft



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

DESIGN CRITERIA

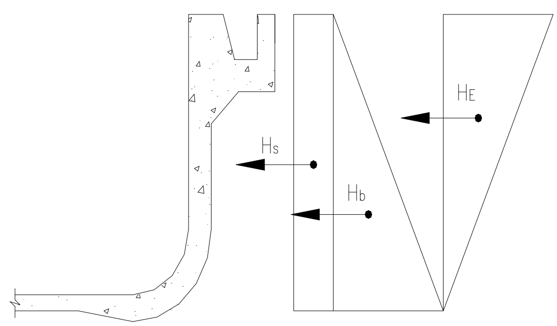
1. THE CRITERICAL DESIGN, FOR REBAR AT MIDDLE OR EQUAL OF EACH FACE, IS POOL WALL AT INWARD SOIL PRESSURE BEFORE RESTRAINED AT TOP AND POOL FILLED.
2. SINCE THE WALL AXIAL LOAD SMALL AND SECTIONS UNDER TENSION-CONTROLLED (ACI 318-08, 10.3.4), ONLY CHECK WALL FLEXURAL CAPACITIES ARE ADEQUATE.
3. SINCE THE SLAB AT FLEXURAL & AXIAL LOADS, THE COMBINED CAPACITY OF FLEXURAL & AXIAL MUST BE CHECKED.

SERVICE LOADS

$H_b = 0.5\, P_a\, (H + t_s)^2$ = 0.95 kips / ft

$H_s = w_s\, P_a\, (H + t_s) / \gamma_b$ = 0.13 kips / ft

$H_E = 0.5\, P_E\, (H + t_s)^2$ = 0.42 kips / ft



FACTORED LOADS

$\gamma H_b = 1.6\, H_b$ = 1.52 kips / ft

$\gamma H_s = 1.6\, H_s$ = 0.21 kips / ft

$\gamma H_E = 1.6\, H_E$ = 0.68 kips / ft

CHECK WALL FLEXURE CAPACITY (ACI 318-08, 15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$M_u = (0.5\, \gamma H_s + 0.33\, \gamma H_b + 0.67\, \gamma H_E)\, H$ = 6.38 ft-kips / ft, (entire lateral loads used conservatively)

P_u = 1.19 kips / ft, (concrete wall self weight)

d = 4.00 in, b = 12 in, A_s = 0.465 in^2 / ft

$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7 b f_c'}\right)\right] = 7.46 \text{ ft-kips / ft} > M_u$$

[Satisfactory]

ρ_{ProvD} = 0.010 < ρ_{MAX} = 0.015
> ρ_{MIN} = 0.004 **[Satisfactory]**

CHECK WALL SHEAR CAPACITY (ACI 318-08, 15.5.2, 11.1.3.1, & 11.2)

$V_u = \gamma H_s + \gamma H_b + \gamma H_E$ = 2.41 kips / ft, (entire lateral loads used conservatively)

$$\phi V_n = 2\phi b d \sqrt{f_c'} = 3.94 \text{ kips / ft} > V_u$$

[Satisfactory]

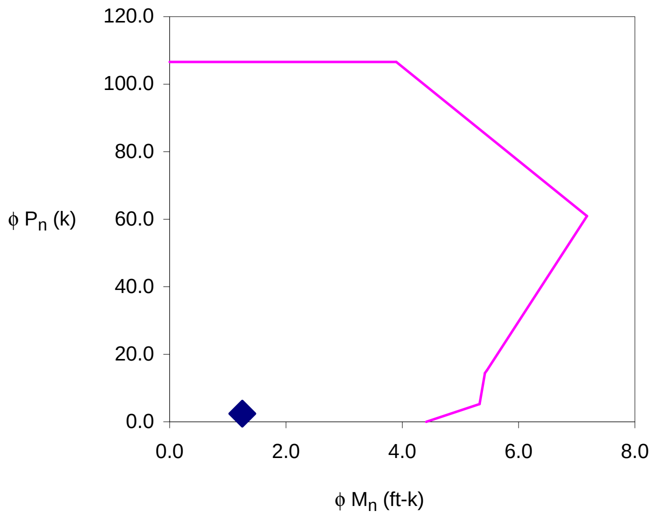
(cont'd)

CHECK SLAB COMBINED CAPACITY OF FLEXURE & AXIAL (ACI 318-08, 10)

ρ_{ProvD} = 0.01033 **<** ρ_{MAX} = 0.08 (for compression, ACI 318-08, 10.9.1)

> ρ_{MIN} = 0.0018 (for flexural, ACI 318-08, 10.5.4)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	106.6	0.0
AT MAXIMUM LOAD	106.6	3.9
AT MIDDLE	60.9	7.2
AT ε_t = 0.002	15.3	5.5
AT BALANCED	14.4	5.4
AT ε_t = 0.005	5.3	5.3
AT FLEXURE ONLY	0.0	4.4

(Note: For middle reforming the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

P_u =	2.41	kips / ft
M_u =	1.25	ft-kips / ft

[Satisfactory]

CHECK REBAR DEVELOPMENT

$$L_d = MAX\left(\frac{\rho_{requird}}{\rho_{provided}}\frac{0.075\psi_t\psi_e\psi_s d_b f_y}{\lambda\sqrt{f_c^{'}}\left(\frac{c+K_{tr}}{d_b}\right)}\ ,\ 12\ \ in\right) = \quad 26\ d_b \quad = \quad 16$$

in, (ACI 318-08, 12.2.3)

< ***L_s*** **[Satisfactory]**

where Bar size # 5 , (governing size)

d_b = 0.625 in

$\rho_{required}$ / ρ_{provid} = 1 ($A_{s,reqd}$ / $A_{s,provd}$, ACI 318-08, 12.2.5)

ψ_t = 1.0 (1.3 for bottom cover more than 12", ACI 318-08, 12.2.4)

ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-08, 12.2.4)

ψ_s = 0.8 (0.8 for # 6 or smaller, 1.0 for other)

λ = 1.0 (0.75 for light weight, ACI 318-08, 12.2.4)

c = 3.3 in, min(d' , 0.5s), (ACI 318-08, 12.2.4)

K_{tr} = (A_{tr} f_{yt} / 1500 s n) = 0 (ACI 318-08, 12.2.4)

(c + K_{tr}) / d_b = 2.5 < 2.5 , (ACI 318-08, 12.2.3)

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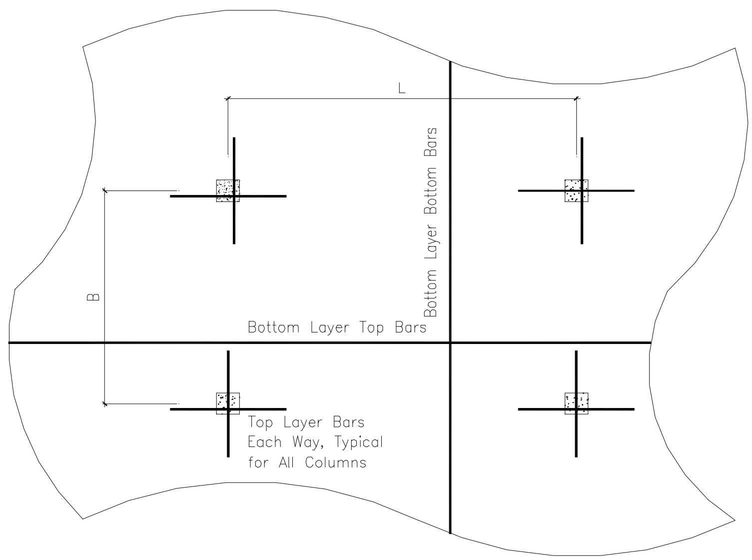
Two-Way Slab Design Based on ACI 318-11 using Finite Element Method

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH
f_c' = 3 ksi
REBAR YIELD STRESS
f_y = 60 ksi
COLUMN SPACING EACH WAY
L = 24 ft
B = 24 ft
SLAB THICKNESS
t = 9.5 in
BENDING DROP PANEL THK. (12.0 ft x 12.0 ft)
t_{drop} = 2.5 in
PUNCHING CAP THICKNESS
t_{cap} = 0 in
COLUMN SIZE (SHORT EDGE)
c = 24 in
DEAD LOAD & SELF WT
DL = 150 psf
LIVE LOAD
LL = 70 psf

TOP BARS AT COLUMNS EACH WAY
6 # 6 @ 12 o.c.
x 8.0 ft. long, with 0.75 in. cover
(All top bars to column strip suggested, if column strip & middle strip used.)

BOTTOM LAYER BOTTOM BARS
5 @ 18 o.c.
BOTTOM LAYER TOP BARS
5 @ 18 o.c.
with 0.75 in. bottom concrete cover
(75% total bottom bars to middle strip & 25% to column strip suggested, if column strip & middle strip used.)

THE DESIGN IS ADEQUATE.

ANALYSIS

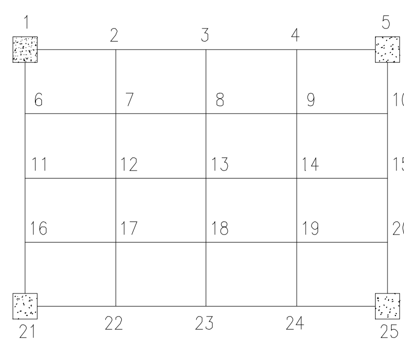
w_c = 150 pcf, (ACI 318-11 8.5.1)

$E_c = w_c^{1.5}\ 33\ f'^{0.5}_c$ = 3321 ksi, (ACI 318-11 8.5.1)

t_e = $(I_e / I_g)^{1/3}\ t = (0.25\ I_g / I_g)^{1/3}\ t = (0.25)^{1/3}\ t$ = 0.63 t = 6.0 in, for Slab only (ACI 318-11 9.5.3.4 & 10.10.4.1)
8.5 in, for Slab & Drop Panel

Joint Number	Δ_u in	R_u kips
1	0	42.05
2	0.29	
3	0.53	
4	0.29	
5	0	42.05
6	0.29	
7	0.46	
8	0.64	
9	0.46	
10	0.29	
11	0.53	
12	0.64	
13	**0.75**	
14	0.64	
15	0.53	
16	0.29	
17	0.46	
18	0.64	
19	0.46	
20	0.29	
21	0	42.05
22	0.29	
23	0.53	
24	0.29	
25	0	42.05

Bending Section	M_u ft-k/ft
1 - 2	12.9
2 - 3	-0.2
1 - 6	12.9
6 - 11	-0.2
3 - 8	-0.5
8 - 13	-6.0
11 - 12	-0.5
12 - 13	-6.0



DETERMINE FACTORED LOAD (ACI 318-11 9.2.1)
w_u = 1.2 DL + 1.6 LL = 0.292 ksf

DETERMINE FLEXURE CAPACITY (ACI 318-11 7.12.2.1, 10.2, 10.5.1)

	Top Bar 6 # 6 @ 12" o.c.	**Bot. Layer Bot.** 5 @ 18" o.c.	**Bot. Layer Top** 5 @ 18" o.c.
d (in)	10.13	8.44	7.81
$\mathbf{A_s}$ (in²/ft)	0.44	0.21	0.21
$\mathbf{A_{s,\,min}}$ (in²/ft)	0.41	0.21	0.21
a (in)	0.86	0.41	0.41
$\mathbf{\phi M_n}$ (ft-k/ft)	19.2	7.7	7.1

CHECK FLEXURE CAPACITY

$M_{u,Top}$ = Max($M_{u,1\text{-}2}$, $M_{u,1\text{-}6}$) = 12.9 ft-k/ft < ϕM_n = 19.2 ft-k/ft [Satisfactory]

$M_{u,Bot,Bot}$ = - Min($M_{u,8\text{-}13}$, $M_{u,12\text{-}13}$) = 6.0 ft-k/ft < ϕM_n = 7.7 ft-k/ft [Satisfactory]

$M_{u,Bot,Top}$ = - Max($M_{u,8\text{-}13}$, $M_{u,12\text{-}13}$) = 6.0 ft-k/ft < ϕM_n = 7.1 ft-k/ft [Satisfactory]

(cont'd)

CHECK LIVE LOAD DEFLECTION (ACI 318-11 Table 9.5b)

$\Delta_{LL} = \Delta_{u,Max}$ LL / (1.2 DL + 1.6 LL) = 0.18 in < L / 360 = 0.80 in

[Satisfactory]

CHECK LONG-TERM DEFLECTION (ACI 318-11 9.5.2.5)

$\Delta_{3DL + LL} = \Delta_{u,Max}$ (3DL + LL) / (1.2 DL + 1.6 LL) = 1.33 in < L / 180 = 1.60 in

[Satisfactory]

CHECK COLUMN PUNCHING CAPACITY (ACI 318-11 11.11.1.2, 11.11.7, & 13.5.3.2)

$P_u = 4\ R_{u,max} =$ 168.2 kips (See Punching.xls Software for More Information.)

$\phi V_n = (2 + y)\phi\sqrt{f'_c}A_p =$ 227.10 kips > P_u [Satisfactory]

where	ϕ =	0.75	(ACI 318-11, Section 9.3.2.3)	
	β_c =	1.00		
	d =	10.1	in	
	b_0 =	4 c + 4 d =	136.5	in
	A_p =	b_0 d =	1382.1	in^2
	y =	MIN(2 , 4 / β_c , 40 d / b_0) =	2.0	

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Two-Way Slab Design Based on ACI 318-11 using Finite Element Method

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH
f_c' = 3 ksi

REBAR YIELD STRESS
f_y = 60 ksi

COLUMN SPACING EACH WAY
L = 24 ft
B = 24 ft

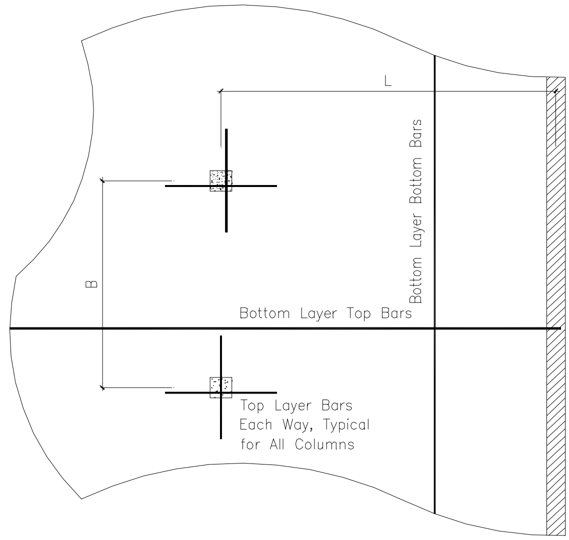
SLAB THICKNESS
t = 9.5 in

BENDING DROP PANEL THK. (12.0 ft x 12.0 ft)
t_{drop} = 2.5 in

PUNCHING CAP THICKNESS
t_{cap} = 0 in

DEAD LOAD & SELF WT
DL = 150 psf

LIVE LOAD
LL = 70 psf



TOP BARS AT COLUMNS EACH WAY
6 # 6 @ 12 o.c.
x 8.0 ft. long, with 2 in. cover
(All top bars to column strip suggested, if column strip & middle strip use

BOTTOM LAYER BOTTOM BARS
5 @ 18 o.c.

THE DESIGN IS ADEQUATE.

BOTTOM LAYER TOP BARS
5 @ 18 o.c.
with 0.75 in. bottom concrete cover
(75% total bottom bars to middle strip & 25% to column strip suggested, if column strip & middle strip used.)

ANALYSIS

w_c = 150 pcf, (ACI 318-11 8.5.1)

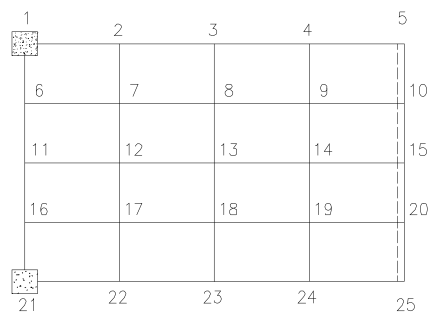
$E_c = w_c^{1.5}\, 33\, f'_c{}^{0.5}$ = 3321 ksi, (ACI 318-11 8.5.1)

$t_e = (I_e / I_g)^{1/3}\, t = (0.25\, I_g / I_g)^{1/3}\, t = (0.25)^{1/3}\, t$ = 0.63

t = 6.0 in, for Slab only (ACI 318-11 9.5.3.4 & 10.10.4.1)
8.5 in, for Slab & Drop Panel

Joint Number	Δ_u in
1	0
2	0.30
3	0.56
4	0.32
5	0
6	0.30
7	0.47
8	0.62
9	0.33
10	0
11	0.54
12	0.64
13	0.68
14	0.34
15	0
16	0.30
17	0.47
18	0.62
19	0.33
20	0
21	0
22	0.30
23	0.56
24	0.32
25	0

Bending Section	M_u ft-k/ft
1 - 2	13.3
2 - 3	-0.6
3 - 4	-2.3
4 - 5	12.9
1 - 6	13.2
6 - 11	-0.3
12 - 13	-6.9
13 - 14	-6.8
7 - 12	-7.3
8 - 13	-5.8
9 - 14	-2.8



DETERMINE FACTORED LOAD (ACI 318-11 9.2.1)
w_u = 1.2 DL + 1.6 LL = 0.292 ksf

DETERMINE FLEXURE CAPACITY (ACI 318-11 7.12.2.1, 10.2, 10.5.1)

	Top Bar 6 # 6 @ 12" o.c.	**Bot. Layer Bot.** 5 @ 18" o.c.	**Bot. Layer Top** 5 @ 18" o.c.
d (in)	8.88	8.44	7.81
$\mathbf{A_s}$ (in^2/ft)	0.44	0.21	0.21
$\mathbf{A_{s,\,min}}$ (in^2/ft)	0.36	0.21	0.21
a (in)	0.86	0.41	0.41
$\phi\mathbf{M_n}$ (ft-k/ft)	16.7	7.7	7.1

CHECK FLEXURE CAPACITY

$M_{u,Top}$ = Max($M_{u,1\text{-}5}$, $M_{u,1\text{-}11}$) = 13.3 ft-k/ft < ϕM_n = 16.7 ft-k/ft [Satisfactory]

$M_{u,Bot,Bot}$ = - Min(M_u) = 7.3 ft-k/ft < ϕM_n = 7.7 ft-k/ft [Satisfactory]

$M_{u,Bot,Top}$ = - Max($M_{u,12\text{-}14}$, $M_{u,7\text{-}12}$, $M_{u,8\text{-}13}$, $M_{u,9\text{-}14}$) = 2.8 ft-k/ft
< ϕM_n = 7.1 ft-k/ft [Satisfactory]

CHECK LIVE LOAD DEFLECTION (ACI 318-11 Table 9.5b)

$\Delta_{LL} = \Delta_{u,Max}$ LL / (1.2 DL + 1.6 LL) = 0.16 in < L / 360 = 0.80 in
[Satisfactory]

CHECK LONG-TERM DEFLECTION (ACI 318-11 9.5.2.5)

$\Delta_{3DL + LL} = \Delta_{u,Max}$ (3DL + LL) / (1.2 DL + 1.6 LL) = 1.20 in < L / 180 = 1.60 in
[Satisfactory]

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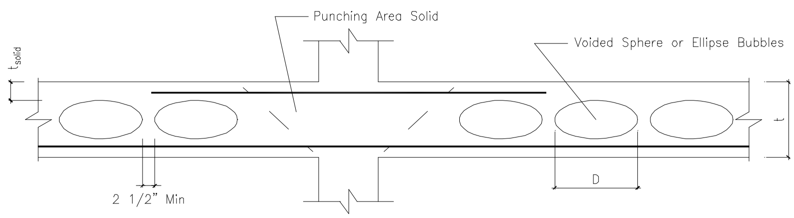
Voided Two-Way Slab Design Based on ACI 318-11

DESIGN CRITERIA

1. The voided sphere or ellipse bubbles within slab can reduce concrete weight, so both seismic mass (ASCE 7 12.7.2) and gravity loads reduced. And the long-term deflection (3 DL + LL) limits may not govern the two-way slab design (ACI 318 9.5).
2. The entire slab bottom formwork can be flat, without girder, beam, drop panel or cap, but the punching area (ACI 318 11.11), or lateral frame diaphragm area (ACI 318 21.11.9), may need to be solid as normal concrete shear transfer.
3. The section forces of voided slab can be determined by a two-way finite element method or by ACI 318 Chapter 13, but PT slab can only be designed by one way method because the secondary moment of PT slab is one way concept. Also, the voided two-way slab is better for depressed floor, or irregular opening, than PT slab.
4. The bottom two direction rebar can be distributed as a regular solid two-way slab, without Waffle slab or hollow core plank limits.



SECTION

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH
f_c' = 4 ksi

REBAR YIELD STRESS
f_y = 60 ksi

TOTAL SLAB THICKNESS
t = 18 in

TOP & BOTTOM SOLID THICKNESS
t_{solid} = 4 in

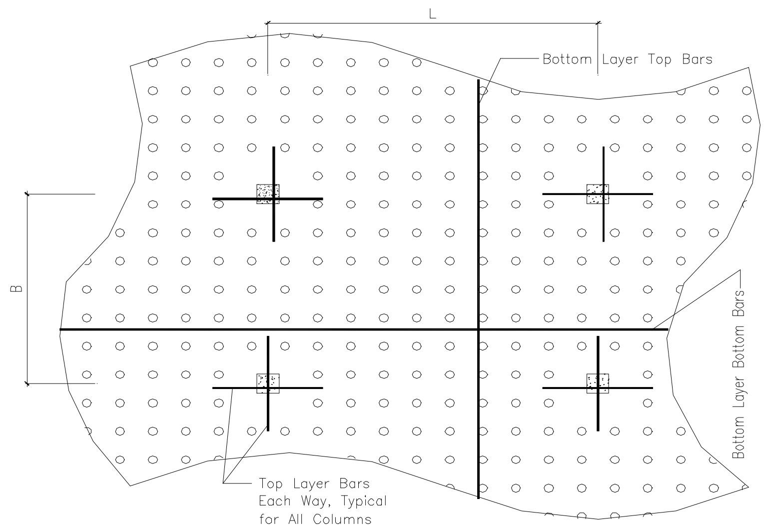
VOIDED BUBBLE HORIZONTAL DIAMETER
D = 20 in

COLUMN SPACING EACH WAY
L = 35 ft
B = 35 ft

COLUMN SIZE (SHORT EDGE)
c = 30 in

SUPERIMPOSED DEAD LOAD, ASD
DL_{sup} = 20 psf

LIVE LOAD
LL = 70 psf



PLAN

TOP BARS AT COLUMNS EACH WAY
17 # 7 @ 6 o.c.
x 11.7 ft. long, with 0.75 in. cover
(All top bars to column strip suggested, if column strip & middle strip used.)

BOTTOM LAYER BOTTOM BARS
\# 6 @ 12 o.c.

THE DESIGN IS ADEQUATE.

BOTTOM LAYER TOP BARS
\# 6 @ 12 o.c.
with 0.75 in. bottom concrete cover
(75% total bottom bars to middle strip & 25% to column strip suggested, if column strip & middle strip used.)

ANALYSIS

DETERMINE SECTION PROPERTY & DEAD LOAD

t_{solid} = 4 in > 0.75 + 1.75 + 0.75 = 3.25 in, top solid min thk
> 0.75 + 1.50 + 0.75 = 3.00 in, bot solid min thk
(inside cover) (2 rebar thick) (top & bot cover) [Satisfactory]

D = 20 in > 10 in, height of voided sphere or ellipse bubble [Satisfactory]

w_c = 150 pcf, (ACI 318-11 8.5.1) V = 2094 in^3, volume of a voided sphere or ellipse bubble

Wt = 173 psf, self weight reduced 23% DL = DL_{sup} + Wt = 193 psf

I_{soild} = 10935 in^4 I_g = 9953 in^4

$E_c = w_c^{1.5}\, 33\, f'_c{}^{0.5}$ = 3834 ksi, (ACI 318-11 8.5.1) $(I_g / I_{soild})\, E_c$ = 3490 ksi, for Finite Element Method

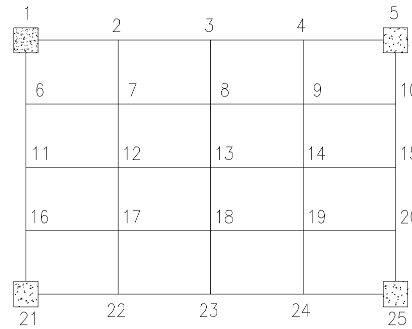
$t_e = (I_e / I_g)^{1/3}\, t = (0.25\, I_g / I_g)^{1/3}\, t = (0.25)^{1/3}\, t$ = 0.63 t = 11.3 in, for Slab only (ACI 318-11 9.5.3.4 & 10.10.4.1)

(cont'd)

DETERMINE SECTION FORCE AND SLAB DEFLECTION USING FINITE ELEMENT METHOD

Joint Number	Δ_u in	R_u kips
1	0	105.33
2	0.49	
3	0.78	
4	0.49	
5	0	105.33
6	0.49	
7	0.69	
8	0.88	
9	0.69	
10	0.49	
11	0.78	
12	0.88	
13	**0.99**	
14	0.88	
15	0.78	
16	0.49	
17	0.69	
18	0.88	
19	0.69	
20	0.49	
21	0	105.33
22	0.49	
23	0.78	
24	0.49	
25	0	105.33

Bending Section	M_u ft-k/ft
1 - 2	80.9
2 - 3	-3.7
1 - 6	80.9
6 - 11	-3.7
3 - 8	-1.7
8 - 13	-20.6
11 - 12	-1.7
12 - 13	-20.6

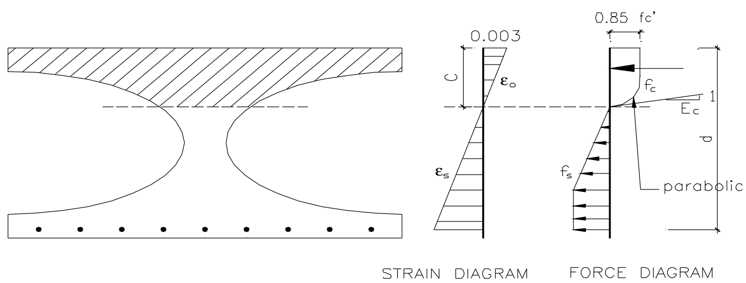


DETERMINE FACTORED LOAD (ACI 318-11 9.2.1)

w_u = 1.2 DL + 1.6 LL = 0.344 ksf

DETERMINE FLEXURE CAPACITY (ACI 318-11 7.12.2.1, 10.2, 10.5.1)

	Top Bar	**Bot. Layer Bot.**	**Bot. Layer Top**
	17 # 7 @ 6" o.c.	6 @ 12" o.c.	6 @ 12" o.c.
d (in)	15.94	16.88	16.13
$\mathbf{A_s}$ (in^2/ft)	1.20	0.44	0.44
$\mathbf{A_{s,\ min}}$ (in^2/ft)	0.64	0.39	0.39
c (in)	2.20	0.84	0.84
$\mathbf{\phi M_n}$ (ft-k/ft)	81.1	32.7	31.3



$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c} \quad , \quad E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] & , \ for \ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c & , \ for \ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s & , \ for \ \ \varepsilon_s \le \varepsilon_t \\ f_y & , \ for \ \ \varepsilon_s > \varepsilon_t \end{cases}$$

CHECK FLEXURE CAPACITY

$M_{u,Top}$ = Max($M_{u,1-2}$, $M_{u,1-6}$) = 80.9 ft-k/ft < ϕM_n = 81.1 ft-k/ft [Satisfactory]

$M_{u,Bot,Bot}$ = - Min($M_{u,8-13}$, $M_{u,12-13}$) = 20.6 ft-k/ft < ϕM_n = 32.7 ft-k/ft [Satisfactory]

$M_{u,Bot,Top}$ = - Max($M_{u,8-13}$, $M_{u,12-13}$) = 20.6 ft-k/ft < ϕM_n = 31.3 ft-k/ft [Satisfactory]

CHECK LIVE LOAD DEFLECTION (ACI 318-11 Table 9.5b)

$\Delta_{LL} = \Delta_{u,Max}$ LL / (1.2 DL + 1.6 LL) = 0.20 in < L / 360 = 1.17 in
[Satisfactory]

CHECK LONG-TERM DEFLECTION (ACI 318-11 9.5.2.5)

$\Delta_{3DL+LL} = \Delta_{u,Max}$ (3DL + LL) / (1.2 DL + 1.6 LL) = 1.87 in < L / 180 = 2.33 in
[Satisfactory]

CHECK COLUMN PUNCHING CAPACITY (ACI 318-11 11.11.1.2, 11.11.7, & 13.5.3.2)

$P_u = 4\ R_{u,max}$ = 421.3 kips (See Punching.xls Software for More Information.)

$\phi V_n = (2+y)\phi\sqrt{f'_c}\,A_p =$ 555.65 kips > P_u [Satisfactory]

where ϕ = 0.75 (ACI 318-11, Section 9.3.2.3)

β_c = 1.00

d = 15.9 in

b_0 = 4 c + 4 d = 183.8 in

A_p = b_0 d = 2928.5 in^2

y = MIN(2 , 4 / β_c , 40 d / b_0) = 2.0

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Engineering International

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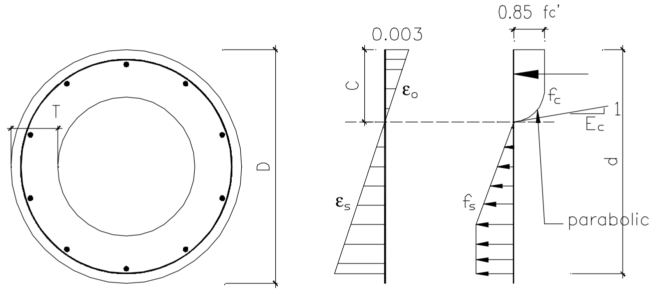
Pipe Concrete Column Design Based on ACI 318-08

INPUT DATA & DESIGN SUMMARY

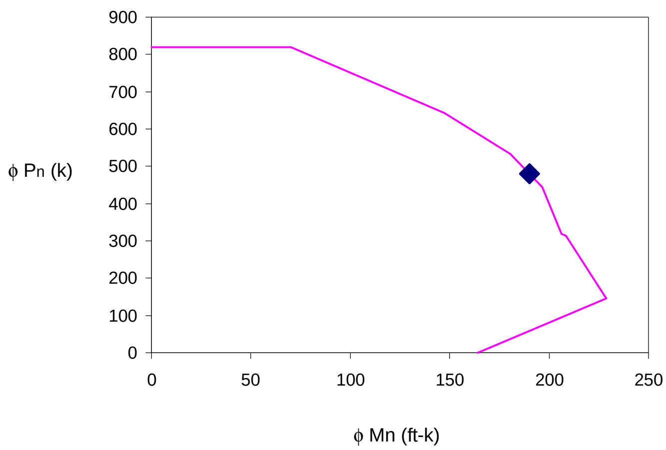
CONCRETE STRENGTH	fc'	=	5	ksi
REBAR YIELD STRESS	fy	=	60	ksi
COLUMN OUTSIDE DIAMETER	D	=	20	in
CONCRETE WEB THICKNESS	T	=	7	in
FACTORED AXIAL LOAD	Pu	=	480	k
FACTORED MAGNIFIED MOMENT	Mu	=	190	ft-k
FACTORED SHEAR LOAD	Vu	=	20	k
COLUMN VERT. REINFORCEMENT	8	#	7	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT #	4	@	12	in o.c.



STRAIN DIAGRAM FORCE DIAGRAM

THE COLUMN DESIGN IS ADEQUATE.

ANALYSIS



$$\varepsilon_O = \frac{2\left(0.85 f'_c\right)}{E_C}\ ,\ E_C = 57\sqrt{f'_c}\ ,\ E_S = 29000ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right] , & for\ \ 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f'_c\ , & for\ \ \varepsilon_C \ge \varepsilon_O \end{cases}$$

$$f_s = \begin{cases} \varepsilon_S E_S\ , & for\ \ \varepsilon_S \le \varepsilon_y \\ f_y\ , & for\ \ \varepsilon_S > \varepsilon_y \end{cases}$$

	ϕ Pn (kips)	ϕ Mn (ft-kips)
AT AXIAL LOAD ONLY	819	0
AT MAXIMUM LOAD	819	70
AT 0 % TENSION	643	148
AT 25 % TENSION	533	180
AT 50 % TENSION	444	197
AT ε_t = 0.002	318	206
AT BALANCED CONDITION	313	208
AT ε_t = 0.005	146	229
AT FLEXURE ONLY	0	164

CHECK FLEXURAL & AXIAL CAPACITY

$\phi P_{max} = 0.85\ \phi\ [0.85 f_c' (A_g - A_{st}) + f_y A_{st}]$ = 819.1 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.1)

where ϕ = 0.65 (ACI 318-08, Sec.9.3.2.2) **>** Pu **[Satisfactory]**

A_g = 286 in^2. A_{st} = 4.80 in^2.

$a = C_b \beta_1$ = 8 in (at balanced strain condition, ACI 10.3.2)

$\phi = \begin{cases} 0.75 + (\varepsilon_t - 0.002)(50), \text{ for Spiral} \\ 0.65 + (\varepsilon_t - 0.002)(250/3), \text{ for Ties} \end{cases}$ = 0.656 (ACI 318-08, Fig. R9.3.2)

where $C_b = d\, \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 10 in ε_t = 0.002069 ε_c = 0.003

d = 17.6 in, (ACI 7.7.1) β_1 = 0.8 (ACI 318-08, Sec. 10.2.7.3)

$\phi M_n = 0.9\ M_n$ = 164 ft-kips @ P_n = 0, (ACI 318-08, Sec. 9.3.2) ,& $\varepsilon_{t,min}$ = 0.004, (ACI 318-08, Sec. 10.3.5)

ϕM_n = 190 ft-kips @ P_u = 480 kips **>** Mu **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.017

ρ_{min} = 0.01 (ACI 318-08, Section 10.9) **[Satisfactory]**

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c)$ = 49 kips, (ACI 318-08 Sec. 11.1.1)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3)

A_0 = 214 in^2. A_v = 0.40 in^2. f_y = 60 ksi

V_c = $2\,(f_c')^{0.5} A_0$ = 30.3 kips, (ACI 318-08 Sec. 11.2.1)

V_s = MIN ($d\, f_y A_v / s$, $4V_c$) = 35.1 kips, (ACI 318-08 Sec. 11.4.6.2)

s_{max} = 14 (ACI 318-08, Section 7.10.5.2) s_{provd} = 12 in

s_{min} = 1 **[Satisfactory]**

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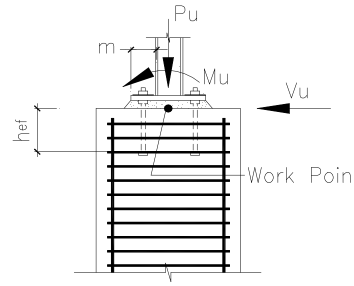
Anchorage to Pedestal Design Based on ACI 318-08 & AISC 360-05

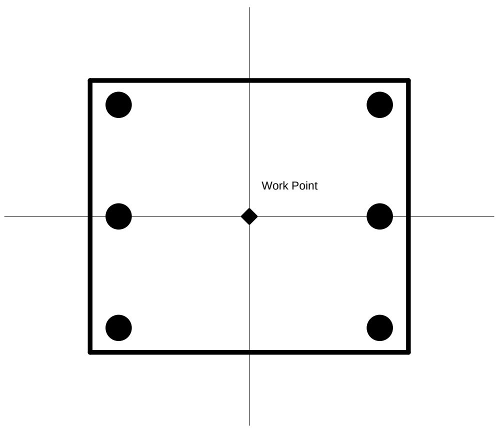
INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	4	ksi
SPECIFIED STRENGTH OF ANCHOR	f_{uta} =	65	ksi
BASE PLATE YIELD STRESS	f_y =	36	ksi
CRITICAL BASE PLATE CANTILEVER	m =	3.3	in
FACTORED AXIAL LOAD AT WORK POINT	P_u =	150	kips
FACTORED SHEAR LOAD AT WORK POINT	V_u =	16	kips, (0 if shear lug used.)
FACTORED MOMENT AT WORK POINT	M_u =	120	ft-kips
EFFECTIVE EMBEDMENT DEPTH	h_{ef} =	28	in
ANCHOR DIAMETER	d =	1	in
BASE PLATE THICKNESS	t =	1.75	in
SEISMIC LOAD ? (ASCE 14.2.2.17)	== >	Yes	
PEDESTAL VERTICAL REBAR SIZE	#	7	
PEDESTAL HORIZONTAL SECTION AREA	A_2 =	576	in^2



ANCHOR LOCATION TO WORK POINT

Anchor	X (in)	Y (in)
1	-8	8
2	-8	0
3	-8	-8
4	8	8
5	8	0
6	8	-8



BASE PLATE SIZE - 20 in x 20 in x 1.75 in (thk.)

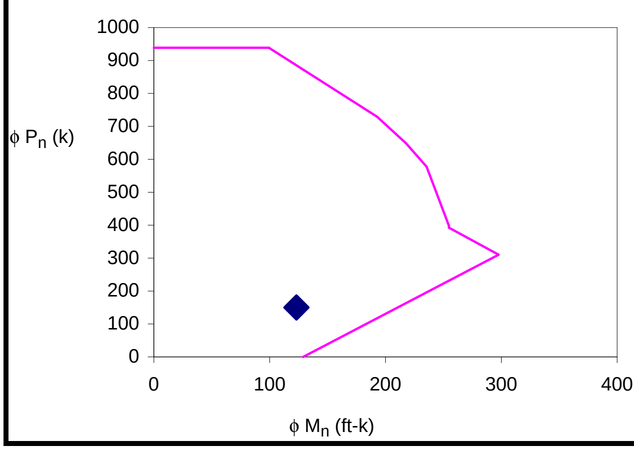
Corner	X (in)	Y (in)
1	-9.75	-9.75
2	-9.75	9.75
3	9.75	9.75
4	9.75	-9.75

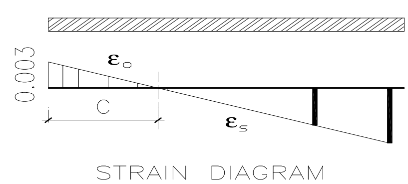
Edge = 1.75 in
(AISC 360-05, Tab J3.4 & J3.5)
A_1 = 400 in^2, base plate area

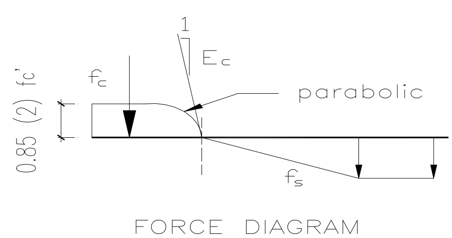
THE BASE PLATE & GROUP ANCHORS DESIGN IS ADEQUATE.

ANALYSIS

CHECK FLEXURAL & AXIAL COMPRESSION CAPACITY OF ANCHORED CONCRETE (ACI 318-08 Sec 9 & Sec 10)







(cont'd)

ϕM_n = 210 ft-kips @ P_u = 150 kips

> $M_{u,c}$ = 123 ft-kips to center of base plate
[Satisfactory]

ϕP_{max} = 939 kips

> P_u = 150 ft-kips [Satisfactory]

$N_{ua,1}$ = 12.50 kips
(The 2.5 times max tension, ASCE 14.2.2.17)

f_p = 4.42 ksi, the max possible concrete compression stress.
(ACI 318-08, 10.17.1 & 9.3.2.4, or AISC Guide - 1)

$$\varepsilon_O = \frac{2f'_c}{E_c}0.85Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right)\ ,\ E_c = 57\sqrt{f'_c}\ ,\ E_s = 29000ksi$$

$$f_c = \begin{cases} 0.85Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right)f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right)-\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]\ ,\ for\ \ 0<\varepsilon_c<\varepsilon_o \\ 0.85Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right)f'_c\ ,\ for\ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \le \varepsilon_y \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_y \end{cases}$$

DETERMINE EACH ANCHOR FORCES WITHOUT CONCTERE COMPRESSION CAPCAITY CONSIDERED

Anchor	**X** (in)	**Y** (in)	X_{CG}^2	Y_{CG}^2	$N_{ua,2}$	V_{ua}
1	-8	8	64	64	5.00	2.67
2	-8	0	64	0	5.00	2.67
3	-8	-8	64	64	5.00	2.67
4	8	8	64	64	-55.00	2.67
5	8	0	64	0	-55.00	2.67
6	8	-8	64	64	-55.00	2.67
Σ			**384**	**256**	**-150**	**16**

X_{CG} = 0 in
Y_{CG} = 0 in

$P_{u,CG}$ = 150.0 kips
$V_{u,CG}$ = 16.0 kips
$M_{u,CG,X}$ = 120.0 ft-kips
$M_{u,CG,Y}$ = 0.0 ft-kips

CHECK GOVERNING ANCHOR TENSILE STRENGTH (ACI 318, D.5.1.2)

$\phi N_s = \phi A_{se}(f_{uta})$ = 22.16 kips > N_{ua} = Max($N_{ua,1}$, $N_{ua,2}$) = 12.50 kips [Satisfactory]

where : ϕ = 0.75 x 0.75 = 0.5625 , (ACI 318-08 D.4.4 & D.3.3.3)
A_{se} = 0.606 in^2

CHECK GOVERNING ANCHOR SHEAR STRENGTH (ACI 318, D.6.1.2b & ASCE 7-05 14.2.2.17)

$\phi V_s = \phi 0.6 A_{se} f_{ut}$ = 11.52 k > V_{ua} = 2.67 kips [Satisfactory]

where : ϕ = 0.65 x 0.75 = 0.4875
(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D.6.1.3)

CHECK TENSION AND SHEAR INTERACTION OF GOVERNING ANCHORS : (ACI 318, D.7)

Since $N_{ua,2}$ > $0.2\ \phi\ N_n$ and
$V_{ua,2}$ > $0.2\ \phi\ V_n$ the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua,2}}{\phi N_n} + \frac{V_{ua,2}}{\phi V_n} = 0.46 < 1.2$$ [Satisfactory]

(cont'd)

CHECK BASE PLATE THICKNESS (AISC Guide - 1, Eq. 3.3.14a)

$t_{reqD} = 1.5m\sqrt{\frac{f_p}{F_y}} =$ 1.73 in < ***t =*** 1.75 in [Satisfactory]

CHECK SPLICE LENGTH OF TENSION ANCHOR WITH REBAR

L_s = 1.3 L_d = 25 d_b = 25 in, (ACI 318-08, 12.15.1,)

< h_{ef} - 2" = 26 in [Satisfactory]

$L_d = MAX\left(\frac{\rho_{requird}}{\rho_{provided}}\frac{0.075\psi_t\psi_e\psi_s d_b f_y}{\lambda\sqrt{f'_c}\left(\frac{c+K_{tr}}{d_b}\right)}\ ,\ 12\ \ in\right) =$ 19 d_b = 19 in

(ACI 318-08, 12.2.3)

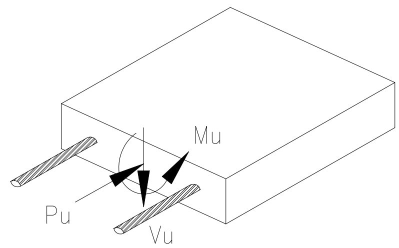
where	$\rho_{required} / \rho_{provided}$	=	0.564	($A_{s,reqd}$ / $A_{s,provd}$, if not apply input zero, ACI 318-08, 12.2.5 & 12.15.1)
	d_b	=	1	in, anchor governing
	f_y	=	65	ksi
	ψ_t	=	1.0	(1.3 for horizontal bar over 12" concrete, ACI 318-08 12.2.4)
	ψ_e	=	1.0	(1.2 for epoxy-coated, ACI 318-08 12.2.4)
	ψ_s	=	1.0	(0.8 for # 6 or smaller, 1.0 for other)
	λ	=	1.0	(0.75 for light weight, ACI 318-08, 12.2.4)
	c	=	2.3	in, (ACI 318-08, 12.2.4)
	K_{tr}	=	(A_{tr} f_{yt} / 1500 s n) = 0	(ACI 318-08, 12.2.3), (50 b_w / 1500 n, for CBC 2001)
	($c + K_{tr}$) / d_b =		2.3	< 2.5 , (ACI 318-08, 12.2.3)

Plate/Shell Element Design Based on ACI 318-08

DESIGN CRITERIA

THE PLATE/SHELL ELEMENT CAN BE FROM BEARING WALL, DIAPHRAGM, TANK, OR OTHER SURFACE STRUCTURES. THE SOFTWARE IS FOR ONE DIRECTION PERPENDICULAR BENDING AND SHEAR DESIGN, SINCE THERE ARE NO INTERACTION AT BOTH DIRECTIONS. FOR IN PLAN LOADS, THE SHEAR WALL DESIGN SOFTWARE SHOULD BE USED.

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	3	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
FACTORED AXIAL LOAD	P_u	=	30	kips / ft		
FACTORED MOMENT	M_u	=	18	ft-kips / ft		
FACTORED SHEAR LOAD	V_u	=	7.2	kips / ft		
THICKNESS OF ELEMENT	t	=	10	in		
ELEMENT REINFORCING (A_s)		#	5	@	18	in o.c.
A_s LOCATION (1=at middle, 2=at each face)			2	at each face		

[THE ELEMENT DESIGN IS ADEQUATE.]

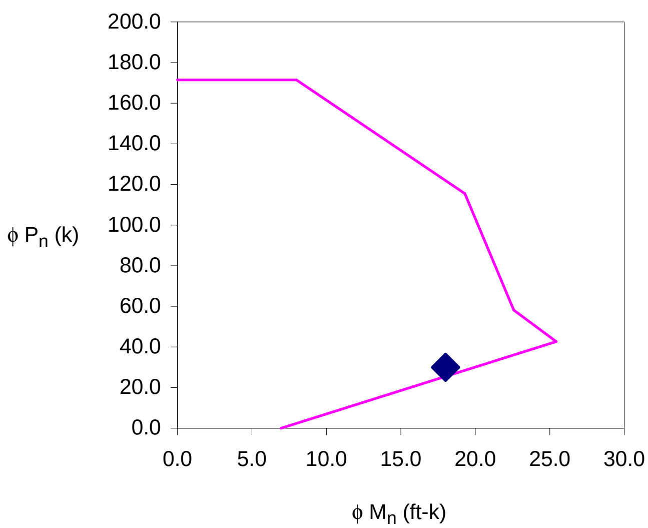
ANALYSIS

CHECK AXIAL & FLEXURE CAPACITY

ρ_{ProvD} = 0.00224 **<** ρ_{MAX} = 0.0400 (tension face only, ACI 318-05 10.3.5 or 10.9.1)

> ρ_{MIN} = 0.0008 (tension face only, ACI 318-05 10.5.1, 10.5.3 or 14.3.2)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	171.5	0.0
AT MAXIMUM LOAD	171.5	8.0
AT MIDDLE	115.5	19.3
AT ε_t = 0.002	59.5	22.5
AT BALANCED	58.1	22.6
AT ε_t = 0.005	42.7	25.4
AT FLEXURE ONLY	0.0	7.0

(Note: For middle reforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-08 SEC.15.5.2, 11.1.3.1, & 11.2)

$\phi V_n = 2\phi bd\sqrt{f_c'}$ = 7.58 kips / ft **>** $\boldsymbol{V_u}$ **[Satisfactory]**

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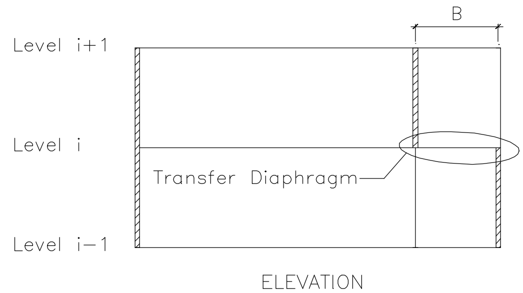
Concrete Diaphragm Design for a Discontinuity of Type 4 out-of-plane offset irregularity

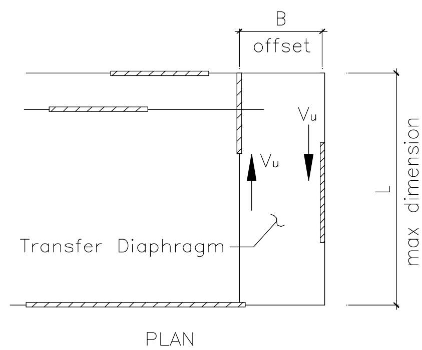


INPUT DATA

OFFSET LATERAL SYSTEM DIMENSION	***B*** =	18	ft
MAX TWO OUTSIDE PERPENDICULAR LATERAL SYSTEM SPAN	***L*** =	32	ft
TRANSFER DIAPHRAGM THICKNESS	***t*** =	8	in
REBAR YIELD STRESS	f_y =	60	ksi
CONCRETE STRENGTH	f_c' =	4	ksi
LIGHTWEIGHT CONCRETE ? (ACI 318, 8.6)		Yes	(Lightweight Concrete)
TRANSFER DIAPHRAGM REINFORCEMENT	# 7	@ 8	in, o.c.
REINF. LOCATION (0 = one layer only, 1 = Top & Bottom)		1	,(top & bottom, each way)
COLLECTOR / CHORD REBARS AROUND TRANS. DIAPHRAGM	10	#	8

(the rebars must be closed trans. diaphragm & developed into vert. lateral system)

FACTORED SHEAR FORCE ON TRANSFER DIAPHRAGM SIDE	V_u =	680	kips
MAX. FACTORED COLLECTOR (DRAG) FORCE	$T_{u,drag}$ =	165	kips
MAX. FACTORED CHORD FORCE	$T_{u,chord}$ =	60	kips



THE TRANSFER DIAPHRAGM DESIGN IS ADEQUATE.

ANALYSIS

CHECK MINIMUM REINFORCEMENT RATIOS (ACI 318-08, 21.11.7 & 7.12)

$(\rho_t)_{provd.}$ = 0.0188 **>** $(\rho_t)_{min.}$ = 0.0018

[Satisfactory]

CHECK DIAPHRAGM SHEAR CAPACITY (ACI 318-08, 21.11.9)

$v_u = 1.25\ \rho\ V_u / L$ = 34.5 kips / ft, (ASCE 7-05, 12.3.3.4 & Table 12.3-2.4)

$\phi v_n = \mathrm{MIN}\ [\ \phi\ A_{cv}\ (2\ \lambda\ (f_c')^{0.5} + \rho_t\ f_y),\ \phi\ 8\ A_{cv}\ (f_c')^{0.5}\ ]$ = 36.4 kips / ft **>** v_u **[Satisfactory]**

where ρ = 1.3 (ASCE 7-05, 12.10.1.1 & 12.3.4.2)

ϕ = 0.75 (ACI 318-08, 9.3.2.3)

A_{cv} = 96 in^2

λ = 0.75 (ACI 318-08, 8.6.1)

CHECK DIAPHRAGM IN-PLANE FLEXURAL CAPACITY, USING DISTRIBUTED REINFORCEMENT ONLY (ACI 318-08, 21.11.9)

$M_u = 1.25\ \rho\ V_u\ B$ = 19890.0 ft-kips, the max value same at both B & L sections, (ASCE 7-05, 12.3.3.4 & Table 12.3-2.4)

ϕM_n = 20011.3 ft-kips, by pure math method **>** M_u **[Satisfactory]**

where β_1 = 0.85 (ACI 318-08, 10.2.7.3)

ε_{cu} = 0.003 (ACI 318-08, 10.3.2)

d_t = MIN(B , L) = 216 in, (ACI 318-08, R10.3.3)

c = 74 in, (ACI 318-08, R10.3.3)

E_s = 29000 ksi, (ACI 318-08, 8.5.2)

e_c = 76 in, (eccentricity of total concrete compression force)

e_s = 61 in, (eccentricity of total steel tension force)

CHECK COLLECTOR / CHORD REBAR CAPACITY (ACI 318-08, 10.2.4)

$T_u = \mathrm{MAX}(\ 1.25\ \rho\ T_{u,chord}\ ,\ \Omega_0\ T_{u,drag})$ = 412.5 kips, (ASCE 7-05, 12.10.2.1, 12.3.4.1, 12.3.3.4 & Table 12.3-2.4)

$\phi T_n = \phi\ f_y\ A_s$ = 426.6 kips **>** T_u **[Satisfactory]**

where Ω_0 = 2.5 (ASCE 7-05 Tab 12.2-1)

A_s = 7.9 in^2

ϕ = 0.90 (ACI 318-08, 9.3.2.1)

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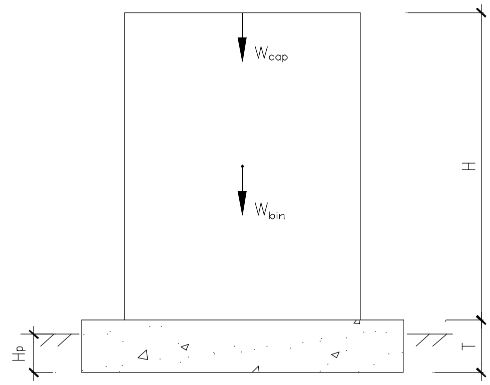
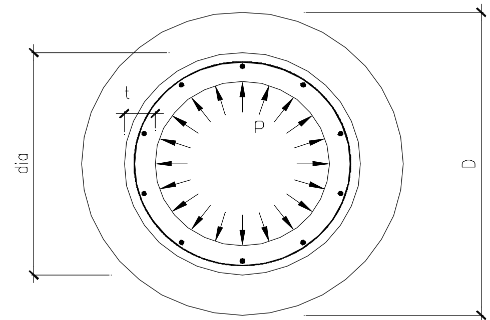
Concrete Silo / Chimney / Tower Design Based on ASCE 7-05, ACI 318-08 & ACI 313-97

INPUT DATA



BIN DEPTH	H =	150	ft
BIN OUTSIDE DIAMETER	dia =	30	ft
BIN CONCRETE THICKNESS	t =	12	in, at bottom
	(	8	in, at top)
FOOTING DIAMETER	D =	61	ft
FOOTING THICKNESS	T =	48	in
SOIL DEPTH TO BOTTOM	H_p =	24	in
WT OF BIN MAX CONTENTS	W_{bin} =	5763.4	kips, (input zero for chimney)
WT OF TOP CAP	W_{cap} =	180.96	kips
MAX HORIZONTAL PRESSURE	p =	9360	psf, (γH for water, or from ACI 313 4-2)
ALLOWABLE SOIL PRESSURE	Q_a =	5.5	ksf
PASSIVE PRESSURE	P_p =	450	psf / ft
SOIL FRICTION COEFFICIENT	μ =	0.35	
CONCRETE STRENGTH	f_c' =	5	ksi
REBAR YIELD STRESS	f_y =	60	ksi



FOOTING REBAR 2 Layers # 10 @ 14 in o.c. each way, at top & bot.

DOWEL / BIN VERTICAL REBARS # 8 @ 12 in o.c.

REBAR LOCATION (1=at middle, 2=at each face) 2 at each face

BIN HORIZONTAL REBARS # 6 @ 12 in o.c.

WALL HORIZONTAL PRESTRESSING TENDONS, (input strands zero for non-prestress)

11 strands @ 24 in o.c.

(each 0.5 in diameter & 0.153 in^2 area)

TENDON YIELD STRENGTH f_{py} = 243 ksi

EFFECTIVE PRESTRESS AFTER ALL LOSSES f_e = 174 ksi

THE CONCRETE DESIGN IS ADEQUATE.

DESIGN SUMMARY

FOOTING 61 ft DIA x 48 in THK. w/ # 10 @ 14" o.c. EACH WAY, AT TOP & BOT.

CONCRETE BIN 12 in THK. w/ # 8 @ 12 in o.c.DOWEL / VERT. BARS AT AT EACH FACE

BIN HORIZONTAL # 6 @ 12 in o.c. AT EACH FACE, AND (11) - STANDS @ 24 in o.c. (THE SECTION UNCRACKED.)

ANALYSIS

DETERMINE LATERAL LOADS

F = Max (0.8 S_1 I / R , 0.03) W / 1.4 = 0.10 ***W*** , ASD (ASCE 7-05 15.4.1.2)

F_{cap} = 0.10 W_{cap} = 18.96 kips, at top

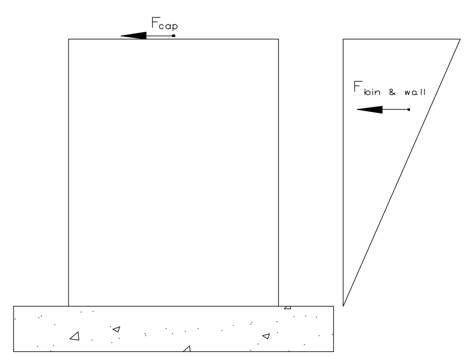
$F_{bin\,\&\,wall}$ = 0.10 (W_{cap} + W_{wall}) = 783.78 kips, at 2/3 H

Where ***S_1*** = 0.55 (from soil report, ASCE 7-05, 11.4.1)

I = 1.00 (ASCE 7-05 15.4.1.1)

R = 3 (ASCE 7-05 Table 15.4-2)

W_{wall} = 1718.1 kips



V = 802.73 kips, total shear at top of footing

M = 81221 ft-kips, total moment at top of footing

COMBINED LOADS AT TOP FOOTING (IBC 1605.3.2 & ACI 318-08 9.2.1)

CASE 1:	DL + LL	P	=	7662	kips	1.2 DL + 1.6 LL	Pu	=	11500	kips
		M	=	0	ft-kips		Mu	=	0	ft-kips
		e	=	0.0	ft, fr cl ftg		eu	=	0.0	ft, fr cl ftg
CASE 2:	DL + LL + E / 1.4	P	=	7662	kips	1.2 DL + 1.0 LL + 1.0 E	Pu	=	8042	kips
		M	=	81221	ft-kips		Mu	=	113710	ft-kips
		e	=	10.6	ft, fr cl ftg		eu	=	14.1	ft, fr cl ftg
CASE 3:	0.9 DL + E / 1.4	P	=	5350	kips	0.9 DL + 1.0 E	Pu	=	5350	kips
		M	=	63222	ft-kips		Mu	=	88511	ft-kips
		e	=	11.8	ft, fr cl ftg		eu	=	16.5	ft, fr cl ftg

(cont'd)

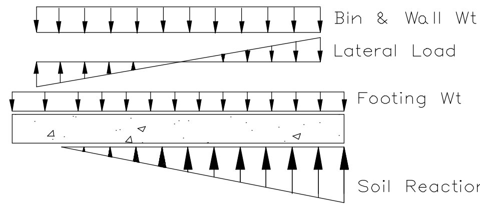
CHECK OVERTURNING FACTOR AT FOOTING EDGE BOTTOM (IBC 09 1605.2.1, 1808.3.1, & ASCE 7-05 12.13.4)

M_R / M_O = 3.4 **>** 1.5 **[Satisfactory]**

Where M_O = M + V T = 84432 k-ft , M_R = Σ(W) 0.5 D = 287186 k-ft

W_{ftg} = (0.15 kcf) T D^2 π / 4 = 1753.5 kips, footing weight.

CHECK SOIL BEARING CAPACITY (ACI 318-08 SEC.15.2.2)



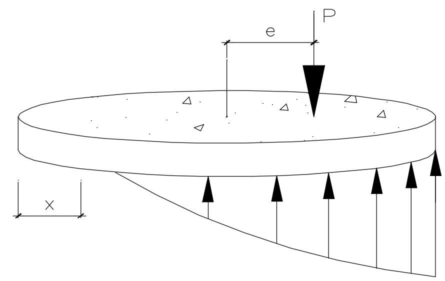
γ_s = 0.11 kcf, soil weight

Service Loads	CASE 1	CASE 2	CASE 3	
P	7662.5	7662.5	5350	k
e	0	10.6	11.817	ft (from center of footing)
P_{ftg} - P_{soil}	1110.5	1110.5	999.48	k, (footing increasing)
Σ P	8773	8773	6349.4	k, (net loads)
e	0	9.2581	9.9572	ft
q_{min}	3.0019	0	0	ksf
x		@ 9.15 ft from edge	@ 9.15 ft from edge	
q_{max}	3.0019	6.8882	5.2453	ksf
$q_{allowable}$	5.5	7.3333	7.3333	ksf

[Satisfactory]

CHECK ENTIRE FLEXURE & SHEAR OF FOOTING

(ACI 318-08 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.2)



$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) \qquad \rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1	CASE 2	CASE 3	
P_u	11500	8042.3	5350	k
e_u	0	14.139	16.544	ft
γ (0.15 T) A	2104.2	2104.2	1578.1	k, (factored footingloads)
Σ P_u	13604	10146	6928.1	k
e_u	0	11.207	12.776	ft
$q_{u, min}$	4.6551	0	0	ksf
x		@ 12.20 ft from edge	@ 15.25 ft from edge	
$q_{u, max}$	4.66	9.22	7.10	ksf

FOOTING MOMENT & SHEAR FOR CASE **1**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	15.50	19.25	23.00	26.75	30.50	34.25	38.00	41.75	45.50	61.00
Tangent (ft)	0.00	53.11	56.70	59.13	60.54	61.00	60.54	59.13	56.70	53.11	0.00
$q_{u,tank}$ (ksf)	0.00	16.27	16.27	16.27	16.27	16.27	16.27	16.27	16.27	16.27	0.00
$M_{u,tank}$ (ft-k)	0	0	3426.1	10857	23531	42067	66657	97109	132805	172505	350760
$V_{u,tank}$ (k)	0	0	914	1,982	3,380	6,557	8,121	9,519	10,587	11,500	11,500
$q_{u,ftg}$ (ksf)	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
$M_{u,ftg}$ (ft-k)	0	3,952	6,095	8,733	11,885	15,565	19,776	24,514	29,767	35,515	64,177
$V_{u,ftg}$ (k)	0	255	572	703	841	1,123	1,264	1,401	1,533	1,849	2,104
$q_{u,soil}$ (ksf)	-4.66	-4.66	-4.66	-4.66	-4.66	-4.66	-4.66	-4.66	-4.66	-4.66	-4.66
$M_{u,soil}$ (ft-k)	0	-25551	-39409	-56461	-76843	-100634	-127860	-158495	-192460	-229618	-414937
$V_{u,soil}$ (k)	0	-1,648	-3,696	-4,547	-5,435	-7,260	-8,169	-9,057	-9,909	-11,956	-13,604
Σ M_u (ft-k)	**0**	**-21599**	**-29888**	**-36872**	**-41427**	**-43002**	**-41427**	**-36872**	**-29888**	**-21599**	**0**
Σ V_u (kips)	**0**	**-1,393**	**-2,210**	**-1,862**	**-1,215**	**420**	**1,215**	**1,862**	**2,210**	**1,393**	**0**

(cont'd)

FOOTING MOMENT & SHEAR FOR CASE **2**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	15.50	19.25	23.00	26.75	30.50	34.25	38.00	41.75	45.50	61.00
Tangent (ft)	0.00	53.11	56.70	59.13	60.54	61.00	60.54	59.13	56.70	53.11	0.00
$q_{u,tank}$ (ksf)	0.00	-33.59	70.58	81.83	93.07	104.31	115.55	126.79	138.03	56.34	0.00
$M_{u,tank}$ (ft-k)	0	0	-526.47	240.15	2969.5	8194.6	16309	27522	41776	58559	131579
$V_{u,tank}$ (k)	0	0	-240	349	1,242	3,694	5,105	6,489	7,640	8,042	8,042
$q_{u,ftg}$ (ksf)	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
$M_{u,ftg}$ (ft-k)	0	3,952	6,095	8,733	11,885	15,565	19,776	24,514	29,767	35,515	64,177
$V_{u,ftg}$ (k)	0	255	572	703	841	1,123	1,264	1,401	1,533	1,849	2,104
$q_{u,soil}$ (ksf)	0.00	-0.62	-1.33	-2.04	-2.75	-3.46	-4.16	-4.87	-5.58	-6.29	-9.22
$M_{u,soil}$ (ft-k)	0	0	-1362.8	-3936.4	-8444.4	-15621	-26178	-40780	-60002	-84300	-195757
$V_{u,soil}$ (k)	0	0	-363	-686	-1,202	-2,815	-3,894	-5,126	-6,479	-10,146	-10,146
Σ M_u (ft-k)	**0**	**3952**	**4206**	**5037**	**6410**	**8139**	**9906**	**11256**	**11541**	**9774**	**0**
Σ V_u (kips)	**0**	**255**	**-31**	**366**	**881**	**2,001**	**2,474**	**2,764**	**2,693**	**-255**	**0**

FOOTING MOMENT & SHEAR FOR CASE **3**

Section	0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)	0	15.50	19.25	23.00	26.75	30.50	34.25	38.00	41.75	45.50	61.00
Tangent (ft)	0.00	53.11	56.70	59.13	60.54	61.00	60.54	59.13	56.70	53.11	0.00
$q_{u,tank}$ (ksf)	0.00	-27.43	53.65	62.41	71.16	79.91	88.66	97.41	106.16	42.57	0.00
$M_{u,tank}$ (ft-k)	0	0	-374.39	107.08	1891.9	5338.2	10711	18155	27633	38804	74662
$V_{u,tank}$ (k)	0	0	-170	219	813	2,446	3,389	4,315	5,085	5,350	5,350
$q_{u,ftg}$ (ksf)	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54
$M_{u,ftg}$ (ft-k)	0	2,964	4,572	6,550	8,914	11,674	14,832	18,386	22,326	26,636	48,133
$V_{u,ftg}$ (k)	0	191	429	527	630	842	948	1,051	1,149	1,387	1,578
$q_{u,soil}$ (ksf)	0.00	-0.04	-0.62	-1.20	-1.78	-2.37	-2.95	-3.53	-4.11	-4.69	-7.10
$M_{u,soil}$ (ft-k)	0	0	-84.591	-732.1	-2517	-6029.8	-11852	-20528	-32544	-48289	-122795
$V_{u,soil}$ (k)	0	0	-23	-173	-476	-1,552	-2,314	-3,204	-4,199	-6,928	-6,928
Σ M_u (ft-k)	**0**	**2964**	**4113**	**5925**	**8289**	**10982**	**13692**	**16012**	**17414**	**17151**	**0**
Σ V_u (kips)	**0**	**191**	**236**	**574**	**967**	**1,736**	**2,022**	**2,161**	**2,036**	**-191**	**0**

FOOTING MOMENT & SHEAR SUMMARY

Section			0	L Edge	1/8 d	2/8 d	3/8 d	Center	5/8 d	6/8 d	7/8 d	R Edge	D
X_u (ft, dist. from left of footing)			0	15.50	19.25	23.00	26.75	30.50	34.25	38.00	41.75	45.50	61.00
Tangent (ft)			0.00	53.11	56.70	59.13	60.54	61.00	60.54	59.13	56.70	53.11	0.00
Uniform Loads	**Case 1**	M_u, (ft-k / ft)	0.0	-406.7	-527.1	-623.6	-684.3	-705.0	-684.3	-623.6	-527.1	-406.7	0.0
		V_u, (k / ft)	0.0	-26.2	-39.0	-31.5	-20.1	6.9	20.1	31.5	39.0	26.2	0.0
	Case 2	M_u, (ft-k / ft)	0.0	74.4	74.2	85.2	105.9	133.4	163.6	190.4	203.6	184.0	0.0
		V_u, (k / ft)	0.0	4.8	-0.6	6.2	14.6	32.8	40.9	46.7	47.5	-4.8	0.0
	Case 3	M_u, (ft-k / ft)	0.0	55.8	72.5	100.2	136.9	180.0	226.2	270.8	307.1	322.9	0.0
		V_u, (k / ft)	0.0	3.6	4.2	9.7	16.0	28.5	33.4	36.6	35.9	-3.6	0.0

CHECK FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	ρ_{provD}
Top Slab	322.9	ft-k / ft	45.37	0.0019	0.0030	0.0243	no limit	0.0020
Bottom Slab	-705.0	ft-k / ft	44.37	0.0019	0.0070	0.0243	18	0.0020

[Satisfactory]

CHECK FLEXURE SHEAR

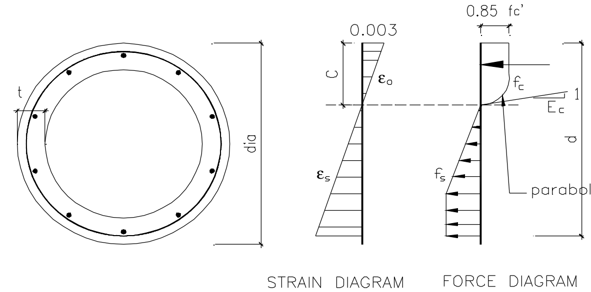
$V_{u,max}$	$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$	check $V_u < \phi\ V_c$
47.5 k / ft	56 k	**[Satisfactory]**

CHECK BIN VERTICAL FLEXURAL & AXIAL CAPACITY

$$\varepsilon_O = \frac{2\left(0.85 f_c'\right)}{E_C}\ ,\quad E_C = 57\sqrt{f_c'}\ ,\quad E_S = 29000ksi$$

$$f_C = \begin{cases} 0.85 f_c'\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right]\ , & for\ \ 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f_c'\ , & for\ \ \varepsilon_C \ge \varepsilon_O \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S\ , & for\ \ \varepsilon_S \le \varepsilon_y \\ f_y\ , & for\ \ \varepsilon_S > \varepsilon_y \end{cases}$$



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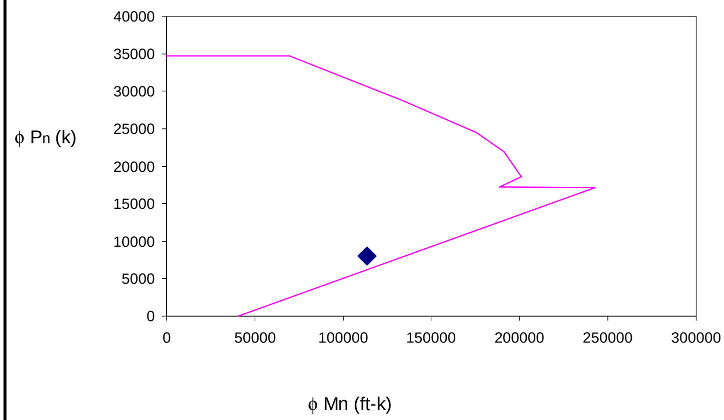


	ϕ Pn (kips)	ϕ Mn (ft-kips)
AT AXIAL LOAD ONLY	34748	0
AT MAXIMUM LOAD	34748	69734
AT 0 % TENSION	28765	133851
AT 25 % TENSION	24488	175599
AT 50 % TENSION	21920	191058
AT ε_t = 0.002	18587	201070
AT BALANCED CONDITION	17229	188611
AT ε_t = 0.005	17125	242597
AT FLEXURE ONLY	0	40448

P_u = 8042 kips

M_u = 113710 ft-kips, max at bottom

(from load combinations)

$\phi P_{max} = 0.85\ \phi\ [\ 0.85\ f_c'\ (A_g - A_{st}) + f_y A_{st}]$ = 34748 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.1)

where ϕ = 0.70 (ACI 318-08, Sec.9.3.2.2) > P_u **[Satisfactory]**

A_g = 13119 in^2. in^2. A_{st} = 47.40 in^2.

$a = C_b \beta_1$ 169 in (at balanced strain condition, ACI 10.3.2)

ϕ = 0.7 + (ε_t - 0.002) (200 / 3), for Spiral; 0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-08, Fig. R9.3.2)

where $C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 212 in ε_t = 0.002069 ε_c = 0.003

d = 357.38 in, (ACI 7.7.1) β_1 = 0.8 (ACI 318-08, Sec. 10.2.7.3)

$\phi M_n = 0.9\ M_n$ = 40448 ft-kips @ Pn = 0, (ACI 318-08, Sec. 9.3.2) ,& et,min = 0.004, (ACI 318-08, Sec. 10.3.5)

ϕM_n = 135380 ft-kips @ P_u = 8042 kips > M_u **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.004

ρ_{min} = 0.004 (ACI 318-08, Section 10.5.1 or 10.9.1) **[Satisfactory]**

CHECK BIN HORIZONTAL SHEAR CAPACITY

$\phi V_n = \phi\ (V_c)$ = 1235 kips, (ACI 318-08 Sec. 11.1.1)

> V_u = 1.4 V = 1123.8 kips, max at bottom **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3)

A_0 = 11640 in^2.

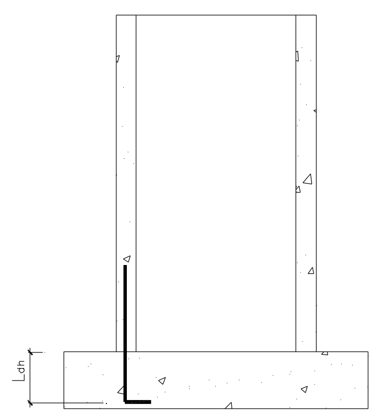
$V_c = 2\ (f_c')^{0.5} A_0$ = 1646.2 kips, (ACI 318-08 Sec. 11.2.1)

CHECK DOWEL DEVELOPMENT

$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f_c'}}\ ,\ 8d_b\ ,\ 6\ in\right) =$$ 12 d_b = 12 in, (ACI 318-08 12.5.2) < 44 in

[Satisfactory]

where			
Bar size	#	8	
d_b	=	1	in
$\rho_{required} / \rho_{provided}$	=	1	(A s,reqd / A s,provd , ACI 318-08, 12.2.5)
f_y	=	60	ksi
f'_c	=	5	ksi
ψ_t	=	1.0	
ψ_e	=	1.0	(1.2 for epoxy-coated, ACI 318-08 12.2.4)
ψ_s	=	1.0	(0.8 for # 6 or smaller, 1.0 for other)
λ	=	1.0	
c	=	3.5	in, min(d' , 0.5s), (ACI 318-08, 12.2.4)
K_{tr}	=	$(A_{tr} f_{yt} / 1500\ s\ n)$ = 0	(ACI 318-08, 12.2.3)
$(c + K_{tr}) / d_b$	=	2.5	< 2.5 , (ACI 318-08, 12.2.3)
η	=	0.7	(#11 or smaller, cover > 2.5" & side >2.0", ACI 318-08 12.5.3)



CHECK BIN LOCAL SHEAR STRESS ON A SQUARE FOOT

$\phi V_n = \phi\ (v_c)$ = 77.28 kips, (ACI 318-08 Sec. 11.1.1)

> v_u = 1.4 p = 13.10 kips **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3)

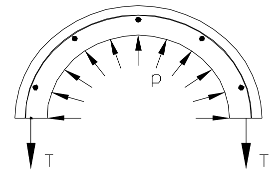
A_0 = 4 x (1'-0") x (0.5 x T) = 288 in^2.

$v_c = 4\ (f_c')^{0.5} A_0$ = 103.0 kips, (ACI 318-08 Sec. 11.11)

p = 9360 psf, (the max perpendicular wall pressure)

(cont'd)

CHECK BIN WALL TENSION STRESS & CRACKING AT HORIZONTAL SERVICE INSIDE PRESSURE



p = 9360 psf, (the max perpendicular wall pressure)

T = 940 lbs / in^2 , by pure math method

T_{fe} = -1017 lbs / in^2 , effective prestressing **>** 0.8 (0.6 f_c') = -2400 lbs / in^2

[Satisfactory] (ACI 318 18.4.1)

T + T_{fe} = -77 lbs / in^2 **<** 7.5 (f_c')$^{0.5}$ = 530 lbs / in^2, (ACI 318 Eq. 9-10 & 18.3.3)

[Uncracked]

CHECK BIN WALL HORIZONTAL TENSION CAPACITY

1.6 T = 1504 lbs / in^2 **<** ϕ (f_y A_s / A_c + f_{py} A_{ps} / A_{pc}) = 1608 lbs / in^2

[Satisfactory]

Where ϕ = 0.90 , (ACI 318-08 R9.3.2)

A_s / A_c = 0.0061

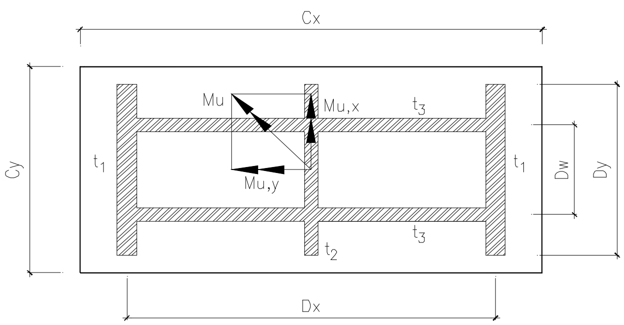
A_{ps} / A_{pc} = 0.0058

Daniel T. Li Engineering International	PROJECT :		PAGE :
	CLIENT :		DESIGN BY :
	JOB NO. :	DATE :	REVIEW BY :

Super Composite Column Design Based on AISC 360-05 & ACI 318-08

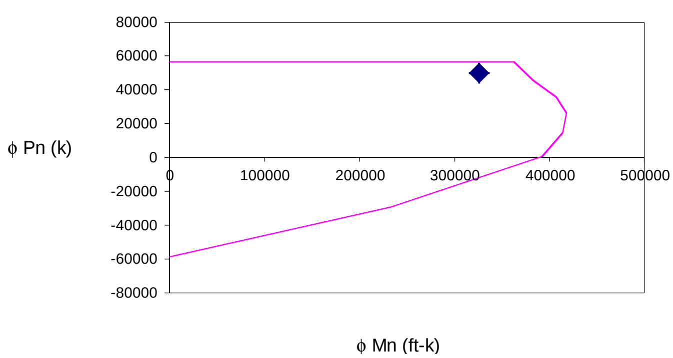
INPUT DATA & DESIGN SUMMARY

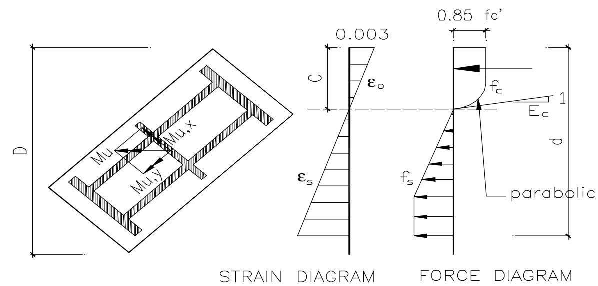
CONCRETE STRENGTH	f_c'	=	5	ksi
STEEL YIELD STRESS	f_y	=	50	ksi
COLUMN EFFECTIVE LENGTH	KL	=	240	ft
CONCRETE SECTION SIZE	C_x	=	240	in
	C_y	=	125	in
STEEL SECTION SIZE	D_x	=	192	in
	D_y	=	100	in
	t_1	=	2	in
	t_2	=	1.5	in
	t_3	=	2	in
	D_w	=	75	in
FACTORED AXIAL LOAD	P_u	=	50000	k
FACTORED MOMENT	$M_{u,x}$	=	310000	ft-k
	$M_{u,y}$	=	100000	ft-k
FACTORED SHEAR LOAD	$V_{u,x}$	=	1800	k
	$V_{u,y}$	=	3200	k



THE COLUMN DESIGN IS ADEQUATE.

ANALYSIS





Capacity Drawings	ϕ	ϕ Pn (k)	ϕ Mn (ft-k)
AT AXIAL LOAD ONLY	0.75	56495	0
AT MAXIMUM LOAD	0.75	56495	362678
AT AXIAL LOAD 45615 k	0.75	45615	383081
AT AXIAL LOAD 35923 k	0.78	35923	407207
AT AXIAL LOAD 26276 k	0.82	26276	417700
AT AXIAL LOAD 14501 k	0.86	14501	413791
AT STEEL STRAIN 0.005	0.9	715	392128
AT AXIAL LOAD -28982 k	0.9	-28982	232902
AT PURE TENSION	0.9	-58680	0

$$\varepsilon_O = \frac{2\left(0.85 f'_c\right)}{E_C}\ ,\quad E_C = 57\sqrt{f'_c}\ ,\quad E_S = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\dfrac{\varepsilon_C}{\varepsilon_O}\right) - \left(\dfrac{\varepsilon_C}{\varepsilon_O}\right)^2\right]\ , & for\quad 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f'_c\ , & for\quad \varepsilon_C \geq \varepsilon_O \end{cases}$$

$$f_s = \begin{cases} \varepsilon_S E_S\ , & for\quad \varepsilon_S \leq \varepsilon_t \\ f_y\ , & for\quad \varepsilon_S > \varepsilon_t \end{cases}$$

CHECK FLEXURAL & AXIAL CAPACITY

$\phi\, P_{max} = \phi_c\, P_n$ = 56495 kips, (AISC 360-05 I2-2 & I2-3)

\> Pu **[Satisfactory]**

where ϕ_c = 0.75 (AISC 360-05 I2.1b & ACI 318-08 9.3.2.2)

A_c = 28696 in^2 I_c = 37543603 in^4

A_s = 1304 in^2 I_s = 1518897 in^4

(cont'd)

C_1 = 0.187 , (AISC 360-05 I2-7) EI_{eff} = 72334735452 ksi-in^4, (AISC 360-05 I2-6)

P_e = 86072 kips, (AISC 360-05 I2-5) P_o = 187158 kips, (AISC 360-05 I2-4)

Balanced : ϕ = 0.75 (AISC 360-05 I2.1b & ACI 318-08 Fig. R9.3.2)

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 151.5 in ε_t = 0.0017 ε_c = 0.003

d = 238.6 in, (ACI 7.7.1) D = 266.8 in

Critical Points	**ϕ**	**ϕ Pn (k)**	**ϕ Mn (ft-k)**
AT AXIAL LOAD ONLY	0.75	56495	0
AT MAXIMUM LOAD	0.75	56495	362678
AT 0 % TENSION	0.75	95998	240093
AT 25 % TENSION	0.75	81038	298887
AT 50 % TENSION	0.75	67975	336762
AT STEEL STRAIN 0.002	0.75	39686	390588
AT BALANCED CONDITION	0.75	45661	383006
AT STEEL STRAIN 0.005	0.9	715	392128
AT FLEXURE ONLY	0.75	0	387404

ϕM_n = 374858 ft-kips @ P_u = 50000 kips **>** M_u = 325730 ft-kips **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08 10.9) ρ_{provd} = 0.043

ρ_{min} = 0.01 (AISC 360-05 I2.1a & ACI 318-08 10.9) **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC I2.1d & ACI 318-08 11.1 & 11.2)

$\phi V_{nx} = \phi\,(V_{cx})$ **>** V_{ux} (ACI 318-08 11.1.1)

$\phi_v V_{ny} = \phi_v\,(V_{ny})$ **>** V_{uy} (AISC 360-05 G2.1) **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 9.3.2.3)

ϕ_v = 1.00 (AISC 360-05 G2.1)

	d	A_0	A_w	$V_c = 2\,(f_c')^{0.5} A_0$	$V_n = 0.6\, f_y A_w C_v$	ϕV_n
x	216	27000		3818.4		2864
y	100		550		16500.0	16500

Note:

1. The minimum Stud Shear Connectors (not shown on this spreadsheet) are 3/4" ϕ @ 12" O.C. in both directions of vertical and horizontal around built-up structural steel shape. (AISC 360-05 I2.1g)
2. The column shall be reinforced, not shown on this spreadsheet, with continuous longitudinal bars (ρ_{sr} = 0.004 min.), and lateral ties or spirals at least 0.009 in^2 per in. (AISC 360-05 I2.1a)

Purchaser's LOGO

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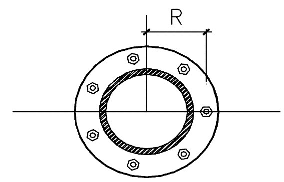
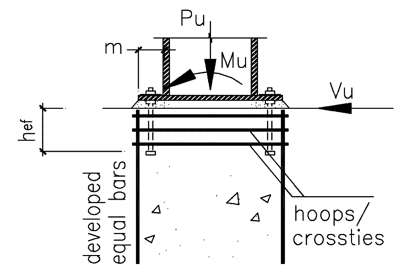
Anchorage Design, with Circular Base Plate, Based on ACI 318-11 & AISC 360-10

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	4	ksi
SPECIFIED STRENGTH OF ANCHOR	f_{uta} =	65	ksi
BASE PLATE YIELD STRESS	f_y =	36	ksi
CRITICAL BASE PLATE CANTILEVER	m =	3.3	in
BASE PLATE THICKNESS	t =	1.75	in
FACTORED AXIAL LOAD	P_u =	3	kips
FACTORED SHEAR LOAD	V_u =	6	kips, (0 if shear lug used.)
FACTORED MOMENT	M_u =	38	ft-kips
SEISMIC LOAD ? (ASCE 14.2.2.17)	== >	Yes	
EFFECTIVE EMBEDMENT DEPTH	h_{ef} =	28	in
ANCHOR DIAMETER	d =	0.75	in
RADIUS ANCHOR LOCATION	R =	8	in
ANCHOR NUMBERS	n =	7	
DEVELOPED VERTICAL REBAR SIZE	#	6	





THE PLATE & ANCHORS DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE MAX ANCHOR FORCES WITHOUT CONCTERE COMPRESSION CAPCAITY CONSIDERED

$L = 2\pi R =$ 50.27 in

$A = n\pi d^2/4 =$ 3.09 in^2

$I = 0.5\,A\,R^2 =$ 98.96 in^4

$T = P_u / A - M_u R / I =$ -35.89 kips / in^2

$N_{ua,2} = -T A / n =$ 15.86 kips / bolt, Tension

$N_{ua,1} =$ 16.28 kips / bolt, (The tensile strength, ASCE 14.2.2.17)

$N_{ua} = \mathrm{Max}(N_{ua,1}, N_{ua,2}) =$ 16.28 kips / bolt

$V_{ua} = 1.5\,(V_u / n) =$ 1.29 kips / bolt

CHECK GOVERNING ANCHOR TENSILE STRENGTH (ACI 318, D.5.1.2)

$\phi N_{sa} = \phi A_{se,N}\left(f_{uta}\right)$ = 16.28 kips = N_{ua} = 16.28 kips [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75 , (ACI 318-11 D.4.4 & D.3.3.4.4)

$A_{se,N}$ = 0.334 in^2

CHECK GOVERNING ANCHOR SHEAR STRENGTH (ACI 318, D.6.1.2b)

$\phi V_{sa} = \phi 0.6 A_{se,N} f_{ut}$ = 8.47 k > V_{ua} = 1.29 kips [Satisfactory]

where : ϕ = 0.65 x 1 = 0.65

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D.6.1.3)

CHECK TENSION AND SHEAR INTERACTION OF GOVERNING ANCHORS : (ACI 318, D.7)

Since $N_{ua,2}$ < $0.2\,\phi\,N_n$ and

$V_{ua,2}$ < $0.2\,\phi\,V_n$ the full tension design strength is permitted.

The interaction equation may be used

$$\frac{N_{ua,2}}{\phi N_n} + \frac{V_{ua,2}}{\phi V_n} = \quad 1.13 \quad < \quad 1.2$$

[Satisfactory]

CHECK BASE PLATE THICKNESS (AISC Guide - 1, Eq. 3.3.14a)

f_p = 4.42 ksi, the max possible concrete compression stress.

(ACI 318-11, 10.14.1 & 9.3.2.4, or AISC Guide - 1)

$$t_{reqD} = 1.5m\sqrt{\frac{f_p}{F_y}} = \quad 1.73 \text{ in} \quad < \quad t = 1.75 \text{ in}$$

[Satisfactory]

(cont'd)

CHECK SPLICE LENGTH OF TENSION ANCHOR WITH REBAR

$L_s = 1.3\,L_d =$ 32 d_b = 24 in, (ACI 318-11, 12.15.1)

< h_{ef} - 2" = 26 in [Satisfactory]

$$L_d = MAX\left(\frac{\rho_{requird}}{\rho_{provided}}\frac{0.075\psi_t\psi_e\psi_s d_b f_y}{\lambda\sqrt{f'_c}\left(\frac{c_b+K_{tr}}{d_b}\right)}\ ,\ 12\ \ in\right) =$$

25 d_b = 18 in

(ACI 318-11, 12.2.3)

where $\rho_{required} / \rho_{provided}$ = 1.000 ($A_{s,reqd}$ / $A_{s,provd}$, if not apply input zero, ACI 318-11, 12.2.5)

d_b	=	0.75	in, anchor governing
f_y	=	65	ksi
ψ_t	=	1.0	(1.3 for horizontal bar over 12" concrete, ACI 318-11 12.2.4)
ψ_e	=	1.0	(1.2 for epoxy-coated, ACI 318-11 12.2.4)
ψ_s	=	0.8	(0.8 for # 6 or smaller, 1.0 for other)
λ	=	1.0	(0.75 for light weight, ACI 318-11, 12.2.4)
c_b	=	1.9	in, (ACI 318-11, 12.2.4)
K_{tr}	=	(A_{tr} f_{yt} / 1500 s n) =	0 (ACI 318-11, 12.2.3)
(c_b + K_{tr}) / d_b =		2.5	< 2.5 , (ACI 318-11, 12.2.3)

Purchaser's LOGO

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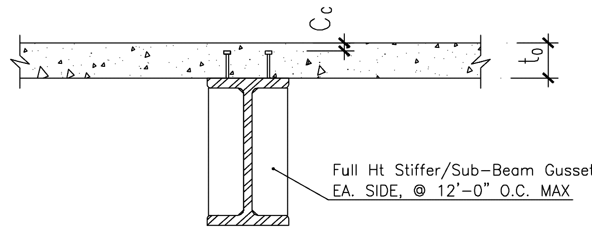
Composite Beam/Collector Design, without Metal Deck, Based on AISC 360-10 & ACI 318-11

INPUT DATA & DESIGN SUMMARY



BEAM/COLLECTOR SECTION **W21X44**

STEEL YIELD STRESS F_y = **50** ksi
CONCRETE STRENGTH f_c' = **5** ksi
CONCRETE COVER C_c = **1** in, 0.75" min., (ACI 318-11 7.7)
t_0 = **4.25** in, 2.20 Min.

BEAM/COLLECTOR SPAN L = **30** ft
SPACING (Tributary Width) B = **28** ft, o.c.
STRONG AXIS POSITIVE MOMENT, *LRFD* M_u = **480** *ft-kips, SD level*
SHEAR LOAD, *LRFD* V_u = **150** *kips, SD level*
COLLOCTOR AXIAL LOAD, *LRFD* P_u = **100** *kips, SD level, at center of W21X44*
SHEAR STUD DIA. (1/2, 5/8, 3/4) ϕ = **3/4** in
STUDS SPACING **2** *rows @* **12** in o.c.

THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE CAMBER/SHORING ON NON-COMPOSITE

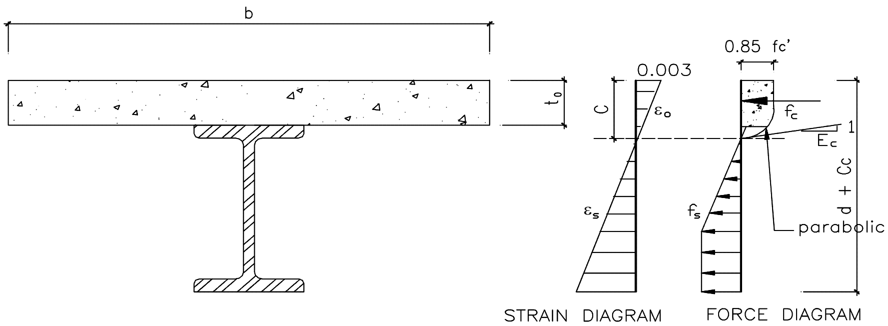
w = **2.497** *kips / ft, floor system self weight, to W21X44, on non-composite*

$\Delta = 5wL^4 / 384\, EI$ = 1.86 *in, deflection of W21X44*

< L / **180** = 2.00 *in* **[Satisfactory]**

Camber = **0.75** Δ = ***1.40*** *in*

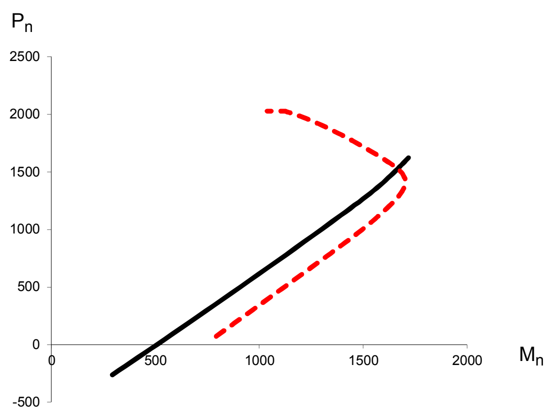
CHECK FLEXURAL & AXIAL CAPACITY (AISC 360-10, I3, ACI 318-11 Chapter 10 & 21)



$$\varepsilon_O = \frac{2\left(0.85 f_c'\right)}{E_C}\ ,\quad E_C = 57\sqrt{f_c'}\ ,\quad E_S = 29000\,ksi$$

$$f_c = \begin{cases} 0.85 f_c'\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right] , & for\quad 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f_c' , & for\quad \varepsilon_C \geq \varepsilon_O \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s , & for\quad \varepsilon_s \leq \varepsilon_t \\ f_y , & for\quad \varepsilon_s > \varepsilon_t \end{cases}$$



Solid Line - Tension Controlled
Dash Line - Compression Controlled

b = *MIN (L / 4 , B)* = *90* *in, (AISC 360-10 I3.1a)*
ϕ = **0.9** *, (AISC 360-10 I3)*
M_n @ P_u / ϕ = *597.5* *ft-kips*
ϕM_n = *537.7* *ft-kips*
> $M_u + 0.5 C_c P_u$ = *484.2* *ft-kips*
[Satisfactory]

CHECK SHEAR CAPACITY (AISC 360-10, G2)

ϕV_n = *195.6* *kips* **>** V_u = *150.0* *kips* **[Satisfactory]**

DETERMINE COMPOSITE PROPERTIES FOR ELASTIC DESIGN

$$n = \frac{E}{E_C} = $$ 6.76 , (ACI 318-11 8.5.1)

$A_{ctr} = b\, t_0 / n$ = 56.5 in^2

A	d	I_x	S_x	Z_x
13	20.7	843	81.6	95.4

(cont'd)

$$y_b = \frac{A_{ctr}(d+0.5t_0)+0.5Ad}{A_{ctr}+A} =$$ 20.5 in, (elastic neutral axis to bottom)

$$I_{tr} = I_x + A(y_b - 0.5d)^2 + \frac{A_{ctr}t_0^2}{12} + A_{ctr}(0.5t_0+d-y_b)^2 =$$ 2573 in^2

$$S_{tr} = \frac{I_{tr}}{y_b} =$$ 126 in^3, referred to steel bottom.

$$S_t = \frac{I_{tr}}{(d + t_0 - y_b)} =$$ 577 in^3, referred to concrete top.

CHECK SHEAR CONNECTOR CAPACITY (ASD)

M_{max} = 320.0 ft-kips > $M_n / \Omega_b = Z_x F_y / \Omega_b$ = 238.0 ft-kips **<== Shear Studs Required**

where Ω_b = 1.67 (AISC 360-10 F1 & F2-1)

$C_f = MIN(0.85 f_c' A_c , F_y A_s)$ = 650 kips, (AISC 360-10 C-I3.1)

$S_{eff} = Min[M_{max} / (0.66 F_y), S_{tr}]$ = 116 in^3, referred to steel bottom.

$$V' = MAX\left[\left(\frac{S_{eff}-S_s}{S_{tr}-S_s}\right)^2, 0.25\right]C_f =$$ 406.54 kips, (AISC 360-10 C-I3-5)

$Q_n = MIN[0.5 A_{sc} (f_c' E_c)^{0.5}, R_g R_p A_{sc} F_u]$ = 16.34 kips, (AISC 360-10 I3.2d)

where w_c = 150 pcf

$E_c = w_c^{1.5}\ 33 (f_c')^{0.5}$ = 4286.8 ksi

A_{sc} = 0.44 in^2

F_u = 58 ksi

R_g = 0.85 (AISC 360-10 Table I3.2b)

R_p = 0.75 (AISC 360-10 Table I3.2b)

$\Sigma Q_n = Q_n N_r X_1 / s$ = 490.05 kips > V' **[Satisfactory]**

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Multi-Story Tilt-Up Wall Design Based on ACI 318-11

DESIGN CRITERIA

1. Multi-story tilt-up wall design may be governed by erection bending forces. To cut tall wall to two different casting panels (i. e. *4 story* + *2 story*) can reduce the maximum section forces, be constructed on a very tight site, and reduce upper panel thickness.
2. Since it is difficult to design multi-story tilt-up wall tension-controlled (ACI 318-11 14.8.2.3) only, this software does NOT use the method of *Alternative Design of Slender Walls* (ACI 318-11 14.8), so there are no P-Δ effects have to be applied (ACI 318-11 14.8.4).

INPUT DATA & DESIGN SUMMARY

FACTORED MAX OUT-OF-PLANE SECTION MOMENT (from erection, wind, and/or seismic loads, the max value of $w_{max} L^2/8$ may be input).

M_u = 12 ft-kips / ft

FACTORED AXIAL LOAD (con-currently at the same section with M_u).

P_u = 35 kips / ft

FACTORED SHEAR LOAD V_u = 7.2 kips / ft

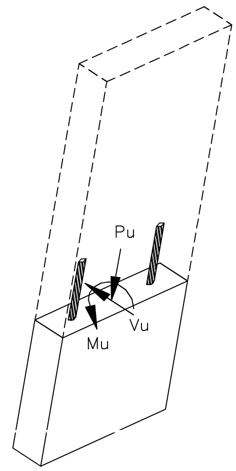
CONCRETE STRENGTH f_c' = 3 ksi

REBAR YIELD STRESS f_y = 60 ksi

THICKNESS OF WALL t = 10 in

WALL REINFORCING (A_s) # 5 @ 18 in o.c.

A_s LOCATION (1=at middle, 2=at each face) 2 at each face



[THE WALL DESIGN IS ADEQUATE.]

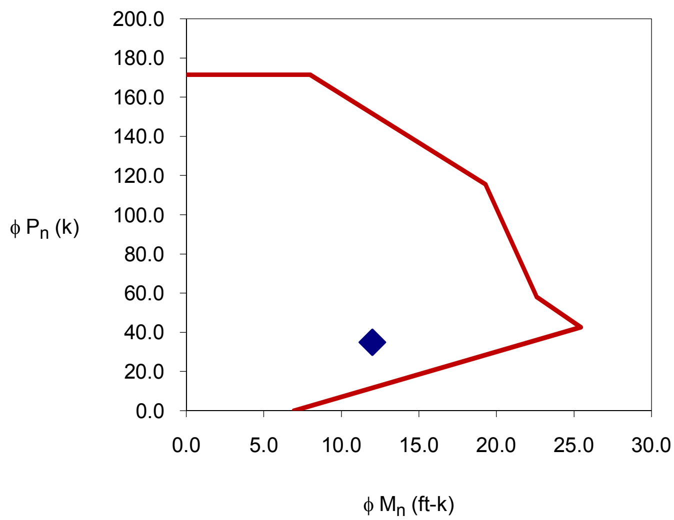
ANALYSIS

CHECK AXIAL & FLEXURE CAPACITY

ρ_{ProvD} = 0.00224 < ρ_{MAX} = 0.0400 (tension face only, ACI 318-11 10.3.5 or 10.9.1)

> ρ_{MIN} = 0.0008 (tension face only, ACI 318-11 10.5.1, 10.5.3 or 14.3.2)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	171.5	0.0
AT MAXIMUM LOAD	171.5	8.0
AT MIDDLE	115.5	19.3
AT ε_t = 0.002	59.5	22.5
AT BALANCED	58.1	22.6
AT ε_t = 0.005	42.7	25.4
AT FLEXURE ONLY	0.0	7.0

(Note: For middle reforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-11 SEC.15.5.2, 11.1.3.1, & 11.2)

$\phi V_n = 2\phi b d \sqrt{f'_c}$ = 7.58 kips / ft > V_u **[Satisfactory]**

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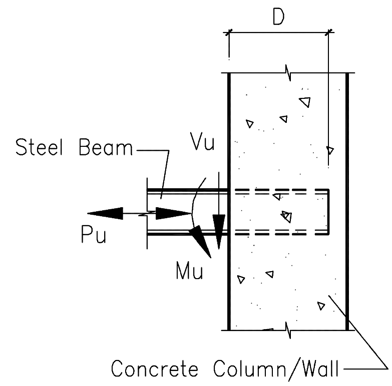
Composite Moment Connection Design Based on ACI 318-11

INPUT DATA & DESIGN SUMMARY

					d	A	b_f
BEAM SHAPE (Tube, Pipe, or WF) & SIZE		W24X192	< ==	W Shape	25.5	56.3	13.0

CONCRETE STRENGTH	f_c' =	3	ksi
FACTORED SHEAR LOAD	V_u =	205	kips
FACTORED MOMENT	M_u =	750	ft-kips
FACTORED VERTICAL LOAD (negative for uplift)	P_u =	691	kips
EMBEDMENT DEPTH	D =	48	in



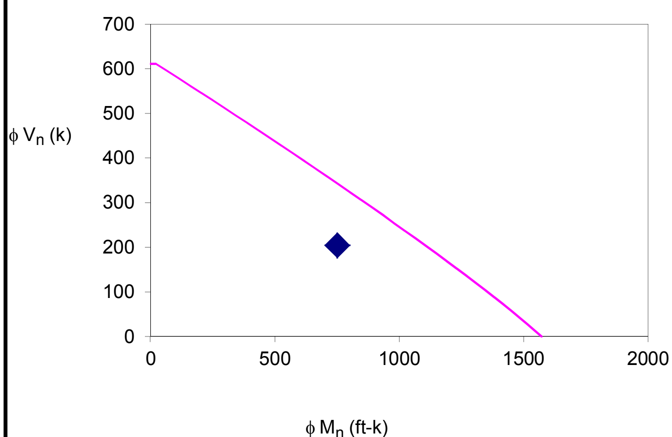
THE MOMENT CONNECTION DESIGN IS ADEQUATE.

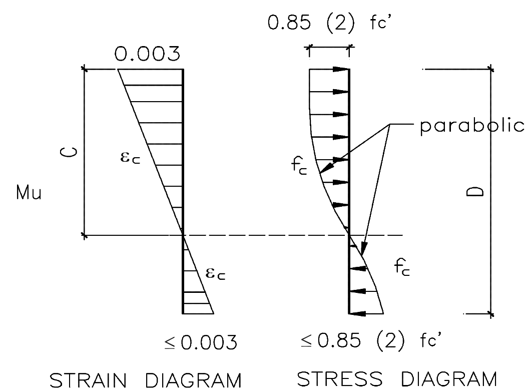
(A_{vf} = **16.0** in^2 , Required Area of Shear Studs or Welded Reinforcement.

Side edge of concrete must be wider than "b_f", and top/bottom concrete height can fully developed vertical bars.)

ANALYSIS

CHECK FLEXURAL & SHEAR CAPACITY (ACI 318 Chapter 9 & 10)





$$\varepsilon_O = \frac{2 f_c'}{E_C} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right)\ ,\ E_C = 57\sqrt{f_c'}$$

$$f_C = \begin{cases} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f_c'\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right]\ ,\ for\ \ 0 < \varepsilon_C < \varepsilon_O \\ 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f_c'\ ,\ for\ \ \varepsilon_C \ge \varepsilon_O \end{cases}$$

ϕM_n = 1103 ft-kips @ V_u = 205 kips

> M_u = 750 ft-kips [Satisfactory]

$\phi V_{n,max}$ = 611.08 kips, when C = 33.0 in

> V_u = 205 ft-kips [Satisfactory]

where ϕ = 0.65 , (ACI 318 9.3.2.4)

Bearing factor = 2, (ACI 318 22.5.5)

b = effective bearing width = 95% b_f = 12.35 in

CHECK HORIZONTAL AXIAL CAPACITY

ϕP_n = End Bering + Friction = 1295.4 kips > P_u = 691 kips [Satisfactory]

where End Bering = 0.65 (2) 0.85 f_c' A = 186.6 kips, (ACI 318 22.5.5)

Friction = 0.75 MAX(0.2f_c' A_c , 800 A_c) = 1108.8 kips, (ACI 318 11.7.5)

A = 56 in^2, end bearing area

A_c = 0.5 (2d + 2b_f) D = 1848 in^2, (0.5 for concrete cracked)

A_{vf} = $P_{u,Friction}$ / (ϕ f_y μ) = 16.0 in^2 , Required Area of Shear Studs or Welded Reinforcement

where ϕ = 0.75 , (ACI 318 9.3.2.3)

μ = 0.70 , (ACI 318 11.7.4.3)

f_y = 60 ks

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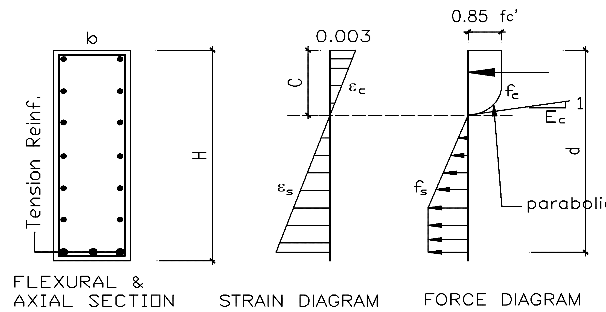
Flexural & Axial Design for Custom Metric Bars Based on Linear Distribution of Strain

DESIGN CRITERIA

This design method is, based on linear distribution of strain of ACI 318-11, for custom metric bars, which can apply to Shear Wall, Beam, and/or Column, including Metric System for different country codes.

INPUT DATA & DESIGN SUMMARY

REBAR YIELD STRESS	f_y =	400	N/mm^2 =	58	ksi
CONCRETE STRENGTH	f_c' =	20	N/mm^2 =	2.9	ksi
SECTION DEPTH	H =	1300	mm =	50.7	in
SECTION WIDTH	b =	300	mm =	11.7	in



SD LEVEL SECTION LOADS

P_u =	1250	kN =	281.0	kips, (axial force)
M_u =	400	kN•m =	293.0	ft-kips
V_u =	520	kN =	116.9	kips

THE SECTION DESIGN IS ADEQUATE.

AXIAL REINFORCEMENT

2	bars (side faces)	- single bar area	130	mm^2	- spacing	150	mm
(2	bars (side faces)	- single bar area	0.20	in^2	- spacing	5.9	in o.c.)

TENSION REINFORCEMENT

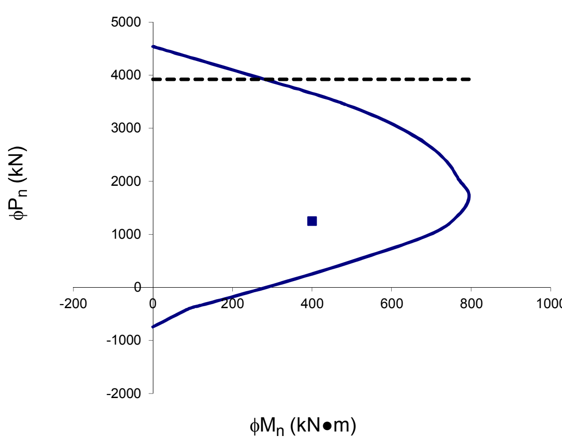
1	Layer	- each layer	3	bars	- single bar area	300	mm^2
(1	Layer	- each layer	3	bars	- single bar area	0.47	in^2)

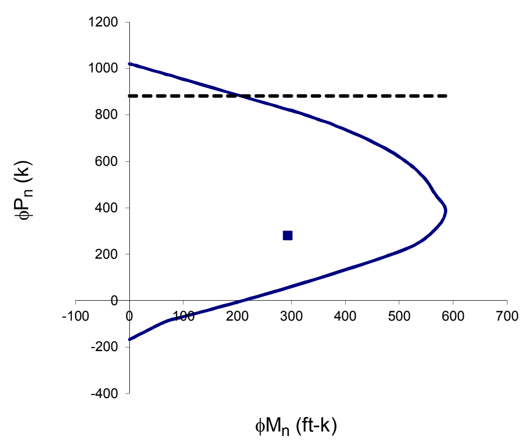
SHEAR REINFORCEMENT

2	legs (side faces)	- single leg area	300	mm^2	- spacing	150	mm
(2	legs (side faces)	- single leg area	0.47	in^2	- spacing	5.9	in o.c.)

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY





ε_c = 0.003 , (ACI 318-11 10.2.3)

ϕ = 0.834 , (for Pu & Mu, ACI 318-11 9.3.2)

d = 1259 mm = 49.6 in

c_b = 755 mm = 29.7 in

(balance point between Tension Controlled and Compression Controlled.)

P_u = 1250 kN = 281 kips

< ϕP_n = 3920 kN = 881 kips, (ACI 318-11 10.3.6.1)

M_u = 400.00 kN•m = 293.04 ft-kips

< ϕM_n = 750.27 kN•m = 549.65 ft-kips, at P_u level.

[Satisfactory]

CHECK SHEAR CAPACITY

V_u = 520 kN = 116.9 kips < $\phi V_n = \phi (V_s + V_c)$ = 533 kN = 119.8 kips

where ϕ = 0.75 (ACI 318-11 9.3.2.3) (ACI 318-11 11.1.1) [Satisfactory]

V_c = $2 (f_c')^{0.5} A_0$ = 142 kN = 31.9 kips, (ACI 318-11 11.2.1)

V_s = MIN ($d f_y A_v / s$, $4V_c$) = 568 kN = 127.8 kips, (ACI 318-11 11.4.7.2)

Purchaser's LOGO
(Please email us your LOGO at purchase.)

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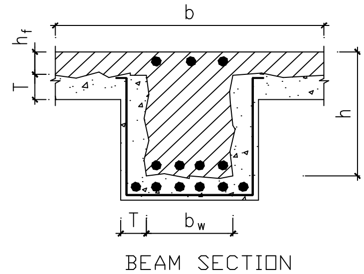
Existing Concrete Beam Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-14

DESIGN CRITERIA

1. The new shotcrete strength can be either higher or lower than existing. The capacity of existing shear reinforcement ignored conservatively.
2. Before new shotcrete, to rough, clear and wet the existing surface is required.

INPUT DATA & DESIGN SUMMARY



EXISTING BEAM SECTION SIZE	b_w =	24	in, (610 mm).
	h_f =	6	in, (152 mm).
	b =	60	in, (1524 mm), (ACI 318-14 6.3.2.1 & 9.2.4.4)
	h =	30	in, (762 mm).
EXISTING BEAM STRENGTH	old f_c' =	2.79	ksi, (19 MPa)
	old F_y =	40	ksi, (276 MPa)
NEW SHOTCRETE THICKNESS	T =	7	in, (178 mm).
NEW STEEL YIELD STRESS	F_y =	60	ksi, (414 MPa)
NEW SHOTCRETE STRENGTH	f_c' =	5	ksi, (34 MPa)
THE MAX CONCRETE STRESS	β =	0.85	f_c' (0.85 on ACI, 1.0 on AISC)
THE MAX CONCRETE STRAINS	ε_{max} =	0.003	, (ASCE 41-17 10.3.3.1)

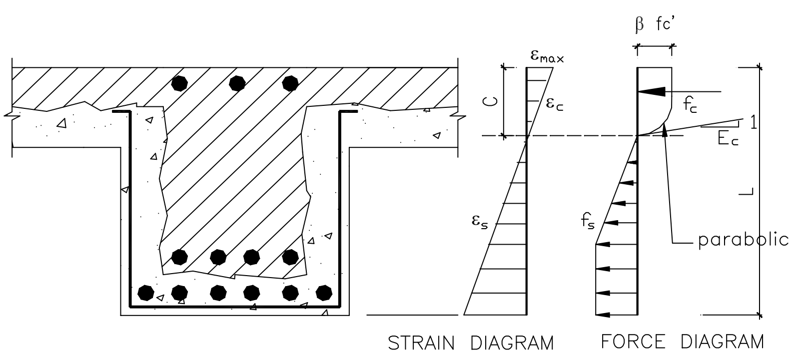
THE DESIGN IS ADEQUATE.

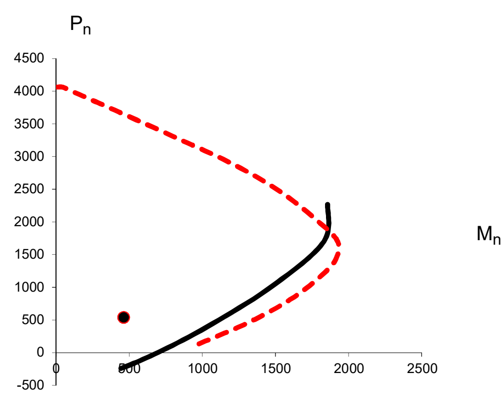
SECTION FORCES	V_u =	196	kips, (872 kN)			
(Strength Level, LRFD)	P_u =	350	kips, (1557 kN)	<== For this P_u, the max ϕM_n =	768.1	ft-kips, (1041 kN-m).
	M_u =	300	ft-kips, (407 kN-m)	<== For this M_u, the max ϕP_n =	3399.6	kips, (15122 kN),
				(P_u / ϕP_n =	0.103	)

OLD TOP REINFORCEMENT (zero for not sure)		3	#	8		
OLD BOTTOM REINFORCEMENT (zero for not sure)		4	#	8		
NEW BOTTOM REINFORCEMENT		5	#	7		
NEW SHEAR REINFORCEMENT	#	4	@	8	in. (203 mm), o.c.	Cover = 0.75 in, (19 mm)

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY





Solid Line - Tension Controlled
Dash Line - Compression Controlled

P_u / ϕ = 538.5 kips, (2395 kN)

M_{nc} @ P_u / ϕ = 1181.7 ft-kips, (1602 kN-m)

> M_u / ϕ = 461.5 ft-kips, (626 kN-m)

[Satisfactory]

where ϕ = 0.650 , (TMS 402-16 9.1.4.1, ACI 318-14 21.2 & AISC 341-16 B3.2)

CHECK SHEAR CAPACITY (ACI 318-14 10 & 22.5)

V_u / ϕ = 261.3 kips, (1162 kN) < $V_c + V_s$ = 311.2 kips, (1384 kN)

[Satisfactory]

where ϕ = 0.75 , (ACI 318-11 21.2)

V_c = 205.8 kips, (915 kN)

V_s = 105.4 kips, (469 kN)

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REVIEW BY :

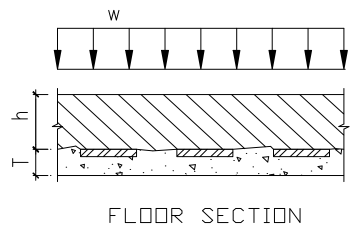
Existing Concrete Floor Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-14

DESIGN CRITERIA

1. The existing concrete floor can be either PT-concrete floor or regular one way slab. For PT-concrete floor, assume that only 50% existing concrete floor weight balanced by left post-tensioned force, without secondary moment changes added back to enhanced floor.
2. The capacity of existing floor regular reinforcement ignored conservatively. Superimposed load supported by new steel plates that epoxy adhered to existing bottom of floor.
3. Before new shotcrete, to rough, clear and wet the existing surface is required.

INPUT DATA & DESIGN SUMMARY

EXISTING FLOOR SECTION THICKNESS	h =	8	*in, (203 mm).*
EXISTING FLOOR STRENGTH	old f_c' =	4.3	*ksi, (30 MPa)*
FLOOR SPAN (larger span for two-way)	L =	30	*ft, (9.1 m).*
SUPERIMPOSED LOAD (on the top of floor)	w =	75	*psf, (3.6 kPa), ASD*



NEW SHOTCRETE THICKNESS	T =	3	*in, (76 mm).*
NEW STEEL YIELD STRESS	F_y =	50	*ksi, (345 MPa)*
NEW SHOTCRETE STRENGTH	f_c' =	5	*ksi, (34 MPa)*
THE MAX CONCRETE STRESS	β =	0.85	*f_c' (0.85 on ACI, 1.0 on AISC)*
THE MAX CONCRETE STRAINS	ε_{max} =	0.003	*, (ASCE 41-17 10.3.3.1)*

THE DESIGN IS ADEQUATE.

NEW BOTTOM REINFORCEMENT 1/8 in thk 5 in wide @ 12 *in. (305 mm), o.c.*

AXIAL EXISTING PT STRESS (zero for not sure) *PT / Area =* 0.175 *ksi, (1.2 MPa), ASD*

ANALYSIS

DETERMINE SECTION FORCES

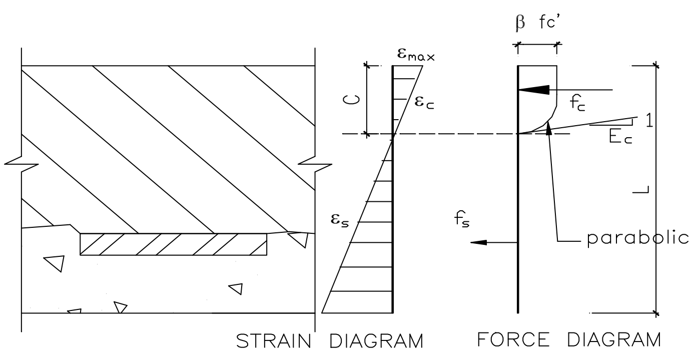
w_{total} = 1.6 w + 1.4 Wt = 120 + 122.5 = 242.5 *psf, (11.6 kPa), SD*

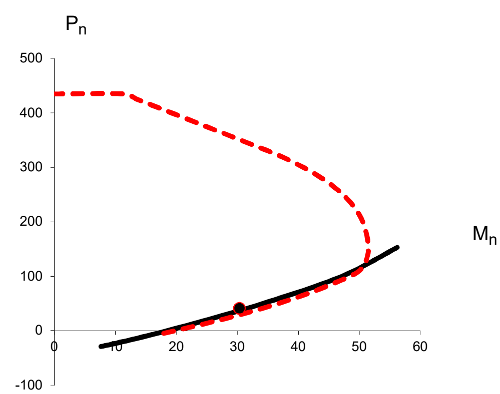
$M_u = (1/8)\, w_{total}\, L^2$ = 27.28 *ft-kips, (37 kN-m), per foot*

$V_u = (1/2)\, w_{total}\, L$ = 3.64 *kips, (16 kN), per foot*

$P_u = 1.6\, (PT / Area)\, (h + T)$ = 36.96 *kips, (164 kN), per foot*

CHECK FLEXURAL & AXIAL CAPACITY





P_u / ϕ = 41.1 *kips, (183 kN)*

M_{nc} @ P_u / ϕ = 32.8 *ft-kips, (45 kN-m)*

> M_u / ϕ = 30.3 *ft-kips, (41 kN-m)*

[Satisfactory]

where ϕ = 0.900 , (ACI 318-14 21.2 & AISC 341-16 B3.2)

Solid Line - Tension Controlled

Dash Line - Compression Controlled

CHECK SHEAR CAPACITY (ACI 318-14 10 & 22.5)

V_u / ϕ = 4.9 *kips, (22 kN)* **<** $V_c + V_s$ = 12.7 *kips, (56 kN)*

where ϕ = 0.75 , (ACI 318-11 21.2) **[Satisfactory]**

V_c = 12.7 *kips, (56 kN)*

V_s = 0.0 *kips, (0 kN)*

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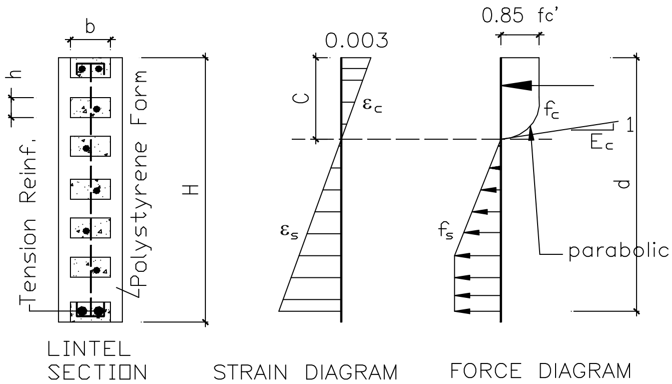
Lintel Design of Insulated Concrete Form (ICF) Based on ACI 318-14 & 2018 IBC

DESIGN CRITERIA

Insulated Concrete Form (ICF) can be waffle grid core, and screen grid core, and/or even flat wall core. This design method is based on linear distribution of strain, which only consider the concrete strength of screen grid core conservatively.

INPUT DATA & DESIGN SUMMARY



REBAR YIELD STRESS	f_y =	60	*ksi, (414 MPa)*
CONCRETE STRENGTH	f_c' =	3	*ksi, (21 MPa)*
OVERALL LINTEL DEPTH	H =	48	*in, (1219 mm)*
LINTEL CORE SIZE	b =	6	*in, (152 mm)*
	h =	12	*in, (305 mm)*

SD LEVEL SECTION LOADS

P_u =	50	*kips, (222 kN), horizontal axial force.*
M_u =	100	*ft-kips, (136 kN-m)*
V_u =	35	*kips, (156 kN)*

VERTICAL REINFORCEMENT IN CONCRETE

1 # 5 @ 24 *in. (610 mm), o.c.*

THE LINTEL DESIGN IS ADEQUATE.

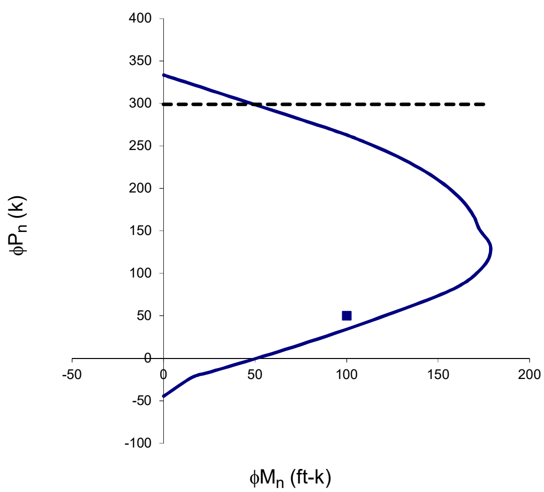
HORIZONTAL REINFORCEMENT WITH CORE SPACING

1 # 5 @ 18 *in. (457 mm), o.c.*

TENSION REINFORCEMENT 1 Layer 2 # 6

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY



ε_c = 0.003 , (ACI 318-14 22.2.2)

ϕ = 0.900 , (for P_u & M_u, ACI 318-14 21.2)

d = 46.9 in

c_b = 27.7 in, (balance point between Tension Controlled and Compression Controlled.)

P_u = 50 kips

< ϕP_n = 299 kips, (ACI 318-14 22.4.2)

M_u = 100 ft-kips

< ϕM_n = 120 ft-kips, at P_u level.

[Satisfactory]

CHECK SHEAR CAPACITY

V_u = 35 kips < $\phi V_n = \phi (V_s + V_c)$ = 39 kips, (ACI 318-14 22.5.1) [Satisfactory]

where ϕ = 0.75 (ACI 318-14 21.2)

$V_c = 2 (f_c')^{0.5} A_0$ = 10.5 kips, (ACI 318-14 22.5.5)

V_s = MIN (d $f_y A_v$ / s , $4V_c$) = 42.1 kips, (ACI 318-14 22.5.10.5)

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Bearing Wall Design of Insulated Concrete Form (ICF) Based on ACI 318-14 & 2018 IBC

DESIGN CRITERIA

1. Insulated Concrete Form (ICF) can be waffle grid core, and screen grid core, and/or even flat wall core. This design method is based on linear distribution of strain, which only consider the concrete strength of screen grid core conservatively.
2. Since it is difficult to design ICF wall tension-controlled (ACI 318-14 11.8.1.1) only, this software does NOT use the method of *Alternative Design of Slender Walls* (ACI 318-14 11.8), so no P-Δ effects have to be applied (ACI 318-11 11.8.1.1).

INPUT DATA & DESIGN SUMMARY

FACTORED MAX OUT-OF-PLANE SECTION MOMENT (from wind or seismic lateral loads, the max value of $w_{max} H^2/8$ may be input).

M_u = 2.3 *ft-kips / ft, (10.2 kN-m / m)*

FACTORED AXIAL LOAD (con-currently at the same section with M_u).

P_u = 20 *kips/ft, (291.7 kN/m)*

FACTORED SHEAR LOAD V_u = 0.98 *kips/ft, (14.3 kN/m)*

CONCRETE STRENGTH f_c' = 3 *ksi, (21 MPa)*

REBAR YIELD STRESS f_y = 60 *ksi, (414 MPa)*

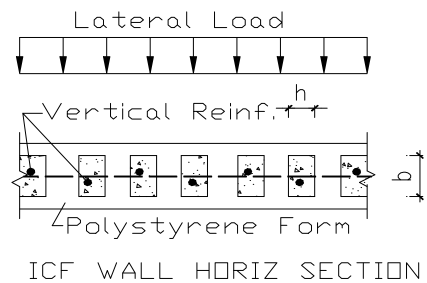
WALL CORE SIZE b = 6 *in, (152 mm)*

h = 12 *in, (305 mm)*

VERTICAL REINFORCEMENT (A_s) WITH CORE SPACING

\# 5 @ 24 *in. (610 mm), o.c.*

A_s LOCATION (1=at middle, 2=at each face) 1 at middle



THE WALL DESIGN IS ADEQUATE.

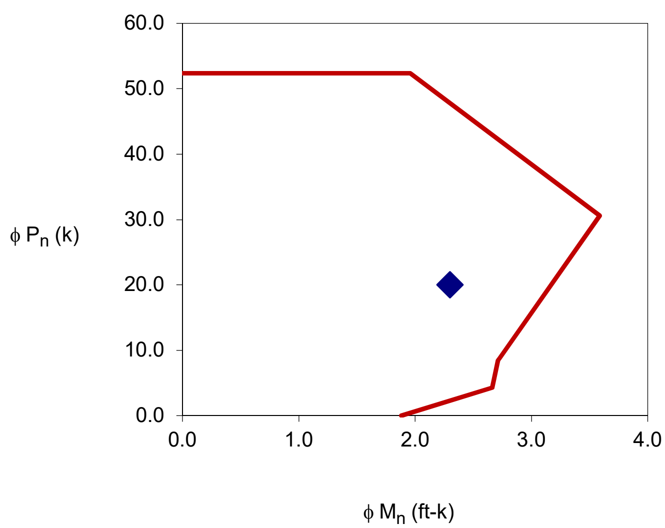
ANALYSIS

CHECK AXIAL & FLEXURE CAPACITY

ρ_{ProvD} = 0.00861 < ρ_{MAX} = 0.0800 (tension face only, ACI 318-14 22.2.3.1 or 10.6.1.1)

\> ρ_{MIN} = 0.0015 (tension face only, ACI 318-14 9.6.1 or 11.6.1)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	52.4	0.0
AT MAXIMUM LOAD	52.4	2.0
AT MIDDLE	30.6	3.6
AT ε_t = 0.002	8.8	2.7
AT BALANCED	8.4	2.7
AT ε_t = 0.005	4.3	2.7
AT FLEXURE ONLY	0.0	1.9

(Note: For middle reinforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

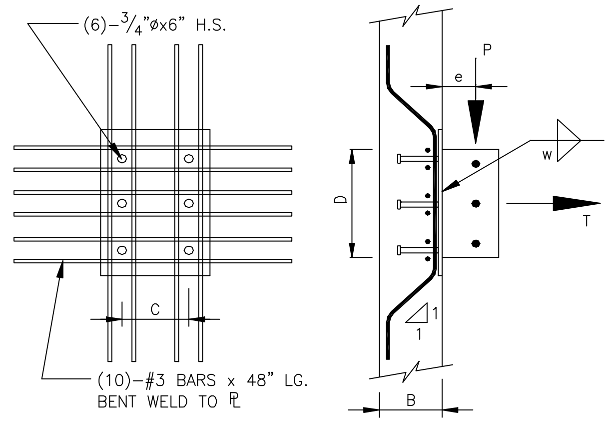
CHECK SHEAR CAPACITY (ACI 318-14 22.5.5)

$\phi V_n = 2\phi b d\sqrt{f'_c}$ = 1.48 kips / ft > V_u **[Satisfactory]**

Design for Connection to Wall Based on AISC 360-16 & ACI 318-14

DESIGN CRITERIA

1. The wall can be tiltup panel or vertical pour/curing concrete wall.
2. The (6) headed studs and (10) bent bars are con-currently required to weld embedded plate, so punching capacity (ACI 318-14 8.4 & 8.5) controls concrete design.
3. The loads (T, P, & P e) may be from beam, girder, collector, or truss.



INPUT DATA & DESIGN SUMMARY

LATERAL TENSION LOAD, ASD	T =	15	*kips, (67 kN)*
GRAVITY SERVICE LOAD	P =	47	*kips, (209 kN)*
DISTANCE FROM WALL FACE	e =	4.75	*in, (121 mm)*
CONCRETE STRENGTH	$f_c{}'$ =	4	*ksi, (28 MPa)*
PLATE STEEL YIELD STRESS	F_y =	50	*ksi, (345 MPa)*
MINIMUM PLATE THICKNESS	t =	1	*in, (25 mm)*
TRIAL WELD SIZE	w =	0.625	*in, (16 mm)*
CONCRETE WALL THICKNESS	B =	8	*in, (203 mm)*
GUSSET DEPTH / EFFECTIVE DEPTH	D =	8.5	*in, (216 mm)*
BOLT SPACING / EFFECTIVE WIDTH	C =	6	*in, (152 mm)*

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK PUNCHING CAPACITY FOR COLUMN (ACI 318-14 8.4 & 8.5)

$$\phi v_c(psi) = \phi(2+y)\sqrt{f_c'} + \phi\left(A_V f_y \sin\alpha\right)/A_P = \quad 190 \text{ psi} \quad > \quad v_u(psi) = \frac{1.5T}{A_P} + \frac{0.5\gamma_v b_1 1.5eP}{J} = \quad 104 \text{ psi}$$

[Satisfactory]

Where ϕ = 0.75 (ACI 318-14 21.2)

β_c = ratio of long side to short side of concentrated load = 1.42

d = B - 2 (0.5 d_b) - 0.75 " cover = 6.875 in

A_p = b_0 d = 388 in^2

b_1 = (C + d) = 12.875 in

A_v = 0 in^2, conservatively

b_2 = (D + d) = 15.375 in

α = 45 ^{0}

b_0 = $2b_1 + 2b_2 = 2(C + d) + 2(D + d)$ = 56.5 in

f_y = 60 ksi

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = \quad 0.3789$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1+\left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] = \quad 0.57 \text{ ft}^4$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right) = \quad 2.0$$

, (if bent bars used, y must be zero per ACI 22.6)

CHECK CAPACITY OF WELDING (AISC 360 J2)

Min allowable weld	=	0.31	in	[Satisfactory]
Max allowable weld	=	0.94	in	[Satisfactory]
t_e	=	0.44	in	
I = 2 (t_e D^3 / 12)	=	45.2	in^4	
Vertical shear = P / A_w = P / 2 D t_e	=	6.3	ksi	
Bending stress = 0.5 P e D / I	=	21.0	ksi	
Tension stress = T / A_w = T / 2 D t_e	=	2.0	ksi	
Resultant Stress = [$(P/A_w)^2 + (0.5\ P\ e\ D / I + T/A_w)^2\ ]^{0.5}$ =		23.8	ksi	
Allow shear F_w / Ω = F_w / 2.0	=	28.6	ksi	
	>	23.8	ksi	[Satisfactory]

θ =	17.7	deg, (AISC 360 J2-5)	
Δ_u =	0.0868	in	
Δ_m =	0.0503	in	
f(p) =	1.2563	, (AISC 360 J2-9)	
F_w =	57.188	ksi, (AISC 360 J2-8)	

CHECK PLATE SHEAR CAPACITY (AISC 360 G2)

P / A = 5.5 ksi < $0.6\ F_y\ C_v / \Omega_v = 0.6\ F_y\ 1.0 / 1.5$ = 20 ksi [Satisfactory]

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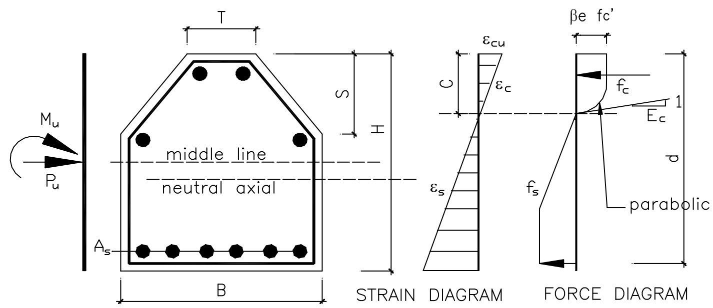
Irregular Section Design of Concrete Beam/Column Based on & ACI 318-19

DESIGN CRITERIA

1. Concrete design is a section design with equilibrium and strain compatibility. For irregular section, no matter the input is pure axial compression at middle of section or not, the moment always exists because **ϕP ~ ϕM** is based on neutral axis, not the middle of height. (ACI 318-19 22.2.1.2)
2. The moment, from middle of section to the neutral axial, is increased, so directly check the middle section compression capacity is inadequate.
3. Conservatively assume that only bottom tension reinforcement used for capacity, so the maximum pure tension, **-ϕP_{max}**, is not at moment zero.

INPUT DATA & DESIGN SUMMARY



SECTION SIZE	*H* =	**36**	*in, (914 mm).*
	B =	**36**	*in, (914 mm).*
	T =	**12**	*in, (305 mm).*
	S =	**24**	*in, (610 mm).*

(Tip: for triangle, input T = 0 and S = H)

STEEL & CONCRETE	F_y =	**60**	*ksi, (414 MPa)*
	f_c' =	**4**	*ksi, (28 MPa)*
	β =	**0.85**	f_c' *(0.85 on ACI, 1.0 on AISC)*

Bending: **6** # **8** *(bottom reinforcement,* A_s *only)*

Shear: # **4** @ **8** *in. (203 mm), o.c.* Cover = **0.75** *in, (19 mm)*

THE DESIGN IS ADEQUATE.

SECTION FORCES	V_u =	**80**	*kips, (356 kN)*
(Strength Level, LRFD)	P_u =	**820**	*kips, (3648 kN)*
	M_u =	**300**	*ft-kips, (407 kN-m)*

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY

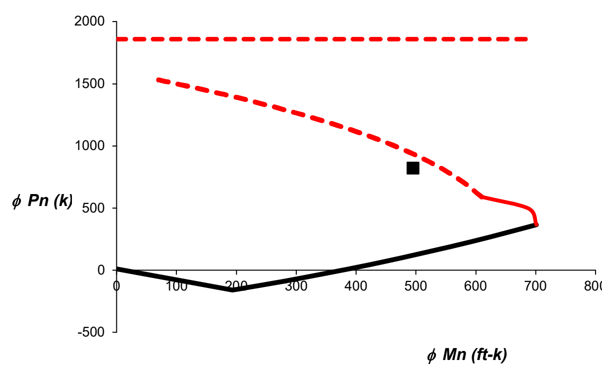
y = 15.14 *in, (385 mm). neutral location from bottom*

$M_{u,\,neutral}$ = **495.2** *ft-kips, (671 kN-m)*

$$f_c=\begin{cases}\beta_e f_c'\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right)-\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] & ,\ for\ \ 0<\varepsilon_c<\varepsilon_o\\ \beta_e f_c' & ,\ for\ \ \varepsilon_c\ge\varepsilon_o\end{cases}\quad ,\quad f_s=\begin{cases}\varepsilon_s E_s & ,\ for\ \ \varepsilon_s\le\varepsilon_{ty}\\ f_y & ,\ for\ \ \varepsilon_s>\varepsilon_{ty}\end{cases}$$

$$\varepsilon_o=\frac{2\left(\beta_e f_c'\right)}{E_c}\ ,\ \varepsilon_{ty}=\frac{f_y}{E_s}\ ,\ \varepsilon_t=\begin{cases}\varepsilon_{ty} & ,\ for\ \ compression\ \ controlled\\ \varepsilon_{ty}+0.003 & ,\ for\ \ tension\ \ controlled\end{cases}$$

$$E_c=57\sqrt{f_c'}\ ,\ E_s=29000ksi\ ,\ \beta_e=0.85\ ,\ \varepsilon_{cu}=0.003$$



Solid Black Line - **Tension Controlled**

Solid Red Line - **Transition**

Dash Line - **Compression Controlled**

ϕ P_{max} = 1860 *kips, (8273 kN)*

> P_u **[Satisfactory]**

ϕ M_n = 541 ft-kips @ P_u = 820 kips

> $M_{u,\,neutral}$ = 495.2 ft-kips **[Satisfactory]**

CHECK SHEAR CAPACITY (ACI 318-19 10 & 22.5)

V_u / ϕ = 106.7 *kips, (474 kN)* < V_c + V_s = 222.3 *kips, (989 kN)*

where ϕ = 0.75 , (ACI 318-19 21.2) **[Satisfactory]**

V_c = 119.5 *kips, (532 kN)*

V_s = 102.8 *kips, (457 kN)*

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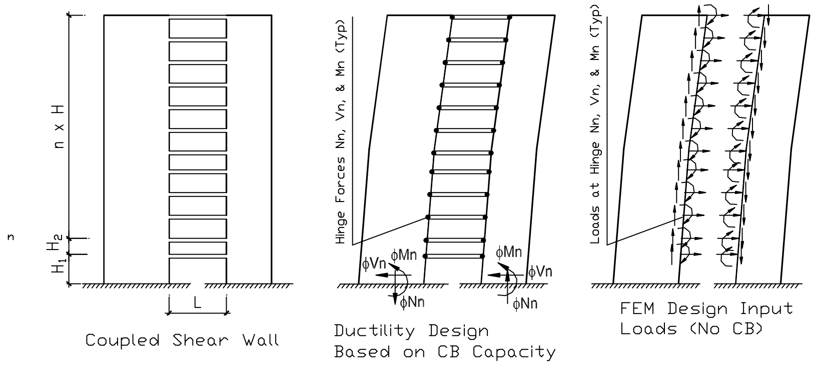
Coupled Shear Walls Design Based on ASCE 7-22 & ACI 318-19

DESIGN CRITERIA

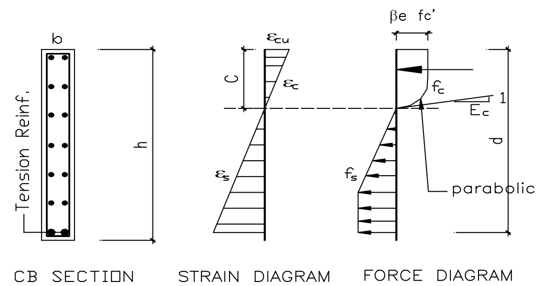
All coupled shear walls (RC, pure Steel, or Composite Plate Shear Wall) have to be ductility designed:

1. *Coupling beams (CB) have to have stronger shear and axial capacities than flexural. No matter what level seismic loads (ASD, SD, MCE, or FLE), the CB plastic hinges only happened at the ends.*
2. *Shear walls have to have the minimum capacities* ϕM_n, ϕN_n, *and* ϕV_n, *at* ***SD*** *design level, based on all CB end hinges with possible maximum flexural force,* $(\phi M_{n,CB})_{max}$.
3. *The CB hinge* $\phi M_{n,CB}$ & $\phi N_{n,CB}$ *always coupled together, no matter which is RC (ACI 318-19 Tab. 21.2.2 & Fig. R10.4.2.1), pure Steel (AISC 360-16 H1), or Filled Composite (AISC 360-16 I 1.1b & I4).*



INPUT DATA & DESIGN SUMMARY

SHEAR WALL WIDTH (smaller)				32	ft, (9.8m)				
CB LENGTH		L =		12	ft, (3.7m)				
CB DEPTH		h =		36	in, (914 mm)				
CB WIDTH		b =		12	in, (305 mm)				
REBAR YIELD STRESS		f_y =		60	ksi, (414 MPa)				
CONCRETE STRENGTH		f_c' =		4	ksi, (28 MPa)				
CB VERTICAL REINF.									
	2	#	4	@	6	in. (152 mm), o.c.			
CB HORIZONTAL REINF.									
	2	#	4	@	8	in. (203 mm), o.c.			
TENSION REINFORCEMENT				1	Layer	3	#	6	
STORY HEIGHT		Typical:	n x H =	12	x	14	ft, (4.3m)		
Total Height:	*204*	*ft, (62.2m)*			H_2 =	12	ft, (3.7m)		
					H_1 =	24	ft, (7.3m)		



THE CB DESIGN IS ADEQUATE.

The Shear Walls Min ReqD Capacities:

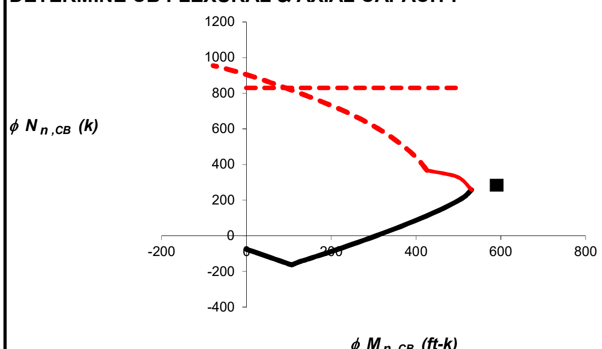
ϕN_n = ***1377.2*** *kips, (6126 kN)*

ϕM_n = ***418452.3*** *ft-kips, (567345 kN-m)*

ϕV_n = ***4014.9*** *kips, (17859 kN)*

ANALYSIS

DETERMINE CB FLEXURAL & AXIAL CAPACITY



Solid Black Line - **Tension Controlled**
Solid Red Line - **Transition**
Dash Line - **Compression Controlled**

The CB end hinge possible maximum flexural forces:

$N_n = \phi N_{n,CB} / \phi$ = ***286.8*** *kips, (1276 kN), horizontal force.*

$M_n = (\phi M_{n,CB})_{max} / \phi$ = ***590.2*** *ft-kips, (800 kN-m)*

CHECK CB STRONG SHEAR WEAK FLEXURAL

$2M_n / L$ = 98 kips < $\phi V_n = \phi (V_s + V_c)$ = 102 kips, (ACI 318-19 22.5.1) [Satisfactory]

where ϕ = 0.75 (ACI 318-19 21.2)

$V_c = 2 (f_c')^{0.5} A_0$ = 27.3 kips, (ACI 318-19 22.5.5)

V_s = MIN ($d f_y A_v / s$, $4V_c$) = 109.3 kips, (ACI 318-19 22.5.10.5)

Purchaser's LOGO

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DESIGN BY :
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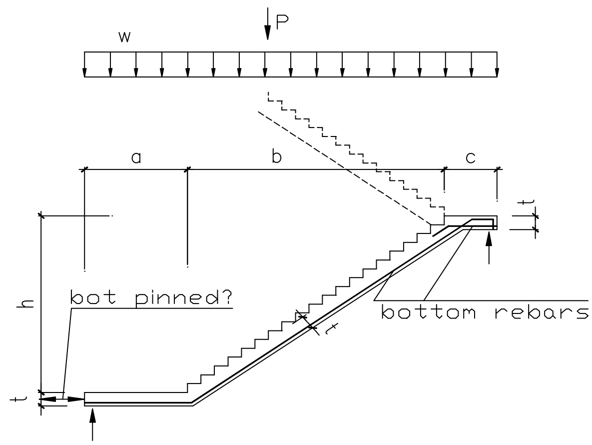
Concrete Stair Design Based on 2021 IBC & ACI 318-19

DESIGN CRITERIA

1. The concrete stair is one way RC slab without stringer and landing beam to save the space.

2. The suggested concrete cover is 1-1/2 in for outside stair, 3/4 in for inside stair, or 3 in against soil. (ACI 318-19 20.6.1.1)

3. The stair concrete can be precast or cast in place. If the horizontal pinned at bottom end, the section axial force may be in tension.



INPUT DATA & DESIGN SUMMARY

DIMENSIONS

h = **12** *ft, (3.7 m)*
a = **6** *ft, (1.8 m)*
b = **16** *ft, (4.9 m)*
c = **1.5** *ft, (0.5 m)*

STAIR WIDTH W = **6** *ft, (1.8 m)*
THICKNESS t = **12** *in, (305 mm)*

THE STAIR DESIGN IS ADEQUATE.

CONCRETE STRENGTH f_c' = **4** *ksi, (28 MPa)*
CONCRETE COVER C_c = **0.75** *in, (19 mm)*
REBAR YIELD STRESS f_y = **60** *ksi, (414 MPa)*
TENSION REINFORCEMENT (bottom rebars)
\# **6** @ **8** *in. (203 mm), o.c.*

LOADS w_{DL} = **172.5** *psf, (8.3 kPa), including self weight.*
w_{LL} = **100** *psf, (4.8 kPa)*
P_{LL} = **0.8** *kips, (3.6 kN)*
Horizontal Pinned at Bottom End **?** **Yes** *(Yes or No)*

ANALYSIS

DETERMINE THE POSSIBLE CRITICAL SECTION FORCES (ACI 318-19 Table 5.3.1)

V_u = 1.6 P_{LL} + 0.5 w_u (a + b + c) = ***27.2*** *kips, this should also be the end vertical connection force.*

N_u = - [0.5 w_u (a + b) + 1.6 P_{LL}] h / $(h^2 + b^2)^{0.5}$, **or** , zero
= ***-15.3*** *kips, axial force, zero or negative because always* **Tension Controlled** *(ACI 318-19 Fig. R21.2.2).*

M_u = 0.25 (a + b + c) 1.6 P_{LL} + 0.125 w_u $(a + b + c)^2$ = ***159.5*** *ft-kips, (M_u & N_u may not be at the same section.)*

where w_u = W (1.2 w_{DL} + 1.6 w_{LL}) = ***2.20*** *kips / ft*

CHECK DEFLECTION / STIFFNESS (ACI 318-19 Table 7.3.1.1 **or** 24.2.2)

L / 20 = ***16.5*** *in* **>** t = ***12.0*** *in* **[Unsatisfactory]**

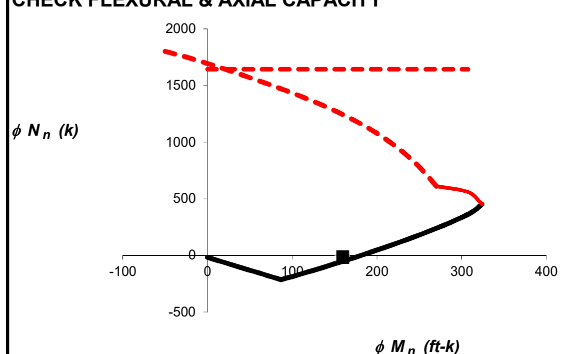
$\Delta_{(3DL + LL)}$ = 5 $w_{(3DL + LL)}$ L^4 / (384 EI) + P_{LL} L^3 / (48 EI) = ***1.29*** *in* **<** L / 240 = ***1.38*** *in*

where *L* = a + $(h^2 + b^2)^{0.5}$ + c = ***330.0*** *in* **[Satisfactory]**

$w_{(3DL + LL)}$ = W (3 w_{DL} + w_{LL}) = ***0.31*** *kips / in*

EI = 57 $(f_c')^{0.5}$ W t^3 / 12 = ***37376604*** *in^2-kips*

CHECK FLEXURAL & AXIAL CAPACITY



Solid Black Line - **Tension Controlled**
Solid Red Line - **Transition**
Dash Line - **Compression Controlled**

ρ_{ProvD} = 0.00506 **<** ρ_{MAX} = 0.0800
(tension face only, ACI 318-19 22.2.3 or 10.9.1)
> ρ_{MIN} = 0.0015
(tension face only, ACI 318-19 6.6.4.3, 9.6.1 or 11.6.1)

[Satisfactory]

CHECK SHEAR CAPACITY

V_u = 27.2 kips **<** $\phi V_n = \phi (V_s + V_c)$ = 37 kips, (ACI 318-19 22.5.1) **[Satisfactory]**

where ϕ = 0.75 (ACI 318-19 21.2)

V_c = 2 $(f_c')^{0.5} A_0$ = 49.5 kips, (ACI 318-19 22.5.5)

V_s = 0.0 kips, (ACI 318-19 22.5.10.5)

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REVIEW BY :

Design for Two-Way Concrete Slab on Wall Based on ACI 318-19 using Finite Element Method

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH

f_c' = 3 *ksi, (21 MPa)*

REBAR YIELD STRESS

f_y = 60 *ksi, (414 MPa)*

WALL SPACING EACH WAY

L = 32 *ft, (9.75 m)*

B = 30 *ft, (9.14 m)*

SLAB THICKNESS

t = 8 *in, (203 mm)*

DEAD LOAD & Wt DL = 100 *psf, (4.8 kPa)*

LIVE LOAD LL = 70 *psf, (3.4 kPa)*

TOP BARS AT WALL EACH WAY (Perpendicular to Wall)

\# 5 @ 10 *in, (254 mm), o.c.*

x 10.7 ft. long, with 0.75 *in, (19 mm), cover*

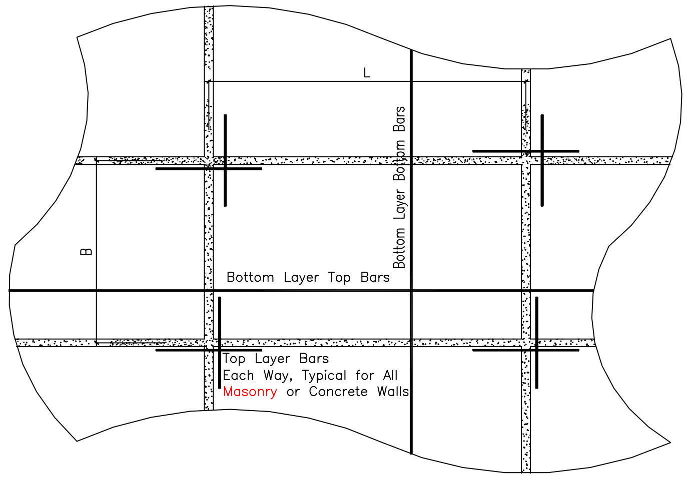
BOTTOM LAYER TOP BARS

\# 5 @ 16 *in, (406 mm), o.c.*

BOTTOM LAYER BOTTOM BARS

\# 5 @ 16 *in, (406 mm), o.c.*

with 0.75 *in, (19 mm), bottom concrete cover*



THE DESIGN IS ADEQUATE.

ANALYSIS

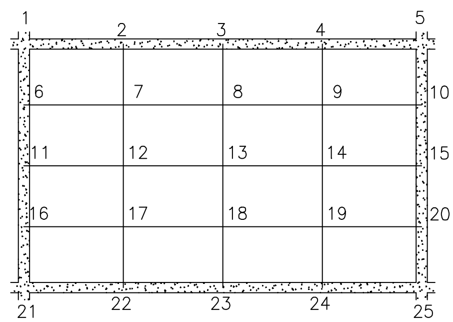
w_c = 150 pcf, (ACI 318-19 19.2.2.1)

$E_c = w_c^{1.5}\, 33\, f'_c{}^{0.5}$ = 3321 ksi, (ACI 318-19 19.2.2.1)

t_e = $(I_e / I_g)^{1/3}\, t = (0.25\, I_g / I_g)^{1/3}\, t = (0.25)^{1/3}\, t$ = 0.63 t = 5.0 in, for Slab only (ACI 318-19 8.3.2, 24.2 & 6)

Joint Number	Δ_u in	R_u kips/ft
1	0.000	-0.258
2	0.000	1.117
3	0.000	2.329
4	0.000	1.117
5	0.000	-0.258
6	0.000	0.924
7	0.422	0.000
8	0.696	0.000
9	0.422	0.000
10	0.000	0.924
11	0.000	2.170
12	0.709	0.000
13	**1.174**	0.000
14	0.709	0.000
15	0.000	2.170
16	0.000	0.924
17	0.422	0.000
18	0.696	0.000
19	0.422	0.000
20	0.000	0.924
21	0.000	-0.258
22	0.000	1.117
23	0.000	2.329
24	0.000	1.117
25	0.000	-0.258

Bending Section	M_u ft-k/ft
1 - 2	6.0
2 - 3	2.0
1 - 6	6.7
6 - 11	1.8
3 - 8	-3.7
8 - 13	-5.8
11 - 12	-3.8
12 - 13	-5.0



DETERMINE FACTORED LOAD (ACI 318-19 5.3)

w_u = 1.2 DL + 1.6 LL = 0.232 ksf

DETERMINE FLEXURE CAPACITY (ACI 318-19 7.6.1.1, 8.6.1.1, & 22)

	Top Bar #5 @ 10" o.c.	**Bot. Layer Bot.** 5 @ 16" o.c.	**Bot. Layer Top** 5 @ 16" o.c.
d (in)	6.31	6.94	6.31
$\mathbf{A_s}$ (in²/ft)	0.37	0.23	0.23
$\mathbf{A_{s,\,min}}$ (in²/ft)	0.25	0.17	0.17
a (in)	0.73	0.46	0.46
$\boldsymbol{\phi M_n}$ (ft-k/ft)	10.0	7.0	6.4

CHECK FLEXURE CAPACITY

$M_{u,Top} = \text{Max}(M_{u,1\text{-}5}, M_{u,1\text{-}21})$ = 6.7 ft-k/ft < ϕM_n = 10.0 ft-k/ft [Satisfactory]

$M_{u,Bot,Bot} = -\text{Min}(M_{u,8\text{-}13}, M_{u,12\text{-}13})$ = 5.8 ft-k/ft < ϕM_n = 7.0 ft-k/ft [Satisfactory]

$M_{u,Bot,Top} = -\text{Max}(M_{u,8\text{-}13}, M_{u,12\text{-}13})$ = 5.0 ft-k/ft < ϕM_n = 6.4 ft-k/ft [Satisfactory]

CHECK LIVE LOAD DEFLECTION (ACI 318-19 Table 24.2.2)

$\Delta_{LL} = \Delta_{u,Max}$ LL / (1.2 DL + 1.6 LL) = 0.35 in < L / 360 = 1.07 in

[Satisfactory]

CHECK LONG-TERM DEFLECTION (ACI 318-19 24.2.4)

$\Delta_{3DL+LL} = \Delta_{u,Max}$ (3DL + LL) / (1.2 DL + 1.6 LL) = 1.87 in < L / 180 = 2.13 in

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-19 13.2.7.2, 7.4.3, & 22.5)

$\phi V_n = 2\phi b d \sqrt{f'_c}$ = 6.22 kips/ft > V_u = 2.33 kips/ft [Satisfactory]

where ϕ = 0.75 (ACI 318-19 21.2)

d = 6.3 in b = 12.0 in

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Design for Two-Way Concrete Slab on Wall Based on ACI 318-19 using Finite Element Method

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH f_c' = 3 *ksi, (21 MPa)*

REBAR YIELD STRESS f_y = 60 *ksi, (414 MPa)*

WALL SPACING EACH WAY

L = 32 *ft, (9.75 m)*

B = 30 *ft, (9.14 m)*

SLAB THICKNESS t = 8 *in, (203 mm)*

DEAD LOAD & SELF WT DL = 100 *psf, (4.8 kPa)*

LIVE LOAD LL = 70 *psf, (3.4 kPa)*

TOP BARS AT WALL EACH WAY (Perpendicular to Wall)

5 @ 10 *in, (254 mm), o.c.*

x 10.7 ft. long, with 2 *in, (51 mm), cover*

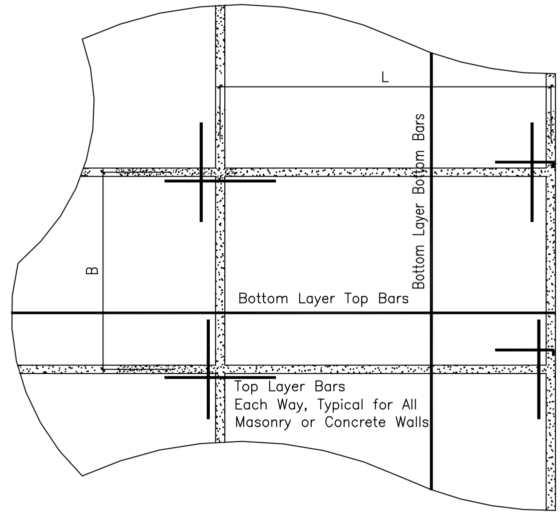
BOTTOM LAYER TOP BARS

5 @ 16 *in, (406 mm), o.c.*

BOTTOM LAYER BOTTOM BARS

5 @ 16 *in, (406 mm), o.c.*

with 0.75 *in, (19 mm), bottom concrete cover*



THE DESIGN IS ADEQUATE.

ANALYSIS

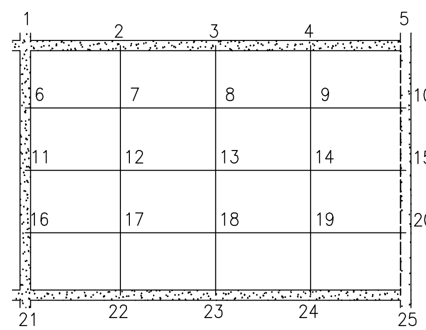
w_c = 150 pcf, (ACI 318-19 19.2.2.1)

$E_c = w_c^{1.5}\, 33\, f'_c{}^{0.5}$ = 3321 ksi, (ACI 318-19 19.2.2.1)

$t_e = (I_e / I_g)^{1/3} t = (0.25\, I_g / I_g)^{1/3} t = (0.25)^{1/3} t =$ 0.63 t = 5.0 in, for Slab only (ACI 318-19 8.3.2, 24.2 & 6)

Joint Number	Δ_u in
1	0.000
2	0.000
3	0.000
4	0.000
5	0.000
6	0.000
7	0.422
8	0.697
9	0.424
10	0.000
11	0.000
12	0.710
13	1.175
14	0.710
15	0.000
16	0.000
17	0.422
18	0.697
19	0.424
20	0.000
21	0.000
22	0.000
23	0.000
24	0.000
25	0.000

Bending Section	M_u ft-k/ft
1 - 2	6.0
2 - 3	2.0
3 - 4	2.0
4 - 5	6.2
1 - 6	6.7
6 - 11	1.8
12 - 13	-5.0
13 - 14	-5.0
7 - 12	-3.5
8 - 13	-5.7
9 - 14	-5.7



DETERMINE FACTORED LOAD (ACI 318-19 5.3)

w_u = 1.2 DL + 1.6 LL = 0.232 ksf

DETERMINE FLEXURE CAPACITY (ACI 318-19 7.6.1.1, 8.6.1.1, & 22)

	Top Bar	**Bot. Layer Bot.**	**Bot. Layer Top**
	# 5 @ 10" o.c.	5 @ 16" o.c.	5 @ 16" o.c.
d (in)	5.06	6.94	6.31
$\mathbf{A_s}$ (in²/ft)	0.37	0.23	0.23
$\mathbf{A_{s,\,min}}$ (in²/ft)	0.20	0.17	0.17
a (in)	0.73	0.46	0.46
$\phi\mathbf{M_n}$ (ft-k/ft)	7.9	7.0	6.4

CHECK FLEXURE CAPACITY

$M_{u,Top}$ = Max($M_{u,1\text{-}5}$, $M_{u,1\text{-}11}$) = 6.7 ft-k/ft < ϕM_n = 7.9 ft-k/ft [Satisfactory]

$M_{u,Bot,Bot}$ = - Min(M_u) = 5.7 ft-k/ft < ϕM_n = 7.0 ft-k/ft [Satisfactory]

$M_{u,Bot,Top}$ = - Max($M_{u,12\text{-}14}$, $M_{u,7\text{-}12}$, $M_{u,8\text{-}13}$, $M_{u,9\text{-}14}$) = 3.5 ft-k/ft

< ϕM_n = 6.4 ft-k/ft [Satisfactory]

CHECK LIVE LOAD DEFLECTION (ACI 318-19 Table 24.2.2)

$\Delta_{LL} = \Delta_{u,Max}$ LL / (1.2 DL + 1.6 LL) = 0.35 in < L / 360 = 1.07 in

[Satisfactory]

CHECK LONG-TERM DEFLECTION (ACI 318-19 24.2.4)

$\Delta_{3DL + LL} = \Delta_{u,Max}$ (3DL + LL) / (1.2 DL + 1.6 LL) = 1.87 in < L / 180 = 2.13 in

[Satisfactory]

Prestressed Concrete Girder Design for Bridge Structure Based on AASHTO 17th Edition & ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c =	6	ksi
REBAR STRENGTH	$f_y^* = f_y$ =	60	ksi
TENDON TENSILE STRENGTH	$f_{su}^* = f_{pu}$ =	270	ksi
TENDON YIELD STRENGTH	f_{py} =	243	ksi

COMPRESSION REINF.	12	#	8	
TENSION REINF.	8	#	8	

SHEAR STIRRUP REINF.
2 legs, # 6 @ 12 in o. c.

PRESTRESSING TENDONS
36 strands (each 0.5 in diameter & 0.153 in² area)

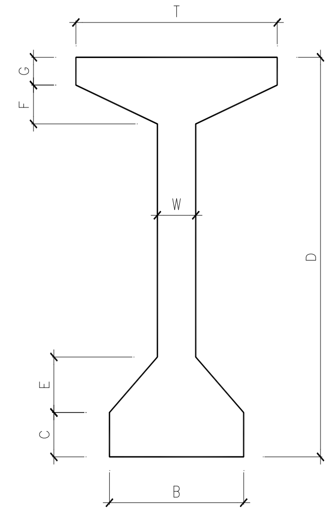
DISTANCE TO CENTROID OF COMPRESSION	d' =	4.5	in
DISTANCE TO CENTROID OF PRESTRESSED	d_p =	64	in
DISTANCE TO CENTROID OF TENSION	d =	68	in
GIRDER SPAN LENGTH	L =	80	ft
GIRDER SPACING	S =	8	ft, o.c.
CONCRETE DECK THICKNESS	t =	8	in

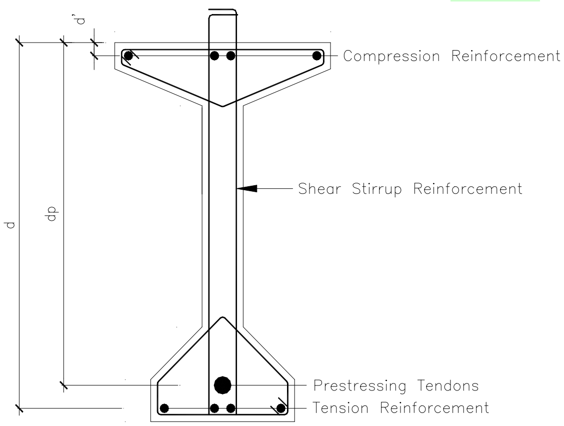
SECTION DIMENSIONS

T =	42	in
B =	28	in
C =	8	in
D =	72	in
E =	10	in
F =	7	in
G =	5	in
W =	8	in

SECTION PROPERTIES

A =	1125	in²
y_t =	34.65	in
y_b =	37.35	in
I =	762125	in⁴
S_t =	21995	in³
S_b =	20405	in³





TENDON FORCE IMMEDIATELY AFTER PRESTRESS TRANSFER
P_i = 1000 kips

TENDON FORCE AT SERVICE LOAD AFTER ALLOWANCE LOSSES
P_e = 800 kips

MOMENT DUE TO SELF-WEIGHT	M_G =	1042	ft-k
MOMENT DUE TO DEAD LOAD	M_D =	1000	ft-k
MOMENT DUE TO LIVE LOAD	M_L =	1354	ft-k
FACTORED SHEAR FORCE	V_u =	250	k
FACTORED TORSIONAL MOMENT	T_u =	52	ft-k

SECTION LOCATION (0, 1 or 2)	0	at midspan
PRESTRESSING METHOD (0, 1 or 2)	2	post-tensioned & bonded
EXPOSURE (0 OR 1)	0	mild exposure

THE DESIGN IS ADEQUATE.

Additional 4 #8 Longitudinal Reinforcement Required for Torsion

CHECK TRANSFER LOAD CONDITION (AASHTO 9.15.1 & 9.15.2.1)

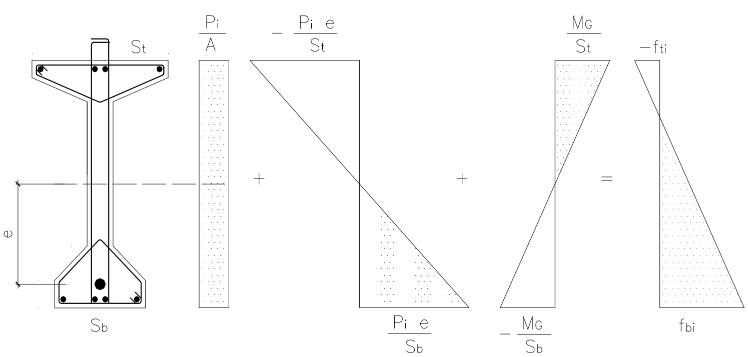
PRESTRESSED ECCENTRICITY	e =	29.35	in
MIN. TOP FIBER STRESS	$-F_{ti}$ =	-0.581	ksi
MAX. BOT. FIBER STRESS	F_{bi} =	3.300	ksi
MAX. ALLOWABLE STRESS	F_{si} =	189.000	ksi

f_{si} = 181.554 ksi < F_{si}
[Satisfactory]

$$f_{ti} = P_i\left(\frac{1}{A} - \frac{e}{S_t}\right) + \frac{M_G}{S_t} = \quad 0.123 \text{ ksi} \quad > \quad -F_{ti}$$
[Satisfactory]

$$f_{bi} = P_i\left(\frac{1}{A} + \frac{e}{S_b}\right) - \frac{M_G}{S_b} = \quad 1.715 \text{ ksi} \quad < \quad F_{bi}$$
[Satisfactory]

$(A_s')_{reqd}$ = 0.000 in² < $(A_s')_{provd}$
(ACI 318-08 R18.4.1) **[Satisfactory]**



CHECK SERVICEABILITY LOAD CONDITION
(AASHTO 9.15.1 & 9.15.2.2)

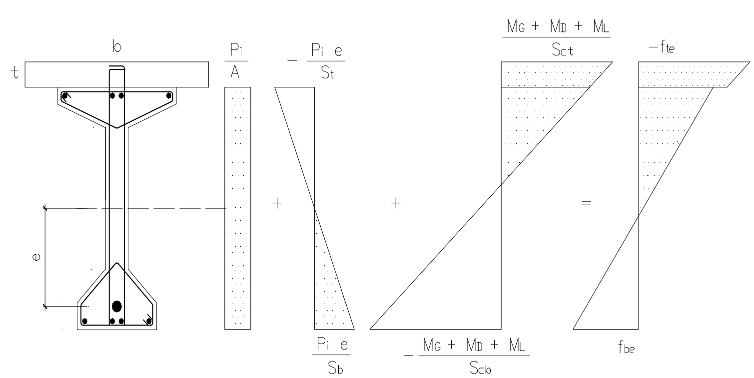
MIN. TOP FIBER STRESS

$F_{te} = 0.6f_c'$ =	3.600	ksi, for total loads
$F_{te,\ G+D} = 0.4f_c'$ =	2.400	ksi, for sustained loads only
$F_{te,\ 0.5(G+D)+L} = 0.4f_c'$ =	2.400	ksi, for live + 50% sustained loads

MAX. BOT. FIBER STRESS
$-F_{be} = -(0, 3, \text{or } 6)(f_c')^{0.5}$ = -0.465 ksi

MAX. ALLOWABLE STRESS
$F_{se} = 0.8f_y$ = 194.400 ksi, after all losses

f_{se} = 145.243 ksi < F_{se}
[Satisfactory]



(Cont'd)

$$f_{te} = P_e\left(\frac{1}{A}-\frac{e}{S_t}\right)+\frac{M_G+M_D+M_L}{S_{ct}} =$$ 0.403 ksi < F_{te} **[Satisfactory]**

$$f_{te,G+D} = P_e\left(\frac{1}{A}-\frac{e}{S_t}\right)+\frac{M_G+M_D}{S_{ct}} =$$ 0.100 ksi < $F_{te,\,G+D}$ **[Satisfactory]**

$$f_{te,0.5(G+D)+L} = 0.5P_e\left(\frac{1}{A}-\frac{e}{S_t}\right)+\frac{0.5(M_G+M_D)+M_L}{S_{ct}} =$$ 0.353 kis < $F_{te,\,0.5(G+D)+L}$ **[Satisfactory]**

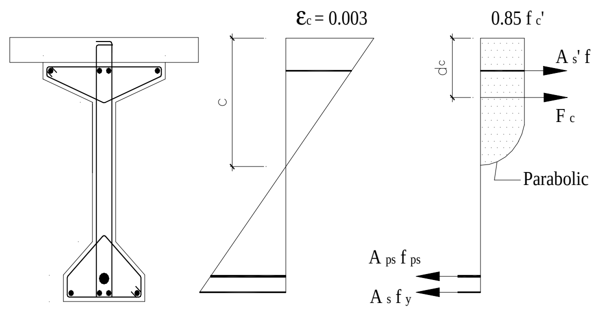
$$f_{be} = P_e\left(\frac{1}{A}+\frac{e}{S_b}\right)-\frac{M_G+M_D+M_L}{S_{cb}} =$$ 0.369 ksi > $-F_{be}$ **[Satisfactory]**

COMPOSITE SECTION PROPERTIES

b =	96	in, (ACI 318-08 8.12.3)
A_c =	1893	in^2
y_{ct} =	26.97	in
y_{cb} =	53.03	in
I_c =	1448000	in^4
S_{ct} =	53691	in^3
S_{cb} =	27305	in^3

CHECK ULTIMATE LOAD CONDITION (AASHTO 9.15.1 & 9.17 ACI 318-08 18.7)

COMPRESSION ZONE FACTOR	β_1 =	0.75	, (ACI 318-08 R10.2.7)
TENDON TYPE FACTOR	γ_p =	0.280	, (ACI 318-08 18.7.2)
RATIO OF TENSION REINF.	ρ =	0.002	, (ACI 318-08 Chapter 2)
RATIO OF COMPR. REINF.	ρ' =	0.003	, (ACI 318-08 Chapter 2)
RATIO OF PRESTR. REINF.	ρ_p =	0.002	, (ACI 318-08 Chapter 2)
INDEX OF TENSION REINF.	ω =	0.022	, (ACI 318-08 18.7.2)
INDEX OF COMPR. REINF.	ω' =	0.033	, (ACI 318-08 18.7.2)
INDEX OF PRESTR. REINF.	ω_p =	0.086	, (ACI 318-08 18.7.2)



FACTORED ULTIMATE MOMENT

$M_u = \gamma\,(\beta_D M_D + \beta_L M_L) = 1.3\,[\,1.0\,(M_G + M_D) + 1.67\,M_L]$, (AASHTO Eq. 3-10)

= 5594.134 ft-k

STRESS IN BONDED TENDONS :

$$f_{ps} = f_{pu}\left[1-\left(\frac{\gamma_p}{\beta_1}\right)\times MIN\left(\rho_p\frac{f_{pu}}{f'_c}+\frac{d\left(\omega-\omega'\right)}{d_p}\ ,\ 0.17\right)\right] =$$ 252.864 ksi

$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c}\ ,\ E_c = 57\sqrt{f'_c}\ ,\ E_s = 29000ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right)-\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]\ ,\ for\ \ 0<\varepsilon_c<\varepsilon_o \\ 0.85 f'_c\ ,\ for\ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

STRESS IN UNBOUNDED TENDONS :

$$f_{ps} = MIN\left(f_{se}+10+\frac{f'_c}{100\rho_p}, f_y, f_{se}+60\right) =$$ Not applicable

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \le \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$

$$f_{ps} = MIN\left(f_{se}+10+\frac{f'_c}{300\rho_p}, f_y, f_{se}+30\right) =$$ Not applicable

$\varepsilon_{s,max}$ =	0.0021	, (ACI 318-08 10.3.4 & 10.3.5)
c =	18.4	in, by pure math method
F_c =	1612.817	kips
d_c =	4.971	in
$A_s\,'f_s'$ =	159.158	kips
$A_s f_y$ =	379.200	kips
$A_{ps} f_{ps}$ =	1392.775	kips

$$\phi M_n = \phi\left[A_{ps}f_{ps}\left(d_p-d_c\right)+A_s f_y\left(d-d_c\right)+A'_s f_y\left(d_c-d'\right)\right] =$$ 7964 ft-k

\> M_u **[Satisfactory]**

$$1.2M_{cr} = 1.2S_b\left[P_e\left(\frac{1}{A_c}+\frac{e}{S_b}\right)+7.5\sqrt{f'_c}\right]-M_G\left(\frac{S_{cb}}{S_b}-1\right) =$$ 4632 ft-k < $\phi\ M_n$ **[Satisfactory]**

(AASHTO 9.18.2 & ACI 318-08 18.8.2)

CHECK SHEAR CAPACITY (AASHTO 9.20, ACI 318-08 11.1 & 11.4)

$$d = MAX\left(0.8h\ \ ,\ d_p\right) =$$ 64.00 in

$$\sqrt{f'_c} = MIN\left(100\ \ ,\ \sqrt{\left(f'_c\right)_{provd}}\right) =$$ 77.46 psi

$$V_c = \begin{cases} MAX\left\{MIN\left[\left(0.6\sqrt{f'_c}+700MIN\left(1\ ,\ \frac{V_u d_p}{M_u}\right)\right)b_w d\ \ ,\ 5b_w d\sqrt{f'_c}\right]\ ,\ 2b_w d\sqrt{f'_c}\right\}\ ,\ for\ \ f_{se}\ge 0.4f_{pu} \\ 2b_w d\sqrt{f'_c}\ \ ,\ for\ \ f_{se}<0.4f_{pu} \end{cases} =$$ 116.39 kips

$$V_s = MIN\left(\frac{A_v f_y d}{S}\ \ ,\ 8b_w d\sqrt{f'_c}\right) =$$ 281.60 kips

$$A_{v(\min)} = \begin{cases} MAX\left(\frac{50b_w S}{f_y}\ ,\ \frac{A_{ps}f_{pu}S\sqrt{\frac{d}{b_w}}}{80df_y}\right)\ ,\ for\ \ f_{se}\ge 0.4f_{pu} \\ \frac{50b_w S}{f_y}\ \ ,\ for\ \ f_{se}<0.4f_{pu} \end{cases} =$$ 0.164 in^2

$$S_{\max} = \begin{cases} MIN\left(0.75d\ \ ,\ 24\right),\ for\ \ V_s \le 4b_w d\sqrt{f'_c} \\ MIN\left(0.375d\ \ ,\ 12\right),\ for\ \ V_s > 4b_w d\sqrt{f'_c} \end{cases} =$$ 12.00 in

$$A_{v,requd} = \begin{cases} no\ \ shear\ \ reinf.\ requd\ \ ,\ for\ \ case1:\ V_u<\frac{\phi V_c}{2} \\ A_{v(\min)}\ \ ,\ for\ \ case2:\ \frac{\phi V_c}{2}\le V_u\le\phi V_c \\ MAX\left(A_{v,cal}\ \ ,\ A_{v(\min)}\right)\ \ ,\ for\ \ case3:\ \phi V_c\le V_u\le\phi\left(V_S+V_c\right) \\ unsatisfactory\ \ ,\ for\ \ case4:\ \phi\left(V_S+V_c\right)\le V_u \end{cases} =$$ 0.523 in^2 **[Satisfactory], Case 3 applicable**

(Cont'd)

CHECK TORSIONAL CAPACITY (AASHTO 9.21, ACI 318-08 11.1 & 11.5)

A_{cp}	=	576	in^2
P_{cp}	=	160	in
f_{pc}	=	1.389	ksi
A_{oh}	=	290	in^2
P_h	=	145	in

T_u > $\phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)\sqrt{1+\frac{f_{pc}}{4\sqrt{f'_c}}} =$ 11.403 ft-kips

Thus, Torsional Reinf. Reqd.

$\frac{A_t}{S} = \frac{T_u}{1.7\phi A_{oh} f_{yv}\cos(37.5°)} =$ 0.013 in^2/in

$A_L = MAX\left[\left(\frac{A_t}{S}\right)P_h\left(\frac{f_{yv}}{f_{yL}}\right)\cot^2(37.5°)\ ,\ \frac{5A_{cp}\sqrt{f'_c}}{f_{yL}} - P_h\left(\frac{f_{yv}}{f_{yL}}\right)\max\left(\frac{A_t}{S}\ ,\ \frac{25b_w}{f_{yv}}\right)\right] =$ 3.19 in^2

Additional 4 #8 Longitudinal Reinforcement Required for Torsion

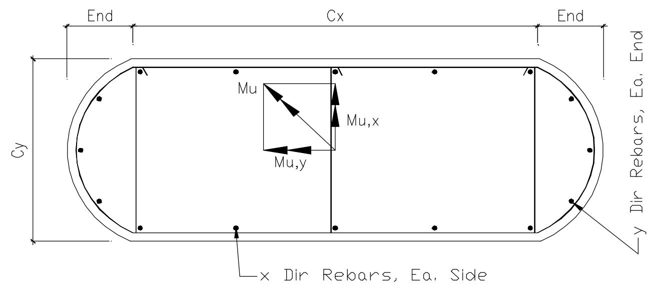
$\left(\frac{A_t}{S}\right)_{Total\ ReqD} = MAX\left(\frac{A_v+2A_t}{S}\ ,\ \frac{50b_w}{f_{yv}}\right) =$ 0.069 in^2/in **<** $\left(\frac{A_t}{S}\right)_{ProvD} =$ 0.073 in^2/in

[Satisfactory]

Bridge Column Design Based on AASHTO 17th & ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SECTION SIZE	C_x	=	120	in
	End	=	12	in
	C_y	=	48	in
FACTORED AXIAL LOAD	P_u	=	8000	k
FACTORED MAGNIFIED MOMENT	$M_{u,x}$	=	18000	ft-k
	$M_{u,y}$	=	18000	ft-k
FACTORED SHEAR LOAD	$V_{u,x}$	=	30	k
	$V_{u,y}$	=	20	k



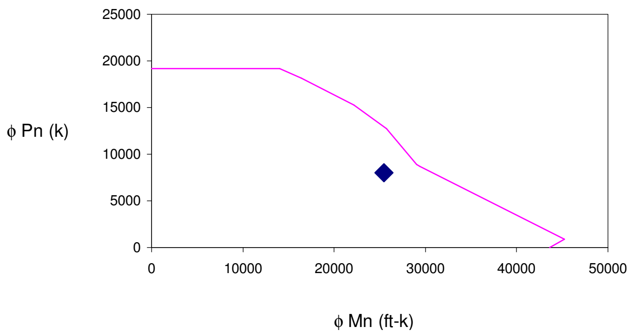
THE COLUMN DESIGN IS ADEQUATE.

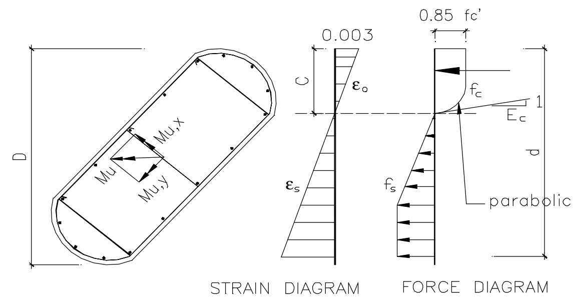
COLUMN VERT. REINFORCEMENT	15	#	18	at x dir.
	5	#	18	at y dir.

(Total 40 # 18)

LATERAL REINF. OPTION (0=Spirals, 1=Ties) 1 Ties
LATERAL REINFORCEMENT # 4 @ 12 in o.c.
3 straight legs # 4 @ 12 in o.c.

ANALYSIS





	φ Pn (k)	φ Mn (ft-k)
AT AXIAL LOAD ONLY	19147	0
AT MAXIMUM LOAD	19147	14042
AT 0 % TENSION	18160	16393
AT 25 % TENSION	15279	22182
AT 50 % TENSION	12730	25771
AT ε_t = 0.002	8897	29007
AT BALANCED CONDITION	8731	29407
AT ε_t = 0.005	885	45199
AT FLEXURE ONLY	0	43623

$$\varepsilon_O = \frac{2\left(0.85 f_c'\right)}{E_C} \ , \ E_C = 57\sqrt{f_c'} \ , \ E_S = 29000ksi$$

$$f_c = \begin{cases} 0.85 f_c'\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right] & , \ for \ \ 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f_c' \ , \ for \ \ \varepsilon_C \geq \varepsilon_O \end{cases}$$

$$f_s = \begin{cases} \varepsilon_S E_S \ , \ for \ \ \varepsilon_S \leq \varepsilon_t \\ f_y \ , \ for \ \ \varepsilon_S > \varepsilon_t \end{cases}$$

CHECK FLEXURAL & AXIAL CAPACITY

ϕP_{max} =F ϕ [0.85 f_c' (A_g - A_{st}) + f_y A_{st}] = 19147 kips., (at max axial load, ACI 318-08, Sec. 10.3.6.2)

where ϕ = 0.65 (ACI 318-08, Sec.9.3.2.2) > P_u **[Satisfactory]**

F = 0.8 A_g = 6565 in^2 A_{st} = 160.00 in^2

ϕ = { 0.75 + (ε_t - 0.002) (50), for Spiral ; 0.65 + (ε_t - 0.002) (250 / 3), for Ties } = 0.656 (ACI 318-08, Fig. R9.3.2)

where C_b = d ε_c / (ε_c + ε_s) = 68.8 in ε_t = 0.0021 ε_c = 0.003

d = 116.3 in, (ACI 7.7.1) D = 119.4 in Cover = 1.5 in, (AASHTO 8.22.1)

ϕ M_n = 30878 ft-kips @ P_u = 8000 kips > M_u = 25456 ft-kips **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-08, Section 10.9) ρ_{provd} = 0.024

ρ_{min} = 0.01 (ACI 318-08, Section 10.9) **[Satisfactory]**

CHECK SHEAR CAPACITY (ACI 318-08 Sec. 11.1.1, 11.2.1, & 11.4.6.2)

ϕ V_n = ϕ (V_s + V_c) (ACI 318-08 Sec. 11.1.1)

> V_u **[Satisfactory]**

where ϕ = 0.75 (ACI 318-08 Sec. 9.3.2.3) f_y = 60 ksi

	d	A_0	A_v	V_c = 2 $(f_c')^{0.5}A_0$	V_s = MIN (d f_y A_v / s , 4V_c)	ϕ V_n
x	140.9	5655	0.40	799.8	281.7	811
y	44.9	5655	1.00	799.8	224.4	768

s_{max} = 24 (ACI 318-08, Section 7.10.5.2) s_{provd} = 12 in

s_{min} = 1 **[Satisfactory]**

Magnified Moment Calculation for Bridge Circular Column Based on AASHTO 17th & ACI 318-08

INPUT DATA & DESIGN SUMMARY

EFFECTIVE LENGTH FACTOR	k	=	1.6	, (ACI 10.10.6.3 or 10.10.7.1)
COLUMN UNSUPPORTED LENGTH	L_u	=	24	ft
LARGER FACTORED MOMENT	M_2	=	1800	ft-k
SMALLER FACTORED END MOMENT	M_1	=	1000	ft-k, (positive if single curvature.)
CONCRETE STRENGTH	f_c'	=	5	ksi
COLUMN DIMENSIONS	C_x	=	120	in
	End	=	12	in
	C_y	=	48	in
FACTORED AXIAL LOAD	P_u	=	8000	k
SUMMATION FOR ALL VERTICAL LOADS IN THE STORY	ΣP_u	=	8000	k
SUMMATION FOR ALL CRITICAL LOADS IN THE STORY	ΣP_c	=	50000	k, (ACI Eq. 10-21)

THE MAGNIFIED MOMENT: M_u = 2378.1 ft-k , Sway

ANALYSIS

MAGNIFIED MOMENT - NONSWAY

A_g = 6565 in^2

I_g = 1906740 in^4

$r = (I_g / A)^{0.5}$ = 17.0 in, ACI 10.11.2

$k L_u / r$ = 27.0388647 **<** $34 - 12(M_1 / M_2)$ = 27.333 < = = Slenderness effect may be ignored. (AASHTO 8.16.5.2 or ACI Eq 10-7)

$E_c = 57000 (f_c')^{0.5}$ = 4030.5 ksi, ACI 8.5.1

$$EI = \frac{0.4 E_c I_g}{1+\beta_d} = \frac{0.4 E_c I_g}{1+0.6} = 0.25 E_c I_g =$$ 2E+09 k-in^2 , ACI 10.10.6.1

$$P_c = \frac{\pi^2 EI}{(k L_u)^2} =$$ 89303 k, ACI Eq (10-14)

$M_{2,min}$ = MAX[M_2 , P_u (0.6+0.03 (2 End + C_x))] = 3280 ft-k, ACI 10.10.6.5

C_m = MAX[0.6 + 0.4 (M_1 / $M_{2, min}$) , 0.4] = 0.722 , ACI Eq(10-16)

$$\delta_{ns} = MAX\left(\frac{C_m}{1-\frac{P_u}{0.75 P_c}} \ , \ 1.0\right) =$$ 1.00 , ACI Eq (10-12)

$M_{u, ns} = \delta_{ns} M_{2, min}$ = 3280.0 ft-k, ACI Eq (10-11) **>** $1.05 M_2$ = 1890.0 ft-k **[Unsatisfactory]** ,(ACI 10.10.5.1)

The column is sway. See calculation as follows.

MAGNIFIED MOMENT - SWAY

$k L_u / r$ = 27.0388647 **>** 22 < = = Slenderness effect must be considered. ACI Eq (10-6)

$$\delta_s = MIN\left[MAX\left(\frac{1}{1-\frac{\Sigma P_u}{0.75 \Sigma P_c}} \ , \ 1.0\right) \ , \ 2.5\right] =$$ 1.27 , ACI Eq (10-21)

L_u / r = 16.90 **<** $35 / [P_u / (f_c' A_g)]^{0.5}$ = 70.90 [Satisfactory]

$M_{2s} = M_2$ = 1800.0 ft-k, as given

M_{2ns} = 5% M_{2s} = 90.0 ft-k, assumed conservatively

$M_{u, s} = M_{2ns} + \delta_s M_{2s}$ = 2378.1 ft-k, ACI Eq (10-19)

Note: For column subject to bending about both principal axis, the moment about each axis shall be magnified separately based on the conditions corresponding to that axis.

Bridge Design for Prestressed Concrete Box Section Based on AASHTO 17th Edition & ACI 318-08

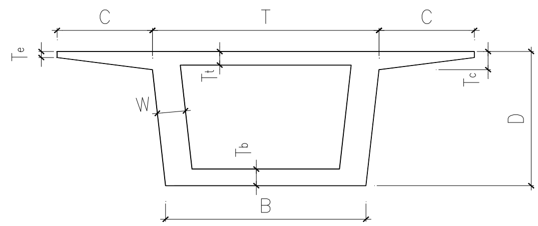
INPUT DATA & DESIGN SUMMARY

BRIDGE SPAN L = 120 ft

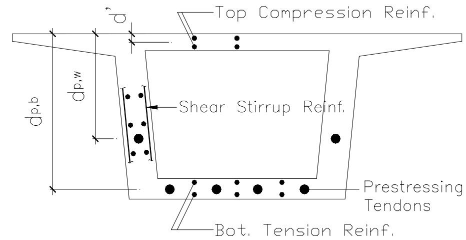
SECTION DIMENSIONS

T =	280	in, within [B , 300"]		$A =$	15529.0	in^2
B =	240	in, within [200", 280"]		$y_t =$	41.82	in
C =	110	in, within [80",180"]		$y_b =$	54.18	in
D =	96	in, within [L/15 , L/25]		$I =$	19766329	in^4
$T_t =$	12	in, within [8",18"] & >T/30		$S_t =$	472656	in^3
$T_b =$	24	in, within [8", 24"] & >B/30		$S_b =$	364825	in^3
W =	24	in, within [8", 24"] & >D/15				
$T_c =$	24	in, within [14", 24"] & >C/12				
$T_e =$	6	in				

SECTION PROPERTIES (right columns above)



CONCRETE STRENGTH	$f'_c =$	6	ksi
REBAR STRENGTH	$f_y^* = f_y =$	60	ksi
TENDON TENSILE STRENGTH	$f_{su}^* = f_{pu} =$	270	ksi
TENDON YIELD STRENGTH	$f_{py} =$	243	ksi
DISTANCE TO CENTROID OF COMPRESSION	$d' =$	6	in
DIST. TO CENTROID OF BOT. PRESTRESSED	$d_{p,b} =$	84	in
DIST. TO CENTROID OF WEB. PRESTRESSED	$d_{p,w} =$	76	in
DISTANCE TO CENTROID OF TENSION	$d =$	80	in



TOP COMPRESSION REINF.	2	#	8	@	12	in o. c.
BOTTOM TENSION REINF.	2	#	8	@	12	in o. c.
WEB HORIZONTAL REINF.	2	#	8	@	12	in o. c.
SHEAR STIRRUP REINF.	2	legs, #	10	@	8	in o. c.

BOTTOM PRESTRESSING TENDONS
16 x 36 strands (each 0.5 in diameter & 0.153 in^2 area)

EACH WEB PRESTRESSING TENDONS
4 x 36 strands (each 0.5 in diameter & 0.153 in^2 area)

TOTAL TENDON FORCE IMMEDIATELY AFTER PRESTRESS TRANSFER P_i = 23794.56 kips

TOTAL TENDON FORCE AT SERVICE LOAD AFTER ALLOWANCE LOSSES P_e = 19035.648 kips

MOMENT DUE TO SELF-WEIGHT	$M_G =$	29116.963	ft-k	FACTORED VERT. SHEAR FORCE	$V_u =$	6886.735	k
MOMENT DUE TO DEAD LOAD	$M_D =$	37500	ft-k	FACTORED TORSIONAL MOMENT	$T_u =$	35868.412	ft-k
MOMENT DUE TO LIVE LOAD	$M_L =$	52500	ft-k	FACTORED LATERAL BENDING MOMENT	$M_{u,y} =$	41320.41	ft-k

SECTION LOCATION (0, 1 or 2) 0 at midspan (Seismic/Wind Horizontal Bending Load)
PRESTRESSING METHOD (0, 1 or 2) 2 post-tensioned & bonded
EXPOSURE (0 OR 1) 0 mild exposure

THE DESIGN IS ADEQUATE.

CHECK TRANSFER LOAD CONDITION (AASHTO 9.15.1 & 9.15.2.1)

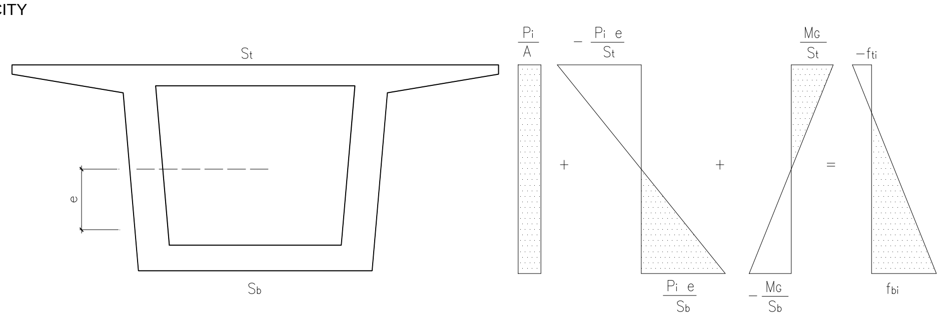
ENTIRE SECTION PRESTRESSED ECCENTRICITY
e = 38.18 in

MIN. TOP FIBER STRESS
$-F_{ti} =$ -0.581 ksi

MAX. BOT. FIBER STRESS
$F_{bi} =$ 3.300 ksi

MAX. ALLOWABLE STRESS
$F_{si} =$ 189.000 ksi

f_{si} = 180.000 ksi
< F_{si}
[Satisfactory]



$$f_{ti} = P_i\left(\frac{1}{A}-\frac{e}{S_t}\right)+\frac{M_G}{S_t} =$$ 0.349 ksi > $-F_{ti}$ **[Satisfactory]**

$(A_s')_{reqd}$ = 0.000 in^2 < $(A_s')_{provd}$
(ACI 318-08 R18.4.1) **[Satisfactory]**

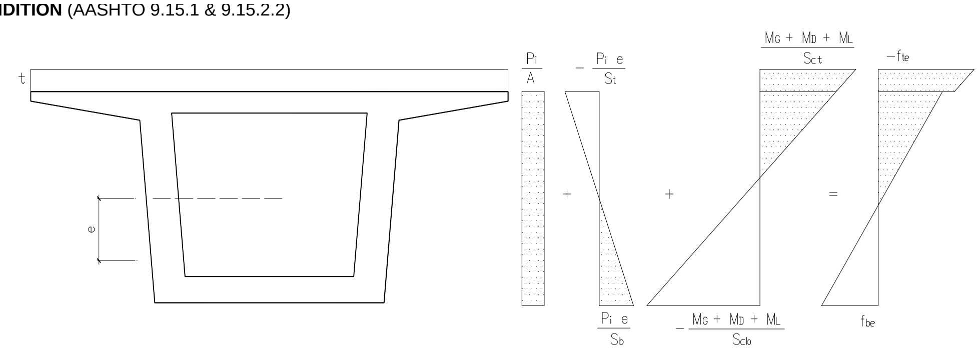
$$f_{bi} = P_i\left(\frac{1}{A}+\frac{e}{S_b}\right)-\frac{M_G}{S_b} =$$ 3.065 ksi < F_{bi} **[Satisfactory]**

CHECK SERVICEABILITY LOAD CONDITION (AASHTO 9.15.1 & 9.15.2.2)

CONCRETE DECK THICKNESS
t = 4 in
(0 for non-composite)

COMPOSITE SECTION PROPERTIES

b =	500	in
$A_c =$	17529.0	in^2
$y_{ct} =$	40.82	in
$y_{cb} =$	59.18	in
$I_c =$	23171159	in^4
$S_{ct} =$	567642	in^3
$S_{cb} =$	391537	in^3



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MIN. TOP FIBER STRESS

$F_{te} = 0.6f_c' =$ 3.600 ksi, for total loads

$F_{te,\,G+D} = 0.4f_c' =$ 2.400 ksi, for sustained loads only

$F_{te,\,0.5(G+D)+L} = 0.4f_c' =$ 2.400 ksi, for live + 50% sustained loads

$\boldsymbol{f_{se}}$ = 144.000 ksi < F_{se} **[Satisfactory]**

MAX. BOT. FIBER STRESS

$-F_{be} = -(0,\ 3,\ \text{or}\ 6)(f_c')^{0.5} =$ -0.465 ksi

MAX. ALLOWABLE STRESS

$F_{se} = 0.8f_y =$ 194.400 ksi, after all losses

$$f_{te} = P_e\left(\frac{1}{A} - \frac{e}{S_t}\right) + \frac{M_G + M_D + M_L}{S_{ct}} =$$ 2.206 ksi < F_{te} **[Satisfactory]**

$$f_{te,G+D} = P_e\left(\frac{1}{A} - \frac{e}{S_t}\right) + \frac{M_G + M_D}{S_{ct}} =$$ 1.096 ksi < $F_{te,\,G+D}$ **[Satisfactory]**

$$f_{te,0.5(G+D)+L} = 0.5P_e\left(\frac{1}{A} - \frac{e}{S_t}\right) + \frac{0.5(M_G + M_D) + M_L}{S_{ct}} =$$ 1.658 kis < $F_{te,\,0.5(G+D)+L}$ **[Satisfactory]**

$$f_{be} = P_e\left(\frac{1}{A} + \frac{e}{S_b}\right) - \frac{M_G + M_D + M_L}{S_{cb}} =$$ -0.433 ksi > $-F_{be}$ **[Satisfactory]**

CHECK ULTIMATE LOAD CONDITION (AASHTO 9.15.1 & 9.17 ACI 318-08 18.7)

FACTORED ULTIMATE MOMENT

$M_{u,x} = \gamma\,(\,\beta_D M_D + \beta_L M_L\,) = 1.3\,[\,1.0\,(\,M_G + M_D) + 1.67\,M_L]$ (AASHTO Eq. 3-10)

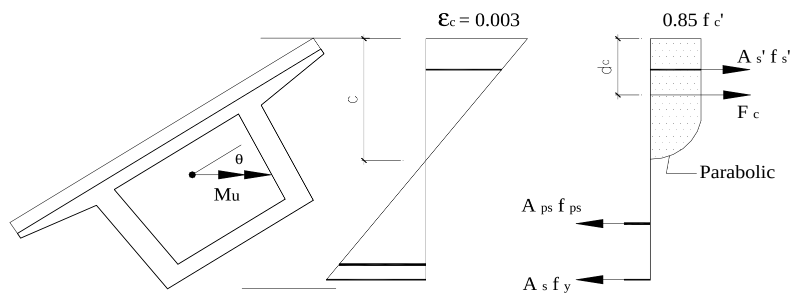
= 200579.551 ft-k

$M_{u,y} =$ 41320.410 ft-k

$M_u = (M_{u,x}^2 + M_{u,y}^2)^{0.5}$

= 204791.437 ft-k

$\theta =$ 11.6 deg



COMPRESSION ZONE FACTOR

$\beta_1 =$ 0.75 , (ACI 318-08 R10.2.7)

TENDON TYPE FACTOR

$\gamma_p =$ 0.280 , (ACI 318-08 18.7.2)

RATIO OF TENSION REINF.	$\rho =$	0.001	, (ACI 318-08 Chapter 2)
RATIO OF COMPR. REINF.	$\rho' =$	0.002	, (ACI 318-08 Chapter 2)
RATIO OF PRESTR. REINF.	$\rho_p =$	0.006	, (ACI 318-08 Chapter 2)
INDEX OF TENSION REINF.	$\omega =$	0.013	, (ACI 318-08 18.7.2)
INDEX OF COMPR. REINF.	$\omega' =$	0.016	, (ACI 318-08 18.7.2)
INDEX OF PRESTR. REINF.	$\omega_p =$	0.237	, (ACI 318-08 18.7.2)

$$\varepsilon_o = \frac{2\left(0.85 f_c'\right)}{E_c}\ ,\ E_c = 57\sqrt{f_c'}\ ,\ E_s = 29000ksi$$

$$f_c = \begin{cases} 0.85 f_c'\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]\ ,\ for\ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f_c'\ ,\ for\ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \le \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$

STRESS IN BONDED TENDONS :

$$f_{ps} = f_{pu}\left[1 - \left(\frac{\gamma_p}{\beta_1}\right) \times MIN\left(\rho_p \frac{f_{pu}}{f_c'} + \frac{d\left(\omega - \omega'\right)}{d_p}\ \ ,\ \ 0.17\right)\right] =$$ 243.940 ksi

STRESS IN UNBOUNDED TENDONS :

$$f_{ps} = MIN\left(f_{se} + 10 + \frac{f_c'}{100\rho_p}, f_y, f_{se} + 60\right) =$$ Not applicable

$$f_{ps} = MIN\left(f_{se} + 10 + \frac{f_c'}{300\rho_p}, f_y, f_{se} + 30\right) =$$ Not applicable

$\varepsilon_{s,max} =$ 0.0021 , (ACI 318-08 10.3.4 & 10.3.5)

c = 98.2 in, by pure math method

$F_c =$ 32282.5 kips

$d_c =$ 47.3 in

$\phi\ M_n =$ 2603343 ft-k

> M_u **[Satisfactory]**

$$1.2M_{cr} = 1.2S_b\left[P_e\left(\frac{1}{A_c} + \frac{e}{S_b}\right) + 7.5\sqrt{f_c'}\right] - M_G\left(\frac{S_{cb}}{S_b} - 1\right) =$$ 136462 ft-k < $\phi\ M_n$ **[Satisfactory]**

(AASHTO 9.18.2 & ACI 318-08 18.8.2)

CHECK SHEAR CAPACITY (AASHTO 9.20, ACI 318-08 11.1 & 11.4)

$$d = MAX\left(0.8h\ \ ,\ \ d_p\right) =$$ 84.00 in

$$\sqrt{f_c'} = MIN\left(100\ \ ,\ \ \sqrt{\left(f_c'\right)_{provd}}\right) =$$ 77.46 psi

$$V_c = \begin{cases} MAX\left\{MIN\left[\left(0.6\sqrt{f_c'} + 700MIN\left(1\ \ ,\ \ \frac{V_u d_p}{M_u}\right)\right)b_w d\ \ ,\ \ 5b_w d\sqrt{f_c'}\right]\ \ ,\ \ 2b_w d\sqrt{f_c'}\right\}\ \ ,\ \ for\ \ f_{se} \ge 0.4f_{pu} \\ 2b_w d\sqrt{f_c'}\ \ ,\ \ for\ \ f_{se} < 0.4f_{pu} \end{cases} =$$ 5441.73 kips

$$V_s = MIN\left(\frac{A_v f_y d}{S}\ \ ,\ \ 8b_w d\sqrt{f_c'}\right) =$$ 3200.40 kips

$$A_{v(\min)} = \begin{cases} MAX\left(\frac{50b_w S}{f_y}\ \ ,\ \ \frac{A_{ps} f_{pu} S\sqrt{\frac{d}{b_w}}}{80 d f_y}\right)\ \ ,\ \ for\ \ f_{se} \ge 0.4f_{pu} \\ \frac{50b_w S}{f_y}\ \ ,\ \ for\ \ f_{se} < 0.4f_{pu} \end{cases} =$$ 1.867 in^2

$$S_{\max} = \begin{cases} MIN\left(0.75d\ \ ,\ \ 24\right),\ \ for\ \ V_s \le 4b_w d\sqrt{f_c'} \\ MIN\left(0.375d\ \ ,\ \ 12\right),\ \ for\ \ V_s > 4b_w d\sqrt{f_c'} \end{cases} =$$ 24.00 in

(Cont'd)

$$A_{v,requd}=\begin{cases} no\ \ shear\ \ re\inf.\ \ requd\ \ ,\ \ for\ \ case1:\ \ V_u<\dfrac{\phi V_c}{2} \\ A_{v(\min)}\ \ ,\ \ for\ \ case2:\ \ \dfrac{\phi V_c}{2}\le V_u\le \phi V_c \\ MAX\left(A_{v,cal}\ \ ,\ \ A_{v(\min)}\right)\ \ ,\ \ for\ \ case3:\ \ \phi V_c\le V_u\le \phi\left(V_S+V_c\right) \\ unsatisfactory\ \ ,\ \ for\ \ case4:\ \ \phi\left(V_S+V_c\right)\le V_u \end{cases} = \quad 4.434 \quad in^2$$

[Satisfactory], Case 3 applicable

CHECK TORSIONAL CAPACITY (AASHTO 9.21, ACI 318-08 11.1 & 11.5)

A_{cp}	=	26880	in^2
P_{cp}	=	752	in
f_{pc}	=	0.708	ksi
A_{oh}	=	25293	in^2
P_h	=	735	in

$$T_u \quad > \quad \phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)\sqrt{1+\frac{f_{pc}}{4\sqrt{f'_c}}} = \quad 5277.761 \ \text{ft-kips}$$

Thus, Torsional Reinf. Reqd.

$$\frac{A_t}{S}=\frac{T_u}{1.7\phi A_{oh} f_{yv}\cos\left(37.5°\right)}= \quad 0.020 \quad in^2/in$$

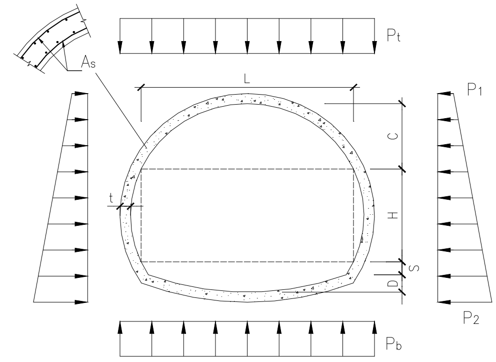
$$A_L=MAX\left[\left(\frac{A_t}{S}\right)P_h\left(\frac{f_{yv}}{f_{yL}}\right)\cot^2\left(37.5°\right)\ \ ,\ \ \frac{5A_{cp}\sqrt{f'_c}}{f_{yL}}-P_h\left(\frac{f_{yv}}{f_{yL}}\right)\max\left(\frac{A_t}{S}\ \ ,\ \ \frac{25b_w}{f_{yv}}\right)\right]= \quad 87.77 \quad in^2$$

$$\left(\frac{A_t}{S}\right)_{Total\ \ ReqD}=MAX\left(\frac{A_v+2A_t}{S}\ \ ,\ \ \frac{50b_w}{f_{yv}}\right)= \quad 0.595 \quad in^2/in \quad < \quad \left(\frac{A_t}{S}\right)_{ProvD}= \quad 0.635 \quad in^2/in$$

[Satisfactory]

Concrete Tunnel Design Based on AASHTO-17th & ACI 318-08

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c' =	5	ksi
REBAR YIELD STRESS	f_y =	60	ksi

TUNNEL DIMENSIONS	L =	32	ft, (AASHTO Fig 2.5)
	H =	16	ft, (AASHTO Fig 2.5)
	C =	9.9	ft
	S =	16	in
	D =	60	in
	t =	20	in

REINFORCING (A_s)

2 layers # 9 @ 8 in o.c. (curvered)

Concrete Cover = 3 in, (AASHTO 8.22.1)

FACTORED SOIL PRESSURE (SD level, including surcharge & seismic ground shaking)

P_t = 1760 psf $P_b = P_t + Wt$ = 2671 psf

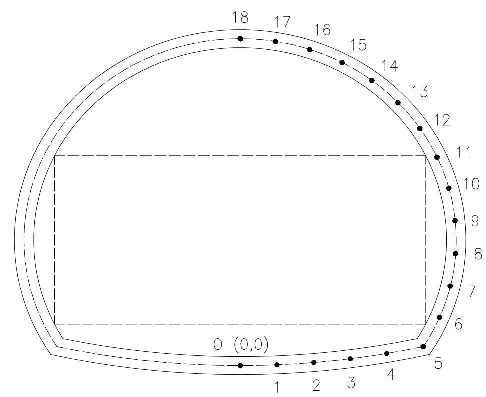
P_1 = 720 ft-pcf, (psf) P_2 = 1820 ft-pcf, (psf)

[THE DESIGN IS ADEQUATE.]

ANALYSIS

DETERMINE TUNNEL FORCES BY FINITE ELEMENT METHOD

Point	X (ft)	Y (ft)	P_u (k)	M_u (ft-k)	V_u (k)
0	0.00	0.00	24.97	124.44	0.00
1	3.40	0.22	25.83	116.28	7.17
2	6.73	0.87	27.51	91.87	11.67
3	9.96	1.94	29.95	52.18	15.69
4	13.03	3.41	33.08	-1.21	19.07
5	15.89	5.26	40.68	-66.10	5.66
6	17.29	7.98	41.81	-83.43	3.82
7	18.23	10.89	42.16	-95.13	1.40
8	18.68	13.92	41.52	-99.43	-1.32
9	18.63	16.98	40.02	-95.39	-4.02
10	18.09	20.00	37.82	-83.08	-6.46
11	17.06	22.88	34.98	-63.29	-8.41
12	15.57	25.56	31.65	-37.55	-9.61
13	13.67	27.96	28.09	-8.14	-9.87
14	11.40	30.01	24.62	22.10	-9.12
15	8.83	31.68	21.60	50.02	-7.37
16	6.02	32.89	19.37	72.60	-4.79
17	3.05	33.64	18.18	87.28	-1.66
18	0.00	33.89	18.18	92.37	0.00



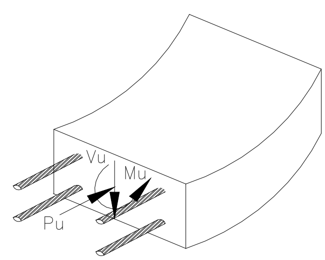
R_t =	18.72	ft, (top circle radius)
R_b =	26.62	ft, (bottom circle radius)
θ_t =	121.95	deg, (between 5 & 18)
θ_b =	36.63	deg, (between 0 & 5)

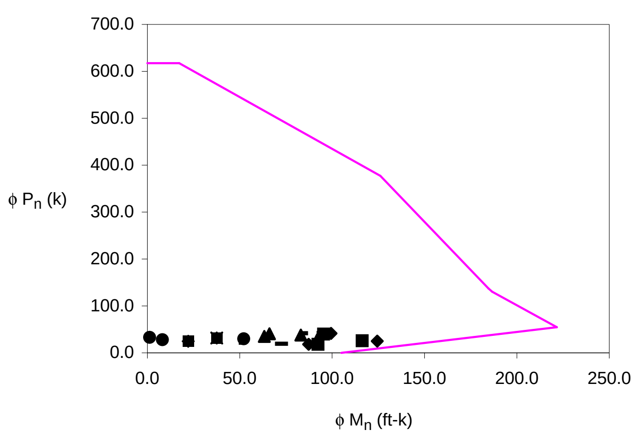
CHECK AXIAL & FLEXURE CAPACITY

ρ_{ProvD} = 0.007605 < ρ_{MAX} = 0.0400 (tension face only, ACI 318-05 10.3.5 or 10.9.1)

\> ρ_{MIN} = 0.0008 (tension face only, ACI 318-05 10.5.1, 10.5.3 or 14.3.2)

[Satisfactory]





	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	617.4	0.0
AT MAXIMUM LOAD	617.4	17.2
AT MIDDLE	377.3	126.1
AT ε_t = 0.002	137.2	184.6
AT BALANCED	130.2	186.6
AT ε_t = 0.005	54.6	221.6
AT FLEXURE ONLY	0.0	105.0

(Note: For middle reforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-08 SEC.15.5.2, 11.1.3.1, & 11.2)

$\phi V_n = 2\phi b d \sqrt{f_c'}$ = 20.92 kips / ft > $V_{u,\,max}$ = 19.07 kips / ft **[Satisfactory]**

Purchaser's LOGO

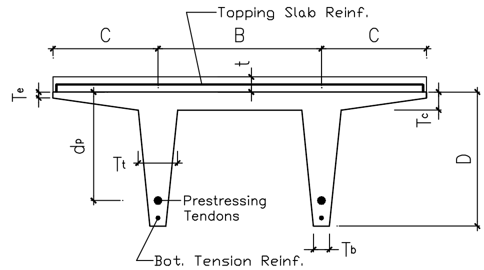
PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Concrete Design for Prestressed Double Tee Section Based on AASHTO 17th Edition & ACI 318-14

INPUT DATA & DESIGN SUMMARY

BEAM SPAN L = 45 ft

UNTOPPED SECTION DIMENSIONS			UNTOPPED SECTION PROPERTIES		
B =	60	in, (1524 mm)	A =	449.0	in^2
C =	30	in, (1524 mm)	y_t =	6.23	in
D =	24	in, (610 mm)	y_b =	17.77	in
T_t =	5.75	in, (146 mm)	I =	22469	in^4
T_b =	3.75	in, (1524 mm)	S_t =	3609	in^3
T_c =	2	in, (51 mm)	S_b =	1264	in^3
T_e =	2	in, (51 mm)	Wt =	468	lbs / ft, (47 psf)



THE DESIGN IS INADEQUATE, SEE ANALYSIS

CONCRETE STRENGTH f'_c = 6 ksi, (41 MPa)

REBAR STRENGTH $f_y^* = f_y$ = 60 ksi, (414 MPa)

TENDON TENSILE STRENGTH $f_{su}^* = f_{pu}$ = 270 ksi, (1862 MPa)

TENDON YIELD STRENGTH f_{py} = 243 ksi, (1675 MPa)

TOTAL PRESTRESSING TENDONS

6 strands (0.5 in dia. & 0.153 in^2 area per strand), at Each Leg

(13 mm) (99 mm^2)

TOTAL TENDON FORCE IMMEDIATELY AFTER PRESTRESS TRANSFER

P_i = 330.48 kips, (double tee), (1470.0 kN)

TOTAL TENDON FORCE AT SERVICE LOAD AFTER ALLOWANCE LOSSES

P_e = 264.384 kips, (double tee), (1176.0 kN)

DIST. TO CENTROID OF BOT. PRESTRESSED d_p = 18.3 in, (465 mm)

BOTTOM TENSION REINF. 1 # 6 at Each Leg

TOPPING t = 2 in, with # 4 @ 12 in. o.c., trans. way

(51 mm) (305 mm)

SUPERIMPOSED LOADS

DL = 30 psf, (1 kPa), ASD

LL = 100 psf, (5 kPa), ASD

Total Loads = 202 psf, (10 kPa), ASD

FACTORED VERT. SHEAR FORCE

V_u = 45.398 kips, (201.9 kN), SD

PRESTRESSING METHOD (0, 1 or 2) 2 post-tensioned & bonded

EXPOSURE (0 OR 1) 0 mild exposure

SECTION LOCATION (0, 1 or 2) 0 at midspan

FACTORED LATERAL BENDING MOMENT

$M_{u,y}$ = 510.732 ft-kips, (692 kN-m), SD

SHEAR STIRRUP REINF. 1 E. Leg, # 4 @ 10 in o. c. (Seismic/Wind Horizontal Bending Load)

(254 mm)

CHECK TRANSFER LOAD CONDITION (AASHTO 9.15.1 & 9.15.2.1)

ENTIRE SECTION PRESTRESSED ECCENTRICITY

e = 12.07 in

MIN. TOP FIBER STRESS

$-F_{ti}$ = -0.581 ksi

MAX. BOT. FIBER STRESS

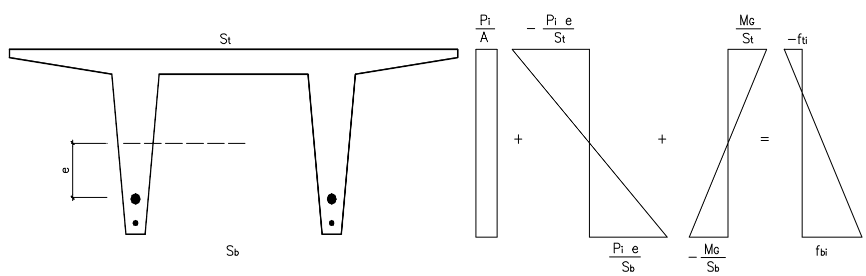
F_{bi} = 3.300 ksi

MAX. ALLOWABLE STRESS

F_{si} = 189.000 ksi

f_{si} = 180.000 ksi < F_{si}

[Satisfactory]



$$f_{ti} = P_i\left(\frac{1}{A} - \frac{e}{S_t}\right) + \frac{M_G}{S_t} =$$ 0.024 ksi > $-F_{ti}$

[Satisfactory]

where M_G = 118.389 ft-k

$$f_{bi} = P_i\left(\frac{1}{A} + \frac{e}{S_b}\right) - \frac{M_G}{S_b} =$$ 2.769 ksi < F_{bi}

[Satisfactory]

CHECK SERVICEABILITY LOAD CONDITION (AASHTO 9.15.1 & 9.15.2.2)

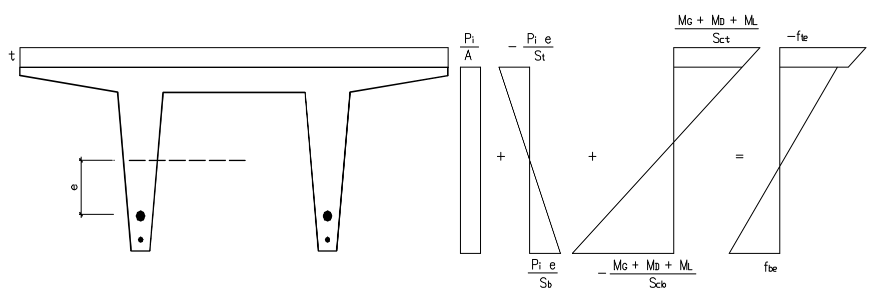
CONCRETE DECK TOPPING

t = 2 in

(0 for non-composite)

COMPOSITE SECTION PROPERTIES

b =	120	in
A_c =	689.0	in^2
y_{ct} =	5.71	in
y_{cb} =	20.29	in
I_c =	30636	in^4
S_{ct} =	5366	in^3
S_{cb} =	1510	in^3



MIN. TOP FIBER STRESS

$F_{te} = 0.6f_c'$ = 3.600 ksi, for total loads

$F_{te,\ G+D} = 0.4f_c'$ = 2.400 ksi, for sustained loads only

$F_{te,\ 0.5(G+D)+L} = 0.4f_c'$ = 2.400 ksi, for live + 50% sustained loads

f_{se} = 144.000 ksi < F_{se}

[Satisfactory]

MAX. BOT. FIBER STRESS

$-F_{be} = -(0, 3, \text{or } 6)(f_c')^{0.5}$ = -0.465 ksi

MAX. ALLOWABLE STRESS

$F_{se} = 0.8f_y$ 194.400 ksi, after all losses

(Cont'd)

$$f_{te} = P_e\left(\frac{1}{A}-\frac{e}{S_t}\right)+\frac{M_G+M_D+M_L}{S_{ct}} =$$ 1.111 ksi < F_{te} **[Satisfactory]** where M_D = 257.607 ft-k, M_L = 253.125 ft-k

$$f_{te,G+D} = P_e\left(\frac{1}{A}-\frac{e}{S_t}\right)+\frac{M_G+M_D}{S_{ct}} =$$ 0.545 ksi < $F_{te,\ G+D}$ **[Satisfactory]**

$$f_{te,0.5(G+D)+L} = 0.5P_e\left(\frac{1}{A}-\frac{e}{S_t}\right)+\frac{0.5(M_G+M_D)+M_L}{S_{ct}} =$$ 0.839 kis < $F_{te,\ 0.5(G+D)+L}$ **[Satisfactory]**

$$f_{be} = P_e\left(\frac{1}{A}+\frac{e}{S_b}\right)-\frac{M_G+M_D+M_L}{S_{cb}} =$$ -1.886 ksi < $-F_{be}$ **[Unsatisfactory]**

CHECK ULTIMATE LOAD CONDITION (AASHTO 9.15.1 & 9.17 ACI 318-14 20.3)

FACTORED ULTIMATE MOMENT

$M_{u,x} = \gamma\ (\beta_D M_D + \beta_L M_L) = 1.3\ [\ 1.0\ (\ M_G + M_D) + 1.67\ M_L]$ (AASHTO Eq. 3-10)

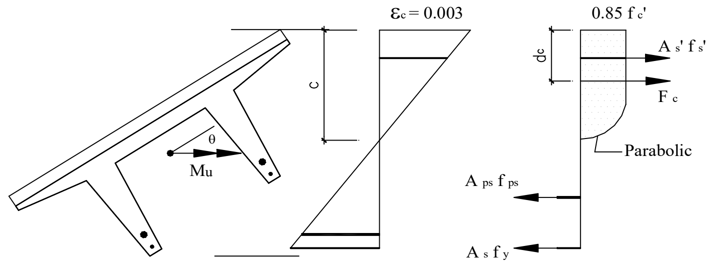
= 1038.329 ft-k

$M_{u,y}$ = 510.732 ft-k

$M_u = (M_{u,x}^2 + M_{u,y}^2)^{0.5}$

= 1157.141 ft-k

θ = 26.2 deg



COMPRESSION ZONE FACTOR

β_1 = 0.75 , (ACI 318-14 22.2.2)

TENDON TYPE FACTOR

γ_p = 0.280 , (ACI 318-14 20.3.2.3.1)

RATIO OF TENSION REINF.	ρ =	0.001	, (ACI 318-14 Chapter 2)
RATIO OF COMPR. REINF.	ρ' =	0.000	, (ACI 318-14 Chapter 2)
RATIO OF PRESTR. REINF.	ρ_p =	0.002	, (ACI 318-14 Chapter 2)
INDEX OF TENSION REINF.	ω =	0.006	, (ACI 318-14 20.3.2.3.1)
INDEX OF COMPR. REINF.	ω' =	0.000	, (ACI 318-14 20.3.2.3.1)
INDEX OF PRESTR. REINF.	ω_p =	0.070	, (ACI 318-14 20.3.2.3.1)

$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c}\ ,\ E_c = 57\sqrt{f'_c}\ ,\ E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right)-\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] \ ,\ for\ \ 0<\varepsilon_c<\varepsilon_o \\ 0.85 f'_c\ ,\ for\ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \le \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$

STRESS IN BONDED TENDONS :

$$f_{ps} = f_{pu}\left[1-\left(\frac{\gamma_p}{\beta_1}\right)\times MIN\left(\rho_p\frac{f_{pu}}{f'_c}+\frac{d\left(\omega-\omega'\right)}{d_p}\ ,\ 0.17\right)\right] =$$ 252.864 ksi

STRESS IN UNBOUNDED TENDONS :

$$f_{ps} = MIN\left(f_{se}+10+\frac{f'_c}{100\rho_p}, f_y, f_{se}+60\right) =$$ Not applicable

$$f_{ps} = MIN\left(f_{se}+10+\frac{f'_c}{300\rho_p}, f_y, f_{se}+30\right) =$$ Not applicable

$\varepsilon_{s,max}$ = 0.0021 , (ACI 318-14 7.3.3 or R21.2.2)

c = 20.9 in, by pure math method

F_c = 320.9 kips

d_c = 8.2 in

$\phi\ M_n$ = 10989 ft-k

> M_u

[Satisfactory]

$$1.2M_{cr} = 1.2S_b\left[P_e\left(\frac{1}{A_c}+\frac{e}{S_b}\right)+7.5\sqrt{f'_c}\right]-M_G\left(\frac{S_{cb}}{S_b}-1\right) =$$ 444 ft-k < $\phi\ M_n$ **[Satisfactory]**

(AASHTO 9.18.2 & ACI 318-11 18.8.2, but ACI 318-14 waived)

CHECK SHEAR CAPACITY (AASHTO 9.20, ACI 318-14 9 & 22)

$$d = MAX\left(0.8h\ ,\ d_p\right) =$$ 19.20 in

$$\sqrt{f'_c} = MIN\left(100\ ,\ \sqrt{\left(f'_c\right)_{provd}}\right) =$$ 77.46 psi

$$V_c = \begin{cases} MAX\left\{MIN\left[\left(0.6\sqrt{f'}+700MIN\left(1\ ,\ \frac{V_u d_p}{M_u}\right)\right)b_w d\ ,\ 5b_w d\sqrt{f'}\right]\ ,\ 2b_w d\sqrt{f'}\right\}\ ,\ for\ \ f\ \ \ge 0.4f \\ 2b_w d\sqrt{f'_c}\ ,\ for\ \ f_{se}<0.4f_{pu} \end{cases} =$$ 178.47 kips

$$V_s = MIN\left(\frac{A_v f_y d}{S}\ ,\ 8b_w d\sqrt{f'_c}\right) =$$ 92.16 kips

$$A_{v(\min)} = \begin{cases} MAX\left(\frac{50b_w S}{f_y}\ ,\ \frac{A_{ps}f_{pu}S}{80df_y}\sqrt{\frac{d}{b_w}}\right)\ ,\ for\ \ f\ \ \ge 0.4f \\ \frac{50b_w S}{f_y}\ ,\ for\ \ f_{se}<0.4f_{pu} \end{cases} =$$ 0.500 in^2

$$S_{\max} \begin{cases} MIN\left(0.75d\ ,\ 24\right),\ for\ \ V_s \le 4b_w d\sqrt{f'_c} \\ MIN\left(0.375d\ ,\ 12\right),\ for\ \ V_s > 4b_w d\sqrt{\ } \end{cases}$$ 14.40 in

(Cont'd)

$$A_{v,requd} = \begin{cases} no \quad shear \quad reinf. \quad requd \quad , \quad for \quad case1: \; V_u < \dfrac{\phi V_c}{2} \\ A_{v(\min)} \quad , \quad for \quad case2: \; \dfrac{\phi V_c}{2} \le V_u \le \phi V_c \\ MAX\left(A_{v,cal} \quad , \quad A_{v(\min)}\right) \quad , \quad for \quad case3: \; \phi V_c \le V_u \le \phi\left(V_S + V_c\right) \\ unsatisfactory \quad , \quad for \quad case4: \; \phi\left(V_S + V_c\right) \le V_u \end{cases} = \quad 0.000 \; in^2$$

No Shear Reinf. Reqd, Case 1 applicable

CHECK CANTILEVER CAPACITY OF SLAB / DECK (AASHTO 9, ACI 318-14 7)

$d_{end\ slab}$ = 3.00 in

ϕ $M_{n,slab}$ = 2612 lbs-ft / ft **>** $M_{u,slab}$ = 515 lbs-ft / ft **[Satisfactory]**

ϕ $V_{n,slab}$ = 4183 lbs / ft **>** $V_{u,slab}$ = 456 lbs / ft **[Satisfactory]**

Daniel T. Li
Engineering International

PROJECT :
CLIENT :
JOB NO. : DATE :

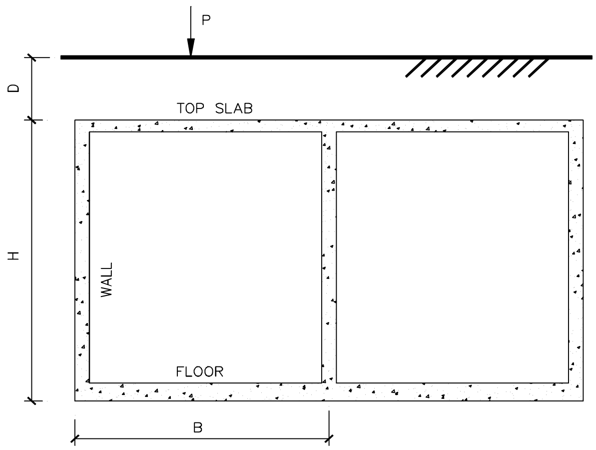
PAGE :
DESIGN BY :
REVIEW BY :

Concrete Box Culvert Design Based on AASHTO 17th & ACI 318-08

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
LATERAL SOIL PRESSURE	P_a	=	45	pcf
(equivalent fluid pressure)				
BACKFILL WEIGHT	γ_b	=	140	pcf
TOP LIVE SURCHARGE	w_s	=	100	psf, vertical
ONE WHEEL LOAD (HS20 Min.)	P	=	18	kips
SEISMIC GROUND SHAKING	P_E	=	20	psf /ft, ASD

(soil pressure, if no report $35S_{DS}$ suggested.)



THICKNESS OF TOP SLAB t_s = 10 in

[THE DESIGN IS ADEQUATE.]

SLAB TRANS REBARS # 6 @ 10 in o.c.
SLAB BAR LOCATION (1=at middle, 2=at top & bot) 2 at top & bottom

THICKNESS OF WALL t_w = 9 in
WALL VERTICAL REBARS # 5 @ 12 in o.c.
WALL BAR LOCATION (1=at middle, 2=at each face) 2 at each face

DEPTH OF FILL.
D = 2.8 ft

DIMENSION
H = 10 ft
B = 8 ft

THICKNESS OF FLOOR t_f = 12 in
FLOOR TRANS REBARS # 6 @ 12 in o.c.
FLOOR BAR LOCATION (1=at middle, 2=at top & bot) 2 at top & bottom

ANALYSIS

CHECK TOP SLAB CAPACITY

$M_u = (1.2\,\gamma_b\,D + 1.6\,w_s)\,B^2 / 8 + 1.6\,P\,I\,B / (4\,E) =$ 15.55 ft-kips / ft, (possible max moment conservatively)

$V_u = (1.2\,\gamma_b\,D + 1.6\,w_s)\,B / 2 + 1.6\,P\,I / E =$ 7.77 kips / ft, (possible max shear force conservatively)

$P_u =$ 0 slab axial force, zero conservatively, since tension controlled. (ACI 318-08 Fig. R9.3.2)

where I = 1.113 Impact Factor (AASHTO 17 3.8.2.3)

E = Min[7 ,Max(4 + 0.12 B , 1.75 D)] = 4.96 ft, point load to load per linear foot

$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7 b f_c'}\right)\right]$$ = 17.06 ft-kips / ft > M_u **[Satisfactory]**

, (ACI 318-08 9 & 10)

ρ_{ProvD} = 0.0058 < ρ_{MAX} = 0.0181 , (ACI 318-08 10.3.5)

> ρ_{MIN} = 0.0033 , (ACI 318-08 10.5) **[Satisfactory]**

$$\phi V_n = 2\phi b d\sqrt{f_c'}$$ = 8.12 kips / ft > V_u **[Satisfactory]**

, (ACI 318-08 9 & 11)

where d = 7.63 in, b = 12 in, A_s = 0.528 in^2 / ft

WALL LATERAL LOADS

$H_b = 0.5\,P_a\,H^2$ = 2.25 kips / ft , $\gamma H_b = 1.6\,H_b$ = 3.60 kips / ft

$H_s = P_a\,D + \text{Max}(36 \text{ lbs/ft}, w_s\,P_a / \gamma_b)\,H$ = 0.49 kips / ft , $\gamma H_s = 1.6\,H_s$ = 0.78 kips / ft

$H_E = 0.5\,P_E\,H^2$ = 1.00 kips / ft , $\gamma H_E = 1.6\,H_E$ = 1.60 kips / ft

(cont'd)

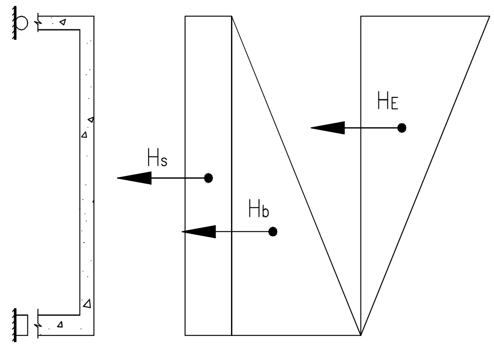
CHECK WALL CAPACITY

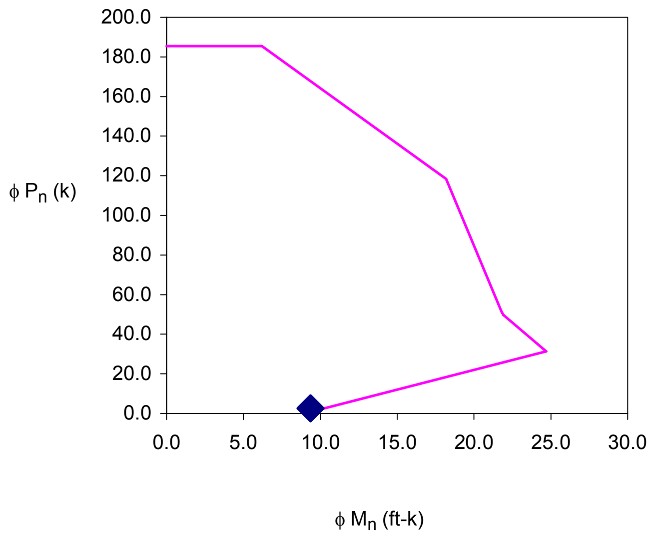
$M_u = (0.1875\,\gamma\,H_s + 0.175\,\gamma\,H_b + 0.100\,\gamma\,H_E)\,H$

= 9.36 ft-kips / ft, (possible max moment conservatively)

$V_u = \gamma\,H_s + \gamma\,H_b + \gamma\,H_E$ = 5.98 kips / ft

P_u = 2.63 kips / ft, (DL only, since tension controlled.)





	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	185.5	0.0
AT MAXIMUM LOAD	185.5	6.2
AT MIDDLE	118.5	18.2
AT ε_t = 0.002	51.5	21.8
AT BALANCED	49.8	21.9
AT ε_t = 0.005	31.3	24.7
AT FLEXURE ONLY	0.0	9.0

(Note: For middle reforming the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

P_u =	2.63	kips / ft
M_u =	9.36	ft-kips / ft

[Satisfactory] , (ACI 318-08 9 & 10)

$\phi V_n = 2\phi b d\sqrt{f'_c}$ = 7.12 kips / ft > V_u **[Satisfactory]**

, (ACI 318-08 9 & 11)

where d = 6.69 in, b = 12 in, A_s = 0.310 in^2 / ft

CHECK BOTTOM FLOOR CAPACITY

$M_u = [1.2\,\gamma_b\,D + 1.6\,w_s + 1.6\,P\,I / (E\,B) + 1.2\,(0.15)\,(t_s - t_f)]\,B^2 / 8$ = 8.85 ft-kips / ft, (max moment conservatively)

$V_u = 4\,M_u / B$ = 4.42 kips / ft, (possible max shear force conservatively)

P_u = 0 floor axial force, zero conservatively, since tension controlled. (ACI 318-08 Fig. R9.3.2)

$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f'_c}\right)\right]$ = 18.33 ft-kips / ft > M_u **[Satisfactory]**

, (ACI 318-08 9 & 10)

ρ_{ProvD} = 0.0038 < ρ_{MAX} = 0.0181 , (ACI 318-08 10.3.5)

> ρ_{MIN} = 0.0033 , (ACI 318-08 10.5) **[Satisfactory]**

$\phi V_n = 2\phi b d\sqrt{f'_c}$ = 10.25 kips / ft > V_u **[Satisfactory]**

, (ACI 318-08 9 & 11)

where d = 9.63 in, b = 12 in, A_s = 0.440 in^2 / ft

Daniel T. Li
Engineering International

PROJECT :			PAGE :	
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Steel Road Plate Design Based on AASHTO 17th Edition & AISC 360-10 using Finite Element Method

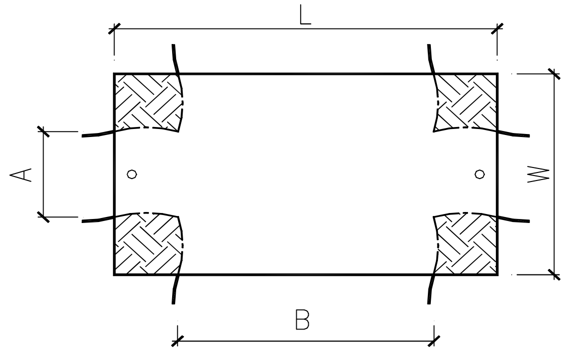
INPUT DATA & DESIGN SUMMARY

MAXIMUM TRENCH WIDTH A = **4** ft W = **6** ft
(A or B zero for one way trench.) B = **6** ft L = **8** ft `

STEEL PLATE THICKNESS t = **1.625** in *(Weight =* 3185 lbs)

TRUCK CONCENTRATED LO/ P = **26** kips, (AASHTO Fig. 3.7.6B)
(26 kips for H20-44, 19.5 kips for H15-44)

STEEL YIELD STRESS F_y = **50** ksi

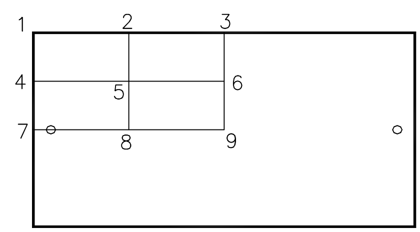


THE DESIGN IS ADEQUATE.
(The Minimum Plate Size: 6 ft x 8 ft x 1.625 in., Wt: 3185 lbs.)

ANALYSIS

IMPACT FACTOR (AASHTO 3.8.2)
$I = 1 + Min[\,0.3\,,\ 50/(L + 125)\,] =$ 1.30
Where $L = Max(A\,,\ B) =$ 6.0 *ft*

JOINT DEFLECTIONS, REACTIONS, & PLATE SECTION FORCES
$P\,I =$ 33.8 kips,(*Truck point load at Joint 9.)*



Joint Number	Δ in	R kips
1	0	-9.246
2	0.60	
3	0.85	
4	0.19	
5	0.69	
6	0.92	
7	0.26	
8	0.74	
9	0.99	

Bending Section	M ft-k/ft
7 - 8	8.63
8 - 9	10.54
3 - 6	16.55
6 - 9	18.26

Minimum W = 6 ft, *for FEM calculation*
Minimum L = 8 ft, *for FEM calculation*
E = 29000 ksi

CHECK BENDING CAPACITY (AISC 360-10, F11)
$M_n\ /\ \Omega_b$ = 19.8 ft-k/ft **>** M = 17.4 ft-k/ft, *(Max section average moment.)*
[Satisfactory]
Where $M_n = Min(\,F_y\,Z\,,\ 1.6\,M_y\,) =$ 33.0 ft-kips
Ω_b = 1.67 , (AISC 360-10, F1)
d = 12.0 in
$Z = d\,t^2 / 4 =$ 7.9 in^3
$M_y = F_y\ d\,t^2 / 6 =$ 22.0 ft-kips

CHECK SHEAR CAPACITY (AISC 360-10, G2)
$V_n\ /\ \Omega_v$ = 2101.8 kips **>** V = 33.8 kips **[Satisfactory]**
Where $V_n = 0.6\,F_y\,A_w\,C_v =$ 3510.0 kips
Ω_b = 1.67 , (AISC 360-10, G1)
C_v = 1.00
$A_w = Min\,(A + 2\,,\ B + 2,\ 8\ ft)\ t =$ 117.0 in^2

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PROJECT :
CLIENT :
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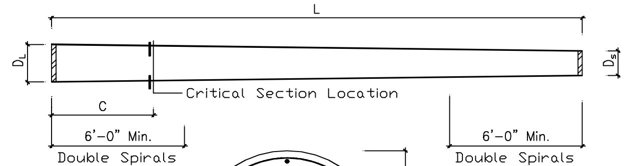
REVIEW BY :

Prestressed Concrete Circular Hollow Pole/Pile Design Based on ACI 318-11 & AASHTO 17th

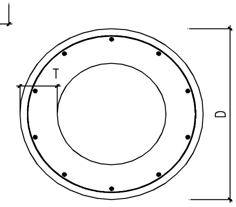
INPUT DATA & DESIGN SUMMARY

OVERALL LENGTH	L =	85	ft
SMALL END DIAMETER	D_S =	11	in
WEB THICKNESS	T_S =	2.76	in
LARGE END DIAMETER	D_L =	29.4	in
WEB THICKNESS	T_L =	3.5	in



LOCATION OF CRITICAL CROSS SECTION C = 10 ft

D =	27.2	in			
T =	3.4	in	A =	255.4	in^2
I =	18491.3	in^4	S =	1357.9	in^3



Critical Cross Section

SHIPPING & ERECTION LOADS AT CRITICAL CROSS SECTION (ASD level)

P = -15.77 kips, axial tension

V = 7.885 kips, shear M = 335.1125 ft-kips, bending

THE DESIGN IS ADEQUATE.

FACTORED ULTIMATE LOADS AT CRITICAL CROSS SECTION (SD level)

P_u = 10 kips, axial M_u = 406 ft-kips, bending

V_u = 12 kips, shear T_u = 150 ft-kips, torsion

CONCRETE STRENGTH	f'_c =	9	ksi
REBAR STRENGTH	$f_y^* = f_y$ =	60	ksi
STRAND TENSILE STRENGTH	$f_{su}^* = f_{pu}$ =	270	ksi
STRAND YIELD STRENGTH	f_{py} =	243	ksi

PRESTRESSING METHOD (0, 1 or 2) 0 pre-tensioned

PRESTRESSING STRAND 16 strands (each 0.5 in dia. & 0.153 in^2 area)

STRAND FORCE IMMEDIATELY AFTER PRESTRESS TRANSFER 0.75 f_{py} , (ACI 318-11 18.5) P_i = 446.1 kips

STRAND FORCE AT SERVICE LOAD AFTER ALLOWANCE LOSSES 0.6 f_{py} , (ACI 318-11 18.6) P_e = 356.9 kips

VERTICAL REINFORCEMENT (0 bars for not apply) 4 # 5

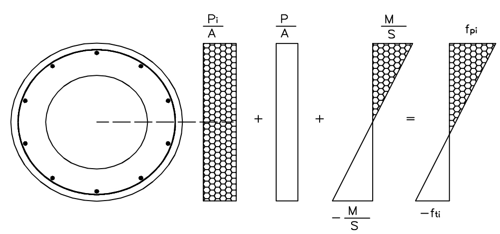
LATERAL REINF. OPTION (0=Spirals, 1=Ties) 0 Spirals

LATERAL REINFORCEMENT # 4 @ 12 in o.c.

ANALYSIS

CHECK TRANSFER (SHIPPING & ERECTION) LOAD CONDITION (ACI 318 18.4.1, 18.5.1, AASHTO 9.15.1 & 9.15.2.1)

TENSION FIBER STRESS	$-F_{ti}$ =	-0.712	ksi, (AASHTO 9.15.2.1)
COMPRESSION FIBER STRESS	F_{pi} =	5.400	ksi, (AASHTO 9.15.2.1)
STRAND ALLOWABLE STRESS	F_{si} =	216.00	ksi, (ACI 318 18.5.1)



f_{si} = 182.25 ksi < F_{si}

[Satisfactory]

$$f_{ti} = \frac{P_i}{A} + \frac{P}{A} - \frac{M}{S} =$$ -1.277 ksi < $-F_{ti}$

[MIN. As' REQUIRED]

$$f_{pi} = \frac{P_i}{A} + \frac{P}{A} + \frac{M}{S} =$$ 4.646 ksi < F_{pi}

[Satisfactory]

$(A_s')_{reqd}$ = 1.171 in^2 < $(A_s')_{provd}$

(ACI 318-11 R18.4.1) **[Satisfactory]**

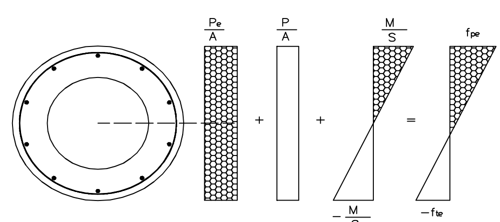
CHECK SERVICEABILITY LOAD CONDITION (ACI 318 18.3.3 & 18.4)

SERVICE LOADS

P = P_u / 1.2 = 8.3 kips, axial

M = M_u / 1.4 = 290.0 ft-kips, bending

TENSION FIBER STRESS	$-F_{te}$ =	-1.138	ksi, (Class U or T, ACI 318 18.3.3)
COMPRESSION FIBER STRESS	F_{pe} =	4.050	ksi, (ACI 318 18.4.2)
STRAND ALLOWABLE STRESS	F_{se} =	216.00	ksi, (ACI 318 18.5.1)



f_{se} = 145.80 ksi < F_{se}

[Satisfactory]

$$f_{te} = \frac{P_e}{A} + \frac{P}{A} - \frac{M}{S} =$$ -1.133 ksi > $-F_{ti}$

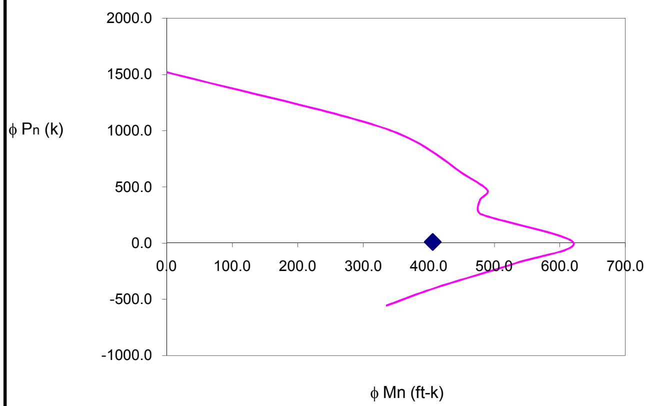
[Satisfactory]

$$f_{pe} = \frac{P_e}{A} + \frac{P}{A} + \frac{M}{S} =$$ 3.993 ksi < F_{pi}

[Satisfactory]

(Cont'd)

CHECK ULTIMATE LOAD CONDITION (AASHTO 9.15.1 & 9.17 ACI 318-11 18.7)



$$\varepsilon_O = \frac{2\left(0.85 f'_C\right)}{E_C} \quad , \quad E_C = 57\sqrt{f'_C} \quad , \quad E_S = 29000ksi$$

$$f_C = \begin{cases} 0.85 f'_C\left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2\right] & , \ for \quad 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f'_C \ , \ for \ \ \varepsilon_C \geq \varepsilon_O \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S \ , \ for \ \ \varepsilon_S \leq \varepsilon_y \\ f_y \ , \ for \ \ \varepsilon_S > \varepsilon_y \end{cases}$$

$$f_{PS} = Min\left(\varepsilon_S E_S + \frac{P_e}{A_{ps}} \ , \ F_{se}\right)$$

ϕM_n =	618.4	ft-kips @ P_u =	10	kips	**>**	M_u	**[Satisfactory]**

CHECK SHEAR & TORSIONAL CAPACITY (ACI 318-11 11.3)

$\phi v_n = \phi (v_c)$ =	142	psi	**>**	v_u	**[Satisfactory]**
where ϕ =	0.75	(ACI 318-11 9.3.2.3)			
$v_c = 2\,(f_c')^{0.5}$ =		190	psi		
$A_v = 0.8\,A$ =		204.3	in^2		
$v_u = [V_u + T_u / (0.5D - T)] / A_v$ =		131	psi		

Purchaser's LOGO

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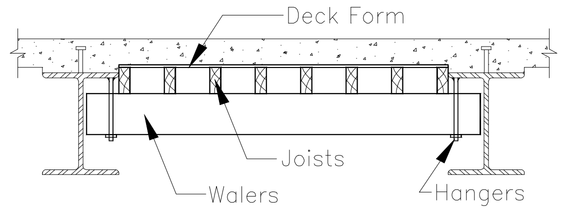
Falsework Design for Steel Girder Bridge Based on NDS 2012 & AASHTO 17th

INPUT DATA & DESIGN SUMMARY

DECK FORM 3/4 in, plywood

JOISTS 1 - 2 x 4
@ 12 *in. O. C., (No. 2, Douglas Fir-Larch)*

WALERS 2 - 2 x 12
@ 48 *in. O. C., (No. 1, Douglas Fir-Larch)*



WALER SPAN (Hanger to Hanger) 8 ft

THE FALSEWORK DESIGN IS ADEQUATE.

CONCRETE DECK THICKNESS 9 in

ANALYSIS

CHECK BENDING CAPACITY OF DECK FORM

$M = w\,L^2 / 8 =$ 259.69 in-lbs / ft < $C_D F_b S =$ 312.50 in-lbs / ft **[Satisfactory]**

where $w =$ 120 + 50 + 3.13 = 173.13 psf
(conc wt, 160 pcf) (LL) (plywood)

$L =$ 12 in

$C_D =$ 1.25 , (NDS 2012 Table 2.3.2)

$F_b S =$ 250 psf, (NDS 2012 Table M9.2-1 or C3.2A)

CHECK JOISTS

$f_b = M / S =$ 1096.8 psi < $C_D C_F C_V C_L F_b =$ 1647.5 psi **[Satisfactory]**

where $w =$ 173.13 x 1.00 + 1.8229 = 174.95 lbs / ft
(Spacing) (joist wt)

$M = w\,L^2 / 10 =$ 279.92 ft-lbs

$L =$ 4.00 ft

$S =$ 3.06 in^3

$C_F =$ 1.50 , (NDS 2012 Section 4.3.6)

$C_V =$ 1.00 , (NDS 2012 Section 5.3.6)

$C_L = (1+F) / 1.9 - [((1+F) / 1.9)^2 - F / 0.95]^{0.5} =$ 0.98 , (NDS 2012 Section 3.3.3)

$F = F_{bE} / F_b{}^* =$ 2.99

$F_b =$ 900.0 psi, (No. 2, Douglas Fir-Larch) <== 3 *of 1 to 6*

$f_v = 3V / 2A =$ 120.0 psi < $C_D F_v =$ 225.0 psi **[Satisfactory]**

where $V = 0.6\,w\,L =$ 419.9 lbs

$A =$ 5.25 in^2

$F_v =$ 180.0 psi, (No. 2, Douglas Fir-Larch)

$\Delta = w\,L^4 / 145\,EI =$ 0.044 in < $L / 360 =$ 0.133 in **[Satisfactory]**

where $w =$ 124.95 lbs / ft, (no live load)

$E =$ 1600 ksi, (No. 2, Douglas Fir-Larch)

$I =$ 5.36 in^4

CHECK WALERS

$f_b = M / S =$ 1185.5 psi < $C_D C_F C_V C_L F_b =$ 1234.1 psi **[Satisfactory]**

where $w =$ 769.77 / 1.00 + 11.719 = 781.49 lbs / ft
(1.1 x Reaction) (Spacing) (waler wt)

$M = w\,L^2 / 8 =$ 6251.9 ft-lbs

$L =$ 8.00 ft

$S =$ 63.28 in^3

$C_F =$ 1.00 , (NDS 2012 Section 4.3.6)

$C_V =$ 1.00 , (NDS 2012 Section 5.3.6)

$C_L = (1+F) / 1.9 - [((1+F) / 1.9)^2 - F / 0.95]^{0.5} =$ 0.99 , (NDS 2012 Section 3.3.3)

$F = F_{bE} / F_b{}^* =$ 4.82 *, (where $L_u =$* 1 *ft, joist spacing for NDS 2012 Table 3.3.3)*

$F_b =$ 1000.0 psi, (No. 1, Douglas Fir-Larch) <== 2 *of 1 to 6*

$f_v = 3V / 2A =$ 138.9 psi < $C_D F_v =$ 225.0 psi **[Satisfactory]**

where $V = 0.5\,w\,L =$ 3126.0 lbs

(Cont'd)

$A =$ 33.75 in^2

$F_v =$ 180.0 psi, (No. 1, Douglas Fir-Larch)

$\Delta = 5\, w\, L^4 / 384\, EI =$ 0.119 in **<** $L / 360 =$ 0.267 in **[Satisfactory]**

where $w =$ 781.49 lbs / ft

$E =$ 1,700 ksi, ()

$I =$ 355.96 in^4

Hanger Load, $T = 1.5\, V =$ 4.69 kips

Minimum Bracket Bearing Area, $A_b = T / 400\ psi =$ 11.72 in^2

Minimum Hanger Diameter, $d = (4T / 24\ ksi\ \pi)^{0.5} =$ 0.50 in

Purchaser's LOGO

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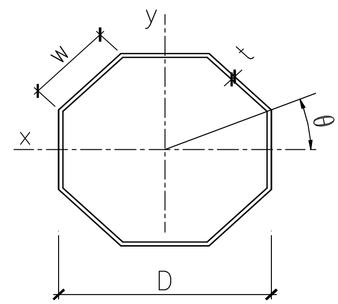
Polygon Section Member (Tubular Steel Pole) Design Based on ASCE 48-11

INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	50	ksi
SECTION DIMENSIONS	t =	0.5	in
	D =	20	in
POLYGON SIDE NUMBER (6, 8, 12, or 16 only)	n =	8	
	(Octagonal 8-Sided Polygon Section)		
AXIAL LOAD, SD level (Factored Section Force)	P =	525	kips
AXIS, x-x, BENDING LOAD, SD level	M_x =	150	ft-kips
AXIS, y-y, BENDING LOAD, SD level	M_y =	0.15	ft-kips
SHEAR LOAD, SD level	V =	150	kips
TORSIONAL LOAD, SD level	T =	20	ft-kips
UNBRACED AXIAL LENGTH	KL =	50	ft



THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE SECTION PROPERTIES (ASCE 48-11 Appendix II)

$BR = Min(\,actual\,,\,4\,t\,)$ = 2 in, effective bend radius.

θ =	22.5	deg			r =	0.364	D =	7.280	in
A_g =	3.32	$D\,t$ =	33.200	in^2	w =	0.414	$(D - t - 2\,BR)$ =	6.417	in
$I_x = I_y$ =	0.438	$D^3\,t$ =	1752.00	in^4					
c_x =	0.541	$(D + t)\,Cos\,\theta$ =	10.246	in	Max. $Q / I\,t$ =	0.618	$/\,D\,t$ =	0.062	
c_y =	0.541	$(D + t)\,Sin\,\theta$ =	4.244	in	Max. c / J =	0.603	$(D + t) / (D^3\,t)$ =	0.0031	

CHECK COMBINED STRESSES (ASCE 48-11 5.2)

$$\left[\left(\frac{P}{A}+\frac{M_x c_y}{I_x}+\frac{M_y c_x}{I_y}\right)^2+3\left(\frac{VQ}{It}+\frac{Tc}{J}\right)^2\right]^{(1/2)} = 26.6 \quad < \quad k\,(F_a \text{ or } F_t) = 27.8 \text{ ksi}$$

[Satisfactory]

Where k = $(0.877\ \phi_c)$ = 0.877 x 0.9 = 0.7893 , from AISC 360-10 E3-3 & E1 than ASCE 48-11 Eq. 5.2-4

F_t = F_y = 50.0 ksi, (ASCE 48-11 Eq. 5.2-1) **>** T / k A_g = 0.0 ksi **[Satisfactory]**

$$F_{a,Beam}=\begin{cases} F_y \ , \ for \ \dfrac{w}{t}\le k_1\dfrac{\Omega}{\sqrt{F_y}} \\ k_3F_y\left(1.0-k_4\dfrac{\sqrt{F_y}}{\Omega}\dfrac{w}{t}\right) \ , \ for \ k_1\dfrac{\Omega}{\sqrt{F_y}}<\dfrac{w}{t}\le k_2\dfrac{\Omega}{\sqrt{F_y}} \\ 0 \ , \ for \ \dfrac{w}{t}>k_2\dfrac{\Omega}{\sqrt{F_y}} \end{cases} = 50.0 \text{ ksi, (ASCE 48-11 5.2.3.2.1)}$$

> P / k A_g = 20.0 ksi **[Satisfactory]**

k_1 = 260 , k_2 = 351 , k_3 = 1.42 , k_4 = 0.00114

Ω = 1.0

$$F_{a,Truss}=\begin{cases} F_y\left(1-0.5\left(\dfrac{KL}{rC_c}\right)^2\right) \ , \ for \ \dfrac{KL}{r}\le C_c \\ \dfrac{\pi^2E}{\left(\dfrac{KL}{r}\right)^2} \ , \ for \ \dfrac{KL}{r}>C_c \end{cases} = 35.2 \text{ ksi, (ASCE 48-11 5.2.3.2.1)}$$

> P / k A_g = 20.0 ksi **[Satisfactory]**

KL / r = 82.4 , C_c = 107.0 , (ASCE 48-11 Eq. 5.2-5)

$$\frac{VQ}{It}\frac{1}{t}+\frac{Tc}{J} = 19.3 \quad < \quad k\,F_v = k\,0.58F_y = 22.9 \text{ ksi}$$

[Satisfactory]

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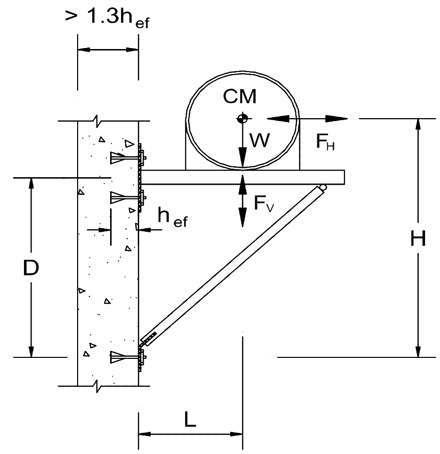
Mounting Design on Concrete Wall/Tunnel Based on FEMA E-74, 2012 IBC, and 2013 CBC Chapter A

INPUT DATA & DESIGN SUMMARY

EQUIPMENT WEIGHT	W	=	3.5	kips
CENTER OF MASS	H	=	3.5	ft
	L	=	2	ft
STRUT DIMENSION	D	=	3	ft
KWIK BOLT-TZ DIAMETER	ϕ	=	3/4	in. Per ICC ESR-1917
ANCHOR DEPTH	h_{ef}	=	4 3/4	in. Per ICC ESR-1917
EDGE DISTANCE	c	=	12	in
ANCHOR MINIMUM SPACING	s	=	10	in



[THE ANCHORAGE, KWIK BOLT-TZ, DESIGN IS ADEQUATE.]

ANALYSIS

TENSION & SHEAR CAPACITY (***SD*** - 2013 CBC 1909A,and ***ASD*** - ICC ESR-1917, 4.2)

$\phi P_{n,t}$ = 7399.5 lbs, *2013 CBC 1905A & ACI 318-11 App.D* P_t = 4933 lbs, $T_{allowable,ASD}$

$\phi V_{n,t}$ = 8838.2 lbs, *2013 CBC 1905A & ACI 318-11 App.D* V_t = 6313 lbs, $V_{allowable,ASD}$

SPACING & EDGE REQUIREMENTS (ICC ESR-1917, Table 3 or 4)

S_{cr} = 7 3/4 in, (Critical Spacing for $K_{edge\text{-}space}$ based on ACI 318-11 Appendix D)

S_{min} = 4 in

C_{cr} = 8 7/8 in, shear, 8 7/8 in, tension

C_{min} = 4 1/8 in, shear, 4 1/8 in, tension

DESIGN LOADS

$F_H = F_p = \Omega_0 (K_H)$ MAX{ $0.3S_{DS}I_pW$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p W$, $1.6S_{DS}I_pW$] } , (ASCE 7-10, Sec. 13.3.1)

= 1.5 (1.3) MAX{ 0.43W , MIN[0.81W , 2.30W]}

= 1.58 W , (SD) = 5.53 kips

= 1.13 W , (ASD) = 3.95 kips

$F_V = K_V W$ = 0.25 W , (SD) = 0.87 kips, up & down

0.18 W , (ASD) = 0.62 kips, up & down

$K_V = K_H\ 0.2\ S_{DS} / 1.4$ = 0.18 (vertical seismic factor)

where

S_{DS} =	0.96	(ASCE 7-10 Sec 11.4.4)
I_p =	1.5	(ASCE Sec. 13.1.3)
a_p =	1	(ASCE Tab. 13.6-1)
R_p =	1.5	(ASCE Tab. 13.6-1)
z =	20	ft
h =	36	ft
Ω_0 =	1.5	(ASCE 7-10 Supp. 1 Tab. 13.6-1)
K_H =	1.3	*(This option only for local jurisdiction amendments to the code.)*

CHECK TENSION CAPACITY

$P_{u,s}$ = 1.50 P_s = 5520 lbs / bolt, SD < $\phi P_{n,t}$ [SATISFACTORY]

$P_s = [(F_V + W) L + F_H H] / (n_t S)$ = 3680 lbs / bolt, ASD < $P_t K_{DSA} K_{seismic} K_{edge\text{-}space}$ [SATISFACTORY]

where n_t = 2 , (total bolts at top tension)

K_{DSA} = 0.8 (DSA/OSHPD adapted ICBO / ICC value)

$K_{seismic}$ = 1 1/3 (allowable increase? CBC 1605A.3.2)

$K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

CHECK SHEAR CAPACITY

$V_{u,s}$ = 1.40 V_s = 5774 lbs / bolt, SD < $\phi V_{n,t}$ [SATISFACTORY]

$V_s = (W + F_V) / n_b$ = 4124 lbs / bolt, ASD < $V_t K_{DSA} K_{seismic} K_{edge\text{-}space}$ [SATISFACTORY]

where n_b = 1 , (total bolts at bottom shear)

$K_{edge\text{-}space}$ = 1.00 (ICC ESR-1917 Sec. 4.2.1 SIM.)

CHECK COMBINED LOADING CAPACITY (ACI 318-11 D1.7 & ICC ESR-1917 Sec. 4.2.2)

$(P_{u,s} / \phi P_{n,t}) + (V_{u,s} / \phi V_{n,t})$ = 0.964 < 1.20 [SATISFACTORY]

$(P_s / P_t) + (V_s / V_t)$ = 0.904 < 1.20 [SATISFACTORY]

where $V_{u,s}$ = 1.40 V_s = 1925 lbs / bolt, SD

$V_s = (W + F_V) / (n_t + n_b)$ = 1375 lbs / bolt, ASD

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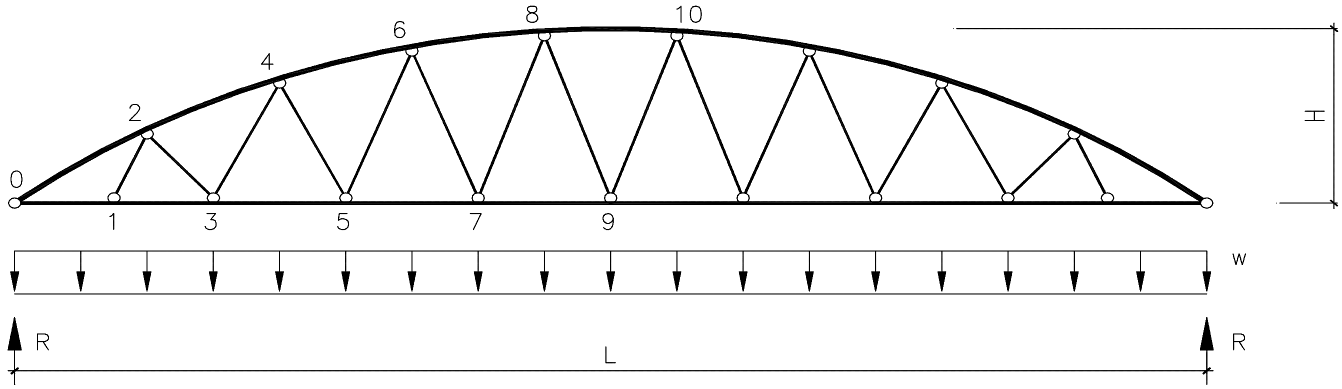
Truss Analysis using Finite Element Method

INPUT DATA & DESIGN SUMMARY



TRUSS SPAN LENGTH L = 150 ft

TRUSS DEPTH H = 40 ft

BOTTOM CHORD DECK LOAD

w = 2 kips / ft, *(including truss weight and impact factor.)*

CONTINUED TOP CHORD SECTION (FLAT)

$A =$ 30.3 in^2

$I_y =$ 119 in^4, in plane

$I_x =$ 3000 in^4, out-of plane

CONTINUED BOTTOM CHORD SECTION

$A =$ 30.3 in^2

$I_x =$ 3000 in^4, in plane

$I_y =$ 119 in^4, out-of plane

ALL WEB MEMBER SECTION

$A =$ 16.379 in^2

MODULUS OF ELASTICITY

$E =$ 29000 ksi

Design Data (Conservative Values)

	Length (ft)	In Plane Support (ft)	P (kips, axial)	M (ft-kips, in plane)	V (kips, in plane)
Top Chord	177.00	25.08	180.21	1.54	0.06
Bottom Chord	150.00	16.67	-9.76 to 12.54	64.72	17.10
Web	40.48	40.48	-32.00		

Reaction ***R =*** 150.00 *kips*

Deflection Δ_{max} = 0.311 *in, (5795 / L)*

ANALYSIS

r = 90.31 ft, radius of top chord

Element	Joint	Joint	L (ft)	P (kips, axial)	M (ft-kips, in plane)	V (kips, in plane)
1	0	1	12.50	8.41	64.72	15.72
2	0	2	25.08	180.21	1.54	0.06
3	1	2	19.09	-18.91	0.00	0.00
4	1	3	12.50	12.54	64.72	15.83
5	2	3	20.41	-32.00	0.00	0.00
6	2	4	20.11	155.49	0.03	0.00
7	3	5	16.67	0.51	50.60	17.10
8	3	4	30.96	-3.86	0.00	0.00
9	4	5	30.96	-29.64	0.00	0.00
10	4	6	17.98	146.52	0.11	0.00
11	5	6	37.41	-4.49	0.00	0.00
12	5	7	16.67	-6.47	51.01	16.69
13	6	7	37.41	-24.09	0.00	0.00
14	6	8	16.99	142.91	0.16	0.00
15	7	8	40.48	-10.09	0.00	0.00
16	7	9	16.67	-9.76	51.79	16.71
17	8	9	40.48	-16.98	0.00	0.00
18	8	10	16.69	141.85	0.16	0.00

Joint	*Δ (in)*
0	0
1	-0.14
2	-0.15
3	-0.20
4	-0.21
5	-0.26
6	-0.26
7	-0.30
8	-0.29
9	-0.31
10	-0.31

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Arch Bridge Analysis using Finite Element Method

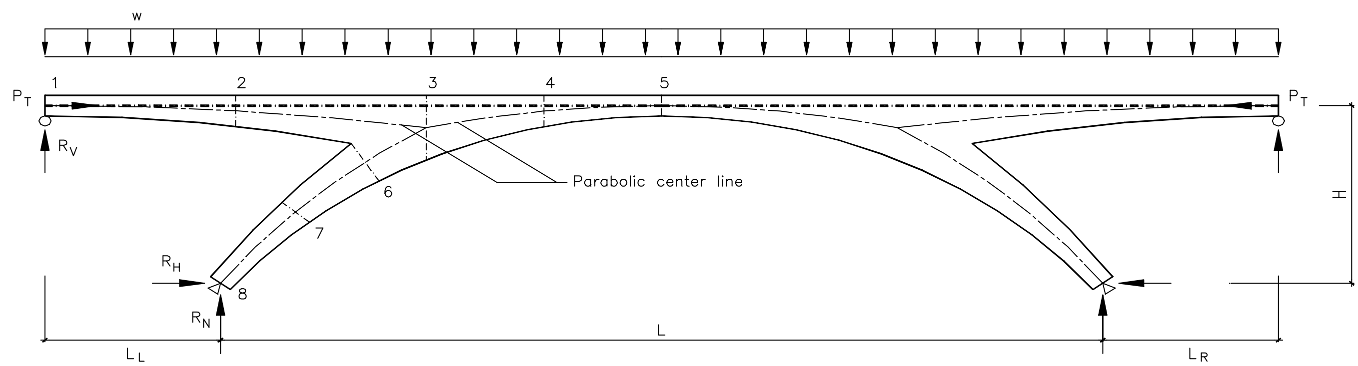
DESIGN CRITERIA

The concrete post-tensioned structural design is actually sections design, no matter box girder, circular column, or other sections.

There are three kind of forces on each section:

1. External loads, *w* only without P_T, section forces. The External loads can be ASD level for serviceability design, or SD level for ultimate strength design.
2. Primary equivalent loads, P_T section forces. The tendon is mentally removed and replaced with all of the loads it exerts on the structure.
3. Secondary section forces from the reactions of primary P_T , at different boundary conditions.

INPUT DATA



DIMENSION

L_L =	5	*ft, (1.52 m)*
L =	130	*ft, (39.62 m)*
L_R =	5	*ft, (1.52 m)*
H =	40	*ft, (12.19 m)*

TOP DECK LOAD

w = 42.1008 *kips/ft, (614.0 kN/m)*

(including bridge weight and impact factor.)

Section	D (in)	A (in^2)	I_x (in^4, in plane)	*D (cm)*	*A (cm^2)*	*I_x (cm^4, in plane)*
1	76	3507.84	2791662	*193*	*22631*	*116197740*
2	113	4307.04	7488499	*287*	*27787*	*311694862*
3	200	6186.24	31119420	*508*	*39911*	*1295288064*
4	107	4177.44	6540969	*272*	*26951*	*272255676*
5	84	3680.64	3584948	*213*	*23746*	*149216781*
6	100	10800	9000000	*254*	*69677*	*374608283*
7	80	8640	4608000	*203*	*55742*	*191799441*
8	60	6480	1944000	*152*	*41806*	*80915389*

TENDON FORCE AFTER ALLOWANCE LOSSES

P_T = 8769.6 *kips, (39007.2 kN)*

MODULUS OF ELASTICITY

E = 4030.509 *ksi, (27789 MPa)*

ANALYSIS & DESIGN SUMMARY

Design Section Forces (Conservative Values)

Section	External Load, w, Only				
	P (kips, axial)	M (ft-kips)	V (kips, in plane)	Δ_x (in)	Δ_y (in)
1	-18.13	0.00	-220.47	0.042	0
2	107.85	5997.86	557.85	0.033	-0.103
3	2260.00	-8471.67	-516.06	0.033	-0.101
4	2095.19	149.39	-219.10	0.024	-0.155
5	2095.19	3715.96	0.00	0.000	-0.194
6	3122.67	2829.70	-63.31	0.036	-0.101
7	3122.67	1414.85	-63.31	0.038	-0.073
8	3122.67	0.00	-63.31	0	0

External load reactions

R_V =	615.91	*kips*
R_H =	2078.66	*kips*
R_N =	2331.15	*kips*

Primary reactions

R_V =	-280.48	*kips*
R_H =	178.11	*kips*
R_N =	280.48	*kips*

Section	Primary equivalent loads, P_T			Secondary section forces from primary reactions		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)	P (kips, axial)	M (ft-kips)	V (kips, in plane)
1	8763.09	0.00	-439.09	22.98	0.00	279.54
2	8663.40	8260.81	-1389.25	53.24	-5258.99	275.38
3	8703.67	31880.12	2075.49	173.25	-13429.48	41.31
4	8932.19	-2792.25	526.77	177.80	-14119.65	10.49
5	8932.19	-11367.14	0.00	177.80	-14290.34	0.00
6	326.72	2698.22	-60.37	326.72	2698.22	-60.37
7	326.72	1349.11	-60.37	326.72	1349.11	-60.37
8	326.72	0.00	-60.37	326.72	0.00	-60.37

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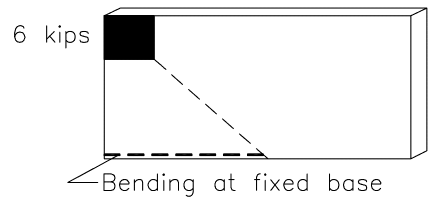
Vehicular Barrier Wall Design Based on ASCE 7-10 & ACI 318-14

DESIGN CRITERIA

1. Vehicular barrier load, 6 kips (ASCE 7-10 4.5.3), is at wall height from 1'-6" to 2'-3" on an area 12 in by 12 in.
2. The maximum load effects are punching/shear when the load at top corner, and base bending when the load at edge with 2'-3" height.

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
THICKNESS OF WALL	t	=	6	in
VERTICAL REINFORCING (A_s)		#	6	@ 12 in o.c.
A_s LOCATION (1=at middle, 2=at each face)			1	at middle



THE WALL DESIGN IS ADEQUATE.

ANALYSIS

CHECK AXIAL & FLEXURE CAPACITY AT FIXED BASE

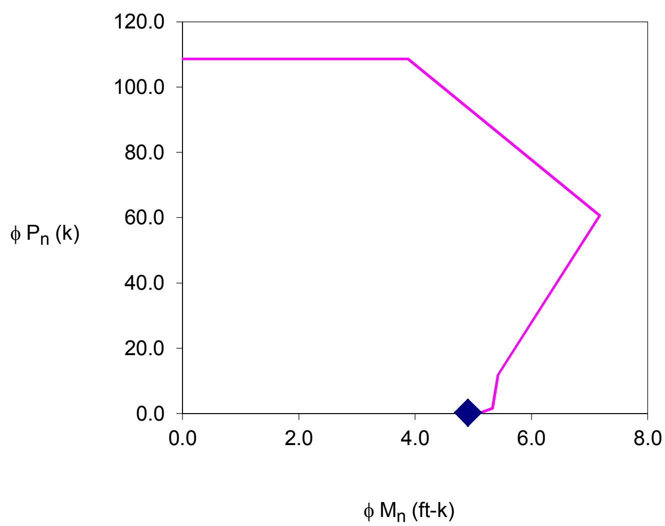
FACTORED MOMENT M_u = 6 kips x 2.25 ft Height / 2.75 ft Spread = 4.9 ft-kips / ft

FACTORED AXIAL LOAD P_u = 0.15 kcf x 2.25 = 0.3375 kips / ft, vertical

ρ_{ProvD} = 0.01222 < ρ_{MAX} = 0.0800 (tension face only, ACI 318-14 22.2.3 or 10.9.1)

> ρ_{MIN} = 0.0015 (tension face only, ACI 318-14 6.6.4.3, 9.6.1 or 11.6.1)

[Satisfactory]



	ϕP_n	ϕM_n
AT AXIAL LOAD ONLY	108.6	0.0
AT MAXIMUM LOAD	108.6	3.9
AT MIDDLE	60.7	7.2
AT ε_t = 0.002	12.7	5.5
AT BALANCED	11.8	5.4
AT ε_t = 0.005	1.6	5.3
AT FLEXURE ONLY	0.0	5.1

(Note: For middle reforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

CHECK PUNCHING/SHEAR CAPACITY (ACI 318-14 13.2.7.2, 7.4.3, & 22.5)

FACTORED SHEAR LOAD V_u = 6 kips / ft, horizontal & perpendicular to wall

$\phi V_n = 2\phi bd\sqrt{f'_c}$ = 6.65 kips / ft > V_u **[Satisfactory]**

where d = 3.00 in

b = 27.00 in, two way sides only

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Footbridge Vibration Design Based on The Structural Engineer, Vol. 94-1, 2016

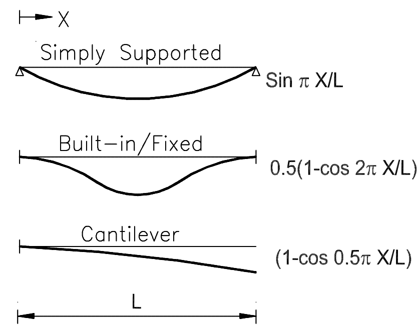
DESIGN CRITERIA

1. Footbridge vibration can be felt strongly by walker/jogger of structures, if the maximum dynamic deflection of single walker with average weight more than (Span / 35000).
2. Typically, footbridge design are controlled by vibration, not by structural capacity. The first resonant frequencies of footbridge between 6.5 and 7.5 Hz fall in the frequency range where human occupants are most sensitive in perceiving vibration.

INPUT DATA & DESIGN SUMMARY

SPAN	L	=	50	ft
MOMENT OF INERTIA	I	=	23917	in^4
MODULUS OF ELASTICITY	E	=	29000	ksi
POISSON'S RATIO	ν	=	0.28	
DAMPING RATIO	ξ	=	0.03	, (ASCE 7-10 16.1.3 & 21.1.3)
FOOTBRIDGE UNIT WEIGHT	w	=	0.05623	kips/ft

EDGE CONNECTION (1, 2, or 3) 1 <== Simply Supported



THE FOOTBRIDGE DESIGN IS ADEQUATE.

ANALYSIS

For Simply Supported

$m^* = \frac{wL}{2g} =$ 0.04 , *generalized mass* $\qquad k^* = \frac{\pi^4 EI}{2L^3} =$ 156 , *equivalent stiffness*

For Built-in/Fixed

$m^* = \frac{3wL}{8g} =$ 0.03 , *generalized mass* $\qquad k^* = \frac{2\pi^4 EI}{L^3} =$ 626 , *equivalent stiffness*

For Cantilever

$m^* = \frac{0.227wL}{g} =$ 0.02 , *generalized mass* $\qquad k^* = \frac{\pi^4 EI}{32L^3} =$ 10 , *equivalent stiffness*

$\omega = \sqrt{\frac{k^*}{m^*}} =$ 59.823 , *circular frequency*

$f = \frac{\omega}{2\pi} =$ 9.521 Hz, *natural frequency* > *7.5*
[Satisfactory]

$T = \frac{1}{f} =$ 0.105 Sec, *natural period*

$c^* = 2\zeta\omega m^* =$ 0.157 , *generalized damping*

$a = \frac{F\omega}{c^*} =$ 57.207 in/sec², *maximum acceleration* where *F =* *150* lbs, *average walker weight*

$y_{\max} = \frac{F}{\omega c^*} =$ 0.016 in, *maximum dynamic deflection* < $\frac{L}{35000} =$ 0.017 in **[Satisfactory]**

References

1. Andrew Robertson, MSc, Ceng, MIStructE, MICE: Simplified dynamic analysis of beams and slabs with tuned mass dampers. The Structural Engineer, Vol. 94-1, 2016.
2. Reza Kashani, PhD, PE: Vibration abatement of rectangular, trapezoidal and irregular-shaped joist-framed floors, using tuned mass dampers. The Structural Engineer, Vol. 94-1, 2016.

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Design of Mechanically Stabilized Earth Wall Based on AASHTO/2015 IBC & TMS 402-13

INPUT DATA & DESIGN SUMMARY

WALL HEIGHT H = 20 ft

WALL THICKNESS t = 12 in

MASONRY STRENGTH f_m' = 1.5 ksi

(modular concrete facing blocks - MBW)

WALL VERTICAL REBAR (fully grouted) f_y = 60 ksi

2 # 5 @ 16 in o.c. at each face

HORIZONTAL BAR MATS L = 18 ft

4 @ 16 in o.c., horizontal each way,

s = 24 in o.c., vertical mat dist.

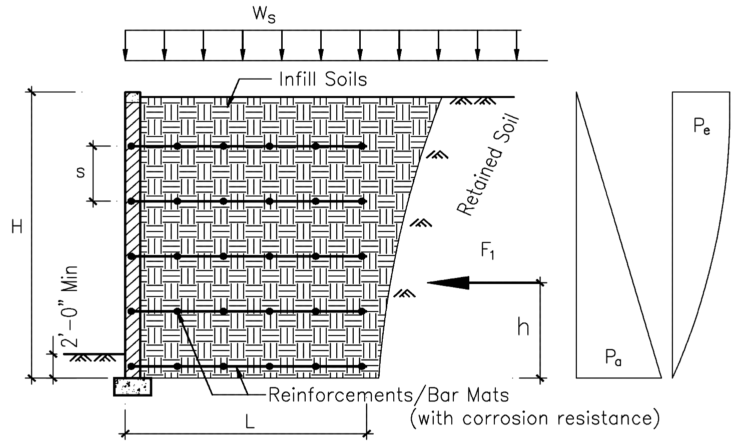
SURCHARGE WEIGHT w_S = 500 psf

SOIL SPECIFIC WEIGHT γ_b = 110 pcf

SOIL INTERNAL FRICTION ANGLE ϕ = 29 deg

HORIZONTAL ACCELERATION (in g) k_h = 0.35 g, (0.5 S_{DS})

ALLOW SOIL PRESSURE Q_a = 4.2 ksf



THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE ACTIVE EARTH PRESSURE

$P_e = 0.75\, k_h\, \gamma_b$ = 29 psf / ft, (FEMA P-750 Page 356)

$P_a = \gamma_b\, K_a$ = 38 psf / ft

where $$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2} = 0.347$$

β = 0 deg, slope of backfill

δ = 0 deg, external friction angle

θ = 90 deg, rack angle of wall face

(Coulomb, AASHTO Figure 5.5.2A)

$F_1 = 0.5\, H^2\,(P_a + P_e)$ = 13 kips / ft, *Total*

$h = (1/3\, P_a + 0.6\, P_e)\, H / (P_a + P_e)$ = 8.96 ft

CHECK SOIL BEARING CAPACITY (AASHTO Figure 5.8.3A)

$V_1 = H\, L\, \gamma_b$ = 39.6 kips / ft, *Vertical*

$F_2 = K_a\, H\, w_s$ = 3.47 kips / ft, *from surcharge weight*

$e = (F_1\, h + F_2\, 0.5\, H) / (V_1 + L\, w_s)$ = 3.187 ft

$\sigma_v = (V_1 + L\, w_s) / (L - 2e)$ = 4.18 ksf < Q_a [Satisfactory]

CHECK SLIDING CAPACITY (2015 IBC 1807.2.3)

$1.1\,(F_1 + F_2)$ = 18.57 kips / ft < $\mathrm{Tan}(\phi)\,(V_1 + L\, w_s)$ = 26.94 kips / ft [Satisfactory]

$1.5\,(F_1 + F_2 - 0.5\, H^2\, P_e)$ = 16.65 kips / ft < $\mathrm{Tan}(\phi)\,(V_1 + L\, w_s)$ = 26.94 kips / ft [Satisfactory]

CHECK OVERTURNING CAPACITY (2015 IBC 1807.2.3)

$1.5\,(h\, F_1 + 0.5\, H\, F_2)$ = 232.3 ft-kips / ft < $0.5\, L\,(V_1 + L\, w_s)$ = 437.40 ft-kips / ft [Satisfactory]

(All forces with safety factor 1.5 conservatively.)

CHECK FLEXURE CAPACITY OF MASONRY WALL (TMS 402-13 8.3.3)

$$M_{allowable} = MIN\left[\frac{1}{2} b_w kd\, F_b\left(d-\frac{kd}{3}\right) - P\left(d-\frac{t_e}{2}\right)\ ,\ A_s F_s\left(d-\frac{kd}{3}\right) + P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right] = 5.18 \text{ ft-kips}$$

> $s^2\,(F_1 + F_2) / H$ = 3.38 ft-kips

[Satisfactory]

where t_e = 11.63 in

d = 9.313 in

b_w = 12 in

E_m = 1050 ksi, 700 f_m' conservative value

E_s = 29000 ksi

(cont'd)

F_b	=	0.495 ksi	n	=	27.62	
F_s	=	32 ksi	k	=	0.29	
A_s	=	0.233 in^2	P	=	1.3	kips, axial force at wall middle
ρ	=	0.002 **>** 0.0007 [Satisfactory]				

CHECK HORIZONTAL BAR MATS (AASHTO 5.8.4.1)

$T_{max} = \sigma_h A_{trib} =$ 0.64 kips / bar **<** $F_s A_s =$ 6.40 kips / bar [Satisfactory]

where σ_h = $F_2 / H + P_a + P_e =$ 0.24 ksf

A_{trib} = 384 in^2

CHECK HORIZONTAL BAR ANCHORAGE IN MASONRY (TMS 402-13 8.1.6.4.1)

$L_{anchor} = 12\ d_b =$ 6 in **<** t_e [Satisfactory]

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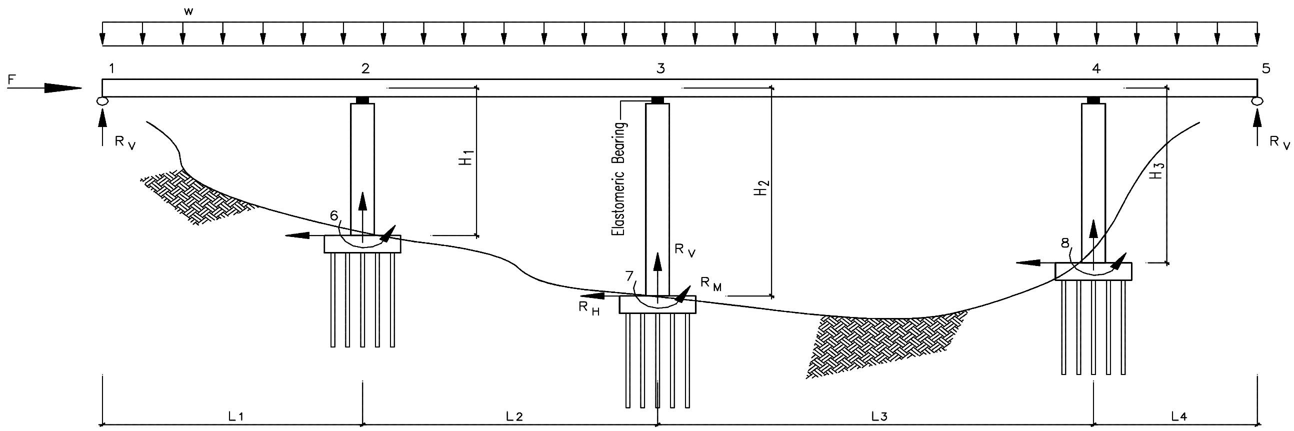
Elastomeric Bearing Bridge Analysis using Finite Element Method



INPUT DATA

DIMENSION

L_1 =	130	ft
L_2 =	260	ft
L_3 =	300	ft
L_4 =	100	ft
H_1 =	82	ft
H_2 =	106	ft
H_3 =	92	ft

Section		E (ksi)	A (in^2)	I_x (in^4, in plane)
Girder	1 - 2	4030.509	4307.04	7488499
Girder	2 - 3	4030.509	4307.04	7488499
Girder	3 - 4	4030.509	4307.04	7488499
Girder	4 - 5	4030.509	4307.04	7488499
Column	6 - 2	3604.997	8640	4608000
Column	7 - 3	3604.997	8640	4608000
Column	8 - 4	3604.997	8640	4608000

LOADS

w =	45	kips / ft, (including bridge weight and impact factor.)
F =	2133	kips, (seismic force.)

ANALYSIS & DESIGN SUMMARY

Reactions

Joints	R_H (kips)	R_V (kips)	R_M (ft-kips)
1	0	1595.48	0
5	0	-416.32	0
6	989.92	9477.35	81173.14
7	454.26	13458.06	48151.30
8	688.83	11435.42	63372.00

Design Section Forces

Section		Left/Bottom End			Middle	Right/Top End		
		P (kips, axial)	V (kips, shear)	M (ft-kips)	M (ft-kips)	P (kips, axial)	V (kips, shear)	M (ft-kips)
Girder	1 - 2	2133.00	1595.48	0.00	-8643.9	2133.00	4254.52	172837.26
Girder	2 - 3	1143.08	5222.84	172837.26	-125881.5	1143.08	6477.16	335899.69
Girder	3 - 4	688.83	6980.89	335899.69	-204984.4	688.83	6519.11	266631.56
Girder	4 - 5	0.00	4916.32	266631.56	77065.8	0.00	-416.32	0.00
Column	6 - 2	9477.35	989.92	81173.14	40586.6	9477.35	-989.92	0.00
Column	7 - 3	13458.06	454.26	48151.30	24075.6	13458.06	-454.26	0.00
Column	8 - 4	11435.42	688.83	63372.00	31686.0	11435.42	-688.83	0.00

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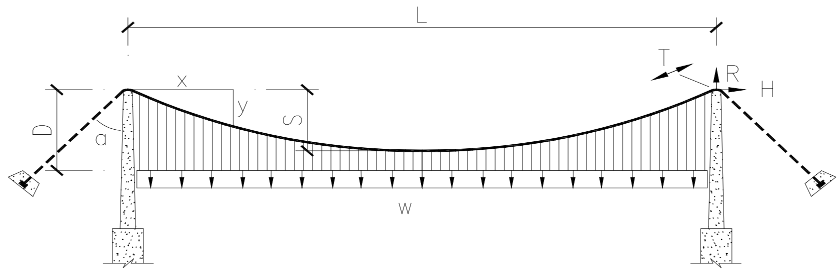
Cable Structure Design Based on ASCE 19-10 & AASHTO 17th

INPUT DATA



MAIN SPAN $L =$ 4500 ft, (1371.60 m)

CABLE MAX SAG $S =$ 450 ft, (137.16 m)

2-CABLE TOTAL LOAD ON STIFFENING DECK/GIRDER

$w =$ 13.5 kips/ft, (197 kN/m), ASD level.

ANGLE OF ANCHOR CABLE

$\alpha =$ 60 ^{o}

CABLE TENSILE STRENGTH $f_{su}{}^* = f_{pu} =$ 270 ksi, (1862 MPa)

SINGLE CABLE EFFECTIVE/NET AREA $A =$ 800 in^2 (516128 mm^2)

ANALYSIS & DESIGN SUMMARY

Check $S / L =$ 1 / 10.0 < 1 / 9

> 1 / 12 [Satisfactory]

Parabolic Main Cable $y = 4S/L^2 \times (L - x) =$ *8.88889E-05 x (4500 - x)* , control suspender ***(D - y)*** to keep parabolic curve.

Main Cable Curve Length = 4617.27 ft, (1407.34 m)

Maximum Main Cable Tension $T = [\ (0.5\ w\ L)^2 + (0.125\ w\ L^2/\ S\ )^2\ ]^{0.5} =$ 81787 kips, (363789 kN), ASD level.

< $2\ A\ f_{pu}\ /\ \Omega_t =$ 103473.1 kips, (460248 kN) [Satisfactory]

where $\Omega_t =$ 1.67 x 2.5 = 4.175 , *AISC 360-10 & Reference page 907.*

Reactions on Tower Top $R = 0.5\ w\ L + T\ \cos\ \alpha =$ 71268.6 kips, (317003 kN), ASD level.

$H = 0.125\ w\ L^2/\ S - T\ \sin\ \alpha =$ 5107.7 kips, (22719 kN), ASD level.

Reference: Engineers' Handbook for Bridge Design, China Communications Press, July 2007.

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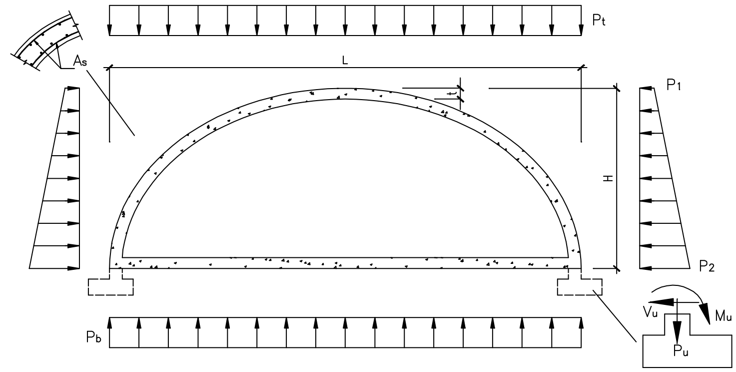
Wildlife Crossing Design Based on AASHTO-17th & ACI 318-14



DESIGN CRITERIA

1. The wildlife crossing can be overpass or underpass, and the structure can be buried bridge or shotcrete tunnel.
2. The wildlife crossing foundation should be RC slab on ground. But the software gives another footing design option, <u>by wall bottom P_u, M_u and V_u</u>, without RC slab.

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c' =	5	*ksi, (34 MPa)*
REBAR YIELD STRESS	f_y =	60	*ksi, (414 MPa)*

DIMENSIONS	L =	40	*ft, (12.19 m)*
	H =	15	*ft, (4.57 m)*
	t =	20	*in, (508 mm)*

REINFORCING (A_s)

2 layers # 8 @ 8 *in o.c., (203 mm), (curvered)*

Concrete Cover = 3 *in, (76 mm), (AASHTO 8.22.1)*

	P_u =	23.42	kip / ft
Footing Option	**M_u** = +/-	82.18	ft-kip / ft
	V_u =	2.22	kip / ft

FACTORED SOIL PRESSURE (SD level, including surcharge & seismic ground shaking)

P_t = 1232 *psf, (59 kPa)* $P_b = P_t$ + Wt = 1935 *psf, (93 kPa)*

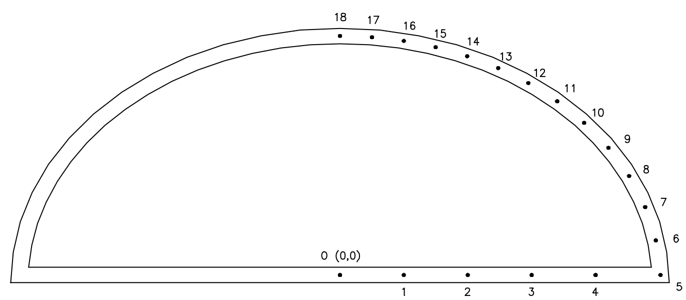
P_1 = 720 *ft-pcf, (psf), (34 kPa)* P_2 = 1820 *ft-pcf, (psf), (87 kPa)*

[THE DESIGN IS ADEQUATE.]

ANALYSIS

DETERMINE SECTION FORCES BY FINITE ELEMENT METHOD

Point	**X** (ft)	**Y** (ft)	**P_u** (k)	**M_u** (ft-k)	**V_u** (k)
0	0.00	0.00	1.61	15.03	0.00
1	3.83	0.00	1.61	52.45	-2.34
2	7.67	0.00	1.61	61.44	5.07
3	11.50	0.00	1.61	41.99	12.49
4	15.33	0.00	1.61	-5.88	19.91
5	19.17	0.00	23.42	-82.18	-2.22
6	19.03	1.61	23.06	-78.61	-3.48
7	18.61	3.19	22.56	-72.91	-4.69
8	17.92	4.73	21.90	-65.02	-5.72
9	16.97	6.20	21.10	-55.02	-6.47
10	15.77	7.57	20.20	-43.22	-6.87
11	14.35	8.84	19.25	-30.10	-6.91
12	12.71	9.98	18.31	-16.32	-6.58
13	10.89	10.97	17.41	-2.67	-5.91
14	8.91	11.81	16.63	10.02	-4.93
15	6.80	12.47	16.00	20.92	-3.71
16	4.59	12.95	15.56	29.30	-2.30
17	2.31	13.24	15.33	34.58	-0.78
18	0.00	13.33	15.33	36.38	0.00

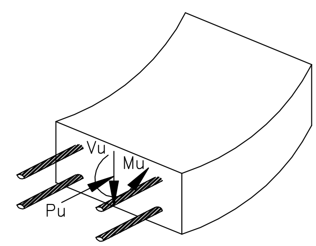


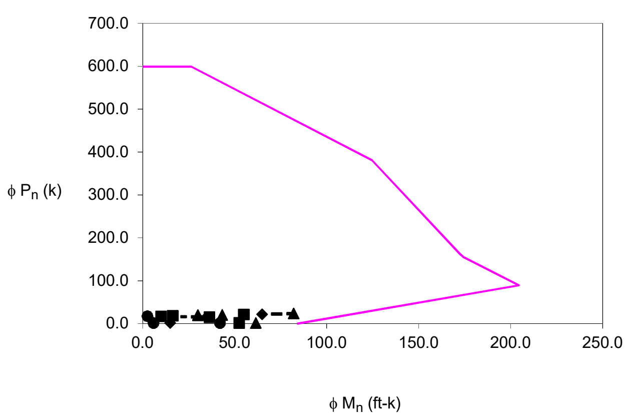
CHECK AXIAL & FLEXURE CAPACITY

ρ_{ProvD} = 0.005985 **<** ρ_{MAX} = 0.0400 (tension face only, ACI 318-14 7.3.3 or R21.2.2)

> ρ_{MIN} = 0.0008 (tension face only, ACI 318-14 9.6.1.2, 9.6.1.3 or Table 11.6.1)

[Satisfactory]





	ϕ P_n	**ϕ M_n**
AT AXIAL LOAD ONLY	599.1	0.0
AT MAXIMUM LOAD	599.1	26.5
AT MIDDLE	380.5	124.7
AT ε_t = 0.002	161.9	172.5
AT BALANCED	155.8	174.1
AT ε_t = 0.005	89.5	204.4
AT FLEXURE ONLY	0.0	84.3

(Note: For middle reforcing the max ϕM_n is at c equal to 0.5 t / β_1, not at balanced condition.)

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-14 9.6.3.1)

$\phi V_n = 2\phi b d\sqrt{f_c'}$ = 21.00 kips / ft **>** $V_{u,\,max}$ = 19.91 kips / ft **[Satisfactory]**

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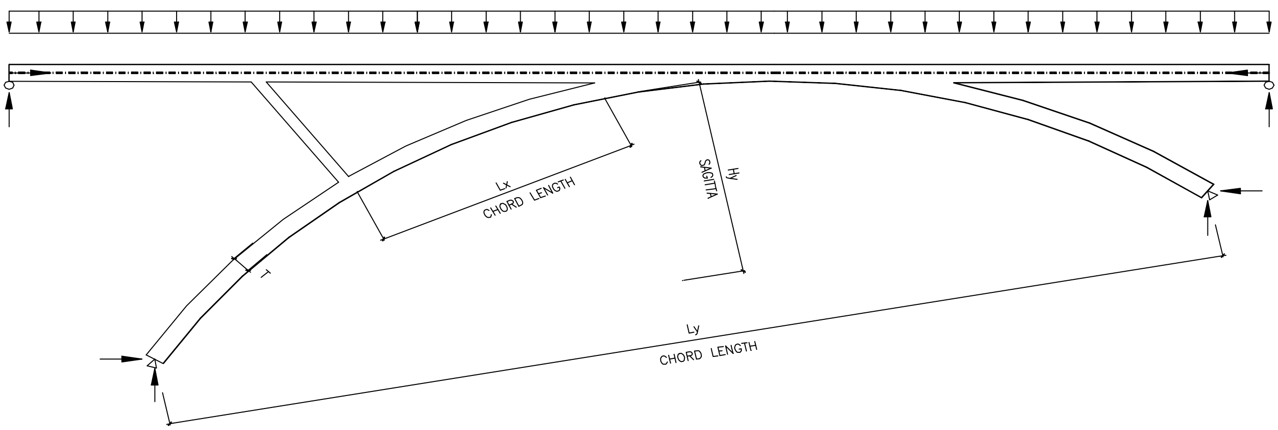
Arch Bridge Limits Analysis Based on ACI 318-14, AISC 360-16 & AASHTO-17th

DESIGN CRITERIA

1. The curved arch is compression concrete member, which should have stability limits for both in-plane direction and out-plane direction.
2. For compression concrete member, the ACI 318-14 & AASHTO-17th do not have slenderness ratio limits. But for solid compression steel member, the AISC 360-16 E2 limits the effective slenderness ratio not exceed 200. Since concrete strength less than steel and concrete cracking, the effective slenderness ratio for compression concrete member should be less than 200. *(80 adequate ?)*
3. This is basic structural buckling concept, no matter if ACI 318-14 ***6.6*** applied, or not, by any FEM analysis.



INPUT DATA & DESIGN SUMMARY

ARCH THICKNESS	T =	**5**	*ft, (1.52 m)*
ARCH WIDTH	W =	**26**	*ft, (7.92 m)*
OUT-PLANE CHORD LENGTH	L_y =	**750**	*ft, (228.60 m)*
OUT-PLANE SAGITTA	H_y =	**98**	*ft, (29.87 m)*
IN-PLANE CHORD LENGTH	L_x =	**250**	*ft, (76.20 m)*
EFFECTIVE LENGTH FACTOR	k =	**0.9**	*, ACI 318-14 Fig. R6.2.5. (If not sure, Input 1.0)*
ALLOWABLE EFFECTIVE SLENDERNESS RATIO	$(kL/r)_{allowable}$ =	**100**	*, (has to be less than 200, since 200 is for solid compression steel.)*

THE DESIGN IS INADEQUATE, SEE ANALYSIS BELOW

ANALYSIS

CHECK OUT-PLANE SLENDERNESS RATIO

$k\,L_{out} / r = 12^{0.5}\, k\,L_{out} / W =$ 93.97 < 100 **[Satisfactory]**

Where $L_{out} = 4\,R\,ATAN(2\,H_y / L_y) =$ 783.69 *ft, (238.87 m), Out-Plane Curved Lateral Unbraced Length*

$R = [(0.5\,L_y)^2 + (H_y)^2] / (2\,H_y) =$ 766.47 *ft, (233.62 m), Arch Radius*

CHECK IN-PLANE SLENDERNESS RATIO

$k\,L_{in} / r = 12^{0.5}\, k\,L_{in} / T =$ 156.58 > 100 **[Unsatisfactory]**

Where $L_{in} = 2\,R\,ASIN(0.5\,L_x / R) =$ 251.12 *ft, (76.54 m), In-Plane Curved Lateral Unbraced Length*

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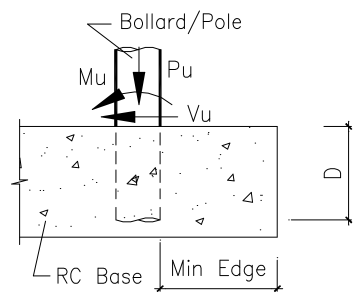
Bollard/Flagpole Anchorage Design Based on ACI 318-14

INPUT DATA & DESIGN SUMMARY

BOLLAED/POLE OUTSIDE DIAMETER	d =	7	in, (178 mm)
CONCRETE STRENGTH	f_c' =	3	ksi, (21 MPa)
FACTORED SHEAR LOAD	V_u =	25	kips, (111.2 kN)
FACTORED MOMENT	M_u =	75	ft-kips, (102 kN-m)
FACTORED VERTICAL LOAD (negative for uplift)	P_u =	120	kips, (533.8 kN)
EMBEDMENT DEPTH	D =	18	in, (457 mm)



THE ANCHORAGE DESIGN IS ADEQUATE.

(A_{vf} = **0.0** in^2 , Required Area of Shear Studs or Welded Reinforcement)

ANALYSIS

CHECK VERTICAL CAPACITY

ϕP_n = End Bering + Friction = 246.3 kips > P_u = 120 kips [Satisfactory]

where End Bering = 0.65 (2) 0.85 f_c' A = 127.6 kips, (ACI 318 14.5.1.1)

Friction = 0.75 MAX(0.2f_c' A_c , 800 A_c) = 118.8 kips, (ACI 318 22.9.4)

A = 38 in^2, end bearing area

$A_c = 0.5\ \pi$ d D = 198 in^2, (0.5 for concrete cracked)

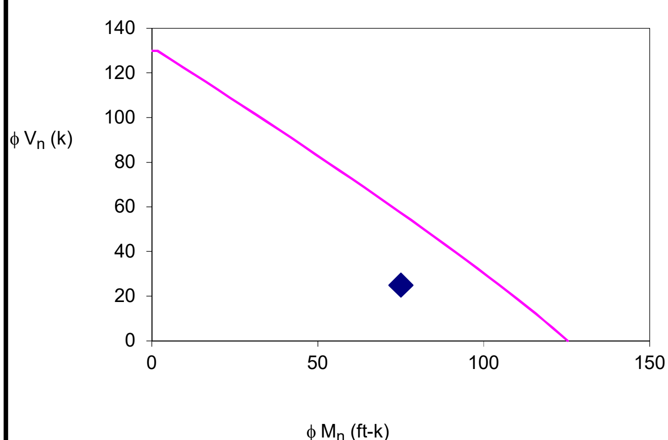
$A_{vf} = P_{u,Friction} / (\phi\ f_y\ \mu)$ = 0.0 in^2 , Required Area of Shear Studs or Welded Reinforcement

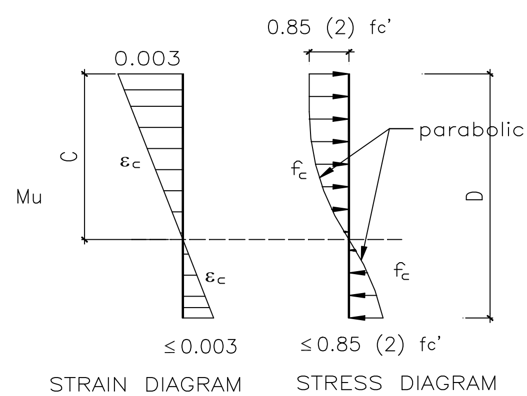
where ϕ = 0.75 , (ACI 318 21.2)

μ = 0.70 , (ACI 318 22.9.4.2)

f_y = 60 ks

CHECK BASE FLEXURAL & SHEAR CAPACITY (ACI 318 21 & 22)





$$\varepsilon_O = \frac{2f'_c}{E_c} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right)\ ,\ E_c = 57\sqrt{f'_c}$$

$$f_c = \begin{cases} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_O}\right) - \left(\frac{\varepsilon_c}{\varepsilon_O}\right)^2\right]\ ,\ for\ \ 0 < \varepsilon_c < \varepsilon_O \\ 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) f'_c\ ,\ for\ \ \varepsilon_c \ge \varepsilon_O \end{cases}$$

(Edge of concrete base must be wider than "D")

ϕM_n = 105 ft-kips @ V_u = 25 kips

> M_u = 75 ft-kips [Satisfactory]

$\phi V_{n,max}$ = 129.89 kips, when C = 12.4 in

> V_u = 25 ft-kips [Satisfactory]

where ϕ = 0.65 , (ACI 318 21.2)

Bearing factor = 2, (ACI 318 14.5.1.1)

b = effective bearing width = 100% d = 7.00 in

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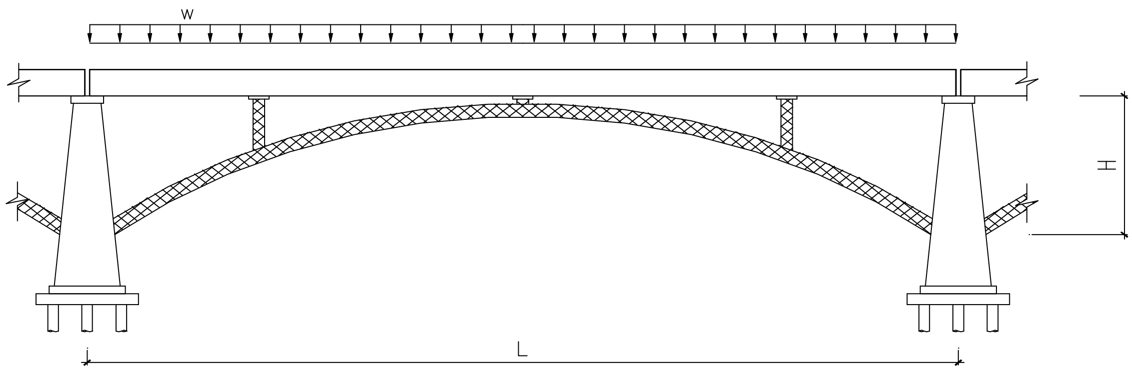
Bridge Design and Repair, by Added New Arch, using Finite Element Method

DESIGN CRITERIA

1. This enhancing method is to reduce existing bridge bending moments by added a new arch structure. The new arch structure does not have to be supported at the column bottom, and the horizontal reactions can be balanced for multi-span bridge, if the passing space under bridge limits.
2. The existing pinned pinned bridge can be steel beam, prestressed concrete girder, or box section member. If the bridge still in service during repairing, two load cases should be checked: one is the service live load added in w_E before arch, another only in w_N.

INPUT DATA



DIMENSION

L = 130 *ft, (39.62 m)*

H = 40 *ft, (12.19 m)*

Section	E (ksi)	A (in^2)	I_x (in^4, in plane)	*E (MPa)*	*A (cm^2)*	*I_x (cm^4, in plane)*
Existing Beam	3605	3680	3584948	*24856*	*23742*	*149216781*
New Arch	4031	1200	225000	*27793*	*7742*	*9365207*
New Column	29000	778	0	*199948*	*5019*	*0*

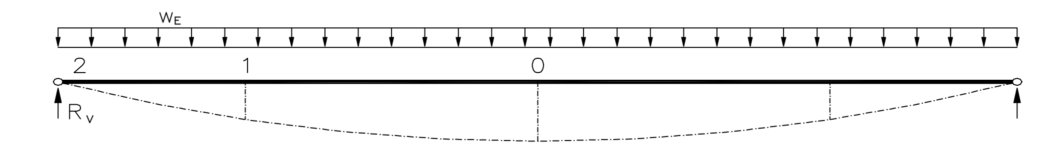
EXISTING BRIDGE LOAD BEFORE ARCH

w_E = 35 *kips/ft, (510.4 kN/m), (including existing bridge weight and impact factor.)*

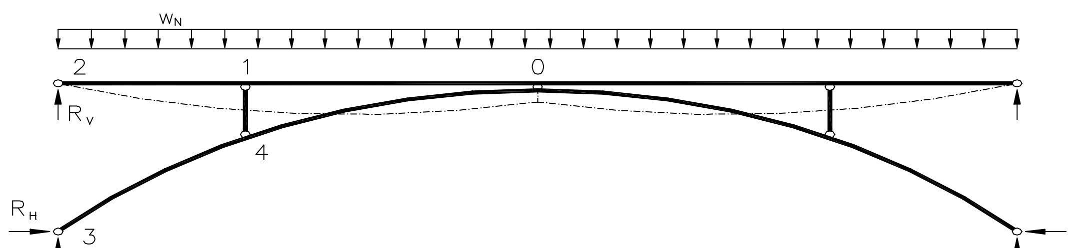
TOTAL BRIDGE LOAD AFTER ARCH

w_N = 55 *kips/ft, (802.1 kN/m), (including all bridge weight and impact factor.)*

ANALYSIS & DESIGN SUMMARY



Existing Bridge Section Moment Before Arch



New Parabolic Arch Reduced Moment

Existing Bridge, before Arch Structure, for load w_E

Element	Design Section Forces		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)
0 - 1	0	73937.50	1137.5
1 - 2	0	55453.13	2275

Existing Bridge, before Arch Structure, for load w_E

Joint	Reaction & Displacements			
	R_H (kips)	R_V (kips)	Δ_x (in)	Δ_y (in)
0			0	17.403
1			0.000	12.400
2	0	2275	0.000	0.000

Enhanced Bridge, after Arch Structure, for load w_N

Element	Design Section Forces by FEM		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)
0 - 1	0.000	334.675	17.249
1 - 2	0.000	334.675	10.298
0 - 4	3012.164	669.350	26.329
1 - 4	1759.953	0.000	0.000
3 - 4	3921.436	669.350	15.134

Enhanced Bridge, after Arch Structure, for load w_N

Joint	Reactions & Displacements by FEM			
	R_H (kips)	R_V (kips)	Δ_x (in)	Δ_y (in)
0			0	1.379
1			0.000	0.704
2	0	904.0477	0.000	0
3	2871.22	2670.952	0	0
4			-0.055	0.694

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Curved Steel HSS (Tube, Pipe) Member Design Based on AISC 360-16

INPUT DATA & DESIGN SUMMARY

MEMBER SHAPE (Tube or Pipe) & SIZE **HSS20X12X5/8** < == **Tube**

STEEL YIELD STRESS F_y = **46** ksi, (317 MPa)

AXIAL COMPRESSION FORCE P_r = **30** kips, (133 kN), ASD

STRONG AXIS EFFECTIVE CURVE LENGTH kL_x = **120** ft, (36.58 m)

WEAK AXIS EFFECTIVE CURVE LENGTH kL_y = **26** ft, (7.92 m)

A	r_x	r_y	S_x
35.0	7.3	4.9	188.0
S_y	I_x	I_y	Z_x
142.0	1880.0	851.0	230.0
Z_y	b	h	t
162.0	20.0	12.0	0.6

STRONG AXIS BENDING MOMENT M_{rx} = **220** ft-kips, (298 kN-m), ASD

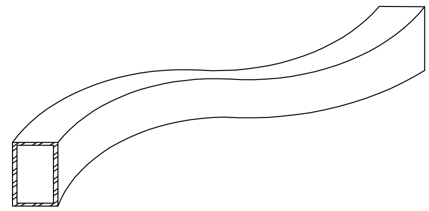
STRONG AXIS BENDING UNBRACED CURVE LENGTH

L_b = **50** ft, (15.24 m), (AISC 360 F2.2.c)

STRONG DIRECTION SHEAR LOAD, ASD V_{strong} = **13** kips, (58 kN)

WEAK AXIS BENDING MOMENT M_{ry} = **30** ft-kips, (41 kN-m), ASD

WEAK DIRECTION SHEAR LOAD, ASD V_{weak} = **10** kips, (44 kN)



TORSIONAL FORCE T_r = **63** ft-kips, (85 kN-m), ASD

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360 H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for \ \frac{P_r}{P_c}\ge 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for \ \frac{P_r}{P_c}< 0.2 \end{cases} = 0.66 < 1.3$$

[Satisfactory]
(2018 IBC, 1605.3.2)

Where $P_c = P_n / \Omega_c$ = 228 / 1.67 = 136.27 kips, (AISC 360 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 881.67 / 1.67 = 527.94 ft-kips, (AISC 360 Chapter F)
> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 621.00 / 1.67 = 371.86 ft-kips, (AISC 360 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK TORSIONAL CAPACITY (AISC 360 H3.1)

$$T_c=\frac{1}{\Omega_T}T_n=\frac{1}{\Omega_T}\begin{cases} \left[2(B-t)(H-t)t-4.5(4-\pi)t^3\right]\begin{bmatrix} 0.6F_y \ , & for \ \frac{h}{t}\le 2.45\sqrt{\frac{E}{F_y}} \\ 0.6F_y 2.45\sqrt{\frac{E}{F_y}}\frac{t}{h} \ , & for \ \frac{h}{t}\le 3.07\sqrt{\frac{E}{F_y}} \\ 0.458\frac{E\pi^2}{(h/t)^2} , & for \ \frac{h}{t}\le 260 \end{bmatrix} , & for \ HSS \ Tube \\ \frac{\pi(D-t)^2 t}{2} Max\left[\frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{(5/4)}} , \frac{0.60E}{\left(\frac{D}{t}\right)^{(3/2)}}\right] , & for \ HSS \ Pipe \end{cases} = 378.1 \text{ ft-kips}$$

> T_r **[Satisfactory]**

Where	B	H	h	t	D	E	Ω_T =	1.67	,ASD
	12.00	20.00	18.13	0.63		29000			

CHECK SHEAR CAPACITY (AISC 360 G2)

$V_{n,strong} / \Omega_v$ = 460.0 / 1.67 = 275.4 kips > V_{strong} = 13.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 276.0 / 1.67 = 165.3 kips > V_{weak} = 10.0 kips **[Satisfactory]**

CHECK COMBINED TORSION, SHEAR, COMPRESSION, AND BENDING CAPACITY (AISC 360 H3.2)

$$\begin{cases} \frac{P_r}{P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)+\left[Max\left(\frac{V_{strong}}{V_{c,strong}} , \frac{V_{weak}}{V_{c,weak}}\right)+\frac{T_r}{T_c}\right]^2 , & for \ \frac{T_r}{T_c}>0.2 \\ Torsion \ \ Neglected , & for \ \frac{T_r}{T_c}\le 0.2 \end{cases} = 0.0 < 1.3$$

[Satisfactory]
(2018 IBC, 1605.3.2)

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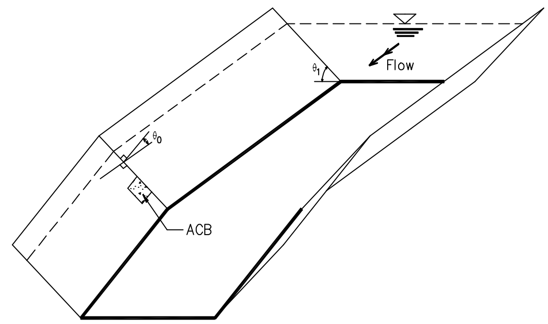
Articulating Concrete Block (ACB) Design Based on NCMA ACB Manual 2nd Edition

DESIGN CRITERIA

1. An ACB system consists of a flexible interlocking matrix of concrete blocks that are commonly used to provide protection against erosion from flowing water.
2. This ACB critical analysis location is at the changed channel bed slope.
3. Tested bed slope not less than the site case, and tested maximum velocity not less than design input value.



INPUT DATA & DESIGN SUMMARY

INFORMATION FROM MANUFACTURER

W = **66** lbs, weight of single block.

S_c = **2.1** , (specific gravity of concrete, assume 2.1)

ΔZ = **0.04** ft, (height of block protrusion above ACB matrix.)

b = **1.25** ft, (block width.)

l_1 = **0.21** ft, (Manual Fig 4.4) , l_2 = **0.88** ft

THE DESIGN IS INADEQUATE, SEE ANALYSIS BELOW

l_3 = **0.33** ft, (Manual Fig 4.4) , l_4 = **0.88** ft

τ_c = **25** lbs/ft^2, (critical shear stress for block on a horizontal surface.)

HYDRAULIC & SITE DESIGN DATA

γ = **62.4** pcf, (the weight of a cubic foot of water.)

R = **4.3** ft, (hydraulic radius.)

V_{avg} = **8.1** ft/s, (cross-section-averaged velocity.)

V_{des} = **11** ft/s, (the maximum velocity expected at the bend.)

ρ = **1.94** slugs/ft^3, (mass density of water.)

θ_1 = **26.57** deg, (channel side slope.)

θ_0 = **0.57** deg, (channel bed slope.)

S_f = **0.007** ft/ft, (energy grade line or bed slope.)

ANALYSIS

DETERMINE SUBMERGED UNIT WEIGHT

$$W_S = W\left(\frac{S_C - 1}{S_C}\right) = \quad 34.57 \text{ lbs}$$

DETERMINE STABILITY NUMBER ON A HORIZONTAL SURFACE

$$\eta_0 = \frac{\tau_{des}}{\tau_c} = \frac{\gamma R S_f \left(\frac{V_{des}}{V_{avg}}\right)^2}{\tau_c} = \quad 0.14$$

DETERMINE ADDITIONAL LIFT AND DRAG FORCES FROM BLOCK PROTRUSION OUT OF THE ACB MATRIX

$$F'_L = F'_D = 0.5(\Delta Z)\, b \rho V_{des}^2 = \quad 5.87 \text{ lbs}$$

DETERMINE ANGLES

$$a_0 = \sqrt{\cos^2\theta_1 - \sin^2\theta_0} = \quad 0.8943$$
, (projection of W_S into subgrade beneath block.)

$$\theta = \arctan\left(\frac{\tan\theta_0}{\tan\theta_1}\right) = \quad 1.1396$$
deg, (angle between side slope projection of W_S and the vertical.)

$$\beta = \arctan\left(\frac{\cos(\theta_0 + \theta)}{\left(\frac{l_4}{l_3} + 1\right)\frac{\sqrt{1 - a_0^2}}{\eta_0\left(\frac{l_2}{l_1}\right)} + \sin(\theta_0 + \theta)}\right) = \quad 19.2941$$
deg, (angle of block projection from downward direction, once in motion.)

$$\delta = 90 - \beta - \theta = \quad 69.5663$$
deg, (angle between drag force and block motion.)

DETERMINE STABILITY NUMBER ON A SLOPED SURFACE

$$\eta_1 = \eta_0\left(\frac{\frac{l_4}{l_3} + \sin(\theta_0 + \theta + \beta)}{\frac{l_4}{l_3} + 1}\right) = \quad 0.1143$$
, (projection of W_S into subgrade beneath block.)

CHECK ACTUAL FACTOR OF SAFETY

$$SF = \frac{\frac{l_2}{l_1} a_0}{\sqrt{1 - a_0^2}\cos\beta + \frac{l_2}{l_1}\eta_1 + \frac{l_3 F_D \cos\delta + l_4 F_L}{l_1 W_S}} = \quad 2.1971 \quad < \quad 2.4$$

[Unsatisfactory]

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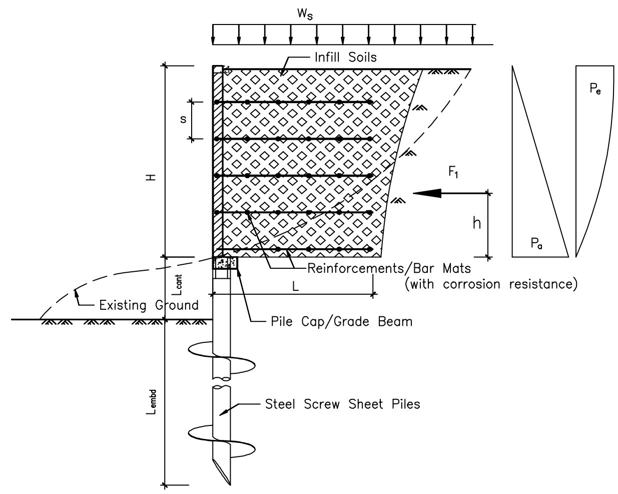
Hybrid Retaining Structural Design Based on 2018 IBC/AASHTO, TMS 402-16 & AISC 360-16

DESIGN CRITERIA

1. Hybrid retaining structure (MSE Wall & Steel Screw Sheet Piles) are typically used in the combined cut and fill soil area, where MSE wall at top filled area, and steel screw sheet piles at bottom with less disturbed soil & quick installation.
2. This is capacity design, not included environmental corrosion parameters.



INPUT DATA (MSE Wall) & DESIGN SUMMARY

WALL HEIGHT	H =	18	ft, (5.49 m)
WALL THICKNESS	t =	12	in, (305 mm)
MASONRY STRENGTH	f_m' =	1.5	ksi, (10 MPa)

(modular concrete facing blocks - MBW)

WALL VERT. REBAR (fully grouted) f_y = 60 ksi, (414 MPa)

2 # 5 @ 16 in o.c. at each face (406 mm o.c. at each face)

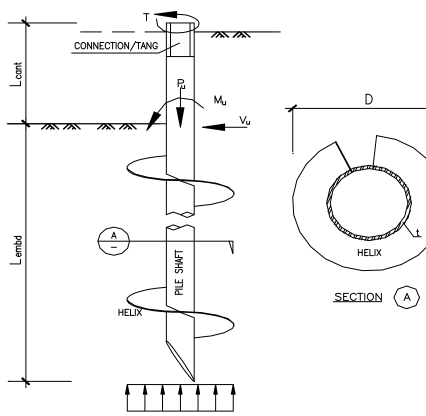
HORIZONTAL BAR MATS L = 18 ft, (5.49 m)

\# 4 @ 16 in, (406 mm), o.c., horiz. E. W.

s = 24 in, (610 mm), o.c., vertical mat dist.

SURCHARGE WEIGHT	w_s =	500	psf, (24 kPa)
SOIL SPECIFIC WEIGHT	γ_b =	110	pcf, (1762 kg/m^3)
SOIL INTERNAL FRICTION ANGLE	ϕ =	29	deg
HORIZONTAL ACCELERATION (in g)	k_h =	0.35	g, (0.5 S_{DS})
ALLOW SOIL PRESSURE	Q_a =	4.2	ksf, (201 kPa)

THE TOP MSE-WALL DESIGN IS ADEQUATE.



INPUT DATA (Steel Screw Sheet Piles) & DESIGN SUMMARY

DIMENTION	L_{cant} =	5	ft, (1.52 m)
	L_{embd} =	50	ft, (15.24 m)
	D =	48	in, (1219 mm)
PILE WEB THICKNESS	t =	0.75	in, (19 mm), zero for no tube.
	ϕ =	12	in, (305 mm)
STEEL YIELD STRESS	F_y =	50	ksi, (345 MPa)

TOP INSTALLATION TORSION *(ASD level)* T = 300 ft-kips, (407 kN-m)

ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT

P_p = 0.25 ksf / ft, (39.2719 [kN/m^2] / m)

PILE SPACING S = 4 ft, O.C., (1.22 m, O.C.)

THE STEEL PILE DESIGN IS ADEQUATE.

ANALYSIS (MSE Wall)

DETERMINE ACTIVE EARTH PRESSURE

$P_e = 0.75\, k_h\, \gamma_b$ = 29 psf / ft, (FEMA P-750 Page 356)

$P_a = \gamma_b\, K_a$ = 38 psf / ft

where $K_a = \dfrac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2}$ = 0.347

(Coulomb, AASHTO Figure 5.5.2A)

β = 0 deg, slope of backfill

δ = 0 deg, external friction angle

θ = 90 deg, rack angle of wall face

$F_1 = 0.5\, H^2\, (P_a + P_e)$ = 11 kips / ft, *Total*

$h = (1/3\, P_a + 0.6\, P_e)\, H / (P_a + P_e)$ =

CHECK SOIL BEARING CAPACITY (AASHTO Figure 5.8.3A)

$V_1 = H\, L\, \gamma_b$ = 35.640 kips / ft, *Vertical*

$F_2 = K_a\, H\, w_s$ = 3.123 kips / ft, *from surcharge weight*

$e = (F_1\, h + F_2\, 0.5\, H) / (V_1 + L\, w_s)$ = 0.630 ft

$\sigma_v = (V_1 + L\, w_s) / (L - 2e)$ = 2.67 ksf < Q_a **[Satisfactory]**

CHECK SLIDING CAPACITY (2018 IBC 1807.2.3)

$1.1\, (F_1 + F_2)$ = 15.382 kips / ft < $\tan(\phi)\, (V_1 + L\, w_s)$ = 24.74 kips / ft **[Satisfactory]**

$1.5\, (F_1 + F_2 - 0.5\, H^2\, P_e)$ = 13.959 kips / ft < $\tan(\phi)\, (V_1 + L\, w_s)$ = 24.74 kips / ft **[Satisfactory]**

CHECK OVERTURNING CAPACITY (2018 IBC 1807.2.3)

$1.5\, (h\, F_1 + 0.5\, H\, F_2)$ = 42.157 ft-kips / ft < $0.5\, L\, (V_1 + L\, w_s)$ = 401.76 ft-kips / ft **[Satisfactory]**

(All forces with safety factor 1.5 conservatively.)

CHECK FLEXURE CAPACITY OF MASONRY WALL (TMS 402 8.3.3)

$$M_{allowable} = MIN\left[\frac{1}{2} b_w kd\, F_b\left(d-\frac{kd}{3}\right) - P\left(d-\frac{t_e}{2}\right)\ ,\ A_s F_s\left(d-\frac{kd}{3}\right) + P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right] =$$

5.219 ft-kips

\> $s^2\, (F_1 + F_2) / H$ = 3.107465 ft-kips

[Satisfactory]

(cont'd)

where	t_e	=	11.625 in				
	d	=	9.3125 in	E_m	=	1050	ksi, 700 f_m' conservative value
	b_w	=	12 in	E_s	=	29000	ksi
	F_b	=	0.495 ksi	n	=	27.619	
	F_s	=	32.000 ksi	k	=	0.286	
	A_s	=	0.233 in^2	P	=	1.170	kips, axial force at wall middle
	ρ	=	0.002 > 0.0007	**[Satisfactory]**			

CHECK HORIZONTAL BAR MATS (AASHTO 5.8.4.1)

$T_{max} = \sigma_h A_{trib} =$ 0.64 kips / bar < $F_s A_s =$ 6.40 kips / bar **[Satisfactory]**

where σ_h = $F_2 / H + P_a + P_e =$ 0.24

A_{trib} = 384 in^2

CHECK HORIZONTAL BAR ANCHORAGE IN MASONRY (TMS 402-16 6.1.9.1)

$L_{anchor} = 12\, d_b =$ 6 in < t_e **[Satisfactory]**

ANALYSIS (Steel Screw Sheet Piles)

DETERMINE MAX LOADS *(LRFD / SD level)*

$M_u =$ **325.825** *ft-kips, (442 kN-m)*

$P_u =$ **20.499** *kips, (91 kN)*

$V_u =$ **65.165** *kips, (290 kN)*

ALLOW SOIL PRESSURE IN BOTTOM END STRATUM $Q_a =$ **4.2** *ksf*

CHECK SOIL END BEARING CAPACITY (ACI 318-19 13.3.1.1)

$0.7\, P_u + t\, \pi\, \phi\, (\gamma_{pile} - \gamma_{soil})\, (L_{embd} + L_{cant}) =$ 18.89 *kips, (84 kN)*

< $0.25\, \pi\, D^2\, Q_a =$ 52.78 *kips, (235 kN)* **[Satisfactory]**

Where $\gamma_{pile} =$ 0.49 *kcf, one cubic foot of pile weight* $\gamma_{soil} =$ 0.07 *kcf, soil/water weight*

CHECK STEEL PILE EMBEDMENT LENGTH, L_{embd}, (IBC/CBC 1807.3)

By trials, use pole depth, $d = L_{embd} =$ 50.00 *ft, (15.24 m)*

Lateral bearing @ bottom, $S_3 = 2\, P_p\, Min(\, d\, ,\, 12'\,) =$ 6.00 *ksf, (287.28156* kN / m^2)

Lateral bearing @ $d / 3$, $S_1 = 2\, P_p\, Min(\, d / 3\, ,\, 12'\,) =$ 4.94 *ksf, (236.44487* kN / m^2)

Require Depth is given by

$$d = \frac{A}{2}\left[1 + \sqrt{1 + \frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad = \quad 29.58$$

ft, (9.02 m) < L_{embd} **[Satisfactory]**

Where $P =$ $0.7\, V_u =$ 45.61547 *kips, (203 kN)* $A = 2.34\, P / (\, \phi\, S_1\,) =$ 21.614971

$h =$ $M_u / V_u + L_{cont} =$ 10 *ft, (3.05 m)*

CHECK TORSIONAL CAPACITY (AISC 360 H3.1)

$$T_c = \frac{1}{\Omega_T} T_n = \frac{1}{\Omega_T} \frac{\pi(\phi - t)^2 t}{2} Max\left[\frac{1.23E}{\sqrt{\frac{L}{\phi}}\left(\frac{\phi}{t}\right)^{(5/4)}} \; , \; \frac{0.60E}{\left(\frac{\phi}{t}\right)^{(3/2)}}\right] = 2022.822$$

ft-kips, (2743 kN-m) > T **[Satisfactory]**

Where $\Omega_T =$ 1.67

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360 H1)

$$\begin{cases} \frac{P_u}{P_c} + \frac{8}{9}\left(\frac{M_u}{M_c}\right) \; , \; for \; \frac{P_u}{P_c} \geq 0.2 \\ \frac{P_u}{2P_c} + \left(\frac{M_u}{M_c}\right) \; , \; for \; \frac{P_u}{P_c} < 0.2 \end{cases} = 0.93$$

< 1.3 **[Satisfactory]**
(2018 IBC, 1605.3.2)

Where $P_c = P_n\, \phi_c =$ 876 x 0.9 = 788.37 kips, (AISC 360 Chapter E)

> P_u **[Satisfactory]** *, where* $kL =$ 0.5 L_{embd}

$M_c = M_n\, \phi_b =$ 396.09 x 0.9 = 356.48 ft-kips, (AISC 360 Chapter F)

> M_u **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360 G2)

$V_n\, \phi_v =$ 795.2 x 0.9 = 715.7 kips > V_u **[Satisfactory]**

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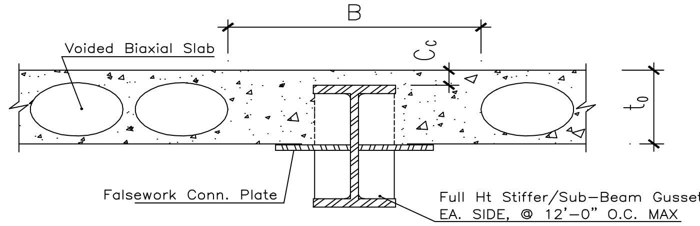
Super Composite Girder Design Based on 2019 CBC/2018 IBC, AISC 360-16 & ACI 318-19

DESIGN CRITERIA

1. Top steel girder in concrete can reduce floor system depth of composite steel-concrete.
2. Voided biaxial slab used to reduce weight and increase vibration stiffness of floor.
3. Top flange within concrete can drag heavy diaphragm force to lateral frames.
4. Flange tapered steel girder supported falsework of self concrete slab.
5. Late poured concrete band, minimum reinforcement, solid section hoops, & camber required (not shown).



THE DESIGN IS ADEQUATE.

INPUT DATA & DESIGN SUMMARY

FLANGE TAPERED STEEL GIRDER SIZE AT CHECKING SECTION

d (in)	t_w (in)	b_f (in)	t_f (in)	A (in^2)	I_x (in^4)
85	2.5	85	4	872.5	1211388
(2159 mm)	(64 mm)	(2159 mm)	(102 mm)	(5629 cm^2)	(50421763 cm^4)

STEEL YIELD STRESS F_y = **50** ksi, (345 MPa)

CONCRETE STRENGTH f_c' = **5** ksi, (34 MPa)

CONCRETE COVER C_c = **2.5** in, 0.75" min., (ACI 318-19 20.5.1) (64 mm) t_0 = **42** in, 7.25 Min. (106.7 cm)

SUPER GIRDER SPAN L = **120** ft, (36.58 m)

SOLID TRIBUTARY WIDTH B = **30** ft, (9.14 m)

SECTION AXIAL LOAD, *LRFD* P_u = **3755.3** kips, (16704 kN)

STRONG AXIS POSITIVE MOMENT, *LRFD* M_u = **112658** ft-kips, (152744 kN-m)

SHEAR LOAD, *LRFD* V_u = **3755.3** kips, (16704 kN)

ANALYSIS

DETERMINE CAMBER/SHORING ON NON-COMPOSITE

w = **45.156** kips / ft, floor system self weight, to steel girder, on non-composite

$\Delta = 5wL^4 / 384\,EI$ = 6.00 in, deflection of steel girder

< L / **180** = 8.00 in **[Satisfactory]**

Camber = **0.75** Δ = **4.50** in

CHECK FLEXURAL & AXIAL CAPACITY (AISC 360 I3, ACI 318-19 Chapter 21 & 22)

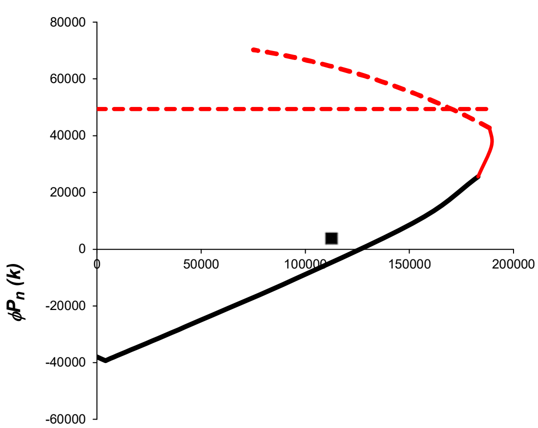
$$f_c=\begin{cases}\beta_e f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right)-\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] & , \ for \ \ 0<\varepsilon_c<\varepsilon_o\\ \beta_e f'_c & , \ for \ \ \varepsilon_c\ge\varepsilon_o\end{cases} \quad , \quad f_s=\begin{cases}\varepsilon_s E_s & , \ for \ \ \varepsilon_s\le\varepsilon_{ty}\\ f_y & , \ for \ \ \varepsilon_s>\varepsilon_{ty}\end{cases}$$

$$\varepsilon_o=\frac{2\left(\beta_e f'_c\right)}{E_c} \quad , \quad \varepsilon_{ty}=\frac{f_y}{E_s} \quad , \quad \varepsilon_t=\begin{cases}\varepsilon_{ty} & , \ for \ \ compression \ \ controlled\\ \varepsilon_{ty}+0.003 & , \ for \ \ tension \ \ controlled\end{cases}$$

$$E_c=57\sqrt{f'_c} \quad , \quad E_s=29000ksi \quad , \quad \beta_e=0.7 \quad , \quad \varepsilon_{cu}=0.003$$

Note:

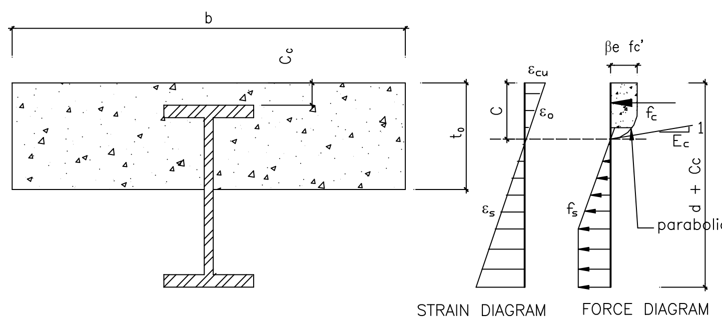
1. Between compression & tension controlled is ***Transition***, which the ϕ should be by linear interpolation. *(ACI 318-19 Tab. 21.2.2 & Fig. R10.4.2.1)*
2. ACI 318-19 changed ϕ *(Fig.R21.2.2b)* and keeps β_e *(22.2.2.4.1)* and ε_{cu} *(Fig. R21.2.2a)* the same. But AISC 360/341-16 has different ϕ (0.9 only) and β_e (1.0 or 0.7).



Solid Black Line - **Tension Controlled**

Solid Red Line - **Transition**

Dash Line - **Compression Controlled**



b = MIN (L / 4 , B) = 360 in, (AISC 360 I3.1a)

ϕ = **0.9** , (AISC 360 I3)

M_n @ P_u / ϕ = 131703.2 ft-kips

ϕM_n = 135895 ft-kips **>** $M_u + 0.5C_c P_u$ = 113049 ft-kips

[Satisfactory]

CHECK SHEAR CAPACITY (AISC 360 G2)

ϕV_n = 5737.5 kips **>** V_u = 3755.3 kips **[Satisfactory]**

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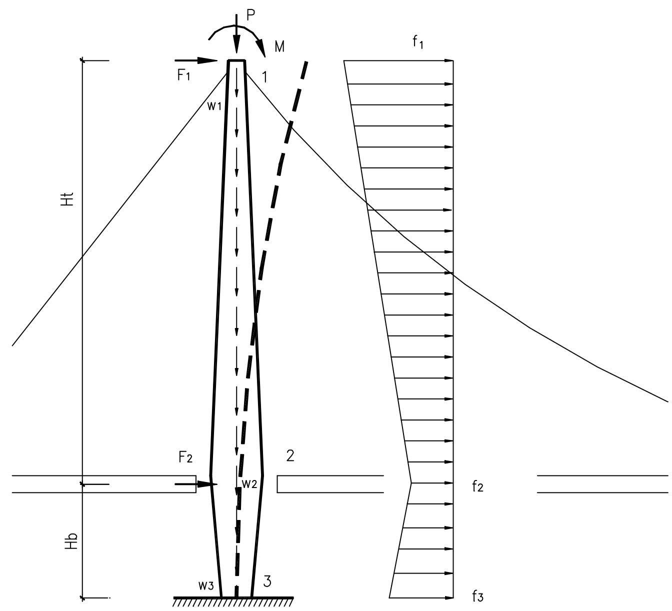
Tower Drift Analysis for Cable Stayed Bridge by Finite Element Method

DESIGN CRITERIA

1. The tower design of cable stayed bridge is always controlled by entire cantilever P-Delta effect, not individual element P-Delta effects, no matter the tower is steel truss, reinforced concrete, or super composite section.
2. The analysis is 2D finite element method for bridge direction plane. The inpu values may be based on single pole or two combined piles.
3. The section changed cantilever bending drifts are based on 1 / ρ = M / (EI), so the magnified moment calculation (ACI 318-19 6.6.4) and any assumed the maximum drift, Δ, with $\delta = \Delta\,(h / H)^{1.9}$ not apply.



INPUT DATA & SUMMARY

DIMENSIONS H_t = **1100** ft, (335.28 m)

H_b = **270** ft, (82.30 m)

MATERIAL & THE MOMENT OF INERTIA

E = **10196.47** *ksi, <== 70320 N / mm²)*

Section	*I*, in^4	*I*, cm^4
1	**14276237390**	594221864104
2	**38253756079**	1592241542721
3	**22313025278**	928738232100

*Note: For truss tower, the equivalent **A** and **I** values may be from (k_{11}-$k_{12}k_{22}^{-1}k_{21}$) by DJ Engineers & Builders Inc.*

LOADS

P = **42800** *kips, (190384 kN)*
M = **290** *ft-kips, (393 kN-m)*
F_1 = **1070** *kips, (4760 kN)*
F_2 = **535** *kips, (2380 kN)*

UNIFORMLY DISTRIBUTED VERTICAL LOADS

w_1 = **230.05** *kips / ft, (3355 kN / m)*
w_2 = **483.6** *kips / ft, (7053 kN / m)*
w_3 = **416.44** *kips / ft, (6073 kN / m)*

UNIFORMLY DISTRIBUTED LATERAL LOADS

f_1 = **34.5072** *kips / ft, (503 kN / m)*
f_2 = **72.5407** *kips / ft, (1058 kN / m)*
f_3 = **62.4656** *kips / ft, (911 kN / m)*

Design Section Forces & Reactions

Section	***M***, (bending)		***P***, kips, (axial)		***V***, kips, (shear)	
	ft-kips	kN-m	kips	kN	kips	kN
2	33973352	46061680	435309	1936352	60481	269035
3	54553521	73964642	556815	2476837	78707	350107

***Δ* = 137 in, (348 cm), the max drift at top.**

THE DEFLECTION IS ADEQUATE.

ANALYSIS

Section			**No P-Δ**	**P-Δ Effect with f, w, P, F, & M** 1 st	**2 nd**	**3 rd**	**4 th**	**5 th**	**...**	**Final**
1	P	kips	42800	42800	42800	42800	42800	42800		42800
	M	ft-kips	290	290	290	290	290	290		290
	V	kips	1070	1070	1070	1070	1070	1070		1070
	Δ	in	118.8	134.5	136.5	136.8	136.8	136.9		136.9
2	P	kips	435309	435309	435309	435309	435309	435309		435309
	M	ft-kips	29400821	33366692	33892784	33962651	33971931	33973163		33973352
	V	kips	60481	60481	60481	60481	60481	60481		60481
	Δ	in	9.5	10.6	10.8	10.8	10.8	10.8		10.8
3	P	kips	556815	556815	556815	556815	556815	556815		556815
	M	ft-kips	48203259	53716037	54442346	54538754	54551559	54553260		54553521
	V	kips	78707	78707	78707	78707	78707	78707		78707
	Δ	in	0	0	0	0	0	0		0

CHECK DEFLECTION (TOP DRIFT)

Δ_{max} = 136.86 *in* < 2 H / ***240*** = 137.00 in

[Satisfactory]

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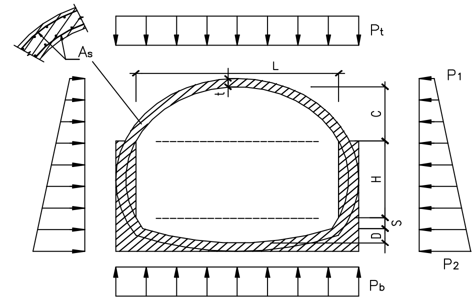
Masonry Vault Design Based on 2018 IBC & TMS 402-16

INPUT DATA & DESIGN SUMMARY



SPECIAL INSPECTION (0=NO, 1=YES)		1	*Yes*
TYPE OF MASONRY (1=CMU, 2=BRICK)		1	*CMU*
MASONRY STRENGTH	f_m' =	1.5	*ksi, (10 MPa)*
REBAR YIELD STRESS	f_y =	60	*ksi, (414 MPa)*
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)		Yes	
VALUT DIMENSIONS	L =	26	*ft, (7.92 m), (AASHTO Fig 2.5)*
	H =	8	*ft, (2.44 m), (AASHTO Fig 2.5)*
	C =	9.6	*ft, (2.93 m)*
	S =	16	*in, (406 mm)*
	D =	32	*in, (813 mm)*
	t =	12	*in, (305 mm)*

ARCH REINFORCEMENT (A_s) 2 # 6 @ 16 in o.c. (at each face)

CHECK SECTION (from 0 to 18, see following image) ==> 12 **[THE VALUT DESIGN, AT CHECK SECTION, IS ADEQUATE.]**

SERVICE SOIL PRESSURE (ASD level, including surcharge & seismic ground shaking)

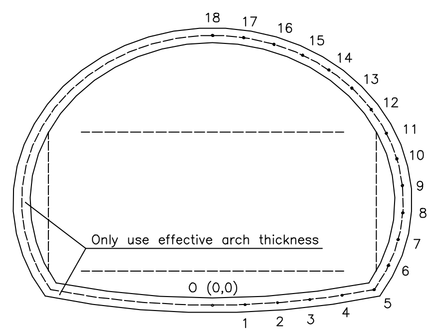
P_t = 550 *psf, (26 kPa)* $P_b = P_t + Wt$ = 946 *psf, (45 kPa)*

P_1 = 225 *ft-pcf, (psf), (11 kPa)* P_2 = 980 *ft-pcf, (psf), (47 kPa)*

ANALYSIS

DETERMINE VALUT FORCES BY FINITE ELEMENT METHOD

Point	X (ft)	Y (ft)	P (k/ft)	M (ft-k/ft)	V (k/ft)
0	0.00	0.00	8.47	39.50	0.00
1	2.66	0.11	8.69	32.92	3.78
2	5.29	0.45	8.98	22.86	5.07
3	7.89	1.01	9.34	9.38	6.34
4	10.43	1.80	9.80	-7.47	7.58
5	12.89	2.79	12.77	-27.62	-0.68
6	13.61	4.82	12.34	-26.15	-0.93
7	14.01	6.94	11.84	-24.15	-1.20
8	14.09	9.09	11.25	-21.56	-1.50
9	13.84	11.23	10.64	-18.33	-1.80
10	13.26	13.30	9.97	-14.44	-2.09
11	12.37	15.27	9.22	-9.93	-2.32
12	11.20	17.07	8.40	-4.93	-2.44
13	9.76	18.68	7.55	0.34	-2.40
14	8.10	20.05	6.74	5.51	-2.16
15	6.24	21.14	6.03	10.18	-1.73
16	4.25	21.95	5.51	13.90	-1.12
17	2.15	22.44	5.23	16.30	-0.39
18	0.00	22.60	5.23	17.13	0.00



R_t = 14.10 ft, (top circle radius)
R_b = 31.19 ft, (bottom circle radius)
θ_t = 113.89 deg, (between 5 & 18)
θ_b = 24.42 deg, (between 0 & 5)

At the check section:
P = 8.40 kips / ft
M = 4.93 ft-kips / ft
V = 2.44 kips / ft

ANALYSIS

VERT. REINF. AREA AT EACH SIDE	A_s	=	0.33	in^2
EFFECTIVE DEPTH (TMS 402 6.1.3.5)	d	=	9.51	in
WIDTH OF SECTION	b_w	=	12.00	in
EFFECTIVE THICKNESS	t_e	=	11.63	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48
REINFORCEMENT RATIO	ρ	=	0.0029
ALLOWABLE STRESS FACTOR	SF	=	1.333

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k=\sqrt{2\rho n+(\rho n)^2}-\rho n \quad = \quad 0.295849$$

THE ALLOWABLE STRESS DUE TO FLEXURE IS

$$F_b=(SF)(0.33f'_m)= \quad 660 \text{ psi}$$

THE ALLOWABLE REINF. STRESS DUE TO FLEXURE IS

$$F_S=(1.33 \ \ or \ \ 1.0)(20) \ \ or \ \ 32= \quad 32000 \text{ psi}$$

DETERMINE THE REGION FOR FLEXURE AND AXIAL LOAD (MDG-3 Tab 12.2.1, Fig 12.2-12 & 13, page 12-25).

$$\frac{M}{Pd}\le\frac{t_e}{6d}$$

1. The section is in compression and not cracked.

$$\frac{M}{Pd}\le\left(\frac{t_e}{2d}-\frac{1}{3}\right)$$

2. The section is cracked but steel is in compression.

$$\frac{M}{Pd}>\left(\frac{t_e}{2d}-\frac{1}{3}\right)$$

3. The section is cracked and steel is in tension.

REGION 3 APPLICABLE FOR (M, P)

CHECK REGION 1 CAPACITY

$$M_m=\frac{b_w t_e^2}{6}F_b-P\frac{t_e}{6}= \quad 13.52 \text{ ft-kips / ft} \quad > \quad M$$

[Not applicable]

CHECK REGION 2 CAPACITY

$$M_m=P\frac{t_e}{2}-\frac{2P^2}{3b_wF_b}= \quad 3.58 \text{ ft-kips / ft} \quad < \quad M$$

[Not applicable]

CHECK REGION 3 CAPACITY (The moment maybe limited by either the masonry compression or steel tension, MDG-3 page 12-25).

$$M_m=MIN\left[\frac{1}{2}b_wkdF_b\left(d-\frac{kd}{3}\right)-P\left(d-\frac{t_e}{2}\right)\ ,\ A_sF_s\left(d-\frac{kd}{3}\right)+P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right]$$

= 5.37 ft-kips / ft > M [Satisfactory]

THE ALLOWABLE SHEAR STRESS IS GIVEN BY (TMS 402 8.2.6)

$$F_v=(SF)1.125\left(\sqrt{f'_m}\right)= \quad 58.09 \text{ psi} \quad > \quad f_v=\frac{V}{t_eb_w}= \quad 17.52 \text{ psi}$$

[Satisfactory]

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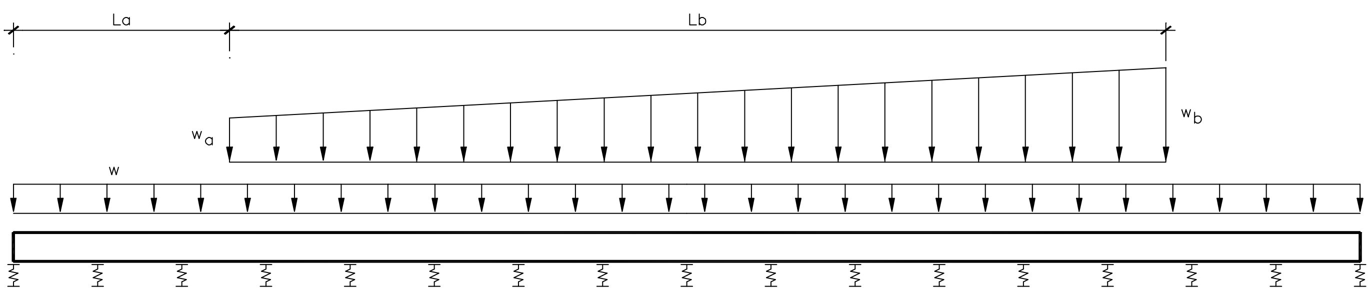
Subgrade Design for Heavy Haul Railway on Soft Soil Based on AASHTO & ACI 318-19

DESIGN CRITERIA

1. The railway subgrade can be reinforced concrete, plain concrete, steel, or even masonry grade beam. But it is assumed elastic at soil–structure interaction.
2. The soil boundary springs are compression only without tension capacity.
3. To check soil capacity, the uniform load, ***w***, should be input net value at the foundation bottom face. But to design subgrade, the ***w*** has to be total gravity value.

INPUT DATA



MODULUS OF SOFT SOIL K_1 = 100 *lb / in³ (27144.6 kN / m³), from soil report*

DIMENSION
- L = 778 *ft, (237.13 m), efficative length*
- B = 18 *ft, (5.49 m), bottom width*
- A = 20736 *in² (133780 cm²), section area*
- I = 15925248 *in⁴ (6.6E+08 cm⁴), moment of inertia*

MODULUS OF ELASTICITY E = 4030.509 *ksi, (27789 MPa) , for concrete input $57(f_c')^{0.5}$, & for steel, 29000 ksi.*

LOADS w = 21.6 *kips/ft, (315.0 kN/m)*

w_a = 15 *kips/ft, (218.8 kN/m)* w_b = 17.805 *kips/ft, (259.7 kN/m)*

L_a = 18 *ft, (5.49 m)* L_b = 700 *ft, (213.36 m)*

ANALYSIS & DESIGN SUMMARY

SPRING VALUE = ***52920.9*** *kips / inch, at grid tributary area of* ***875.25*** *ft²*

SETTLEMENTS
- *Left End =* ***0.0065*** *in*
- *Maximum Value =* ***0.0725*** *in, (from left end* ***632.13*** *ft)*
- *Right End =* ***0.0072*** *in*

TOTAL VERTICAL SPRING REACTION = ***28286.55*** *kips, upward positive*

TOTAL MOMENT SPRING REACTION AT CENTER OF SUBGRADE = ***85903.13*** *ft-kips, counterclockwise positive*

SOIL PRESSURE
- *Left End =* ***394.80*** *psf*
- *Maximum Value =* ***4384.68*** *psf, (from left end* ***632.13*** *ft)*
- *Right End =* ***432.57*** *psf*

DESIGN VALUES OF SUBGRADE SECTION
- *N =* ***144.22*** *kips, axial compression (45% B w_{max} ?), changeable*
- *M =* ***6383.85*** *ft-kips, (from left end* ***729.38*** *ft)*
- *V =* ***179.60*** *kips, (vertical shear from left end* ***0.00*** *ft)*
- *L =* ***778.00*** *ft*

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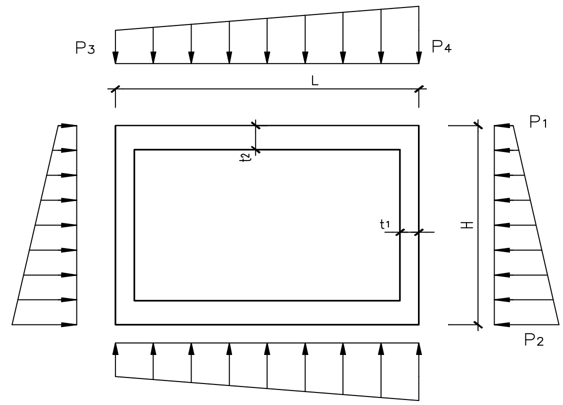
Underground Rectangular Section Design using Finite Element Method

DESIGN CRITERIA

1. The closed box section can be horizontal, vertical, or even sloped section.
   The web can be concrete, masonry or steel, with different thicknesses.
2. The soil pressure around box are balanced, so no movement assumed.



INPUT DATA & DESIGN SUMMARY

DIMENSIONS	L =	15	*ft, (4.57 m)*
	H =	12	*ft, (3.66 m)*
	t_1 =	8	*in, (203 mm)*
	t_2 =	10	*in, (254 mm)*

SERVICE SOIL PRESSURE (ASD level, including surcharge & seismic ground shaking)

P_1 =	300	*psf, (14 kPa)*	P_3 =	250	*psf, (12 kPa)*
P_2 =	500	*psf, (24 kPa)*	P_4 =	650	*psf, (31 kPa)*

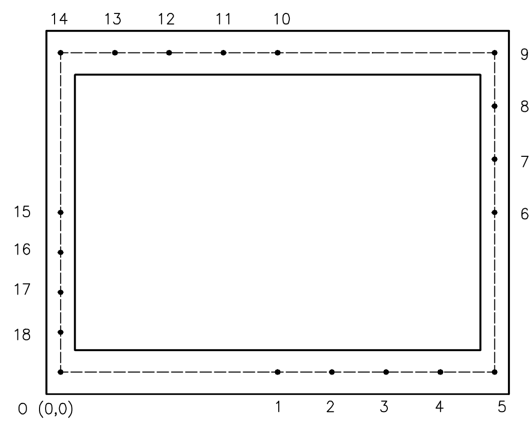
THE MOMENT OF INERTIA E = 10196 *ksi, <== (70317 N / mm^2)*

The sides web (t_1) design forces:			*The top & bottom web (t_2) design forces:*		
P (axial) =	***3.92***	*kips / ft, (38.9 kN/m)*	P (axial) =	***2.67***	*kips / ft, (57.2 kN/m)*
M =	***5.07***	*ft-kips / ft, (30.5 kN-m / m)*	*M =*	***6.85***	*ft-kips / ft, (22.6 kN-m / m)*
V =	***2.12***	*kips / ft, (45.9 kN/m)*	*V =*	***3.14***	*kips / ft, (31.0 kN/m)*

ANALYSIS

DETERMINE DEFLEXTIONS & SECTION FORCES BY FINITE ELEMENT METHOD

Point	X (ft)	Y (ft)	Δ_x(in)	Δ_y(in)	Element	P (k/ft)	M (ft-k/ft)	V (k/ft)
0	0.00	0.00	0.000	0.000	*1* (0 , 1)	2.67	6.59	1.64
1	7.17	0.00	0.000	0.006	*2*	2.67	6.78	0.21
2	8.96	0.00	0.000	0.002	*3*	2.67	6.40	1.10
3	10.75	0.00	0.000	-0.004	*4*	2.67	4.42	2.08
4	12.54	0.00	0.000	-0.013	*5*	2.67	4.94	3.14
5	14.33	0.00	0.000	-0.023	*6*	3.92	5.07	1.15
6	14.33	5.58	0.010	-0.023	*7*	3.92	1.48	0.40
7	14.33	7.44	0.013	-0.023	*8*	3.92	1.27	1.08
8	14.33	9.31	0.017	-0.023	*9*	3.92	4.45	1.71
9	14.33	11.17	0.019	-0.023	*10*	2.13	6.59	1.58
10	7.17	11.17	0.019	-0.028	*11*	2.13	6.85	0.59
11	5.38	11.17	0.019	-0.024	*12*	2.13	5.79	1.31
12	3.58	11.17	0.019	-0.017	*13*	2.13	3.43	1.95
13	1.79	11.17	0.019	-0.009	*14*	2.13	4.54	2.50
14	0.00	11.17	0.019	0.000	*15*	2.83	4.64	1.08
15	0.00	5.58	0.009	0.000	*16*	2.83	1.52	0.25
16	0.00	4.19	0.007	0.000	*17*	2.83	1.17	0.84
17	0.00	2.79	0.004	0.000	*18*	2.83	2.06	1.47
18	0.00	1.40	0.001	0.000	*19* (18 , 0)	2.83	5.02	2.12



Check Deflections:

δ_x(max) = **H / *14623***	<	***H / 480***	
δ_y(max) = **L / *11080***	<	***L / 480***	
	[Satisfactory]		

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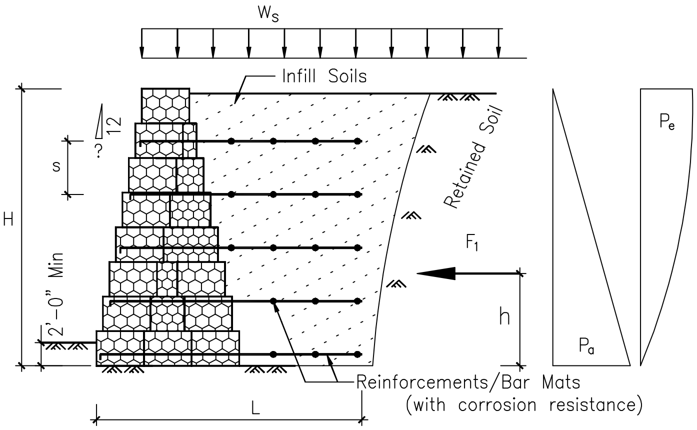
Design of Gabion Retaining Wall Based on AASHTO 17th & 2018 IBC

DESIGN CRITERIA

1. Gabion baskets, which are galvanized steel, double twisted, woven wire mesh filled with stone, have to be designed & made by manufacturer with durability capacity. (ICC report may be required.)
2. Gabion retaining wall, without pore water pressure, are green structure, not only reduced lateral loads but also easy construction..

INPUT DATA & DESIGN SUMMARY

WALL HEIGHT	H =	15	ft, (4.57 m)
WALL TOP THICKNESS	t =	24	in, (610 mm)
WALL OPEN SIDE SLOPE		2 / 12	
WALL SOIL SIDE SLOPE		3 / 12	
REBAR STRENGTH	f_y =	60	ksi
HORIZONTAL BAR MATS	L =	18	ft, (5.49 m)
		(input L = 0 for no bar mat)	
# 4	@	16	in, (406 mm), o.c., horiz. E. W.
	s =	24	in, (610 mm), o.c., vertical mat dist.
SURCHARGE WEIGHT	w_s =	500	psf
WALL SPECIFIC WEIGHT	γ_{wall} =	135	pcf, (2162 kg/m^3)
SOIL SPECIFIC WEIGHT	γ_b =	110	pcf, (1762 kg/m^3)
SOIL INTERNAL FRICTION ANGLE	ϕ =	29	deg
HORIZONTAL ACCELERATION (in g)	k_h =	0.35	g, (0.5 S_{DS})
ALLOW SOIL PRESSURE	Q_a =	3	ksf, (144 kPa)



THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE ACTIVE EARTH PRESSURE

$P_e = 0.75\, k_h\, \gamma_b$ = 29 psf / ft, (FEMA P-750 Page 356)

$P_a = \gamma_b\, K_a$ = 38 psf / ft

where $K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2}$ = 0.347

(Coulomb, AASHTO Figure 5.5.2A)

β	=	0	deg, slope of backfill
δ	=	0	deg, external friction angle
θ	=	90	deg, rack angle of wall face

$F_1 = 0.5\, H^2\, (P_a + P_e)$ = 8 kips / ft, *Total*

$h = (1/3\, P_a + 0.6\, P_e)\, H / (P_a + P_e)$ = 6.72 ft

CHECK SOIL BEARING CAPACITY (AASHTO Figure 5.8.3A)

$W = [0.25\, L\, (\text{slope1} + \text{slope2}) / 12 + t\,]\, H\, \gamma_{wall}$ = 7.21 kips / ft, *vertical wall weight*

$t_1 = H\, \text{slope1} / 12$ = 30.0 in , $t_2 = H\, \text{slope2} / 12$ = 45.0 in , $t_b = t_1 + t + t_2$ = 8.25 ft, *bottom wall thickness*

$F_2 = K_a\, H\, w_s$ = 2.60 kips / ft, *from surcharge weight*

$M = F_1\, h + F_2\, 0.5\, H - \gamma_{wall}\, H\, [0.25\, t_1^2\, 2/3 + t\, (t_1 + 0.5\, t) + 0.25\, t_2\, (t_1 + t + 1/3\, t_2)\,]$ = 43.02 ft-kips, *overturning*

$e = M / W - 0.5\, t_b$ = 1.84 ft

$$q_{MAX} = \begin{cases} \frac{W\left(1+\frac{6e}{t_b}\right)}{t_b}, & for \quad e \le \frac{t_b}{6} \\ \frac{2W}{3(0.5t_b - e)}, & for \quad e > \frac{t_b}{6} \end{cases} = 2.10 \text{ ksf} < Q_a$$

[Satisfactory]

CHECK SLIDING CAPACITY (2018 IBC 1807.2.3)

$1.1\,(F_1 + F_2)$ = 11.2 kips / ft < $\text{Tan}(\phi)\,(W + H\, L\, \gamma_b\,)$ = 20.46 kips / ft [Satisfactory]

$1.5\,(F_1 + F_2 - 0.5\, H^2\, P_e)$ = 10.3 kips / ft < $\text{Tan}(\phi)\,(W + H\, L\, \gamma_b\,)$ = 20.46 kips / ft [Satisfactory]

CHECK OVERTURNING CAPACITY (2018 IBC 1807.2.3)

$1.5\,(h\, F_1 + 0.5\, H\, F_2)$ = 105 ft-kips / ft < $0.5\, L\, (H\, L\, \gamma_b + L\, w_s\,) + W\, (0.5\, t_b - e)$ = 112.34 ft-kips / ft [Satisfactory]

(All forces with safety factor 1.5 conservatively.)

CHECK HORIZONTAL BAR MATS (AASHTO 5.8.4.1)

$T_{max} = \sigma_h\, A_{trib}$ = 0.64 kips / bar < $F_s\, A_s$ = 6.40 kips / bar [Satisfactory]

where σ_h = $F_2 / H + P_a + P_e$ = 0.24 ksf

A_{trib} = 384 in^2

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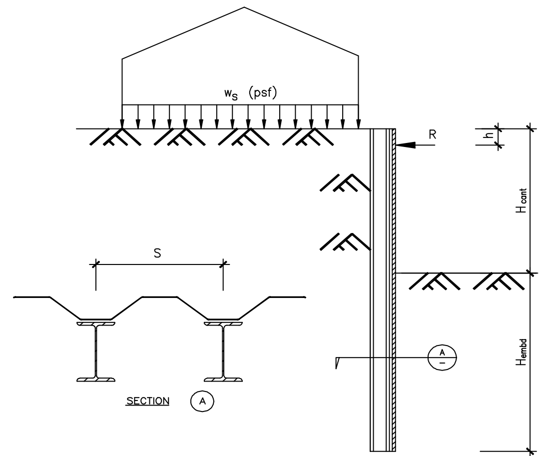
Steel Retaining Wall Design Based on 2018 IBC / 2019 CBC & AISC 360-16

DESIGN CRITERIA

1. There are two kind of steel retaining wall: one is steel sheets only, and another is steel piles (WF or HSS) with steel sheets. This software is to design the 2nd one. The retaining capacity are based on driven piles, so the steel sheets are only retaining and blocking the soil and water between piles.
2. To reduce the top bending drift of steel retaining wall, a restrained horizontal beam is one option. The top horizontal restrained force can be input based on restrained beam stiffness, or conservatively input zero if not sure.

INPUT DATA & DESIGN SUMMARY



HEIGHT OF CANTILEVER H_{cant} = 10 ft

SURCHARGE WEIGHT w_s = 500 psf

ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT

P_p = 300 psf / ft

LATERAL SOIL PRESSURE P_a = 35 pcf (equivalent fluid pressure)

SEISMIC GROUND SHAKING P_E = 450 psf / ft (for H_{cant} > 12 ft only)

SOIL SPECIFIC WEIGHT γ_b = 110 pcf

STEEL PILE (Tube, Pipe, or WF) & SIZE W14X257 (W Shape)

STEEL F_y = 50 ksi

PILE EFFECTIVE DIAMETER D = 16 in, *(input b_f if not sure.)*

STEEL PILE SPACING S = 3 ft, o.c.

RESTRAINED FORCE R = 5 kips / ft

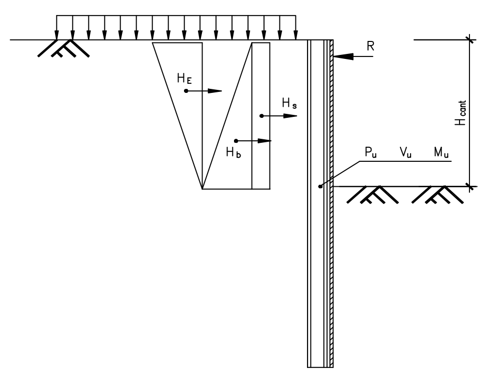
h = 2 ft

THE STEEL PILE DESIGN IS ADEQUATE.

(H_{embd} = 25.47 ft. Min. Req'D)

ANALYSIS

DETERMINE PILE SECTION FORCES AT CANTILEVER BOTTOM



$H_b = 0.5\ S\ P_a\ (H_{cant})^2$ = 5.25 kips, ASD

$H_s = w_s\ S\ P_a\ (H_{cant})\ /\ \gamma_b$ = 4.77 kips, ASD

$H_E = 0.5\ S\ P_E\ (H_{cant})^2$ = 67.50 kips, ASD

$P = S\ D\ w_s + 0.25\ \pi\ \gamma_c\ D^2\ H_{cant}$ = 4.09 kips, ASD

$V = H_b + H_E + H_s$ = 62.52 kips, ASD

$M = (H_b /3 + 2H_E /3 + H_s /2)\ H_{cant}$ = 266.82 ft-kips, ASD

CHECK STEEL PILE LIMITATIONS

$14\ t_{min}$ = 16.52 in > b_f = 16 in

[Satisfactory] (IBC Table 1810.3.5.3)

t_{min} = 1.18 in > 3/8 in

d = 16.4 in > 8 in **[Satisfactory]** (IBC Table 1810.3.5.3)

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360 H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) \ , \ for \ \dfrac{P_r}{P_c}\geq 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) \ , \ for \ \dfrac{P_r}{P_c}< 0.2 \end{cases} = \quad 0.22 \quad < \quad 1.00$$

[Satisfactory]

DETERMINE PILE EMBEDMENT LENGTH, H_{embd} , (IBC 1807.3)

By trials, use pole depth, $d = H_{embd}$ = 25.47 ft

Lateral bearing @ bottom, $S_3 = 2\ P_p\ Min(\ d\ ,\ 12'\ )$ = 7.20 ksf

Lateral bearing @ $d/3$, $S_1 = 2\ P_p\ Min(\ d/3\ ,\ 12'\ )$ = 5.09 ksf

Require Depth is given by

$$d=\frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad = \quad 25.47 \text{ ft}$$

[Satisfactory]

Where $P = V$ = 62.52 kips

$A = 2.34\ P\ /\ (D\ S_1)$ = 21.54

$h = M\ /\ V$ = 4.27 ft

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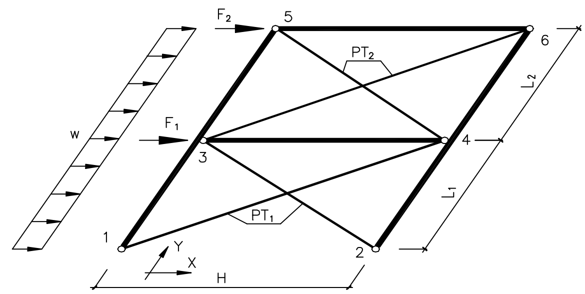
Cantilever Diaphragm Analysis using Tension-Only Braced Frame

DESIGN CRITERIA

1. The horizontal tension-only brace is light weight and simple connection details. Therefore, it is suitable for both new diaphragms and existing seismic retrofits, including glass bridge deck (un-approved seismic material by the Authority Having Jurisdiction, ASCE 7-16 11.1.4). The horizontal frame chords and webs can be steel, wood, and/or concrete. But the tension-only brace has to be steel cable/rod prestressed tension forces (PT).
2. The PT are always required to reduce lateral drifts, because the tension-only brace, without PT, cannot be lateral frame member. The chords & webs design has to include both the primary section forces and the secondary section forces. The secondary section forces are very important, which cannot directly be gotten, from 3D/2D FEM analysis with iteration, by input internal PT as external loads.

INPUT DATA & SUMMARY



HORIZONAL DIMENSIONS

L_1 = **14** *ft, (4.3 m)* H = **20** *ft, (6.1 m)*

L_2 = **12** *ft, (3.7 m)*

HORIZONAL LOADS

F_1 = **10** kips, (horiz. to right) w = **1.1** kips/ft

F_2 = **11** kips, (horiz. to right)

HORIZONAL MEMBERS

	A (in^2)	I_{Horz} (in^4)	E (ksi)
Chord (1 - 5)	**13.3**	**248**	**29000**
Chord (2 - 6)	**14**	**195**	**29000**
Web (3 - 4)	**6.49**	**199**	**29000**
Web (5 - 6)	**360**	**12000**	**1800**

	A (in^2)	E (ksi)	PT (kips)
PT_1	**0.459**	**29950**	**16.524**
PT_2	**0.306**	**29950**	**11.016**

LATERAL HORIZONAL DRIFTS

Δ_3 = ***1.42*** *in, (horiz. to right)*

Δ_5 = ***2.52*** *in, (horiz. to right)*

Δ_4 = ***1.34*** *in, (horiz. to right)*

Δ_6 = ***2.50*** *in, (horiz. to right)*

Horizonal Reactions

$R_{1,X}$ = ***-49.65*** *in, (horiz. to left)*

$R_{1,Y}$ = ***-44.51*** *in, (- Y dir)*

$R_{2,X}$ = ***0.05*** *in, (horiz. to right)*

$R_{2,Y}$ = ***44.51*** *in, (+ Y dir)*

DESIGN SECTION FORCES (not included gravity/vertical bending)

Chord	$N_{(1-5)}$ =	***-5.7***	k,(compre.)	$N_{(2-6)}$ =	***-54.0***	k,(compre.)
	$V_{(1-5)}$ =	***16.2***	kips	$V_{(2-6)}$ =	***15.8***	kips
	$M_{(1-5)}$ =	***132.8***	ft-kips	$M_{(2-6)}$ =	***181.7***	ft-kips
Web	$N_{(3-4)}$ =	***-93.8***	k,(compre.)	$N_{(5-6)}$ =	***-19.3***	k,(compre.)
	$V_{(3-4)}$ =	***0.0***	kips	$V_{(5-6)}$ =	***0.0***	kips
	$M_{(3-4)}$ =	***0.0***	ft-kips	$M_{(5-6)}$ =	***0.0***	ft-kips
PT	$N_{(1-4)}$ =	***67.3***	k,(tension)	$N_{(3-6)}$ =	***40.9***	k,(tension)
	$N_{(2-3)}$ =	***16.5***	k,(tension)	$N_{(4-5)}$ =	***11.0***	k,(tension)

ANALYSIS

Joint	Coordinates (ft) X	Y	Load & Reaction (k) F_x / R_x	F_y / R_y	Joint I Deflection (in) X	Y
1	0	0	-41.95	-35.03	0.00	0.00
2	20	0	0.05	53.99	0.00	0.00
3	0	14	39.58	-3.81	1.42	0.00
4	20	14	-22.98	-3.81	1.34	-0.02
5	0	26	34.75	-5.67	2.52	0.00
6	20	26	-9.45	-5.67	2.50	-0.03

Element	End Joint I	J	A (in^2)	I (in^4)	E (ksi)	Length (ft)	Primary M (ft-k) I	J	Secondary M (ft-k) I	J	Axial (k) Primary	Secondary	Shear (k) Primary	Secondary
1	1	4	0.459	0	29950	24.41	0.00	0.00	0.00	0.00	50.79	0.00	0.00	0.00
2	2	3	0.459	0	29950	24.41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	3	6	0.306	0	29950	23.32	0.00	0.00	0.00	0.00	29.90	0.00	0.00	0.00
4	4	5	0.306	0	29950	23.32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	1	3	13.3	248	29000	14.00	0.00	-56.73	0.00	189.52	5.91	0.00	-0.34	-13.54
6	2	4	14	195	29000	14.00	0.00	7.79	0.00	-189.52	-53.99	0.00	0.05	13.54
7	3	4	6.49	0	29000	20.00	0.00	0.00	0.00	0.00	-64.49	-29.33	0.00	0.00
8	3	5	13.3	248	29000	12.00	56.73	0.00	-189.52	0.00	-5.67	0.00	0.39	15.79
9	4	6	14	195	29000	12.00	-7.79	0.00	189.52	0.00	-21.05	0.00	-0.05	-15.79
10	5	6	360	0	1800	20.00	0.00	0.00	0.00	0.00	-35.14	15.79	0.00	0.00

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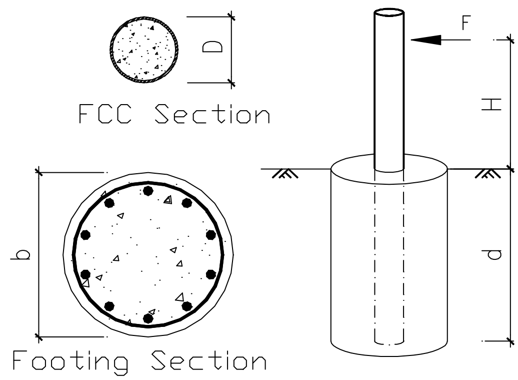
Vehicle Security Barrier Design Based on AASHTO-17th, 2018 IBC, AISC 341-16, & ACI 318-19

DESIGN CRITERIA

1. To be against hostile threat in public and urban environments, the vehicle security barriers are not only stop sign, but also have to support vehicle impact forces. The top cantilever column can be the filled composite column (FCC, AISC 341-16 H7), and the bottom foundation may use the pole footing (2018 IBC 1807.3).
2. The vehicle impact force should be from testing based on ASTM F2656, IWA 14-1, and/or PAS 68. But the testing results have to be converted to an equivalent point load to design, which, at service level (ASD level), not less than 6 times of vehicle total gravity weight.



INPUT DATA & DESIGN SUMMARY

IMPACT POINT HEIGHT	H =	**3**	*ft, (0.91 m)*
COLUMN OUTSIDE DIAMETER	D =	**8**	*in, (203 mm).*
COLUMN STEEL THICKNESS	t_s =	**1**	*in, (25 mm).*
STEEL YIELD STRESS	F_y =	**50**	*ksi, (345 MPa)*
CONCRETE STRENGTH	f_c' =	**5**	*ksi, (34 MPa)*
THE MAX CONCRETE STRESS	β =	**0.85**	f_c' *(0.85 on ACI, 1.0 on AISC)*
SINGLE COLUMN IMPACT FORCE	F =	**37**	*kips, (165 kN), ASD*
DIAMETER OF POLE FOOTING	b =	**4.5**	*ft, (1.37 m)*
LATERAL SOIL CAPACITY	P_P =	**0.35**	*ksf / ft, (55 kPa / m)*
RESTRAINED @ GRADE ?(1=yes,0=no)		**1**	Yes
FOOTING DEPTH	d =	5.31	*ft, (1.62 m)*

THE DESIGN IS ADEQUATE.

ANALYSIS

DESIGN COLUMN FOOTING (2018 IBC 1807.3)

Lateral bearing @ bottom, $S_3 = 2\,P_P\,Min(\,d\,,\,12'\,)$ = 3.71 ksf

Lateral bearing @ $d/3$, $S_1 = 2\,P_P\,Min(\,d/3\,,\,12'\,)$ = 1.24 ksf

Require Depth is given by

$$d=\begin{cases}\dfrac{A}{2}\left[1+\sqrt{1+\dfrac{4.36h}{A}}\right] & for\quad nonconstrained\\ \sqrt{\dfrac{4.25Ph}{bS_3}} & for\quad constrained\end{cases} = 5.307 \text{ ft}$$

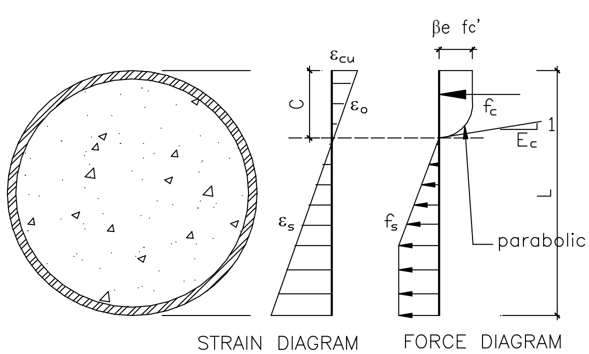
[Satisfactory]

Where $P = F =$ 37.00 kips

$A = 2.34\,P/(b\,S_1) =$ 7.73

$h = H =$ 3.00 ft

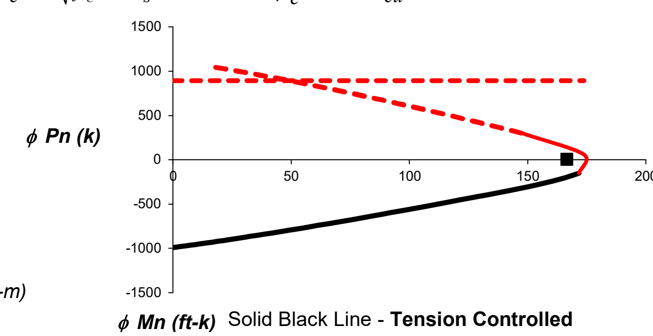
CHECK COLUMN FLEXURAL & AXIAL CAPACITY



$$f_c=\begin{cases}\beta_e f_c'\left[2\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right)-\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right)^2\right] & , \ for\ \ 0<\varepsilon_c<\varepsilon_o\\ \beta_e f_c' \ , \ for\ \ \varepsilon_c\ge\varepsilon_o\end{cases} \quad , \quad f_s=\begin{cases}\varepsilon_s E_s \ , \ for\ \ \varepsilon_s\le\varepsilon_{ty}\\ f_y \ , \ for\ \ \varepsilon_s>\varepsilon_{ty}\end{cases}$$

$$\varepsilon_o=\frac{2\left(\beta_e f_c'\right)}{E_c}\ ,\ \varepsilon_{ty}=\frac{f_y}{E_s}\ ,\ \varepsilon_t=\begin{cases}\varepsilon_{ty} \ , \ for\ \ compression\ \ controlled\\ \varepsilon_{ty}+0.003 \ , \ for\ \ tension\ \ controlled\end{cases}$$

$$E_c=57\sqrt{f_c'}\ ,\ E_s=29000ksi\ ,\ \beta_e=1.0\ ,\ \varepsilon_{cu}=0.003$$



Solid Black Line - **Tension Controlled**

Solid Red Line - **Transition**

Dash Line - **Compression Controlled**

P_u = 4.9 *kips, (22 kN)*

ϕM_{nc} @ P_u = 173.3 *ft-kips, (235 kN-m)*

> M_u = 166.5 *ft-kips, (226 kN-m)*

[Satisfactory]

where ϕ = 0.9 , (AISC 341-16 B3.2)

CHECK COLUMN SHEAR CAPACITY (AISC 341-16 H7.5b)

V_u/ϕ = 61.7 *kips, (274 kN)* **<** $\kappa\,A_{stl}\,F_y$ = 989.6 *kips, (4402 kN)*

where A_{stl} = 22.0 in^2 (14188 mm^2) **[Satisfactory]**

κ = 0.900 , (AISC 341-16 H7-8)

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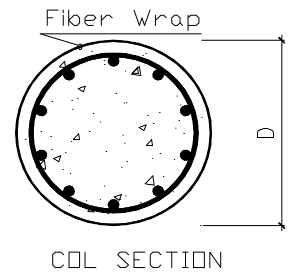
Column Repair Design of Carbon Fiber Wrap Based on 2018 IBC, ACI 318-19, & AASHTO-17th

DESIGN CRITERIA

1. Carbon fiber wrap and polymer are flexible cover, which only have elastic tension-only capacity, without yield stress cap as steel. The tension-only stress (E ε) should be based on the ICC reports, or local jurisdiction amendments to the code.
2. The special inspection should always be required. The lap connection length of carbon fiber wrap has to be more than 0'-8", and staggered for multi-layers.



INPUT DATA & DESIGN SUMMARY

FIBER WRAP MODULUS OF ELASTICITY	E =	34800	ksi, (239938 MPa)
COLUMN OUTSIDE DIAMETER	D =	40	in, (1016 mm).
FIBER WRAP EFFECTIVE THICKNESS	t_{fiber} =	0.00658	in, (0.1671 mm).
CURRENT REBAR YIELD STRESS	F_y =	47	ksi, (324 MPa)
CURRENT CONCRETE STRENGTH	f_c' =	5	ksi, (34 MPa)
THE MAX CONCRETE STRESS	β =	0.85	f_c' (0.85 on ACI, 1.0 on AISC)

THE DESIGN IS ADEQUATE.

SECTION FORCES	V_u =	115	kips, (512 kN)
(Strength Level, LRFD)	P_u =	4000	kips, (17793 kN)
	M_u =	1400	ft-kips, (1898 kN-m)

<== For this P_u, the max ϕM_n = 1437.1 *ft-kips, (1948 kN-m).*

<== For this M_u, the max $P_c = \phi P_n$ = 3085.8 *kips, (13726 kN),*

P_{re} / P_c = 1.296 *, (AISC 341-16 G3.4a)*

COLUMN VERTICAL REINFORCEMENT 12 # 7 Cover 0.75 *in, (19 mm) to fiber wrap.*

(Input zero if not count vertical bars)

ANALYSIS

CHECK SHEAR CAPACITY (AISC 341-16 H7.5b)

V_u / ϕ = 153.3 *kips, (682 kN)* **<** V_c = $2\,(f_c')^{0.5} A_0$ = 177.7 *kips, (791 kN)*

where A_0 = 1256.6 in^2 (8107 cm^2) **[Satisfactory]**

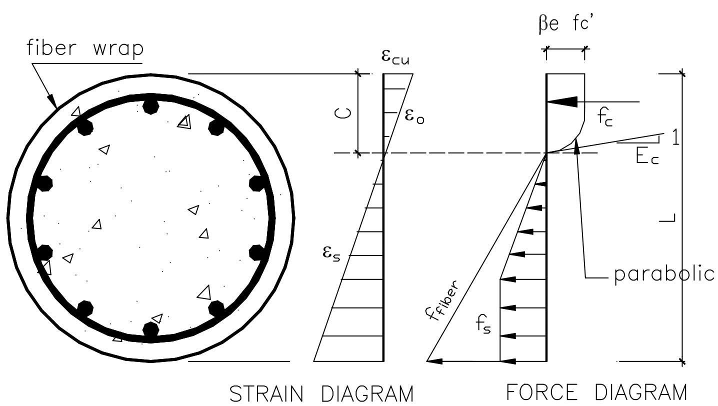
ϕ = 0.75 , (ACI 318-19 21.2)

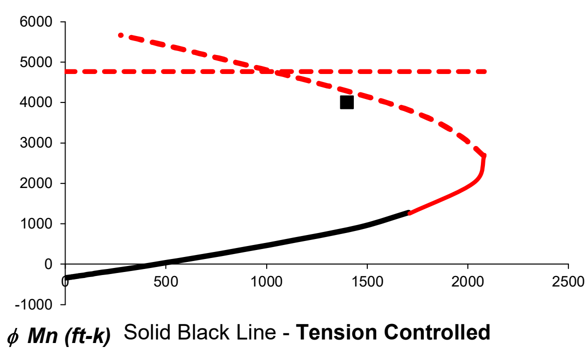
CHECK FLEXURAL & AXIAL CAPACITY

$$f_c = \begin{cases} \beta_e f_c'\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] &, \; for \; 0 < \varepsilon_c < \varepsilon_o \\ \beta_e f_c' &, \; for \; \varepsilon_c \ge \varepsilon_o \end{cases} \quad , \quad f_s = \begin{cases} \varepsilon_s E_s &, \; for \; \varepsilon_s \le \varepsilon_{ty} \\ f_y &, \; for \; \varepsilon_s > \varepsilon_{ty} \end{cases}$$

$$\varepsilon_o = \frac{2\left(\beta_e f_c'\right)}{E_c} \;,\; \varepsilon_{ty} = \frac{f_y}{E_s} \;,\; \varepsilon_t = \begin{cases} \varepsilon_{ty} &, \; for \; compression \; controlled \\ \varepsilon_{ty} + 0.003 &, \; for \; tension \; controlled \end{cases}$$

$$E_c = 57\sqrt{f_c'} \;,\; E_s = 29000 ksi \;,\; \beta_e = 1.0 \;,\; \varepsilon_{cu} = 0.003$$





Solid Black Line - **Tension Controlled**
Solid Red Line - **Transition**
Dash Line - **Compression Controlled**

P_u = 4000.0 *kips, (17793 kN)*

ϕM_{nc} @ P_u = 1596.8 *ft-kips, (2165 kN-m)*

> M_u = 1400.0 *ft-kips, (1898 kN-m)*

[Satisfactory]

where ϕ = 0.9

(AISC 341-16 B3.2 or ACI 318-19 21.2)

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Post-Compression Structure Analysis using Finite Element Method

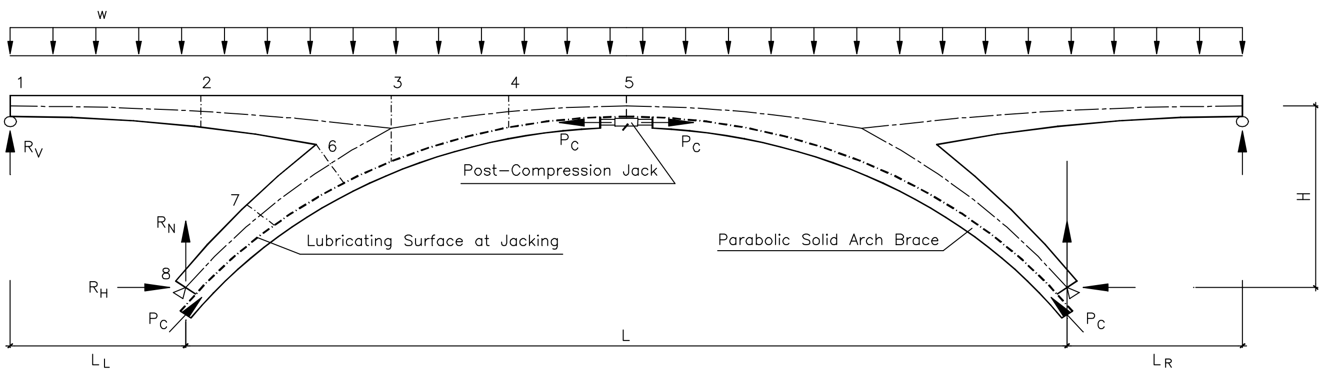
DESIGN CRITERIA

The effective post-compression force, P_c, should balance 60% - 80% DL, based on upward uniform force, ($8 P_C H / L^2$) .

There are three kind of forces on each section:

1. External loads, w only without P_C, section forces. The External loads can be ASD level for serviceability design, or SD level for ultimate strength design.
2. Primary equivalent loads, P_C section forces. The compression brace is mentally removed and replaced with all of the loads it exerts on the structure.
3. Secondary section forces from the reactions of primary P_C , at different boundary conditions.

INPUT DATA



DIMENSION

L_L =	5	ft, (1.52 m)
L =	130	ft, (39.62 m)
L_R =	5	ft, (1.52 m)
H =	40	ft, (12.19 m)

Section	D (in)	A (in^2)	I_x (in^4, in plane)	D (cm)	A (cm^2)	I_x (cm^4, in plane)
1	76	3507.84	2791662	193	22631	116197740
2	113	4307.04	7488499	287	27787	311694862
3	200	6186.24	31119420	508	39911	1295288064
4	107	4177.44	6540969	272	26951	272255676
5	84	3680.64	3584948	213	23746	149216781
6	100	10800	9000000	254	69677	374608283
7	80	8640	4608000	203	55742	191799441
8	60	6480	1944000	152	41806	80915389

DOWNWARD LOAD

w = 42.1008 kips/ft, (614.0 kN/m)

(including structural weight and impact factor.)

POST-COMPRESSION FORCE AFTER ALLOWANCE LOSSES

P_C = 1334.069 kips, (5933.9 kN)

MODULUS OF ELASTICITY

E = 4030.509 ksi, (27789 MPa)

ANALYSIS & DESIGN SUMMARY

Design Section Forces (Conservative Values)

Section	External Load, w, Only				
	P (kips, axial)	M (ft-kips)	V (kips, in plane)	Δ_x (in)	Δ_y (in)
1	-18.13	0.00	-220.47	0.042	0
2	107.85	5997.86	557.85	0.033	-0.103
3	2260.00	-8471.67	-516.06	0.033	-0.101
4	2095.19	149.39	-219.10	0.024	-0.155
5	2095.19	3715.96	0.00	0.000	-0.194
6	3122.67	2829.70	-63.31	0.036	-0.101
7	3122.67	1414.85	-63.31	0.038	-0.073
8	3122.67	0.00	-63.31	0	0

External load reactions

R_V =	615.91	kips
R_H =	2078.66	kips
R_N =	2331.15	kips

Primary reactions

R_V =	89.56	kips
R_H =	-1187.07	kips
R_N =	-1539.40	kips

Section	Primary equivalent loads, P_C			Secondary section forces from primary reactions		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)	P (kips, axial)	M (ft-kips)	V (kips, in plane)
1	-7.34	0.00	-89.26	-7.34	0.00	-89.26
2	-17.00	1679.20	-87.93	-17.00	1679.20	-87.93
3	-1297.51	4107.84	323.58	-146.34	1028.56	-34.90
4	-1197.09	-1297.79	135.00	-150.18	1611.53	-8.86
5	-1197.09	-3495.36	0.00	-150.18	1755.70	0.00
6	-1654.30	-1374.22	-83.63	-168.00	2203.00	-49.29
7	-1935.79	-3243.24	177.80	-168.00	1101.50	-49.29
8	-1935.79	730.32	177.80	-168.00	0.00	-49.29

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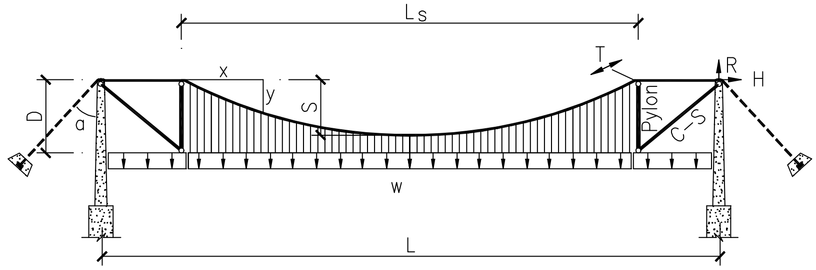
Hybrid Suspension Bridge Design Based on ASCE 19-10 & AASHTO 17th

INPUT DATA



MAIN SPAN	L =	5400	ft, (1645.92 m)
SUSPENSION	L_S =	4500	ft, (1371.60 m)
CABLE MAX SAG	S =	450	ft, (137.16 m)
PYLON HEIGHT	D =	455	ft, (138.68 m)

2-CABLE TOTAL LOAD ON STIFFENING DECK/GIRDER

w = 13.5 kips/ft, (197 kN/m), ASD level.

ANGLE OF ANCHOR CABLE α = 60 °

CABLE TENSILE STRENGTH $f_{su}^* = f_{pu}$ = 270 ksi, (1862 MPa)

SINGLE CABLE EFFECTIVE/NET AREA A = 800 in^2 (516128 mm^2)

ANALYSIS & DESIGN SUMMARY

Check S / L_S = 1 / 10.0 < 1 / 9
> 1 / 12 [Satisfactory]

Parabolic Main Cable $y = 4S/L_S^2\ x\ (L - x)$ = *8.88889E-05 x (4500 - x)* , control suspender ***(D - y)*** to keep parabolic curve.

Main Cable Curve Length = 4617.27 ft, (1407.34 m)

Maximum Main Cable Tension $T = [\ (0.5\ w\ L_S)^2 + (0.125\ w\ L_S^2 / S\ )^2\ ]^{0.5}$ = 81787.2 kips, (363789 kN), ASD level.

< $2\ A\ f_{pu}\ /\ \Omega_t$ = 103473.1 kips, (460248 kN) [Satisfactory]

where Ω_t = 1.67 x 2.5 = 4.175 *, AISC 360 & Reference page 907.*

Pylons $C_P = 0.5\ w\ L_S$ = 30375.0 kips, (135108 kN), ASD level. (Compression)

Cable-Stayed (C-S) L_{C-S} = 639.9 ft, (195.05 m)

$T_{C-S} = 0.25\ w\ (L + L_S)\ L_{C-S} / D$ = 46993.5 kips, (209027 kN), ASD level. (Tension)

Reactions on Tower Top $R = T_{C-S}\ D / L_{C-S} + T \cos\alpha$ = 74306.1 kips, (330514 kN), ASD level.

$H = T\ (1 - \sin\alpha\ ) + (T_{C-S}^2 - C_P^2\ )^{0.5}$ = 46814.7 kips, (208232 kN), ASD level.

Reference:

1. Reference: Engineers' Handbook for Bridge Design, China Communications Press, July 2007.
2. Roumen V. Mladjov: Hybrid Suspension Bridges for Super-Long Spans. STRUCTURE, Oct. 2021.

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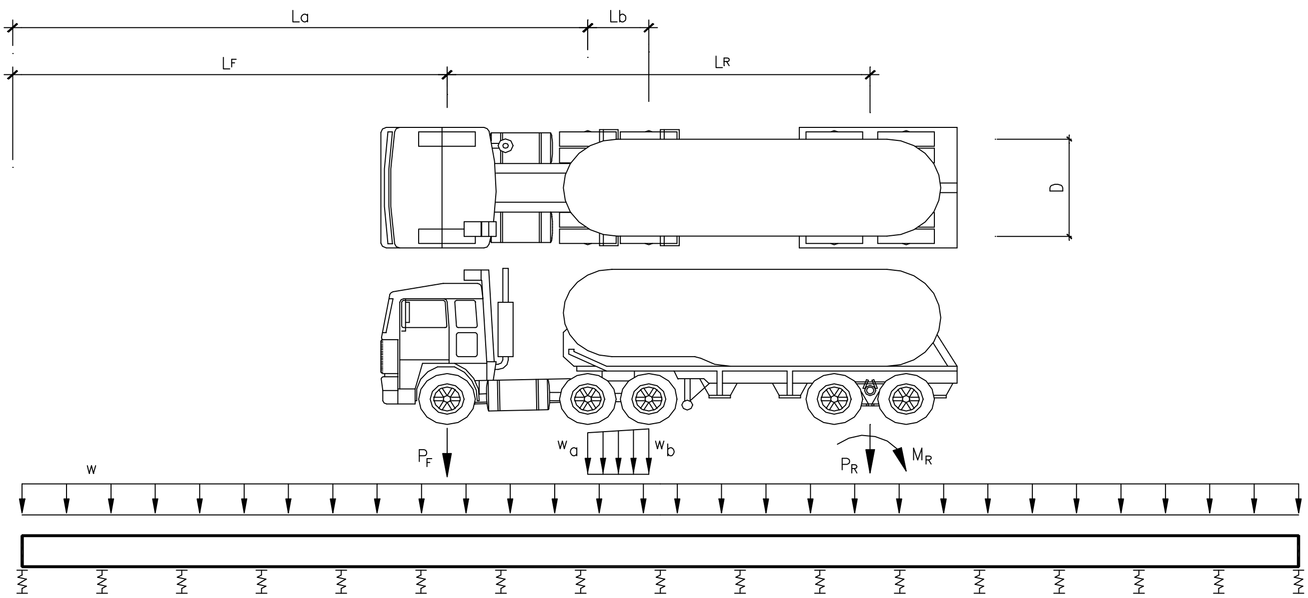
Analysis of Concrete Floor Slabs on Grade Subjected to Heavy Loads Based on AASHTO/ACI 318-19

DESIGN CRITERIA

1. The ASD level input loads (P_F, P_R, M_R, w_a & W_b) should include impact factor and can be zero to cover more kind of wheels loads.
   The dimensions (L_F & L_a) may input large values if not at the slab edge.
2. The soil boundary springs are compression only without tension capacity. It is assumed elastic at soil–structure interaction.
3. To check soil capacity, the uniform load, ***w***, should be input net value at the foundation bottom face. But to design slab, the ***w*** has to be total gravity value.



INPUT DATA

MODULUS OF SUBGRADE	K_1 =	200	lb/in^3 (54289.2 kN/m^3), from soil report				
CONCRETE STRENGTH	f_c' =	6	ksi, (41 MPa)				
MODULUS OF ELASTICITY	E =	4415.2	ksi, (30442 MPa)	, for concrete $57(f_c')^{0.5}$			
LOADS	w =	162.5	pcf, (2603 kg/m^3)	D =	6.5	ft, (1.98 m), out side wheel distance	
	w_a =	9	kips/ft, (131.3 kN/m)	w_b =	11	kips/ft, (160.4 kN/m)	
	L_a =	22	ft, (6.71 m)	L_b =	4	ft, (1.22 m)	
	P_F =	10	kips, (44.5 kN)	P_R =	40	kips, (177.9 kN), compression	
			(uplift, downward negative)	M_R =	20	ft-kips, (27.1 kN-m)	
	L_F =	10	ft, (3.05 m)	L_R =	44	ft, (13.41 m)	
SLAB THICKNESS	t =	8	in, (203 mm)	L =	98	ft, (29.87 m), effective total length	
REBARS COVER	C_C =	4	in, (102 mm), to bottom	B =	142	in, (360.68 cm), effective width, ACI 318-19 9.2.4.4	
# 5	@	18	in o.c., (457 mm)	A =	1136	in^2 (7329.02 cm^2), effective section area	
				I =	6058.67	in^4 (252181 cm^4), moment of inertia	

ANALYSIS & DESIGN SUMMARY

SPRING VALUE = ***2103.55*** kips / inch, at grid tributary area of ***72.48*** ft^2

SETTLEMENTS
- Left End = ***0.0022*** in
- Maximum Value = ***0.0191*** in, (from left end ***24.50*** ft)
- Right End = ***0.0021*** in

TOTAL VERTICAL SPRING REACTION = ***258.45*** kips, upward positive

TOTAL MOMENT SPRING REACTION AT CENTER OF SLAB = ***1607.33*** ft-kips, counterclockwise positive

SOIL PRESSURE
- Left End = ***62.62*** psf
- Maximum Value = ***555.42*** psf, (from left end ***24.50*** ft)
- Right End = ***62.40*** psf

DESIGN VALUES OF SLAB SECTION L = ***98.00*** ft

N_u = 1.5 N = 1.5 x ***0*** kips = ***0.00*** kips / ft, axial compression

V_u = 1.5 V = 1.5 x ***4.98*** kips = ***0.63*** kips / ft, (vertical shear from left end ***55.13*** ft)

M_u = 1.5 M = 1.5 x ***21.66*** ft-kips = ***2.75*** ft-kips / ft, (from left end ***55.13*** ft)

[Satisfactory] < $\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - N_u}{1.7 b f_c'}\right)\right] =$ ***3.34*** ft-kips / ft

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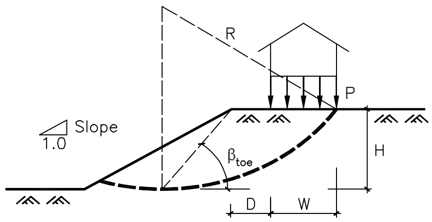
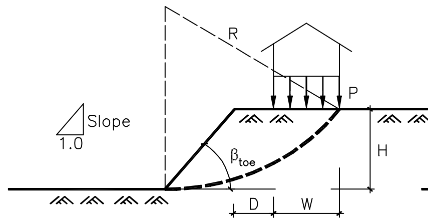
Seismic Slope Stability Analysis Based on Mononobe-Okabe Method, AASHTO 17th & 2021 IBC

DESIGN CRITERIA

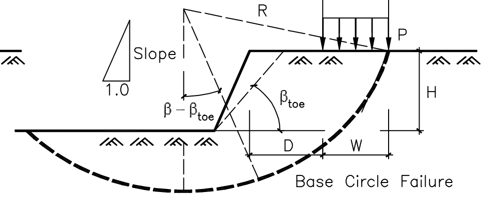
1. Assume that building/structure load, P, may cause three kind of failures: Slope Circle Failure, Toe Circle Failure, or Base Circle Failure. All failures started at the edge of building.
2. For wild fired mountain (tree total lost slope), the effective unit weight of soil, γ_{eff}, should input the both values of ($\gamma_{saturated}$ - γ_{water}) and $\gamma_{saturated}$ no matter if current soil is saturated or not, to check the worse condition.
3. The default cut angle of toe circle failure, β_{toe}, is 53°. When slope angle greater than it, the failure will be base circle failure, but less will be slope circle failure. User may change the default 53°, and input P zero load with different D and W values, to check general slope stability.



Slope Circle Failure



Toe Circle Failure



Base Circle Failure

INPUT DATA & DESIGN SUMMARY

SOIL EFFECTIVE UNIT WEIGHT	γ_{eff} =	38	pcf, (609 kg/m³)
SOIL INTERNAL FRICTION ANGLE	ϕ =	30	deg
SOIL EFFECTIVE COHESION	c =	0.4	ksf, (19 kPa)
BUILDING PRESSURE (LOAD)	P =	4.2	ksf, (201 kPa)

THE DESIGN IS ADEQUATE.

Factor of Safety, ***F = 1.51***

DEMENSION H = 10 ft, (3.05 m) D = 48 ft, (14.63 m)

W = 30 ft, (9.14 m) R = 370.82 ft, (113.02 m)

SLOPE (Vertical : Horizontal Ratio) 1.05 : 1.0 *(β = 46.397* < β_{toe} = ***53*** *deg , Slope Circle Failure apply.)*

HORIZONTAL SEISMIC COEFFICIENT k_h = 0.24 VERTICAL SEISMIC COEFFICIENT k_v = 0.12

(ψ = tan⁻¹ [k_h / (1-k_v)] = 15.26 deg)

ANALYSIS

(Active Earth Pressures Coefficient of Coulomb Analysis)

$$K_{ai} = \frac{sin^2(90-\alpha_i+\phi)}{sin^2(90-\alpha_i)\,sin(90-2\alpha_i)\left[1+\sqrt{\dfrac{\sin(\phi+\alpha_i)\sin\phi}{sin(90-2\alpha_i)\sin(90-\alpha_i)}}\right]^2}$$

(Seismic Active Earth Pressures Coefficient of Mononobe-Okabe Analysis)

$$K_{aei} = (1-k_v)Min(0.6\ ,\ \frac{\cos^2(\phi-\psi-\alpha_i+90)}{\cos\psi\cos^2(90-\alpha_i)\cos(2\alpha_i+\psi)\left[1+\sqrt{\dfrac{\sin(\phi+\alpha_i)\sin(\phi-\psi)}{\cos(2\alpha_i+\psi)\cos(\alpha_i-90)}}\right]^2})$$

$(K_{aei} / K_{ai})_{Max}$ = 1.58

(seismic increased)

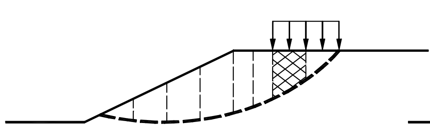
(Modified Bishop's Method of Analysis)

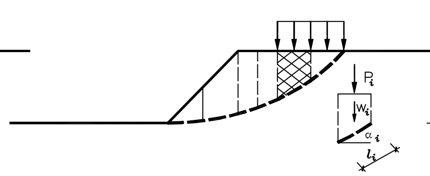
$$F = \frac{\sum_i \dfrac{cl_i+(w_i-u_il_i)\tan\phi}{\cos\alpha_i+\dfrac{\sin\alpha_i\tan\phi}{F}}}{\sum_i w_i\sin\alpha_i} = \frac{\sum_i \dfrac{cl_i\cos\alpha_i+(w_i+P_i)\tan\phi}{Max\left(\cos\alpha_i+\dfrac{\sin\alpha_i\tan\phi}{F}\ ,\ 0.2\right)}}{\sum_i(w_i+P_i)\sin\alpha_i} =$$

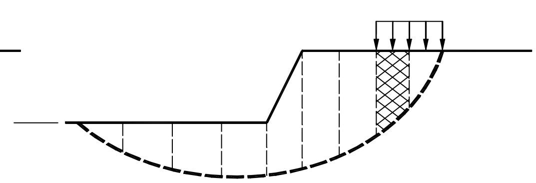
2.39 , *F / [$(K_{aei} / K_{ai})_{Max}$] =* ***1.51***

> ***1.5***

[Satisfactory] (2021 IBC 1807.2.3 & AASHTO 17th 5.2.2.3)







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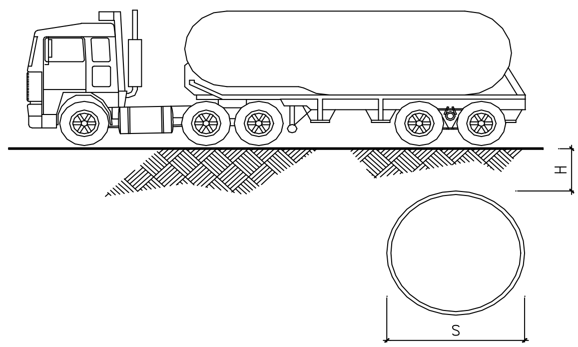
Flexible Pipe Cover Design Based on AASHTO / NCSPA Design Manual

INPUT DATA & DESIGN SUMMARY



COVER DEPTH OVER CROWN H = 2 *ft, (0.61 m)*

SPAN or DIAMETER S = 8 *ft, (2.44 m)*

PLASTIC MOMENT STRENGTH of FLEXIBLE PIPE (Corrugated Steel Pipe)

M_p = 0.79 *ft-kips / ft, (3.51 kN-m / m), from NCSPA Table 8.6 or ICC Reports*

CORRUGATION DEPTH d = 1 *in, (25 mm)*

0.0833 *ft, (0.0254 m)*

AXLE LOAD FOR THE DESIGN VEHICLE AL = 22.3 *kips, (99.2 kN)*

(AL is the load supported on a single axle or on tandem axles if the spacing between the axles is less than 1/3 of the span of the culvert.)

THE COVER DESIGN IS ADEQUATE.

COMPACTION LEVEL (from 90% to 95%) 92 %

(equal to 92-percent standard AASHTO)

ANALYSIS

F_p = ***1.00*** , factor of safety against the development of a plastic hinge (dimensionless)

where F_p = 1.3 for $0 < H/S \le 0.1$

1.6 - 3(H/S) for $0.1 < H/S < 0.2$

1.0 for $H/S > 0.2$

c = ***87.4*** , coefficient whose value depends on the degree of compaction of the backfill

where c = 69 for a compaction level equal to 90-percent standard AASHTO

115 for a compaction level equal to 95-percent standard AASHTO

$$M_{reqD} = \frac{AL\, d\, F_p}{c}\left(\frac{S}{H}\right)^2 =$$ ***0.34*** *ft-kips / ft* **<** M_p = ***0.79*** *ft-kips / ft*

[Satisfactory]

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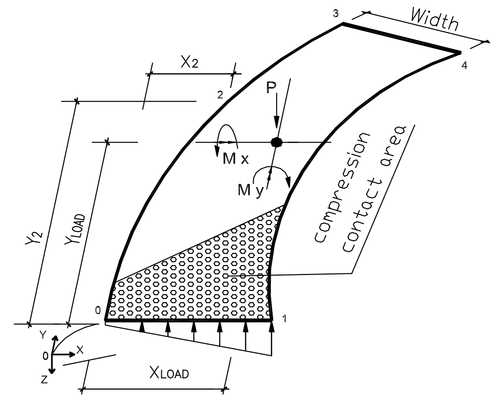
Curved Rigid Footing Design Based on ASCE 41-17 & ACI 318-19

DESIGN CRITERIA

1. Soil resists compression only, without tension capacity. Soil bearing pressure distribution is a plane, and a triangular distribution block when un-full area compression under the footing.

*2. The input **P**, **M**$_x$, & **M**$_y$ are net loads on the footing bottom face, which the location (**X**$_{LOAD}$ & **Y**$_{LOAD}$) is directly from top structure analysis and not has to be at the centroid of footing bottom section.*

3. The most footing of dam (between joints) or multi-story building (box foundation) is rigid relative to the soil.



INPUT DATA & DESIGN SUMMARY

ALLOWABLE SOIL PRESSURE Q_a = **5** *ksf, (239 kPa)*

DIMENSIONS

Width = **60** *ft, (18.3m)*

Length = *163.08 ft, (49.7m), edge curve 0-2-3*

103.80 ft, (31.6m), edge curve 1-4

Point	***0***	***1***	***2***	***3***	***4***
X	0	59.97	**30**	**70**	104.53
Y	0	1.82	**100**	**140**	90.93

Point **2** (**X_2**, **Y_2**) is any arc location of the outside edge.

THE FOOTING DESIGN IS ADEQUATE.

The Maximum Soil Pressure:

P_{max} = ***2.256*** *ksf, (108 kPa)*

at X = ***2.940*** *ft*

Y = ***2.759*** *ft*

LOADS P = **800** *kips, (3559 kN), ASD*

M_y = **100** *ft-kips, (136 kN-m), rotation about the Y axis*

M_x = **60** *ft-kips, (81 kN-m), rotation about the X axis*

X_{LOAD} = **5** *ft, (1.5m)*

Y_{LOAD} = **20** *ft, (6.1m)*

A = *8006.34 ft^2*

(full footing area)

A_C = *504.06 ft^2*

(compression contact area)

ANALYSIS

CHECK THE CENTER OF GRAVITY WITHIN FOOTING

$X_{CG} = X_1 + M_y / P$ = *5.13 ft* $Y_{CG} = Y_1 - M_x / P$ = *19.93 ft*

R = 160.57 **>** *108.08 ft* θ = 174.813 **>** *126.08* deg

< *162.08 ft* **<** *180.79* deg [Satisfactory]

DETERMINE EQUATION OF SOIL PRESSURE DISTRIBUTION PLANE

$$\begin{vmatrix} X\ (ft) & Y\ (ft) & Z\ (ksf) & 1 \\ 2.940 & 2.759 & 2.256 & 1 \\ 69.557 & 135.993 & -3.720 & 1 \\ 101.356 & 92.349 & -3.454 & 1 \end{vmatrix} = 0$$

CHECK SOIL BEARING CAPACITY

Corner	***0***	***1***	***2***	***3***	***4***	***at Max***
P (ksf)	***2.26***	***0.53***	***0.00***	***0.00***	***0.00***	***2.26***

< Q_a [Satisfactory]

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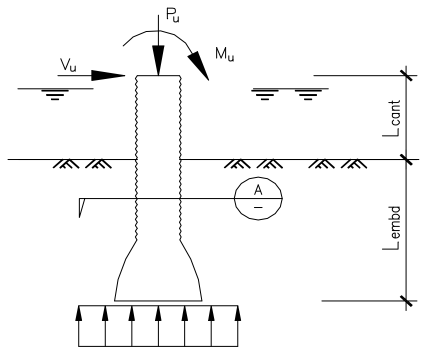
CSP Deep Foundation Design Based on 2021 IBC, ACI 318-19 & AASHTO 17th



DESIGN CRITERIA

1. Corrugated Steel Pipe (CSP) is concrete form, without removed after construction, which good for both concrete cover and skin friction in an upward direction.
2. The deep foundation is designed to rest on an underlying stratum of rock or satisfactory soil and is used when unsatisfactory soil/water exists. If deep foundation bottom end not belled, the deep foundation has to be drilled into hard stratum 3'-0' minimum.
3. All vertical/axial loads are supported by the bottom end hard stratum. The lateral loads are supported by the embedded soil, L_{embd}, based on 2021 IBC Chapter 18.

INPUT DATA & DESIGN SUMMARY

DIMENTION L_{embd} = **18** ft, (5.49 m)

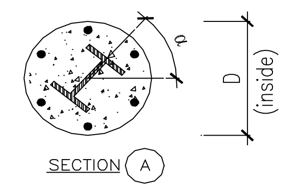
L_{cant} = **1.5** ft, (0.46 m)

D = **22** in, (559 mm)

WF SECTION => **W12X120**

d	t_w	b_f	t_f
13.1	0.71	12.30	1.11

α = **43** Deg



STEEL YIELD STRESS F_y = **60** ksi, (414 MPa)

CONCRETE STRENGTH f_c' = **2.5** ksi, (17 MPa)

THE MAX CONCRETE STRESS β = **0.85** f_c' (0.85 on ACI, 1.0 on AISC)

VERTICAL REINFORCEMENT **6** # **6** (zero for no bars.)

C_C = **2** in, (51 mm), vert. rebars concrete cover.

TOP LOADS (LRFD / SD level) M_u = **250** ft-kips, (339 kN-m)

P_u = **131** kips, (583 kN)

V_u = **9.3** kips, (41 kN)

THE DESIGN IS ADEQUATE.

ALLOW SOIL PRESSURE IN BOTTOM END STRATUM Q_a = **50** ksf, (2394 kN/m^2)

ALLOWABLE LATERAL SOIL-BEARING PRESSURE IN EMBEDMENT P_p = **0.25** ksf / ft, (39.272 $[kN/m^2]/m$)

ANALYSIS

CHECK SOIL END BEARING CAPACITY (ACI 318-19 13.3.1.1)

$0.7\,P_u + 0.25\,\pi\,D^2\,(\gamma_c - \gamma_s)\,(L_{embd} + L_{cant})$ = 95.82 kips, (426 kN)

< $0.25\,\pi\,D^2\,Q_a$ = 131.99 kips, (587 kN) **[Satisfactory]**

Where γ_c = 0.15 kcf, one cubic foot of deep foundation weight γ_s = 0.07 kcf, soil/water weight

CHECK DEEP FOUNDATION EMBEDMENT LENGTH, L_{embd}, (IBC/CBC 1807.3)

By trials, use pole depth, $d = L_{embd}$ = 18.00 ft, (5.49 m)

Lateral bearing @ bottom, $S_3 = 2\,P_p\,Min(\,d\,,\,12'\,)$ = 6.00 ksf, (287.28 kN/m^2)

Lateral bearing @ $d/3$, $S_1 = 2\,P_p\,Min(\,d/3\,,\,12'\,)$ = 2.17 ksf, (103.74 kN/m^2)

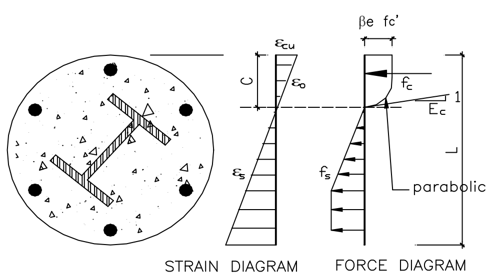
Require Depth is given by

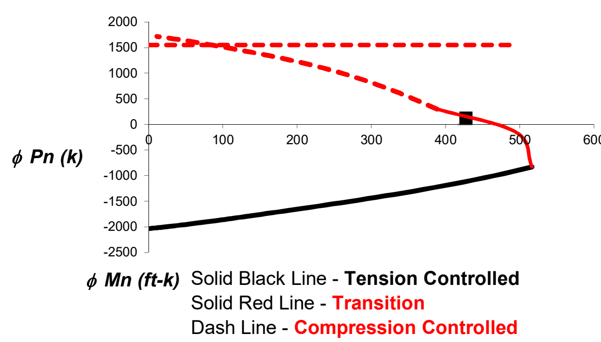
$$d = \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] \quad for \quad nonconstrained \quad =$$ 12.98 ft, (3.96 m) < L_{embd} **[Satisfactory]**

Where $P = 0.7\,V_u$ = 6.51 kips, (29 kN) $A = 2.34\,P/(D\,S_1)$ = 3.8351

$h = M_u / V_u + L_{cont}$ = 28.382 ft, (8.65 m)

CHECK FLEXURAL & AXIAL CAPACITY





$\phi\,M_n$ = 429 ft-kips @ P_u = 131 kips kips

> $[V_u\,(L_{cant} + 0.67\,L_{embd}) + M_u\,]$ = = 426.7 ft-kips, (579 kN-m) **[Satisfactory]**

CHECK SHEAR CAPACITY (only concrete, ACI 318 13.2.7.2 & 22.5)

V_u/ϕ = 12.4 kips, (55 kN) < $2\,A_c\,(f_c')^{0.5}$ = 38.0 kips, (169 kN) **[Satisfactory]**

where A_c = 380.1 in^2 (245246 mm^2) ϕ = 0.75

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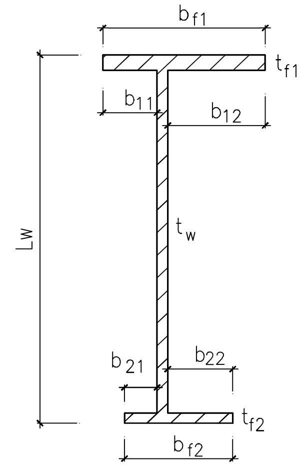
Masonry Shear Wall Design Based on TMS 402-08 / 2009 IBC (both ASD and SD)

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES) 1 Yes
(This option only for local jurisdiction amendments to the code, not part of TMS.)
TYPE OF MASONRY (1=CMU, 2=BRICK) 1 CMU
MASONRY STRENGTH f_m' = 3 ksi
REBAR YIELD STRESS f_y = 60 ksi
ALLOWABLE 30% INCREASING ? (TMS 2.1.2.3) Yes
SEISMIC PERFORMANCE CATEGORY D Seismic D
(C,D,E, 0=WIND, 5=GRAVITY)



SERVICE AXIAL LOAD P = 300 kips, at middle of L_w
SERVICE SHEAR LOAD V_x = 210 kips, (in-plane force)
SERVICE MOMENT LOAD M_x = 6200 ft-kips, (top flange, bf1, compression)
M_y = 500 ft-kips, (out-of-plane, left b11 & b21, compression)

EFFECTIVE HEIGHT OF WALL h_w = 16 ft
LENGTH OF SHEAR WALL L_w = 30 ft, (within vertical control joints)

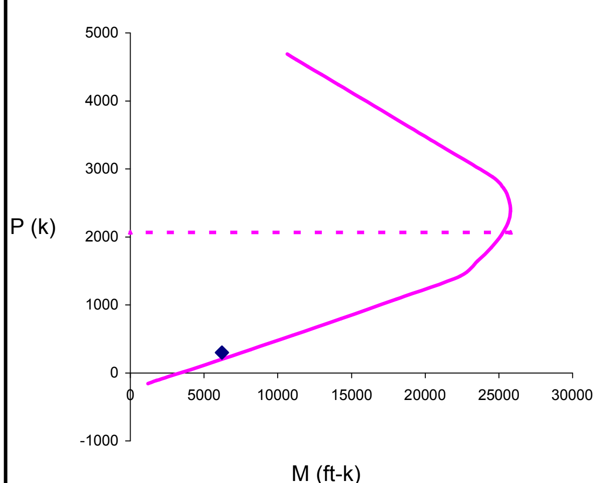
THE WALL DESIGN IS ADEQUATE.

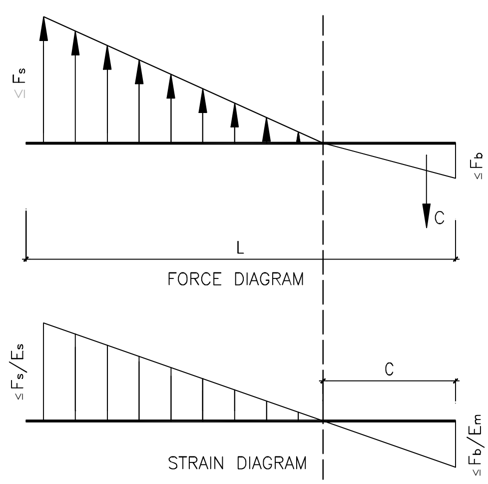
THICKNESS OF WALL t_w = 8 in
REINFORCING OF WALL 2 # 6 at each ends, with 8 in center to edge.
A_{sh}, Horizontal 1 # 6 @ 16 in o.c.
A_{sv}, Vertical 1 # 6 @ 24 in o.c.

TOP FLANGE (COMPRESSION) b_{11} = 48 in , b_{12} = 24 in , b_{f1} = 80 in ,(TMS 1.9.4.2.3)
t_{f1} = 12 in , 2 # 5 @ 48 in o.c., Vertical

BOTTOM FLANGE b_{21} = 36 in , b_{22} = 48 in , b_{f2} = 92 in, (TMS 1.9.4.2.3)
t_{f2} = 8 in , 2 # 5 @ 48 in o.c., Vertical

ANALYSIS
CHECK FLEXURAL & AXIAL CAPACITY BY ALLOWABLE STRESS DESIGN (ASD)



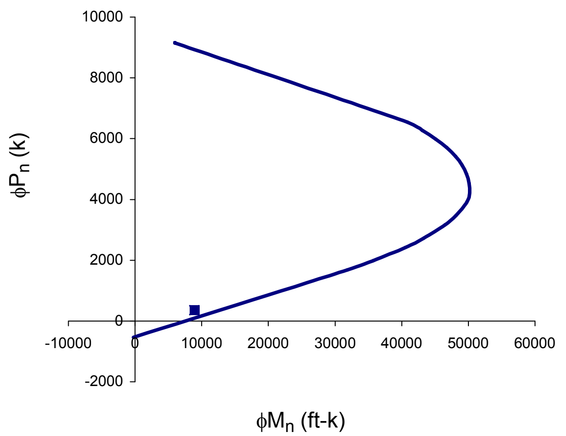


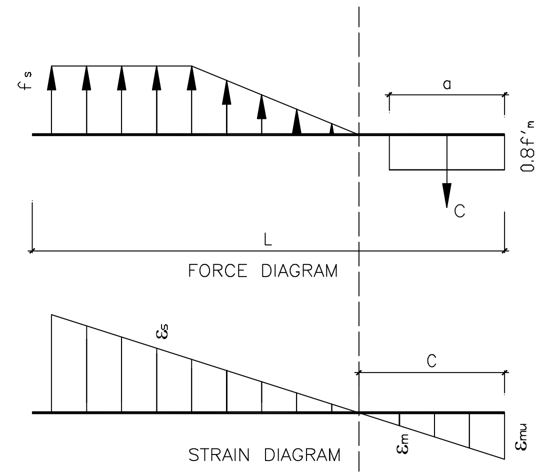
P (load) = 300 kips < P (allowable) = P_a = 2069.56 kips
M (resultant) = $(M_x^2 + M_y^2)^{0.5}$ = 6220.13 ft-kips < M (allowable) = 7571.53 ft-kips [Satisfactory]

Where E_m =	2700	ksi, (TMS 1.8.2.2.1)	A_n =	4235	in^2
E_s =	29000	ksi, (TMS 1.8.2.1)	A_{st} =	9.94	in^2
Scale Factor =	1.333	, (TMS 2.1.2.3)	$f_s \geq$	0	ksi, (TMS 2.3.2.2.1)
F_b =	1.320	ksi, (TMS 2-17)	h / r =	87	, neglected conservativly flanges.
F_s =	32.00	ksi, (TMS 2.3.2.1)	P_a =	2069.56	kips, (TMS 2.3.3.2.1)

(cont'd)

CHECK FLEXURAL & AXIAL CAPACITY BY STRENGTH DESIGN (SD)





P_u = 1.2 P = 360 kips

$M_u = (1/0.7)\,(M_x^2 + M_y^2)^{0.5}$ = 8885.9 ft-kips < ϕM_n = 12763.3 ft-kips, at P_u level.

[Satisfactory]

Where ε_{mu} = 0.0025 , (TMS 3.3.2.c) d = 363 in

ϕ = 0.9 , (TMS 3.1.4.1) f_m' = 3 ksi

CHECK SHEAR CAPACITY (ASD)

$$F_{v,\ \text{without reinf.}} = \begin{cases} (SF)\,MIN\left[\frac{1}{3}\left(4-\frac{M_T}{Vd}\right)\sqrt{f_m'}\ ,\ \left(80-\frac{45M_T}{Vd}\right)\right]\ ,\ for\ \frac{M_T}{Vd}<1.0 \\ (SF)\,MIN(\sqrt{f_m'}\ ,\ 35)\ ,\ for\ \frac{M_T}{Vd}\geq 1.0 \end{cases} = 48\ \text{psi} < 1.5\ f_v$$

(Shear reinf. reqd to carry full shear load.)
(factor 1.5 from TMS 402 1.17.3.2.6.1.2)

$$F_{v,\ \text{Maximum}} = \begin{cases} (SF)\,MIN\left[\frac{1}{2}\left(4-\frac{M_T}{Vd}\right)\sqrt{f_m'}\ ,\ \left(120-\frac{45M_T}{Vd}\right)\right]\ ,\ for\ \frac{M_T}{Vd}<1.0 \\ (SF)\,MIN(1.5\sqrt{f_m'}\ ,\ 75)\ ,\ for\ \frac{M_T}{Vd}\geq 1.0 \end{cases} = 101 > f_v = 77\ \text{psi}$$

[Satisfactory]

CHECK MINIMUM REINFORCEMENTS

$1.5\ \frac{V}{F_s d}=$	0.33	in^2 / ft	<	$\frac{A_v}{s}=$	0.33	in^2 / ft	[Satisfactory]	(TMS 402 1.17.3.2.6.1.2)
$A_{sh,min}$ =	0.137	in^2/ft	<	$A_{sh,actual}$ =	0.330	in^2/ft	[Satisfactory]	(TMS 1.17.3.2.6)
$S_{sh,max}$ =	24	in	>	$S_{sh,actual}$ =	16	in	[Satisfactory]	(TMS 1.17.3.2.6)
$A_{sv,min}$ =	0.064	in^2/ft	<	$A_{sv,actual}$ =	0.220	in^2/ft	[Satisfactory]	(TMS 1.17.3.2.6)
$S_{sv,max}$ =	24	in	>	$S_{sv,actual}$ =	24	in	[Satisfactory]	(TMS 1.17.3.2.6)

CHECK MAXIMUM REINFORCEMENT PERCENTAGE

$$\rho_{max} = \frac{n f_m'}{2 f_y\left(n+\frac{f_y}{f_m'}\right)} = 0.0087 > \rho = 0.0003$$ [Satisfactory]

(TMS 402 2.3.3.4)

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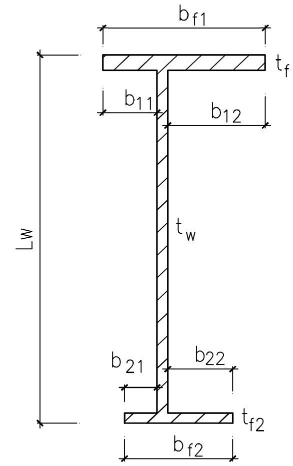
REVIEW BY :

Masonry Shear Wall Design Based on TMS 402-08 / 2010 CBC (both ASD and SD)

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)		1	Yes
(This option only for local jurisdiction amendments to the code, not part of TMS.)			
TYPE OF MASONRY (1=CMU, 2=BRICK)		1	CMU
MASONRY STRENGTH	f_m' =	3	ksi
REBAR YIELD STRESS	f_y =	60	ksi
ALLOWABLE 30% INCREASING ? (TMS 2.1.2.3)			Yes
SEISMIC PERFORMANCE CATEGORY		D	Seismic D
(C,D,E, 0=WIND, 5=GRAVITY)			

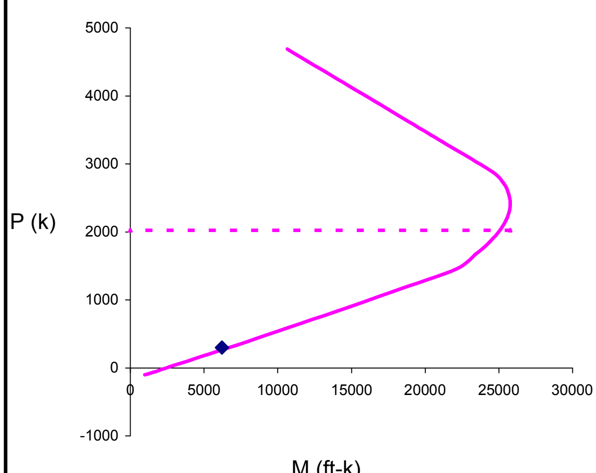


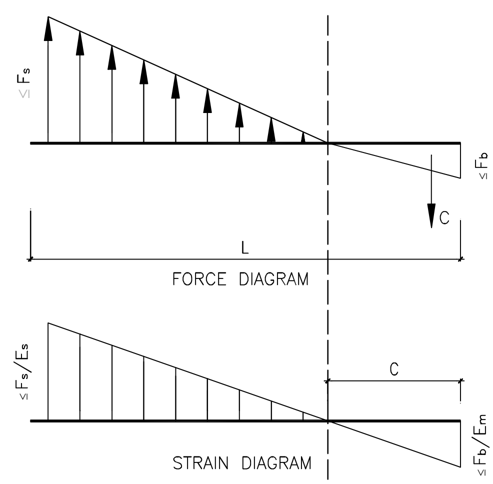
SERVICE AXIAL LOAD	P =	300	kips, at middle of L_w
SERVICE SHEAR LOAD	V_x =	200	kips, (in-plane force)
SERVICE MOMENT LOAD	M_x =	6200	ft-kips, (top flange, bf1, compression)
	M_y =	500	ft-kips, (out-of-plane, left b11 & b21, compression)
EFFECTIVE HEIGHT OF WALL	h_w =	16	ft
LENGTH OF SHEAR WALL	L_w =	30	ft, (within vertical control joints)

THE WALL DESIGN IS ADEQUATE.

THICKNESS OF WALL	t_w =	8	in							
REINFORCING OF WALL		2	#	6	at each ends, with	8	in center to edge.			
A_{sh}, Horizontal	1		#	6	@	16	in o.c.			
A_{sv}, Vertical	1		#	4	@	24	in o.c.			
TOP FLANGE (COMPRESSION)		b_{11} =	48	in ,	b_{12} =	24	in ,	b_{f1} =	80	in ,(TMS 1.9.4.2.3)
t_{f1} =	12	in ,	2	#	5	@	48	in o.c., Vertical		
BOTTOM FLANGE		b_{21} =	36	in ,	b_{22} =	48	in ,	b_{f2} =	92	in, (TMS 1.9.4.2.3)
t_{f2} =	8	in ,	2	#	5	@	48	in o.c., Vertical		

ANALYSIS
CHECK FLEXURAL & AXIAL CAPACITY BY ALLOWABLE STRESS DESIGN (ASD)





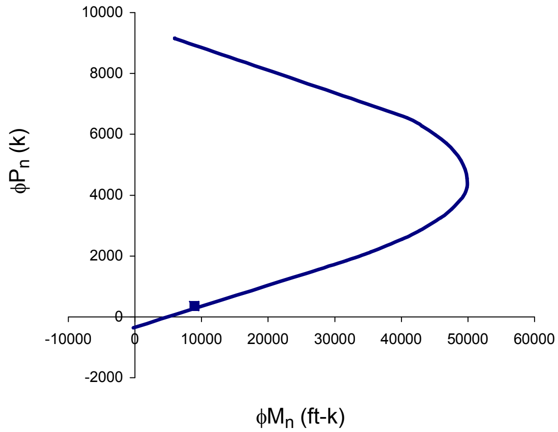
P (load) = 300 kips < P (allowable) = P_a = 2027.82 kips

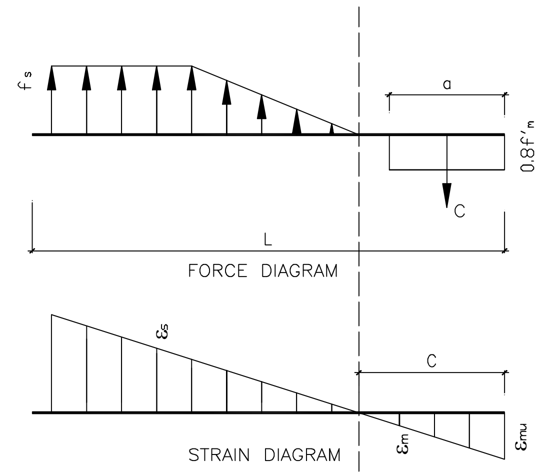
M (resultant) = $(M_x^2 + M_y^2)^{0.5}$ = 6220.13 ft-kips < M (allowable) = 6699.19 ft-kips [Satisfactory]

Where	E_m =	2700	ksi, (TMS 1.8.2.2.1)	A_n =	4235	in^2
	E_s =	29000	ksi, (TMS 1.8.2.1)	A_{st} =	6.66	in^2
	Scale Factor =	1.333	, (TMS 2.1.2.3)	$f_s \geq$	0	ksi, (TMS 2.3.2.2.1)
	F_b =	1.320	ksi, (TMS 2-17)	h / r =	87	, neglected conservativly flanges.
	F_s =	32.00	ksi, (TMS 2.3.2.1)	P_a =	2027.82	kips, (TMS 2.3.3.2.1)

(cont'd)

CHECK FLEXURAL & AXIAL CAPACITY BY STRENGTH DESIGN (SD)





P_u = 1.2 P = 360 kips

$M_u = (1/0.7)\,(M_x^2 + M_y^2)^{0.5}$ = 8885.9 ft-kips < ϕM_n = 10200.8 ft-kips, at P_u level.

[Satisfactory]

Where	ε_{mu} =	0.0025	, (TMS 3.3.2.c)	d =	363 in
	ϕ =	0.9	, (TMS 3.1.4.1)	f_m' =	3 ksi

CHECK SHEAR CAPACITY (ASD)

$$F_{v,\ \text{without reinf.}} = \begin{cases} (SF)\,MIN\left[\frac{1}{3}\left(4-\frac{M_T}{Vd}\right)\sqrt{f'_m}\ ,\ \left(80-\frac{45M_T}{Vd}\right)\right]\ ,\ for\ \frac{M_T}{Vd}<1.0 \\ (SF)\,MIN(\sqrt{f'_m}\ ,\ 35)\ ,\ for\ \frac{M_T}{Vd}\geq 1.0 \end{cases} = 47\ \text{psi} < 1.5\ f_v$$

(Shear reinf. reqd to carry full shear load.)
(factor 1.5 from TMS 402 1.17.3.2.6.1.2)

$$F_{v,\ \text{Maximum}} = \begin{cases} (SF)\,MIN\left[\frac{1}{2}\left(4-\frac{M_T}{Vd}\right)\sqrt{f'_m}\ ,\ \left(120-\frac{45M_T}{Vd}\right)\right]\ ,\ for\ \frac{M_T}{Vd}<1.0 \\ (SF)\,MIN(1.5\sqrt{f'_m}\ ,\ 75)\ ,\ for\ \frac{M_T}{Vd}\geq 1.0 \end{cases} = 100 > f_v = 73\ \text{psi}$$

[Satisfactory]

CHECK MINIMUM REINFORCEMENTS

$1.5\ \frac{V}{F_s d}$ =	0.31	in^2 / ft	<	$\frac{A_v}{s}$ =	0.33	in^2 / ft	[Satisfactory]	(TMS 402 1.17.3.2.6.1.2)
$A_{sh,min}$ =	0.137	in^2/ft	<	$A_{sh,actual}$ =	0.330	in^2/ft	[Satisfactory]	(TMS 1.17.3.2.6)
$S_{sh,max}$ =	24	in	>	$S_{sh,actual}$ =	16	in	[Satisfactory]	(TMS 1.17.3.2.6)
$A_{sv,min}$ =	0.064	in^2/ft	<	$A_{sv,actual}$ =	0.100	in^2/ft	[Satisfactory]	(TMS 1.17.3.2.6)
$S_{sv,max}$ =	24	in	>	$S_{sv,actual}$ =	24	in	[Satisfactory]	(TMS 1.17.3.2.6)
$A_{(sh+sv),min}$ =	0.003	in^2/in^2	<	$A_{(sh+sv),actual}$ =	0.005	in^2/in^2	[Satisfactory]	(CBC 10, 2106A.1.1.1.1.1)

CHECK MAXIMUM REINFORCEMENT PERCENTAGE

$$\rho_{max} = \frac{n f'_m}{2 f_y\left(n + \frac{f_y}{f'_m}\right)} = 0.0087 > \rho = 0.0003$$

[Satisfactory]

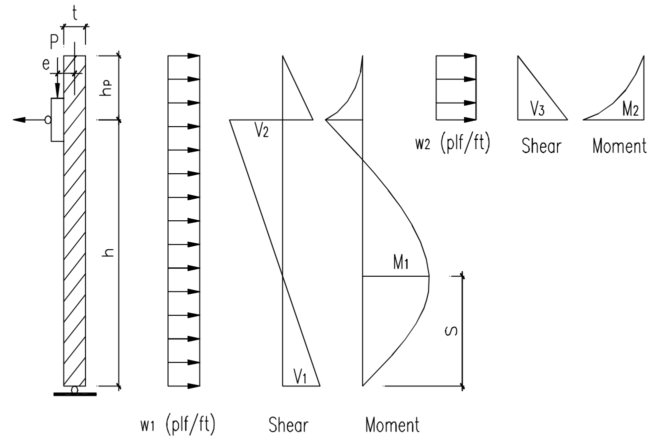
(TMS 402 2.3.3.4)

Daniel T. Li Engineering International	PROJECT :		PAGE :	
	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

Allowable Stress Design of Masonry Bearing Wall Based on CBC 10 Chapter A

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SERVICE GRAVITY LOAD	P	=	625	lbs / ft
SERVICE LATERAL LOAD	w_1	=	26.7	plf / ft
SERVICE PARAPET LOAD	w_2	=	80.2	plf / ft
THICKNESS OF WALL	t	=	8	in
PARAPET HEIGHT	h_p	=	2	ft
WALLHEIGHT	h	=	16	ft
ECCENTRICITY	e	=	6	in
MASONRY SPECIFIC WEIGHT	γ_m	=	130	pcf



[THE WALL DESIGN IS ADEQUATE.]

WALL HORIZ. REINF.	1	#	5	@	24	in o.c. (at middle)
WALL VERT. REINF.	1	#	5	@	24	in o.c. (at middle)

ANALYSIS

VERT. REINF. AREA AT EACH SIDE	A_s	=	0.16	in^2
EFFECTIVE DEPTH (ACI, 1.13.3.5)	d	=	3.82	in
WIDTH OF SECTION	b_w	=	12.00	in
EFFECTIVE THICKNESS	t_e	=	7.63	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48
REINFORCEMENT RATIO	ρ	=	0.0034
ALLOWABLE STRESS FACTOR	SF	=	1.333

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k=\sqrt{2\rho n+(\rho n)^2}-\rho n \quad = \quad 0.31554$$

THE ALLOWABLE STRESS DUE TO FLEXURE IS

$$F_b=(SF)(0.33f'_m)= \quad 660 \text{ psi}$$

THE ALLOWABLE REINF. STRESS DUE TO FLEXURE IS

$$F_S=(1.33\ \ or\ \ 1.0)(24000\ \ or\ \ 20000)= \quad 32000 \text{ psi}$$

THE DISTANCE FROM BOTTOM TO M_1 IS

$$S=h+h_p-\left[\frac{(h+h_p)^2}{2h}-\frac{Pe}{hw_1}\right] = 8.6 \text{ ft}$$

THE GOVERNING MOMENTS AND AXIAL FORCES ARE

$$M_1=\frac{1.05}{2w_1h^2}\left[Pe+\frac{w_1}{2}\left(h^2-h_p^2\right)\right]^2 = 1038 \text{ ft-lbs/ft}$$

$$P_1=P+(wall\ \ weight) = 1439 \text{ lbs / ft}$$

$$M_2=\frac{w_2h_p^2}{2} = 160 \text{ ft-lbs/ft}$$

$$P_2=P+(wall\ \ weight) = 798 \text{ lbs / ft}$$

THE GOVERNING SHEAR FORCES ARE

$$V_1=(h+h_p)w_1-\frac{(h+h_p)^2w_1}{2h}+\frac{Pe}{h} = 230 \text{ lbs / ft}$$

$$V_2=hw_1-V_1 = 197 \text{ lbs / ft}$$

$$V_3=h_pw_2 = 160 \text{ lbs / ft}$$

THE GOVERNING SHEAR STRESS IN MASONRY IS

$$f_v=\frac{MAX(V_1\ ,\ V_2\ ,\ V_3)}{t_eb_w} = 2.51 \text{ psi}$$

DETERMINE THE REGION FOR FLEXURE AND AXIAL LOAD (MDG Tab 12.2.1, Fig 12.2-12 & 13, page 12-25).

$$\frac{M}{Pd}\le\frac{t_e}{6d} \qquad \frac{M}{Pd}\le\left(\frac{t_e}{2d}-\frac{1}{3}\right) \qquad \frac{M}{Pd}>\left(\frac{t_e}{2d}-\frac{1}{3}\right)$$

1. Wall is in compression and not cracked.
2. Wall is cracked but steel is in compression.
3. Wall is cracked and steel is in tension.

REGION 3 APPLICABLE FOR (M1, P1)

REGION 2 APPLICABLE FOR (M2, P2)

(cont'd)

CHECK REGION 1 CAPACITY

$$M_m = \frac{b_w t_e^2}{6} F_b - P\frac{t_e}{6} = \begin{cases} 6251 \text{ ft-lbs / ft} & M_1 \quad \text{Not applicable} \\ 6319 \text{ ft-lbs / ft} & M_2 \quad \text{Not applicable} \end{cases}$$

CHECK REGION 2 CAPACITY

$$M_m = P\frac{t_e}{2} - \frac{2P^2}{3b_w F_b} = \begin{cases} 443 \text{ ft-lbs / ft} & M1 \quad \text{Not applicable} \\ 249 \text{ ft-lbs / ft} & M2 \quad \text{Satisfactory} \end{cases}$$

CHECK REGION 3 CAPACITY (The moment maybe limited by either the masonry compression or steel tension, MDG page 12-25).

$$M_m = MIN\left[\frac{1}{2} b_w kd\, F_b\left(d - \frac{kd}{3}\right) - P\left(d - \frac{t_e}{2}\right) \;,\; A_s F_s\left(d - \frac{kd}{3}\right) + P\left(\frac{t_e}{2} - \frac{kd}{3}\right)\right]$$

$$= \begin{cases} 1356 \text{ ft-lbs / ft} & M_1 \quad \text{Satisfactory} \\ 1356 \text{ ft-lbs / ft} & M_2 \quad \text{Not applicable} \end{cases}$$

THE ALLOWABLE SHEAR STRESS IS GIVEN BY

$$F_v = (SF)\,MIN\left(\sqrt{f'_m} \;,\; 50\right) \quad = \quad 51.64 \text{ psi} \qquad f \qquad \text{Satisfactory}$$

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Engineering International

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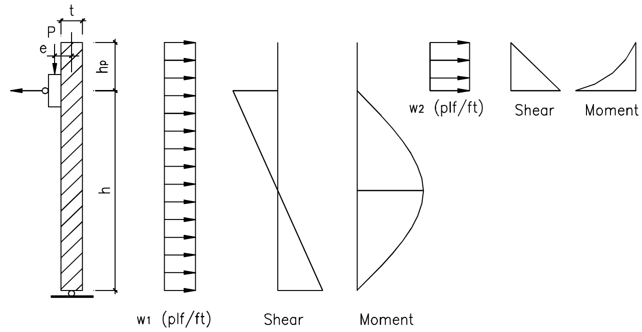
DESIGN BY :
REVIEW BY :

Strength Design of Masonry Bearing Wall Based on CBC 10 Chapter A

INPUT DATA & DESIGN SUMMARY



TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
SERVICE DEAD LOAD	P_{DL}	=	625	lbs / ft		
LATERAL LOAD (E/1.4 or W)	w_1	=	26.7	plf / ft		
LATERAL LOAD (E/1.4 or W)	w_2	=	80.2	plf / ft		
THICKNESS OF WALL	t	=	8	in		
PARAPET HEIGHT	h_p	=	2	ft		
WALL HEIGHT	h	=	24.17	ft		
ECCENTRICITY	e	=	6	in		
MASONRY SPECIFIC WEIGHT	γ_m	=	130	pcf		
WALL HORIZ. REINF.	2	#	5	@	16	in o.c. (at each face)
WALL VERT. REINF.	2	#	5	@	16	in o.c. (at each face)
SEISMIC PARAMETER	S_{DS}	=	1.246			

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

VERT. REINF. AREA AT EA. SIDE	A_s =	0.23	in^2/ft	EFFECTIVE THICKNESS	t_e =	7.63	in
EFFECTIVE DEPTH	d =	5.57	in	MASONRY ELASTICITY MODULUS	E_m =	1350	ksi
WIDTH OF SECTION	b_w =	12.00	in	STEEL ELASTICITY MODULUS	E_s =	29000	ksi
GROSS MOMENT OF INERTIA	I_g =	444	in^4/ft	MODULAR RATIO	n =	21.48	

CHECK REINFORCING RATIO (ACI 530-05 3.3.3.5, page CC-51)

$\rho = A_s / d\, b_w$ = 0.0035 < ρ_{MAX} = 0.0082

[Satisfactory]

$$\rho_{MAX} = \begin{cases} \dfrac{0.64 f'_m \left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \dfrac{P}{bd}}{f_y} & for\ \ bars\ \ middle \\[2ex] \dfrac{0.64 f'_m \left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \dfrac{P}{bd}}{f_y - \min\left\{\varepsilon_{mu} - \dfrac{d'}{d}\left(\varepsilon_{mu} + \alpha\varepsilon_y\right), \varepsilon_y\right\} E_s} & for\ \ bars\ \ each\ \ face \end{cases} = 0.0082$$

where ε_{mu} = 0.0025 , (ACI 530-05 3.3.2 c)

α = 1.5 , (ACI 530-05 3.3.3.5.1 a)

ε_y = f_y / E_s = 0.0021 , (ACI 530-05 3.3.3.5.1 a)

P = $D + 0.75\, L + 0.525\, Q_E$ = 2.63 kips/ ft, (ACI 530-05 3.3.3.5.1 d)

CHECK WALL AXIAL STRESS (ACI 530-05 3.3.5.4)

$1.2\,(P_w + P_f) / A_g$ = 24.2 psi < $0.05\, f_m'$ = 75 psi [Satisfactory]

where P_w = $(0.5\, h + h_p)\, \gamma_m\, t$ = 1221 lbs / ft , P_f = 625 lbs / ft

DETERMINE CRACKING MOMENT (ACI 530-05 Tab 3.1.8.2.1)

f_r = 150 psi, (ACI 530-05 Tab 3.1.8.2.1)

$M_{cr} = S\, f_r = (b_w\, t_e^2 / 6)\, f_r$ = 1455 ft-lbs/ft

CHECK CAPACITY OF LOAD COMBINATION (0.9 - 0.2S_{DS}) D + E_h , (IBC 06 1605.2.1 & ASCE 7-05 12.4.2)

$P_u = (0.9 - 0.2 S_{DS})\, (P_{DL} + P_W)$ = 1201 lbs/ft

DEPTH OF THE COMPRESSIVE STRESS BLOCK

$a = (P_u + A_s\, f_y) / (0.80\, f_m'\, b_w)$ = 1.05 in

DEPTH OF NEUTRAL AXIS

$c = a / 0.80$ = 1.32 in

EFFECTIVE AREA OF REINFORCING STEEL

$A_{se} = (P_u + A_s\, f_y) / f_y$ = 0.25 in^2/ft

CRACKED MOMENT OF INERTIA

$I_{cr} = n\, A_{se}(d-c)^2 + bc^3 / 3$ = 107 in^4/ft

THE MOMENT AND DEFLECTION AT THE MID-HEIGHT OF THE WALL ARE GIVEN BY

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_u = 5M_{cr}h^2/(48E_mI_g) + 5(M_u - M_{cr})h^2/(48E_mI_{cr})$ =	0		1.283	1.377	1.384	in
$M_u = w_u h^2/8 + P_{uf}\, e/2 + P_u \delta_u$ =	2870	> Mcr	2999	3008	3009	ft-lbs/ft
		[Satisfactory]	=> Eq (3-31) Applicable			

(cont'd)

CHECK MOMENT CAPACITY OF THE WALL (ACI 530-05 3.3.5)

$\phi M_n = \phi[A_{se}f_y(d-a/2) - P_u(d-t_e/2)] =$ 5571 ft-lbs/ft **>** M_u [Satisfactory]

$\phi M_n = \phi A_{se}f_y(d-a/2) =$ <= Not applicable

where ϕ = 0.9, (ACI 530-05 3.1.4.1)

CHECK DEFLECTION LIMITATION (ACI 530-05 3.3.5.5)

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_s = 5M_{cr}h^2/(48E_mI_g) + 5(M_{ser} - M_{cr})h^2/(48E_mI_{cr}) =$	0		0.728	0.787	0.792	in
$M_{ser} = wh^2/8 + P_f\ e/2 + P\delta_s =$	2106	> Mcr	2187	2193	2194	ft-lbs/ft
		[Satisfactory]	=> Eq (3-31) Applicable			

0.007 h = 2.03 in **>** δ_s [Satisfactory]

CHECK CAPACITY OF LOAD COMBINATION (1.2 + 0.2S_{DS}) D + E_h , (IBC 06 1605.2.1 & ASCE 7-05 12.4.2)

$P_u = (1.2+0.2S_{DS})(P_{DL} + P_w) =$ 2675 lbs/ft

DEPTH OF THE COMPRESSIVE STRESS BLOCK

$a = (P_u + A_s\ f_y) / (0.80\ f_m'\ b_w) =$ 1.15 in

DEPTH OF NEUTRAL AXIS

$c = a/ 0.80 =$ 1.44 in

EFFECTIVE AREA OF REINFORCING STEEL

$A_{se} = (P_u + A_s\ f_y) / f_y =$ 0.28 in^2/ft

CRACKED MOMENT OF INERTIA

$I_{cr} = n\ A_{se}(d-c)^2 + bc^3 / 3 =$ 104 in^4/ft

THE MOMENT AND DEFLECTION AT THE MID-HEIGHT OF THE WALL ARE GIVEN BY

$w_u = 1.4\ w_1 =$ 37.4 plf / ft

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_u = 5M_{cr}h^2/(48E_mI_g) + 5(M_u - M_{cr})h^2/(48E_mI_{cr}) =$	0		1.376	1.605	1.651	in
$M_u = w_uh^2/8 + P_{uf}\ e/2 + P_u\delta_u =$	2956	> Mcr	3263	3314	3324	ft-lbs/ft
		[Satisfactory]	=> Eq (3-31) Applicable			

CHECK MOMENT CAPACITY OF THE WALL (ACI 530-05 3.3.5)

$\phi M_n = \phi[A_{se}f_y(d-a/2) - P_u(d-t_e/2)] =$ 5871 ft-lbs/ft **>** M_u [Satisfactory]

$\phi M_n = \phi A_{se}f_y(d-a/2) =$ <= Not applicable

where ϕ = 0.9, (ACI 530-05 3.1.4.1)

CHECK DEFLECTION LIMITATION (ACI 530-05 3.3.5.5)

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_s = 5M_{cr}h^2/(48E_mI_g) + 5(M_{ser} - M_{cr})h^2/(48E_mI_{cr}) =$	0		0.728	0.787	0.792	in
$M_{ser} = wh^2/8 + P_f\ e/2 + P\delta_s =$	2106	> Mcr	2187	2193	2194	ft-lbs/ft
		[Satisfactory]	=> Eq (3-31) Applicable			

0.007 h = 2.03 in **>** δ_s [Satisfactory]

CHECK SHEAR CAPACITY (ACI 530-05 3.3.4.1.2.1)

$\phi V_n = \phi\ 2.25\ A_{mv}\ (f_m')^{0.5} =$ 6383 lbs/ft **>** $V_u = 1.4\ [w_1h/2 + w_2(h + 0.5h_P)h_P/h + P_{DL}e/h] =$ 704 lbs/ft

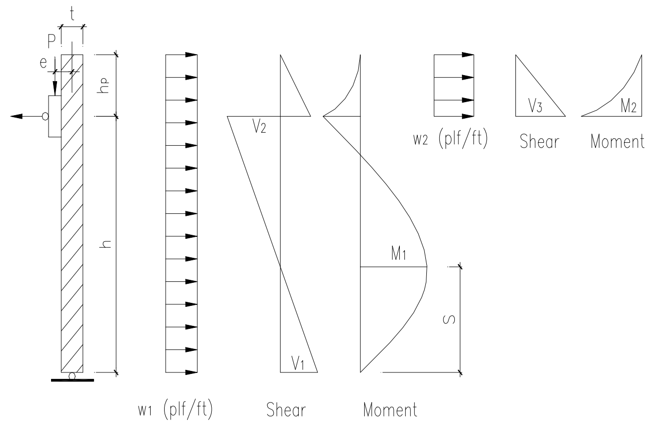
where ϕ = 0.8 [Satisfactory]

CHECK PARAPET BENDING CAPACITY

ϕM_n **>** M_u [Satisfactory]

Allowable Stress Design of Masonry Bearing Wall Based on TMS 402-08 / IBC 09

INPUT DATA & DESIGN SUMMARY



SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes		
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes			
SERVICE GRAVITY LOAD	P	=	625	lbs / ft		
SERVICE LATERAL LOAD	w_1	=	26.7	plf / ft		
SERVICE PARAPET LOAD	w_2	=	80.2	plf / ft		
THICKNESS OF WALL	t	=	8	in		
PARAPET HEIGHT	h_p	=	2	ft		
WALL HEIGHT	h	=	20	ft		
ECCENTRICITY	e	=	6	in		
MASONRY SPECIFIC WEIGHT	γ_m	=	130	pcf		
WALL HORIZ. REINF.	1	#	5	@	16	in o.c. (at middle)
WALL VERT. REINF.	1	#	5	@	16	in o.c. (at middle)

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

VERT. REINF. AREA AT EACH SIDE	A_s	=	0.23	in^2
EFFECTIVE DEPTH (TMS 1.15.3.5)	d	=	3.82	in
WIDTH OF SECTION	b_w	=	12.00	in
EFFECTIVE THICKNESS	t_e	=	7.63	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48
REINFORCEMENT RATIO	ρ	=	0.0051
ALLOWABLE STRESS FACTOR	SF	=	1.333

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k=\sqrt{2\rho n+(\rho n)^2}-\rho n \quad = 0.37059$$

THE ALLOWABLE STRESS DUE TO FLEXURE IS

$$F_b=(SF)\left(0.33 f'_m\right)= \quad 660 \text{ psi}$$

THE ALLOWABLE REINF. STRESS DUE TO FLEXURE IS

$$F_S=(1.33 \;\; or \;\; 1.0)(24000 \;\; or \;\; 20000)= \quad 32000 \text{ psi}$$

THE DISTANCE FROM BOTTOM TO M_1 IS

$$S=h+h_p-\left[\frac{(h+h_p)^2}{2h}-\frac{Pe}{hw_1}\right] \quad = 10.5 \text{ ft}$$

THE GOVERNING MOMENTS AND AXIAL FORCES ARE

$$M_1=\frac{1.05}{2w_1h^2}\left[Pe+\frac{w_1}{2}\left(h^2-h_p^2\right)\right]^2 \quad = 1541 \text{ ft-lbs/ft}$$

$$P_1=P+(wall \quad weight) \quad = 1623 \text{ lbs / ft}$$

$$M_2=\frac{w_2h_p^2}{2} \quad = 160 \text{ ft-lbs/ft}$$

$$P_2=P+(wall \quad weight) \quad = 798 \text{ lbs / ft}$$

THE GOVERNING SHEAR FORCES ARE

$$V_1=(h+h_p)w_1-\frac{(h+h_p)^2w_1}{2h}+\frac{Pe}{h} \quad = 280 \text{ lbs / ft}$$

$$V_2=hw_1-V_1 \quad = 254 \text{ lbs / ft}$$

$$V_3=h_pw_2 \quad = 160 \text{ lbs / ft}$$

THE GOVERNING SHEAR STRESS IN MASONRY IS

$$f_v=\frac{MAX\,(V_1 \;,\; V_2 \;,\; V_3)}{t_eb_w} \quad = 3.06 \text{ psi}$$

DETERMINE THE REGION FOR FLEXURE AND AXIAL LOAD (MDG-3 Tab 12.2.1, Fig 12.2-12 & 13, page 12-25).

$$\frac{M}{Pd}\le\frac{t_e}{6d}$$

1. Wall is in compression and not cracked.

$$\frac{M}{Pd}\le\left(\frac{t_e}{2d}-\frac{1}{3}\right)$$

2. Wall is cracked but steel is in compression.

$$\frac{M}{Pd}>\left(\frac{t_e}{2d}-\frac{1}{3}\right)$$

3. Wall is cracked and steel is in tension.

REGION 3 APPLICABLE FOR (M1, P1)

REGION 2 APPLICABLE FOR (M2, P2)

(cont'd)

CHECK REGION 1 CAPACITY

$$M_m = \frac{b_w t_e^2}{6} F_b - P\frac{t_e}{6} = \begin{cases} 6232 \text{ ft-lbs / ft} > M_1 & \text{[Not applicable]} \\ 6319 \text{ ft-lbs / ft} > M_2 & \text{[Not applicable]} \end{cases}$$

CHECK REGION 2 CAPACITY

$$M_m = P\frac{t_e}{2} - \frac{2P^2}{3b_w F_b} = \begin{cases} 497 \text{ ft-lbs / ft} < M1 & \text{[Not applicable]} \\ 249 \text{ ft-lbs / ft} > M2 & \text{[Satisfactory]} \end{cases}$$

CHECK REGION 3 CAPACITY (The moment maybe limited by either the masonry compression or steel tension, MDG-3 page 12-25).

$$M_m = MIN\left[\frac{1}{2}b_w kd\, F_b\left(d - \frac{kd}{3}\right) - P\left(d - \frac{t_e}{2}\right)\ ,\ A_s F_s\left(d - \frac{kd}{3}\right) + P\left(\frac{t_e}{2} - \frac{kd}{3}\right)\right]$$

$$= \begin{cases} 1560 \text{ ft-lbs / ft} > M_1 & \text{[Satisfactory]} \\ 1560 \text{ ft-lbs / ft} > M_2 & \text{[Not applicable]} \end{cases}$$

THE ALLOWABLE SHEAR STRESS IS GIVEN BY

$$F_v = (SF)\,MIN\left(\sqrt{f'_m}\ ,\ 50\right) = 51.64 \text{ psi} > f_v \quad \text{[Satisfactory]}$$

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

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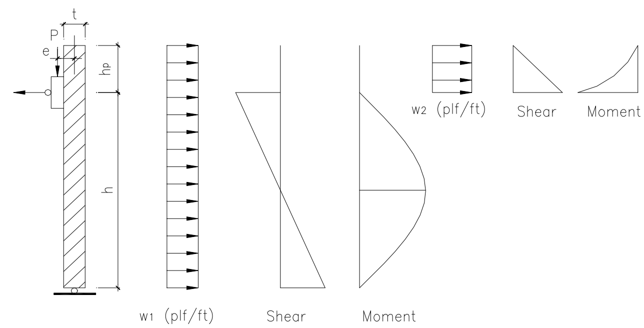
REVIEW BY :

Strength Design of Masonry Bearing Wall Based on TMS 402-08 / IBC 09

INPUT DATA & DESIGN SUMMARY

TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU		
MASONRY STRENGTH	f_m'	=	1.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
SERVICE DEAD LOAD	P_{DL}	=	625	lbs / ft		
LATERAL LOAD (E/1.4 or W)	w_1	=	26.7	plf / ft		
LATERAL LOAD (E/1.4 or W)	w_2	=	80.2	plf / ft		
THICKNESS OF WALL	t	=	8	in		
PARAPET HEIGHT	h_p	=	2	ft		
WALL HEIGHT	h	=	24.17	ft		
ECCENTRICITY	e	=	6	in		
MASONRY SPECIFIC WEIGHT	γ_m	=	130	pcf		
WALL HORIZ. REINF.	2	#	5	@	16	in o.c. (at each face)
WALL VERT. REINF.	2	#	5	@	16	in o.c. (at each face)
SEISMIC PARAMETER	S_{DS}	=	1.246			



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

VERT. REINF. AREA AT EA. SIDE	A_s	=	0.23	in^2/ft	EFFECTIVE THICKNESS	t_e =	7.63	in
EFFECTIVE DEPTH	d	=	5.57	in	MASONRY ELASTICITY MODULUS	E_m =	1350	ksi
WIDTH OF SECTION	b_w	=	12.00	in	STEEL ELASTICITY MODULUS	E_s =	29000	ksi
GROSS MOMENT OF INERTIA	I_g	=	444	in^4/ft	MODULAR RATIO	n =	21.48	

CHECK REINFORCING RATIO (TMS 402-08 3.3.3.5, page CC-61)

$\rho = A_s / d\, b_w = 0.0035$ <
$$\rho_{MAX} = \begin{cases} \dfrac{0.64 f'_m \left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha \varepsilon_y}\right) - \dfrac{P}{bd}}{f_y} & for\ \ bars\ \ middle \\ \dfrac{0.64 f'_m \left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha \varepsilon_y}\right) - \dfrac{P}{bd}}{f_y - \min\left\{\varepsilon_{mu} - \dfrac{d'}{d}\left(\varepsilon_{mu} + \alpha \varepsilon_y\right), \varepsilon_y\right\} E_s} & for\ \ bars\ \ each\ \ face \end{cases} = 0.0082$$

[Satisfactory]

where ε_{mu} = 0.0025 , (TMS 402-08 3.3.2 c)
α = 1.5 , (TMS 402-08 3.3.3.5.1 a)
ε_y **= f_y / E_s =** 0.0021 , (TMS 402-08 3.3.3.5.1 a)
P = D + 0.75 L + 0.525 Q_E = 2.63 kips/ ft, (TMS 402-08 3.3.3.5.1 d)

CHECK WALL AXIAL STRESS (TMS 402-08 3.3.5.3)

$1.2 (P_w + P_f) / A_g$ = 24.2 psi < $0.05 f_m'$ = 75 psi [Satisfactory]

where P_w = $(0.5\, h + h_p)\, \gamma_m\, t$ = 1221 lbs / ft , P_f = 625 lbs / ft

DETERMINE CRACKING MOMENT (TMS 402-08 Tab 3.1.8.2)

f_r = 150 psi, (TMS 402-08 Tab 3.1.8.2)

$M_{cr} = S\, f_r = (b_w\, t_e^2 / 6)\, f_r$ = 1455 ft-lbs/ft

CHECK CAPACITY OF LOAD COMBINATION (0.9 - 0.2S_{DS}) D + E_h , (IBC 09 1605.2.1 & ASCE 7-05 12.4.2)

$P_u = (0.9 - 0.2 S_{DS}) (P_{DL} + P_w)$ = 1201 lbs/ft

DEPTH OF THE COMPRESSIVE STRESS BLOCK

$a = (P_u + A_s\, f_y) / (0.80\, f_m'\, b_w)$ = 1.05 in

DEPTH OF NEUTRAL AXIS

$c = a / 0.80$ = 1.32 in

EFFECTIVE AREA OF REINFORCING STEEL

$A_{se} = (P_u + A_s\, f_y) / f_y$ = 0.25 in^2/ft

CRACKED MOMENT OF INERTIA

$I_{cr} = n\, A_{se}(d-c)^2 + b c^3 / 3$ = 107 in^4/ft

THE MOMENT AND DEFLECTION AT THE MID-HEIGHT OF THE WALL ARE GIVEN BY

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_u = 5M_{cr}h^2/(48E_mI_g) + 5(M_u - M_{cr})h^2/(48E_mI_{cr})$ =	0		1.283	1.377	1.384	in
$M_u = w_uh^2/8 + P_{uf}\, e/2 + P_u\delta_u$ =	2870	> Mcr	2999	3008	3009	ft-lbs/ft
		[Satisfactory]		=> Eq (3-32) Applicable		

(cont'd)

CHECK MOMENT CAPACITY OF THE WALL (TMS 402-08 3.3.5)

$\phi M_n = \phi[A_{se}f_y(d\text{-}a/2) - P_u(d\text{-}t_e/2)] =$ 5571 ft-lbs/ft $>$ M_u [Satisfactory]

$\phi M_n = \phi A_{se}f_y(d\text{-}a/2) =$ <= Not applicable

where f = 0.9, (TMS 402-08 3.1.4.1)

CHECK DEFLECTION LIMITATION (TMS 402-08 3.3.5.4)

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_s = 5M_{cr}h^2/(48E_mI_g) + 5(M_{ser} - M_{cr})h^2/(48E_mI_{cr}) =$	0		0.728	0.787	0.792	in
$M_{ser} = wh^2/8 + P_f\, e/2 + P\delta_s =$	2106	> Mcr	2187	2193	2194	ft-lbs/ft
		[Satisfactory]	=> Eq (3-32) Applicable			

0.007 h = 2.03 in $>$ δ_s [Satisfactory]

CHECK CAPACITY OF LOAD COMBINATION (1.2 + 0.2S_{DS}) D + E_h , (IBC 09 1605.2.1 & ASCE 7-05 12.4.2)

$P_u = (1.2+0.2S_{DS})(P_{DL} + P_W) =$ 2675 lbs/ft

DEPTH OF THE COMPRESSIVE STRESS BLOCK

$a = (P_u + A_s\, f_y) / (0.80\, f_m'\, b_w) =$ 1.15 in

DEPTH OF NEUTRAL AXIS

$c = a/ 0.80 =$ 1.44 in

EFFECTIVE AREA OF REINFORCING STEEL

$A_{se} = (P_u + A_s\, f_y) / f_y =$ 0.28 in^2/ft

CRACKED MOMENT OF INERTIA

$I_{cr} = n\, A_{se}(d\text{-}c)^2 + bc^3 / 3 =$ 104 in^4/ft

THE MOMENT AND DEFLECTION AT THE MID-HEIGHT OF THE WALL ARE GIVEN BY

$w_u = 1.4\, w_1 =$ 37.4 plf / ft

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_u = 5M_{cr}h^2/(48E_mI_g) + 5(M_u - M_{cr})h^2/(48E_mI_{cr}) =$	0		1.376	1.605	1.651	in
$M_u = w_uh^2/8 + P_{uf}\, e/2 + P_u\delta_u =$	2956	> Mcr	3263	3314	3324	ft-lbs/ft
		[Satisfactory]	=> Eq (3-32) Applicable			

CHECK MOMENT CAPACITY OF THE WALL (TMS 402-08 3.3.5)

$\phi M_n = \phi[A_{se}f_y(d\text{-}a/2) - P_u(d\text{-}t_e/2)] =$ 5871 ft-lbs/ft $>$ M_u [Satisfactory]

$\phi M_n = \phi A_{se}f_y(d\text{-}a/2) =$ <= Not applicable

where f = 0.9, (TMS 402-08 3.1.4.1)

CHECK DEFLECTION LIMITATION (TMS 402-08 3.3.5.4)

	1st Cycle		2nd Cycle	3rd Cycle	Final	
$\delta_s = 5M_{cr}h^2/(48E_mI_g) + 5(M_{ser} - M_{cr})h^2/(48E_mI_{cr}) =$	0		0.728	0.787	0.792	in
$M_{ser} = wh^2/8 + P_f\, e/2 + P\delta_s =$	2106	> Mcr	2187	2193	2194	ft-lbs/ft
		[Satisfactory]	=> Eq (3-32) Applicable			

0.007 h = 2.03 in $>$ δ_s [Satisfactory]

CHECK SHEAR CAPACITY (TMS 402-08 3.3.4.1.2.1)

$\phi V_n = \phi\, 2.25\, A_{mv}\, (f_m')^{0.5} =$ 6383 lbs/ft $>$ $V_u = 1.4\,[w_1h/2 + w_2(h + 0.5h_P)h_P/h + P_{DL}e/h] =$ 704 lbs/ft

where $\phi = 0.8$ [Satisfactory]

CHECK PARAPET BENDING CAPACITY

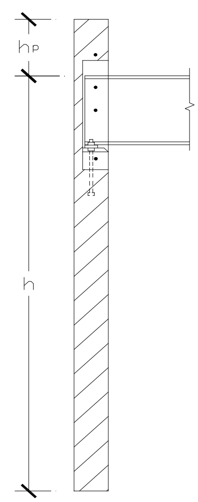
ϕM_n $>$ M_u [Satisfactory]

Design for Girder at Masonry Wall Based on TMS 402-08

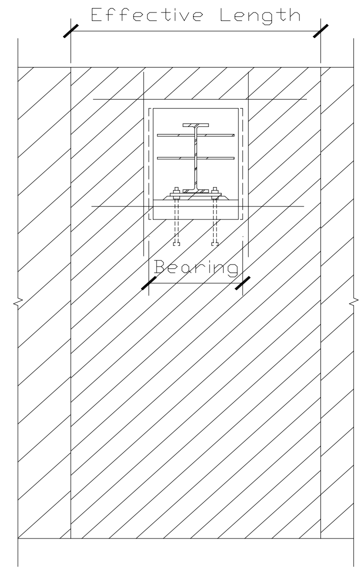
INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
GIRDER SERVICE LOAD	P_G	=	32	kips
ECCENTRICITY	e	=	3	in
BEARING LENGTH	L_{br}	=	20	in
SERVICE LATERAL LOAD	w_1	=	25	psf
SERVICE PARAPET LOAD	w_2	=	45	psf
THICKNESS OF WALL	t	=	8	in
PARAPET HEIGHT	h_p	=	2	ft
WALL HEIGHT	h	=	15	ft

WALL VERT. REINF. (A_{sv}) 2 # 7
@ 16 in o.c. (at each face)

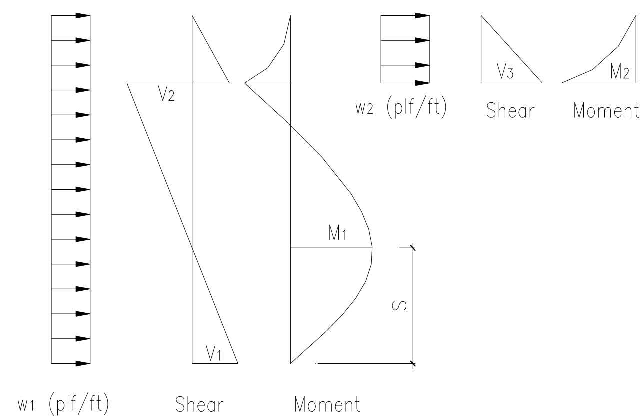


SECTION OF GIRDER AT MASONRY



POCKET ELEVATION

[THE WALL DESIGN IS ADEQUATE.]



ANALYSIS

EFFECTIVE THICKNESS t_e = 7.63 in

EFFECTIVE LENGTH (TMS 402, 1.9.5)

$L_e = 2\,L_{br}$ = 40.00 in

LOAD DISTRIBUTION (TMS 402 1.9.7)

$P = P_G / (0.25\ h)$ = 8533 lbs / ft

CHECK BEARING CAPACITY (TMS 402, 2.1.8)

$f_{br} = P_G / (t_e\ L_{br})$ = 210 psi

< $0.25\ f_m'$ = 375 psi

[Satisfactory]

REINF. AREA AT EACH SIDE	A_s	=	0.45	in^2
EFFECTIVE DEPTH (TMS, 1.15.3.5)	d	=	5.44	in
WIDTH OF SECTION	b_w	=	12.00	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi
MODULAR RATIO	n	=	21.48	

REINFORCEMENT RATIO ρ = 0.0069

ALLOWABLE STRESS FACTOR SF = 1.333

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \quad = 0.41584$$

THE ALLOWABLE STRESS DUE TO FLEXURE IS

$$F_b = (SF)(0.33 f'_m) = \quad 660 \text{ psi}$$

THE ALLOWABLE REINF. STRESS DUE TO FLEXURE IS

$$F_S = (1.33\ \ or\ \ 1.0)(20000\ \ or\ \ 24000) = \quad 32000 \text{ psi}$$

THE DISTANCE FROM BOTTOM TO M_1 IS

$$S = h + h_p - \left[\frac{(h+h_p)^2}{2h} - \frac{Pe}{h w_1} \right] \quad = 13.1 \text{ ft}$$

THE GOVERNING MOMENTS AND AXIAL FORCES ARE

$$M_1 = \frac{1}{2 w_1 h^2} \left[Pe + \frac{w_1}{2}\left(h^2 - h_p^2\right) \right]^2 \quad = 2131 \text{ ft-lbs/ft}$$

$$P_1 = P + (wall\quad weight) \quad = 8875 \text{ lbs / ft}$$

(cont'd)

THE GOVERNING SHEAR FORCES ARE

$$V_1 = \left(h + h_p\right) w_1 - \frac{\left(h+h_p\right)^2 w_1}{2h} + \frac{Pe}{h}$$ = 326 lbs / ft

$$M_2 = \frac{w_2 h_p^2}{2}$$ = 90 ft-lbs/ft

$$P_2 = P + \left(wall \quad weight\right)$$ = 8707 lbs / ft

$$V_2 = h w_1 - V_1$$ = 49 lbs / ft

$$V_3 = h_p w_2$$ = 90 lbs / ft

THE GOVERNING SHEAR STRESS IN MASONRY IS

$$f_v = \frac{MAX\left(V_1 \ , \ V_2 \ , \ V_3\right)}{t_e b_w}$$ = 3.56 psi

DETERMINE THE REGION FOR FLEXURE AND AXIAL LOAD (MDG-3 Tab 12.2.1, Fig 12.2-12 & 13, page 12-25).

$$\frac{M}{Pd} \le \frac{t_e}{6d}$$

1. Wall is in compression and not cracked.

$$\frac{M}{Pd} \le \left(\frac{t_e}{2d} - \frac{1}{3}\right)$$

2. Wall is cracked but steel is in compression.

$$\frac{M}{Pd} > \left(\frac{t_e}{2d} - \frac{1}{3}\right)$$

3. Wall is cracked and steel is in tension.

REGION 3 APPLICABLE FOR (M1, P1)

REGION 1 APPLICABLE FOR (M2, P2)

CHECK REGION 1 CAPACITY

$$M_m = \frac{b_w t_e^2}{6} F_b - P\frac{t_e}{6} = \begin{cases} 5463 \text{ ft-lbs / ft} > M_1 \\ 5481 \text{ ft-lbs / ft} > M_2 \end{cases}$$

5463 ft-lbs / ft > M_1 **[Not applicable]**

5481 ft-lbs / ft > M_2 **[Satisfactory]**

CHECK REGION 2 CAPACITY

$$M_m = P\frac{t_e}{2} - \frac{2P^2}{3 b_w F_b} = \begin{cases} 2269 \text{ ft-lbs / ft} > M1 \\ 2236 \text{ ft-lbs / ft} > M2 \end{cases}$$

2269 ft-lbs / ft > M1 **[Not applicable]**

2236 ft-lbs / ft > M2 **[Not applicable]**

CHECK REGION 3 CAPACITY (The moment maybe limited by either the masonry compression or steel tension, MDG-3 page 12-25).

$$M_m = MIN\left[\frac{1}{2} b_w kd F_b\left(d - \frac{kd}{3}\right) - P\left(d - \frac{t_e}{2}\right) \ , \ A_s F_s\left(d - \frac{kd}{3}\right) + P\left(\frac{t_e}{2} - \frac{kd}{3}\right)\right]$$

$$= \begin{cases} 2298 \text{ ft-lbs / ft} > M_1 \\ 2321 \text{ ft-lbs / ft} > M_2 \end{cases}$$

2298 ft-lbs / ft > M_1 **[Satisfactory]**

2321 ft-lbs / ft > M_2 **[Not applicable]**

CHECK ALLOWABLE SHEAR STRESS (TMS 402 2.3.5.2.2)

$$F_v = \left(SF\right) MIN\left(\sqrt{f'_m} \ , \ 50\right)$$ = 51.64 psi > f_v **[Satisfactory]**

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

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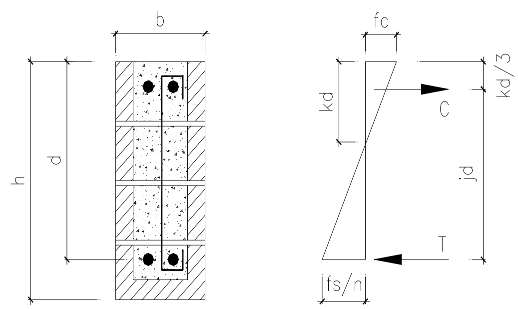
Masonry Beam Design Based on TMS 402-08

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			0	No, (reduced fm' by 0.5)	
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU	
MASONRY STRENGTH	f_m'	=	1.5	ksi	
REBAR YIELD STRESS	f_y	=	60	ksi	
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes		
SERVICE SHEAR LOAD	V	=	4.56	k	
SERVICE MOMENT LOAD	M	=	13.68	ft-k	
WIDTH	b	=	8	in	
EFFECTIVE DEPTH	d	=	45	in	
CLEAR SPAN	Lc	=	12	ft	
LOAD TYPE (1=SEISMIC, 0=WIND, 5=GRAVITY)			1	Seismic	
VERTICAL REINF. 1	#	4	@	8	in o.c.
TENSION REINFORCEMENT		2	#	6	



[THE BEAM DESIGN IS ADEQUATE.]

ANALYSIS

ALLOWABLE STRESS FACTOR	SF	=	0.667		
ALLOWABLE REINF. STRESS	(1.33 or 1.0) F_s	=	32	ksi	
ALLOWABLE MASONRY STRESS	F_b=(SF)(0.33f_m')	=	0.33	ksi	
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi, (TMS 402 1.8.2.2.1)	
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi, (TMS 402 1.8.2.1)	
EFFECTIVE WIDTH	b_w	=	7.63	in	[Satisfactory, Lc < 32 bw]
MODULAR RATIO	n	=	21.48		
TENSION REINFORCEMENT RATIO	ρ	=	0.003		

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k=\sqrt{2\rho n+(\rho n)^2}-\rho n \quad = \quad 0.281$$

THE LEVER-ARM FACTOR IS

$$j=1-\frac{k}{3} \quad = \quad 0.906$$

THE TENSILE STRESS IN REINFORCEMENT DUE TO FLEXURE IS

$$f_s=\frac{M}{A_s jd} \quad = \quad 4.57 \text{ ksi} \quad < \quad F_s$$

[SATISFACTORY]

THE COMPRESSIVE STRESS IN THE EXTREME FIBER DUE TO FLEXURE IS

$$f_b=\frac{2M}{jkb_w d^2} \quad = \quad 0.08 \text{ ksi} \quad < \quad F_b$$

[SATISFACTORY]

THE SHEAR STRESS IN MASONRY IS

$$f_v=\frac{V}{b_w d} \quad = \quad 13 \text{ psi} \begin{cases} < F_v=(SF)MIN(\sqrt{f'_m}\ ,\ 50)= 25.82 \text{ psi} \\ \text{[SATISFACTORY]} \\ < F_v=(SF)MIN(3\sqrt{f'_m}\ ,\ 150)= 77.46 \text{ psi} \end{cases}$$

(Sec. 2.3.5.2.1)

[SATISFACTORY]

CHECK THE MINIMUM AREA OF SHEAR REINFORCEMENT REQUIRED :

$$\frac{V}{F_s d}= \quad 0.04 \text{ in}^2\text{ / ft} \quad < \quad \frac{A_v}{s}= \quad 0.30 \text{ in}^2\text{ / ft}$$

(No shear reinf. Reqd)

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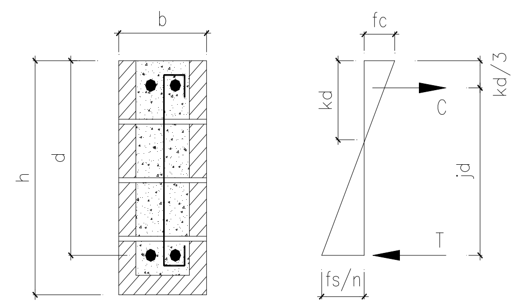
Masonry Beam Design Based on UBC 97

INPUT DATA & DESIGN SUMMARY



SPECIAL INSPECTION (0=NO, 1=YES)				1	Yes, (Sec. 2107.1.2)	
TYPE OF MASONRY (1=CMU, 2=BRICK)				1	CMU	
MASONRY STRENGTH		f_m'	=	1.5	ksi	
REBAR YIELD STRESS		f_y	=	60	ksi	
ALLOWABLE INCREASING ? (UBC/CBC 1612.3.2)				Yes		
SERVICE SHEAR LOAD		V	=	15.1	k	
SERVICE MOMENT LOAD		M	=	83	ft-k	
WIDTH		b	=	12	in	
EFFECTIVE DEPTH		d	=	40	in	
CLEAR SPAN		Lc	=	12	ft	
LOAD TYPE (1=SEISMIC, 0=WIND, 5=GRAVITY)				5	Gravity Only	
VERTICAL REINF.	1	#	4	@	8	in o.c.
TENSION REINFORCEMENT			2	#	7	

[THE BEAM DESIGN IS ADEQUATE.]

ANALYSIS

ALLOWABLE STRESS FACTOR	SF	=	1.333		
ALLOWABLE REINF. STRESS	(1.33 or 1.0) F_s	=	32	ksi	
ALLOWABLE MASONRY STRESS	F_b=(SF)(0.33f_m')	=	0.66	ksi	
MASONRY ELASTICITY MODULUS	E_m	=	750 fm' = 1125		ksi, (Eq. 6-4. 2106.2.12.1)
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi	
EFFECTIVE WIDTH	b_w	=	11.63	in	[Satisfactory, Lc < 32 bw]
MODULAR RATIO	n	=	25.78		
TENSION REINFORCEMENT RATIO	ρ	=	0.0026		

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k=\sqrt{2\rho n+(\rho n)^2}-\rho n \quad = \quad 0.304$$

THE LEVER-ARM FACTOR IS

$$j=1-\frac{k}{3} \quad = \quad 0.899$$

THE TENSILE STRESS IN REINFORCEMENT DUE TO FLEXURE IS

$$f_s=\frac{M}{A_s jd} \quad = \quad 23.09 \text{ ksi} \quad < \quad F_s$$

[SATISFACTORY]

THE COMPRESSIVE STRESS IN THE EXTREME FIBER DUE TO FLEXURE IS

$$f_b=\frac{2M}{jkb_w d^2} \quad = \quad 0.39 \text{ ksi} \quad < \quad F_b$$

[SATISFACTORY]

THE SHEAR STRESS IN MASONRY IS

$$f_v=\frac{V}{jb_w d} \quad = \quad 36 \text{ psi} \quad \begin{cases} < F_v=(SF)MIN(\sqrt{f'_m}\ ,\ 50)= 51.64 \text{ psi} \\ < F_v=(SF)MIN(3\sqrt{f'_m}\ ,\ 150)= 154.92 \text{ psi} \end{cases}$$

(Sec. 2107.2.17)

[SATISFACTORY]

[SATISFACTORY]

CHECK THE MINIMUM AREA OF SHEAR REINFORCEMENT REQUIRED :

$$\frac{V}{F_s d}= 0.14 \text{ in}^2\text{/ ft} \quad < \quad \frac{A_v}{s}= 0.30 \text{ in}^2\text{/ ft}$$

(No shear reinf. Reqd)

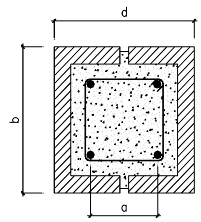
Daniel T. Li
Engineering International

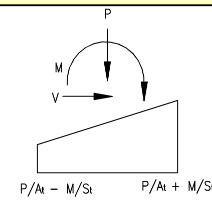
PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Masonry Column Design Based on CBC 10 Chapter A

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			0	No, (reduced fm' by 0.5)	
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU	
MASONRY STRENGTH	f_m'	=	1.5	ksi	
REBAR YIELD STRESS	f_y	=	60	ksi	
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes		
SEISMIC DESIGN CATEGORY (5=Gravity)			3	D	
SERVICE AXIAL LOAD	P	=	50	k	
SERVICE SHEAR LOAD	V	=	9.5	k	
MOMENT AT MIDHEIGHT	M	=	8.2	ft-k	
EFFECTIVE WIDTH	b	=	15.63	in	
EFFECTIVE DEPTH	d	=	15.63	in	
EFFECTIVE HEIGHT	h	=	15	ft	
VERTICAL REINF. (EACH SIDE)		2	#	8	
HORIZ. TIES	2 legs	# 4	@	8	in o.c.





DISTANCE BETWEEN COL. REINF. a = 11.13 in
(ACI 530, 1.13.3.5)

[THE COLUMN DESIGN IS ADEQUATE.]

ANALYSIS

TOTAL REINFORCEMENT AREA	A_s	=	3.16	in^2
EFFECTIVE COLUMN AREA	A_n	=	244	in^2
NET EFFECTIVE MOMENT OF INERT	I_n	=	4973	in^4
RADIUS OF GYRATION	r	=	4.51	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48	
REINFORCEMENT RATIO	ρ	=	0.013	
ALLOWABLE STRESS FACTOR	SF	=	0.667	
MAX. TIES SPACING (2106A.5.3.2)	S_{max}	=	8	in

TRANSFORMED COLUMN AREA

$$A_t = A_n\left(1+\left(2n-1\right)\rho\right) = \quad 377 \ in^2$$

CHECK VERTICAL REINFORCEMENT LIMITATION (ACI 530, 2.1.6.4)

A_s = 3.16 in^2 > $0.0025A_n$ = 0.61 in^2 **[Satisfactory]**

< $0.04A_n$ = 9.77 in^2 **[Satisfactory]**

ALLOWABLE STRESS DUE TO AXIAL LOAD ONLY

$$F_a = \left(SF\right)\left(0.25 f'_m\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 0.230 \ \text{ksi}$$

[for h/r < 99]

AXIAL STRESS AT MIDHEIGHT OF THE COLUMN

$$f_a = \frac{P+\left(half \quad col. \quad weight\right)}{A_t} = \quad 0.137 \ \text{ksi}$$

< Fa, [Satisfactory]

ALLOWABLE STRESS DUE TO FLEXURE

$$F_b = \left(SF\right)\left(0.33 f'_m\right) = \quad 0.330 \ \text{ksi}$$

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = \left(1.33 \quad or \quad 1.0\right)\left(20 \quad or \quad 24\right) = \quad 32.0 \ \text{ksi}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_T = M + \left(0.1\right)\left(\frac{Pd}{2}\right) = \quad 11.5 \ \text{ft-kips}$$

TRANSFORMED MOMENT OF INERTIA

$$I_t = I_n + \left(2n-1\right)A_s\left(\frac{a}{2}\right)^2 = \quad 9080 \ in^4$$

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = \frac{M_T d}{2I_t} = \quad 0.118 \ \text{ksi}$$

< fa, [Satisfactory, the section is uncracked]

MAX.STRESS COMBINED AXIAL & FLEXURE

$$f_m = f_a + f_b = \quad 0.256 \ \text{ksi}$$

< Fb, [Satisfactory]

MAX. REINF. STRESS COMBINED AXIAL & FLEXURE

$$f_S = 2n\left(f_a + \frac{af_b}{d}\right) = \quad 9.5 \ \text{ksi}$$

< Fs, [Satisfactory]

AXIAL LOAD AT BASE OF THE COLUMN

$$P_t = P + \left(full \quad col. \quad weight\right) = \quad 53.435 \ \text{k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(\left(SF\right)0.25 f'_m A_n + 0.65 F_S A_S\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 116.51 \ \text{k}$$

[for h/r < 99] **> Pt, [Satisfactory]**

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$$f_v = \frac{V}{bd} = \quad 39 \ \text{psi}$$

(Sec. 2.3.5.2.1)

> $F_v = \left(SF\right)MIN\left(\sqrt{f'_m} \ , \ 50\right) =$ 25.82 psi

(Shear reinf. reqd to carry full shear load.)

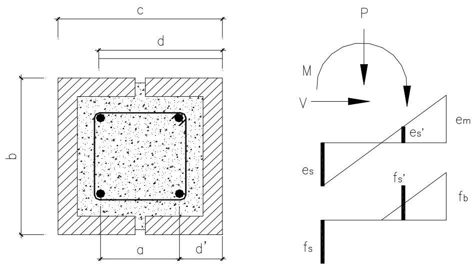
< $F_v = \left(SF\right)MIN\left(3\sqrt{f'_m} \ , \ 150\right) =$ 77.46 psi **[Satisfactory]**

$\frac{V}{F_s d} =$ 0.23 in^2 / ft < $\frac{A_v}{s} =$ 0.60 in^2 / ft **[Satisfactory]**

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

Masonry Column Design Based on CBC 10 Chapter A



INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SEISMIC DESIGN CATEGORY			4	E or F
(1 = B, 2 = C, 3 = D, 4 = E or F, 0 = WIND, 5 = GRAVITY)				
SERVICE AXIAL LOAD	P	=	11.5	k, @ top of col.
MAX SHEAR LOAD	V	=	20	k
MOMENT AT MIDHEIGHT	M	=	106	ft-k, @ mid of col
EFFECTIVE WIDTH	b	=	23.63	in
EFFECTIVE DEPTH	c	=	23.63	in
EFFECTIVE HEIGHT	h	=	29	ft

VERTICAL REINF. (EACH SIDE) 3 # 8

HORIZ. TIES 2 legs # 4 @ 8 in o.c.

DISTANCE BETWEEN COL. REINF. a = 19.13 in
(ACI 530, 1.13.3.5)

[THE COLUMN DESIGN IS ADEQUATE.]

ANALYSIS

REINFORCEMENT AREA AT ONE SIDE	A_s	=	2.37	in^2
EFFECTIVE COLUMN AREA	A_n	=	558	in^2
NET EFFECTIVE MOMENT OF INERTIA	I_n	=	25982	in^4
RADIUS OF GYRATION	r	=	6.82	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi
MODULAR RATIO	n	=	21.48	
TRANSFORMED AREA	$A_t = A_n(1-\rho+n\rho+n\rho')$	=	668	in^2

REINFORCEMENT RATIO	$\rho = \rho'$	=	0.005	
DISTANCE	d' = 2.25 , d =		21.38	in
ALLOWABLE STRESS FACTOR	SF	=	1.333	
MAX. TIES SPACING (2106A.5.3.2)	S_{max}	=	8	in

NEUTRAL AXIS DEPTH FACTOR

$k = \{[n\rho+(2n-1)\rho']^2+2[n\rho+(2n-1)\rho(d'/d)]\}^{0.5} - [n\rho+(2n-1)\rho']$ = 0.278

LEVER-ARM FACTOR $j = 1-k/3$ = 0.907

CHECK VERTICAL REINFORCEMENT LIMITATION (ACI 530, 2.1.6.4)

$A_{s,total}$ = 4.74 in^2 > $0.005A_e = 0.005bd$ = 2.53 in^2 **[Satisfactory]**

< $0.04A_e = 0.04bd$ = 20.21 in^2 **[Satisfactory]**

AXIAL LOAD AT MIDDLE OF THE COLUMN

$$P_{Mid} = P + (half\quad col.\quad weight) =$$ 19.090 k

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_{Mid} = M + (0.1)\left(\frac{Pd}{2}\right) =$$ 107.1 ft-kips

CHECK IF THERE IS TENSILE STRESS IN CROSS SECTION

P_{Mid} / A = 34 psi < $M_{Mid} / (bc^2/6)$ = 585 psi
(tensile exist)

AXIAL LOAD AT BASE OF THE COLUMN

$$P_t = P + (full\quad col.\quad weight) =$$ 26.681 k

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(0.25 f'_m A_n + 0.65 F_S A_{st}\right)\left(1.0 - \left(\frac{h}{140r}\right)^2\right) =$$ 245.71 k **> Pt, [Satisfactory]**

[for h/r < 99]

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = \left(1.33_{wind\ \&\ seismic\ only}\right)(20\quad or\quad 24) =$$ 32.0 ksi

ALLOWABLE STRESS DUE TO FLEXURE

$$F'_b = \left(SF - \frac{P_{Mid}}{P_a}\right)\left(0.33 f'_m\right) =$$ 0.622 ksi

THE CORRESPONDING STRAIN IN THE TENSILE BARS IS

$$e_S = \frac{F_S}{E_S} =$$ 0.0011

THE STRAIN IN THE EXTREME COMPRESSION FIBER IS

$$e_m = MIN\left[\left(\frac{kd}{d-kd}\right)e_s\ ,\ \frac{F'_b}{E_m}\right] =$$ 0.0004253
(steel governs)

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = e_m E_m =$$ 0.574 ksi **< Fb', [Satisfactory]**

MOMENT DUE TO THE MASONRY

$$M_m = \frac{1}{2} bkd\, f_b\left(d - \frac{kd}{3}\right) =$$ 65.21 ft-kips

STRAIN IN THE COMPRESSION BARS

$$e'_S = \left(\frac{kd - d'}{kd}\right)e_m =$$ 0.0002644

STRESS IN THE COMPRESSION BARS

$$f'_S = 2E_S e'_S =$$ 15.334 ksi

(cont'd)

MOMENT DUE TO THE COMPRESSION BARS

$$M'_S = f'_S A_S (d - d') = \quad 57.94 \text{ ft-kips}$$

ALLOWABLE BENDING MOMENT

$$M = M'_S + M_m = \quad 123.15 \text{ ft-kips}$$

> Mt, [Satisfactory]

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$$f_v = \frac{V}{jbd} = \quad 39 \text{ psi} \quad \begin{cases} < \quad F_v = (SF)\,MIN(\sqrt{f'_m}\;,\; 50) = \quad 51.64 \text{ psi} \\ \quad\quad \textbf{[Satisfactory]} \\ < \quad F_v = (SF)\,MIN(3\sqrt{f'_m}\;,\; 150) = \quad 154.92 \text{ psi} \quad \textbf{[Satisfactory]} \end{cases}$$

(Sec. 2107.2.17)

$$\frac{V}{F_s d} = \quad 0.32 \text{ in}^2 / \text{ft} \quad < \quad \frac{A_v}{s} = \quad 0.60 \text{ in}^2 / \text{ft}$$

(No shear reinf. Reqd)

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

Daniel T. Li
Engineering International

PROJECT :			PAGE :	
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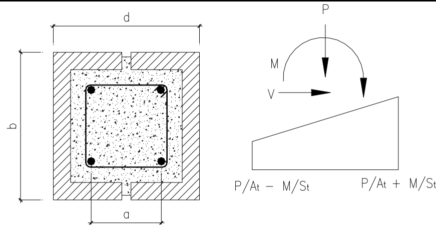
JOB NO. :		DATE :	REVIEW BY :	

Masonry Column Design Based on TMS 402-08 / IBC 09

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			0	No, (reduced fm' by 0.5)
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SEISMIC DESIGN CATEGORY (5=Gravity)			2	C
SERVICE AXIAL LOAD	P	=	50	k
SERVICE SHEAR LOAD	V	=	9.5	k
MOMENT AT MIDHEIGHT	M	=	8.2	ft-k
EFFECTIVE WIDTH	b	=	15.63	in
EFFECTIVE DEPTH	d	=	15.63	in
EFFECTIVE HEIGHT	h	=	15	ft
VERTICAL REINF. (EACH SIDE)		2	#	6
HORIZ. TIES	2 legs #	4	@	8 in o.c.



DISTANCE BETWEEN COL. REINF. a = 11.38 in
(TMS 402, 1.15.3.5)

[THE COLUMN DESIGN IS ADEQUATE.]

ANALYSIS

TOTAL REINFORCEMENT AREA	A_s	=	1.76	in^2
EFFECTIVE COLUMN AREA	A_n	=	244	in^2
NET EFFECTIVE MOMENT OF INERT	I_n	=	4973	in^4
RADIUS OF GYRATION	r	=	4.51	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48	
REINFORCEMENT RATIO	ρ	=	0.007	
ALLOWABLE STRESS FACTOR	SF	=	0.667	
MAX. TIES SPACING (1.17.4)	S_{max}	=	16	in

TRANSFORMED COLUMN AREA

$$A_t = A_n\left(1+\left(2n-1\right)\rho\right) = \quad 318 \text{ in}^2$$

CHECK VERTICAL REINFORCEMENT LIMITATION (TMS 402 1.14.1.2)

A_s = 1.76 in^2 > $0.0025A_n$ = 0.61 in^2 **[Satisfactory]**

< $0.04A_n$ = 9.77 in^2 **[Satisfactory]**

ALLOWABLE STRESS DUE TO AXIAL LOAD ONLY

$$F_a = \left(SF\right)\left(0.25 f'_m\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 0.230 \text{ ksi}$$

[for h/r < 99]

AXIAL STRESS AT MIDHEIGHT OF THE COLUMN

$$f_a = \frac{P+\left(half\quad col.\quad weight\right)}{A_t} = \quad 0.163 \text{ ksi}$$

< Fa, [Satisfactory]

ALLOWABLE STRESS DUE TO FLEXURE

$$F_b = \left(SF\right)\left(0.33 f'_m\right) = \quad 0.330 \text{ ksi}$$

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = \left(1.33 \quad or \quad 1.0\right)\left(20 \quad or \quad 24\right) = \quad 32.0 \text{ ksi}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_T = M + \left(0.1\right)\left(\frac{Pd}{2}\right) = \quad 11.5 \text{ ft-kips}$$

TRANSFORMED MOMENT OF INERTIA

$$I_t = I_n + \left(2n-1\right)A_s\left(\frac{a}{2}\right)^2 = \quad 7365 \text{ in}^4$$

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = \frac{M_T d}{2 I_t} = \quad 0.146 \text{ ksi}$$

< fa, [Satisfactory, the section is uncracked]

MAX.STRESS COMBINED AXIAL & FLEXURE

$$f_m = f_a + f_b = \quad 0.308 \text{ ksi}$$

< Fb, [Satisfactory]

MAX. REINF. STRESS COMBINED AXIAL & FLEXURE

$$f_S = 2n\left(f_a + \frac{a f_b}{d}\right) = \quad 11.5 \text{ ksi}$$

< Fs, [Satisfactory]

AXIAL LOAD AT BASE OF THE COLUMN

$$P_t = P + \left(full \quad col. \quad weight\right) = \quad 53.435 \text{ k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(\left(SF\right)0.25 f'_m A_n + 0.65 F_S A_S\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 89.75 \text{ k}$$

[for h/r < 99] **> Pt, [Satisfactory]**

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$$f_v = \frac{V}{bd} = \quad 39 \text{ psi}$$

(TMS 402 2.3.5.2.1)

> $F_v = \left(SF\right)MIN\left(\sqrt{f'_m}\ ,\ 50\right) =$ 25.82 psi

(Shear reinf. reqd to carry full shear load.)

< $F_v = \left(SF\right)MIN\left(3\sqrt{f'_m}\ ,\ 150\right) =$ 77.46 psi **[Satisfactory]**

$$\frac{V}{F_s d} = \quad 0.23 \text{ in}^2\text{ / ft} \quad < \quad \frac{A_v}{s} = \quad 0.60 \text{ in}^2\text{ / ft}$$

[Satisfactory]

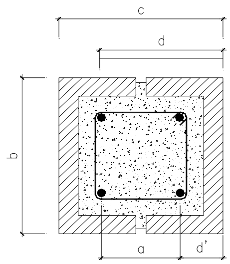
Technical References:

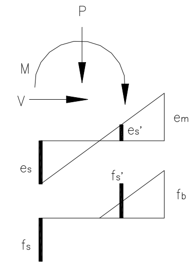
1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

Daniel T. Li
Engineering International

PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Masonry Column Design Based on TMS 402-08 / IBC 09





INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SEISMIC DESIGN CATEGORY			4	E or F
(1 = B, 2 = C, 3 = D, 4 = E or F, 0 = WIND, 5 = GRAVITY)				
SERVICE AXIAL LOAD	P	=	11.5	k, @ top of col.
MAX SHEAR LOAD	V	=	20	k
MOMENT AT MIDHEIGHT	M	=	106	ft-k, @ mid of col
EFFECTIVE WIDTH	b	=	23.63	in
EFFECTIVE DEPTH	c	=	23.63	in
EFFECTIVE HEIGHT	h	=	29	ft
VERTICAL REINF. (EACH SIDE)	3	#	8	
HORIZ. TIES	2 legs # 4	@	8	in o.c.

DISTANCE BETWEEN COL. REINF. a = 19.13 in
(TMS 402, 1.15.3.5)

[THE COLUMN DESIGN IS ADEQUATE.]

ANALYSIS

REINFORCEMENT AREA AT ONE SIDE	A_s	=	2.37	in^2
EFFECTIVE COLUMN AREA	A_n	=	558	in^2
NET EFFECTIVE MOMENT OF INERTIA	I_n	=	25982	in^4
RADIUS OF GYRATION	r	=	6.82	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi
MODULAR RATIO	n	=	21.48	
TRANSFORMED AREA	$A_t = A_n(1-\rho+n\rho+n\rho')$	=	668	in^2

REINFORCEMENT RATIO	$\rho = \rho'$	=	0.005	
DISTANCE	d' = 2.25 , d =		21.38	in
ALLOWABLE STRESS FACTOR	SF	=	1.333	
MAX. TIES SPACING (1.17.4)	S_{max}	=	8	in

NEUTRAL AXIS DEPTH FACTOR
$k = \{[n\rho+(2n-1)\rho']^2+2[n\rho+(2n-1)\rho(d'/d)]\}^{0.5} - [n\rho+(2n-1)\rho']$ = 0.278

LEVER-ARM FACTOR $j = 1-k/3$ = 0.907

CHECK VERTICAL REINFORCEMENT LIMITATION (TMS 402 1.14.1.2)

$A_{s,total}$ = 4.74 in^2 > $0.005A_e = 0.005bd$ = 2.53 in^2 **[Satisfactory]**

< $0.04A_e = 0.04bd$ = 20.21 in^2 **[Satisfactory]**

AXIAL LOAD AT MIDDLE OF THE COLUMN

$$P_{Mid} = P + (half \quad col. \quad weight) = \quad 19.090 \text{ k}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_{Mid} = M + (0.1)\left(\frac{Pd}{2}\right) = \quad 107.1 \text{ ft-kips}$$

CHECK IF THERE IS TENSILE STRESS IN CROSS SECTION

P_{Mid} / A = 34 psi < M_{Mid} / (bc^2/6) = 585 psi
(tensile exist)

AXIAL LOAD AT BASE OF THE COLUMN

$$P_t = P + (full \quad col. \quad weight) = \quad 26.681 \text{ k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(0.25 f'_m A_n + 0.65 F_S A_{st}\right)\left(1.0 - \left(\frac{h}{140r}\right)^2\right) = \quad 245.71 \text{ k}$$

> Pt, [Satisfactory]

[for h/r < 99]

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = \left(1.33_{wind \ \& \ seismic \ only}\right)(20 \quad or \quad 24) = \quad 32.0 \text{ ksi}$$

ALLOWABLE STRESS DUE TO FLEXURE

$$F'_b = \left(SF - \frac{P_{Mid}}{P_a}\right)\left(0.33 f'_m\right) = \quad 0.622 \text{ ksi}$$

THE CORRESPONDING STRAIN IN THE TENSILE BARS IS

$$e_S = \frac{F_S}{E_S} = \quad 0.0011$$

THE STRAIN IN THE EXTREME COMPRESSION FIBER IS

$$e_m = MIN\left[\left(\frac{kd}{d-kd}\right)e_s \ , \ \frac{F'_b}{E_m}\right] = \quad 0.0004253$$

(steel governs)

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = e_m E_m = \quad 0.574 \text{ ksi}$$

< Fb', [Satisfactory]

MOMENT DUE TO THE MASONRY

$$M_m = \frac{1}{2} bkd \, f_b\left(d - \frac{kd}{3}\right) = \quad 65.21 \text{ ft-kips}$$

STRAIN IN THE COMPRESSION BARS

$$e'_s = \left(\frac{kd - d'}{kd}\right)e_m = \quad 0.0002644$$

STRESS IN THE COMPRESSION BARS

$$f'_s = 2E_S e'_S = \quad 15.334 \text{ ksi}$$

(cont'd)

MOMENT DUE TO THE COMPRESSION BARS

$M'_S = f'_S A_S (d - d') =$ 57.94 ft-kips

ALLOWABLE BENDING MOMENT

$M = M'_S + M_m =$ 123.15 ft-kips

> Mt, [Satisfactory]

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$f_v = \dfrac{V}{jbd}$ = 39 psi

(TMS 402 2.3.5.2.1)

< $F_v = (SF) MIN(\sqrt{f'_m}\ ,\ 50) =$ 51.64 psi **[Satisfactory]**

< $F_v = (SF) MIN(3\sqrt{f'_m}\ ,\ 150) =$ 154.92 psi **[Satisfactory]**

$\dfrac{V}{F_s d} =$ 0.32 in^2 / ft < $\dfrac{A_v}{s} =$ 0.60 in^2 / ft **(No shear reinf. Reqd)**

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

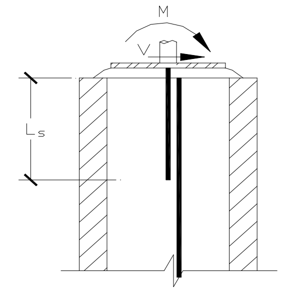
Design for Bending Post at Top of Wall, Based on TMS 402-08

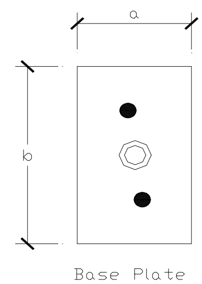
INPUT DATA & DESIGN SUMMARY

MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SERVICE BENDING LOAD	M	=	0.6	ft-kips
SERVICE SHEAR LOAD	V	=	0.116	kips
WALL THICKNESS	T	=	8	in
ANCHORAGE REBARS	2	#	4	@ middle of wall
BASE PLATE YIELD STRESS	F_y	=	36	ksi
BASE PLATE WIDTH	a	=	6	in
BASE PLATE LENGTH	b	=	10	in

(REQUIRED BASE PLATE THK. t = 3/8 in & MIN. REBAR SPLICE LENGTH Ls = 21 in)

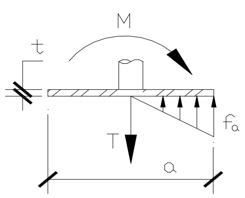
[THE ANCHORAGE DESIGN IS ADEQUATE.]





ANALYSIS

DETERMINE BASE PLATE THICKNESS (AISC 13th, F6-1)



$$t = \sqrt{\frac{4\Omega_o M}{b\left(\frac{4}{3}\right)F_y}} = \quad 3/8 \text{ in}$$

where (4/3) is seismic/wind factor, typical.

CHECK MASONRY BEARING CAPACITY (TMS 402 2.1.8)

$$T = \frac{2M}{\left(\frac{2}{3}\right)(0.5a)} = \quad 7.20 \text{ kips}$$

$$f_a = \frac{2T}{b(0.5a)} = \quad 480 \text{ psi} \quad < \quad \left(\frac{4}{3} \text{ or } 1.0\right)0.25 f'_m = \quad 500 \text{ psi}$$

[SATISFACTORY]

CHECK REBAR CAPACITY (TMS 402 2.3.2.1)

T = 7.20 kips < (4/3 or 1.0) F_S A_S = (4/3) (24 ksi) As = 12.80 kips [SATISFACTORY]

CHECK SHEAR CAPACITY (TMS 402 2.1.4.3.2)

$$V_{allow} = MIN\left(1.25 A_{pv}\sqrt{f'_m}\ ,\ 350\sqrt[4]{f'_m A_b}\ ,\ 2.5 A_{pt}\sqrt{f'_m}\ ,\ 0.36 f_y A_b\right) = \quad 1.46 \text{ kips} \quad > \quad V$$

[SATISFACTORY]

where L_{be} = 3.56 in (rebar edge distance)

$A_{pt} = A_{pv} = \pi L_{be}^2$ = 39.87 in^2, only one projected area used conservatively. (TMS 402 1.16.2 & 1.16.3)

DETERMINE LAP SPLICE LENGTH (TMS 402 2.1.9.7.1)

L_d = ψ_e MAX[0.13 d_b^2 f_y γ / (K $f_m'^{\ 0.5}$) , 12] = 40 d_b = 20.14 in

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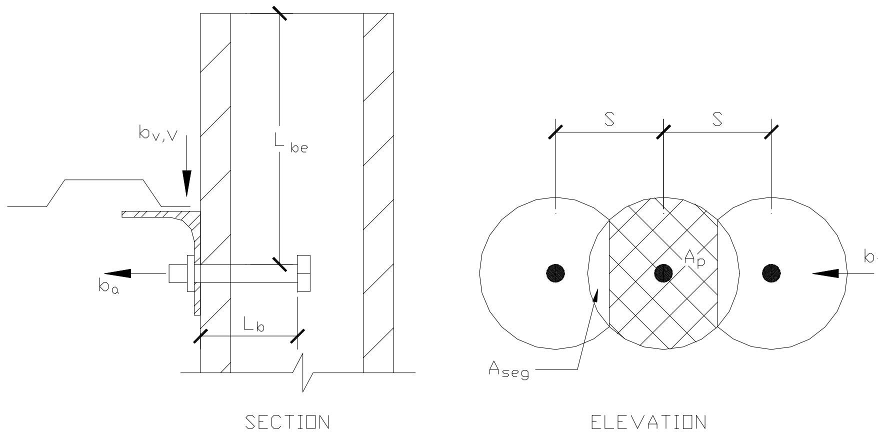
Fastener Anchorage in Combined Stresses Based on TMS 402-08 / IBC 09

INPUT DATA & DESIGN SUMMARY



MASONRY STRENGTH
f_m' = 1.5 ksi

FASTENER YIELD STRESS
f_y = 60 ksi

TENSION STRESS, ASD
b_a = 0.42 kips / ft

SHEAR STRESS, VERTICAL
$b_{v,V}$ = 1 kips / ft

SHEAR STRESS, HORIZONTAL
$b_{v,H}$ = 0.85 kips / ft

WALL THICKNESS
b = 8 in

FASTENER DIAMETER	ϕ	=	1/2	in
EFFECTIVE EMBEDMENT	L_b	=	5	in
EDGE DISTANCE TO WALL TC	L_{be}	=	16	in
FASTENER SPACING	S	=	12	in, o.c.
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	

[THE ANCHORAGE DESIGN IS ADEQUATE.]

ANALYSIS

CHECK MIN. EMBEDMENT (TMS 402 1.16.6)

$L_{b,min}$ = MIN[4ϕ , 2] 2.00 in < L_b [SATISFACTORY]

CHECK TENSION CAPACITY FOR A FASTENER (TMS 402 2.1.4.3.1.1)

B_a = MIN[$1.25A_{pt}(f_m')^{0.5}$, $0.6A_bf_y$] = 3.80 kips / fastener

> $k\ S\ b_a$ [SATISFACTORY]

Where L = MIN[L_b , L_{be}] = 5.00 in, conservative value

$\theta = COS^{-1}(0.5S / L)$ = 0.00 rad

$A_{seg} = L^2$ [θ - 0.5 SIN(2θ)] = 0.00 in^2

$A_{pt} = \pi L^2 - 2A_{seg}$ = 78.54 in^2 (TMS 402 1.16.2)

$A_b = \pi \phi^2 / 4$ = 0.20 in^2

k = 3/4

CHECK SHEAR CAPACITY (TMS 402 2.1.4.3.2)

B_v = MIN[$1.25A_{pv}(f_m')^{0.5}$, $350(A_bf_m')^{1/4}$, $2.5A_{pt}(f_m')^{0.5}$, $0.36A_bf_y$] = 1.45 kips / fastener

> $S\ b_{v,V}$, Gravity only [SATISFACTORY]

> $k\ S\ b_v$, Combined shear [SATISFACTORY]

Where $\theta = COS^{-1}(0.5S / L_{be})$ = 1.19 rad

$A_{seg} = L_{be}^2$ [θ - 0.5 SIN(2θ)] = 303.72 in^2

$A_{pv} = 0.5(\pi L_{be}^2 - 2A_{seg})$ = 98.41 in^2 (TMS 402 1.16.3)

$b_v = (b_{v,V} + b_{v,H})^{0.5}$ = 1.312 kips / ft

CHECK COMBINED SHEAR AND TENSION CAPACITY (TMS 402 2.1.4.3.3)

$S\ b_a / B_a + S\ b_v / B_v$ = 1.02 < 1.33 [SATISFACTORY]

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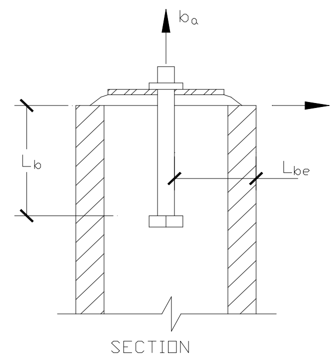
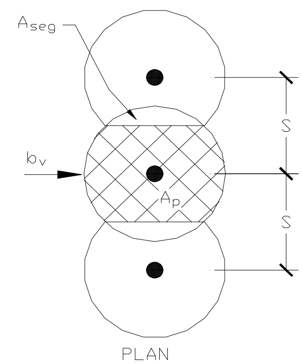
Fastener Anchorage in Tension & Perpendicular Shear Based on TMS 402-08 / IBC 09



INPUT DATA & DESIGN SUMMARY

MASONRY STRENGTH	f_m'	=	1.5	ksi
FASTENER YIELD STRESS	f_y	=	60	ksi
SERVICE TENSION LOAD	b_a	=	0.9	kips / ft
SERVICE SHEAR LOAD	b_v	=	0.4	kips / ft
WALL THICKNESS	b	=	8	in
FASTENER DIAMETER	ϕ	=	3/4	in
EFFECTIVE EMBEDMENT	L_b	=	5	in
FASTENER SPACING	S	=	8	in
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	

[THE ANCHORAGE DESIGN IS ADEQUATE.]



ANALYSIS

CHECK MIN. EMBEDMENT (TMS 402 1.16.6)

$L_{b,min}$ = MIN[4ϕ , 2] = 2.00 in < L_b [SATISFACTORY]

CHECK TENSION CAPACITY (TMS 402 2.1.4.3.1.1)

B_a = MIN[$1.25A_{pt}(f_m')^{0.5}$, $0.6A_bf_y$] = 1.80 kips / fasteners

> k S b_a [SATISFACTORY]

Where L_{be} = 3.44 in

L = MIN[L_b , L_{be}] = 3.44 in, conservative value

$\theta = COS^{-1}(0.5S / L)$ = 0.00 rad

$A_{seg} = L^2 [\theta - 0.5 SIN(2\theta)]$ = 0.00 in^2

$A_{pt} = \pi L^2 - 2 A_{seg}$ = 37.18 in^2 (TMS 402 1.16.2)

$A_b = \pi \phi^2 / 4$ = 0.44 in^2

k = 3/4

CHECK SHEAR CAPACITY (TMS 402 2.1.4.2.3)

B_v = MIN[$1.25A_{pv}(f_m')^{0.5}$, $350(A_bf_m')^{1/4}$, $2.5A_{pt}(f_m')^{0.5}$, $0.36A_bf_y$] = 1.78 kips / fasteners

> k S b_v [SATISFACTORY]

Where $A_{pv} = A_{pt}$ = 37.18 in^2, since L = MIN[L_b , L_{be}] used above, (TMS 402 1.16.3)

CHECK COMBINED SHEAR AND TENSION CAPACITY (TMS 402 2.1.4.3.3)

$S\, b_a / B_a + S\, b_v / B_v$ = 0.48 < 1.33 [SATISFACTORY]

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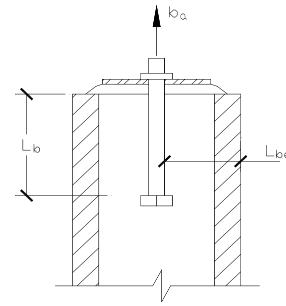
Fastener Anchorage in Tension & Parallel Shear Based on TMS 402-08 / IBC 09

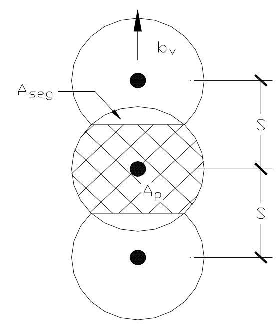


SECTION

INPUT DATA & DESIGN SUMMARY

MASONRY STRENGTH	f_m'	=	1.5	ksi
FASTENER YIELD STRESS	f_y	=	60	ksi
UPLIFT STRESS, ASD	b_a	=	0.5	kips / ft
SHEAR STRESS IN WALL DIR	b_v	=	0.2	kips / ft
WALL THICKNESS	b	=	8	in
FASTENER DIAMETER	ϕ	=	1/2	in
EFFECTIVE EMBEDMENT	L_b	=	5	in
FASTENER SPACING	S	=	16	in, o.c.
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	

[THE ANCHORAGE DESIGN IS ADEQUATE.]



PLAN

ANALYSIS

CHECK MIN. EMBEDMENT (TMS 402 1.16.6)

$L_{b,min}$ = MIN[4ϕ , 2] = 2.00 in < L_b [SATISFACTORY]

CHECK TENSION CAPACITY (TMS 402 2.1.4.3.1.1)

B_a = MIN[$1.25A_{pt}(f_m')^{0.5}$, $0.6A_bf_y$] = 1.93 kips / fastener

\> k S b_a [SATISFACTORY]

Where L_{be} = 3.57 in

L = MIN[L_b , L_{be}] = 3.57 in, conservative value

$\theta = COS^{-1}(0.5S / L)$ = 0.00 rad

$A_{seg} = L^2$ [θ - 0.5 SIN(2θ)] = 0.00 in^2

$A_{pt} = \pi L^2 - 2A_{seg}$ = 39.93 in^2 (TMS 402 1.16.2)

$A_b = \pi \phi^2 / 4$ = 0.20 in^2

k = 3/4

CHECK SHEAR CAPACITY (TMS 402 2.1.4.2.3)

B_v = MIN[$1.25A_{pv}(f_m')^{0.5}$, $350(A_bf_m')^{1/4}$, $2.5A_{pt}(f_m')^{0.5}$, $0.36A_bf_y$] = 1.45 kips / fastener

\> k S b_v [SATISFACTORY]

Where $A_{pv} = A_{pt}$ = 39.93 in^2, since L = MIN[L_b , L_{be}] used above, (TMS 402 1.16.3)

CHECK COMBINED SHEAR AND TENSION CAPACITY (TMS 402 2.1.4.3.3)

S b_a / B_a + S b_v / B_v = 0.53 < 1.33 [SATISFACTORY]

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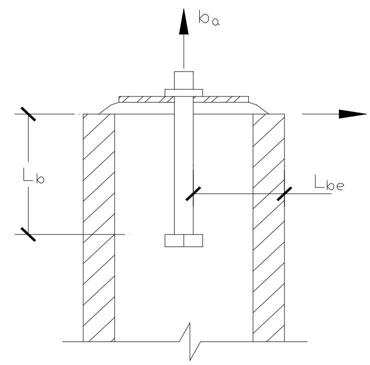
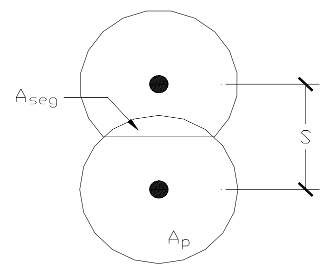
Double Fastener Anchorage in Tension & Shear Based on TMS 402-08 / IBC 09



INPUT DATA & DESIGN SUMMARY

MASONRY STRENGTH	f_m'	=	1.5	ksi
FASTENER YIELD STRESS	f_y	=	60	ksi
SERVICE TENSION LOAD	b_a	=	0.91	kips / 2 fasteners
SERVICE SHEAR LOAD	b_v	=	0.728	kips / 2 fasteners
WALL THICKNESS	b	=	8	in
FASTENER DIAMETER	ϕ	=	3/4	in
EFFECTIVE EMBEDMENT	L_b	=	7	in
FASTENER SPACING	S	=	6	in
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	

[THE ANCHORAGE DESIGN IS ADEQUATE.]



ANALYSIS

CHECK MIN. EMBEDMENT (TMS 402 1.16.6)

$L_{b,min}$ = MIN[4ϕ , 2] = 2.00 in < L_b [SATISFACTORY]

CHECK TENSION CAPACITY (TMS 402 2.1.4.3.1.1)

B_a = 2 MIN[$1.25A_{pt}(f_m')^{0.5}$, $0.6A_bf_y$] = 3.50 kips / 2 fasteners

\> k b_a [SATISFACTORY]

Where L_{be} = 3.44 in

L = MIN[L_b , L_{be}] = 3.44 in, conservative value

$\theta = COS^{-1}(0.5S / L)$ = 0.51 rad

$A_{seg} = L^2 [\ \theta - 0.5\ SIN(2\theta)]$ = 1.00 in^2

$A_{pt} = \pi L^2 - A_{seg}$ = 36.18 in^2 (TMS 402 1.16.2)

$A_b = \pi \phi^2 / 4$ = 0.44 in^2

k = 3/4

CHECK SHEAR CAPACITY (TMS 402 2.1.4.2.3)

B_v = MIN[$1.25A_{pv}(f_m')^{0.5}$, $350(A_bf_m')^{1/4}$, $2.5A_{pt}(f_m')^{0.5}$, $0.36A_bf_y$] = 3.50 kips / 2 fasteners

\> k b_v [SATISFACTORY]

Where $A_{pv} = A_{pt}$ = 36.18 in^2, since L = MIN[L_b , L_{be}] used above, (TMS 402 1.16.3)

CHECK COMBINED SHEAR AND TENSION CAPACITY (TMS 402 2.1.4.3.3)

$b_a / B_a + b_v / B_v$ = 0.47 < 1.33 [SATISFACTORY]

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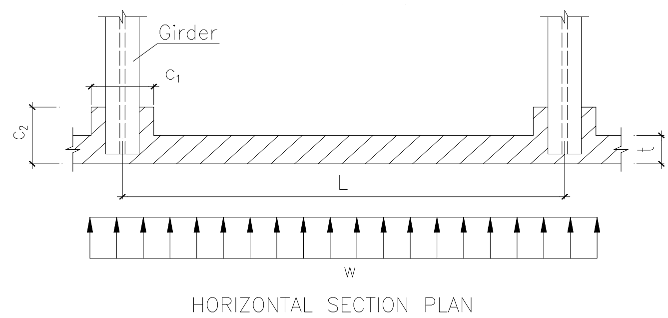
Masonry Wall Design at Horizontal Bending, Based on TMS 402-08

INPUT DATA & DESIGN SUMMARY



SPECIAL INSPECTION (0=NO, 1=YES)			0	No
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SERVICE LATERAL LOAD	w	=	45	psf
THICKNESS OF WALL	t	=	8	in
WALL HEIGHT	h	=	24	ft
PILASTER SPACING	L	=	10	ft
PILASTER SIZE	c_1	=	24	in
	c_2	=	24	in

[THE WALL DESIGN IS ADEQUATE.]

WALL HORIZ. REINF. (A_{sv}) 1 # 4 @ 48 in o.c. (at middle)

ANALYSIS

DESIGN CRITERIA

1. Pilaster spacing less than one half the unsupported vertical span of out-of-plane wall. (MDG-3,page 11-8)

L = 10 ft < 0.5 h = 12 ft [Satisfactory]

2. Pilaster stiffness greater than that the tributary area of wall. (TMS 402 1.7.4)

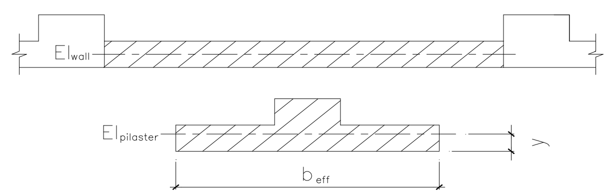
$b_{eff} = c_1 + 12\,t =$ 120 in, (TMS 402, 1.7.6.1 & 1.9.4.2.3)

y = 7.4 in

E $I_{pilaster}$ = 52809 E ,(TMS 402, 1.9.2)

E I_{wall} = 4096 E ,(TMS 402, 1.9.2)

E $I_{pilaster}$ > E I_{wall} [Satisfactory]



CHECK WALL HORIZONTAL BENDING CAPACITY

d	=	3.82	in, effective depth	E_m	=	1350	ksi
b_w	=	12	in	E_s	=	29000	ksi
t_e	=	7.63	in, effective thickness	n	=	21.48	, modular ratio
A_s	=	0.05	in^2 / ft	ρ	=	0.0011	, reinforcement ratio
				SF	=	0.667	, allowable stress factor

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n = \quad 0.1944$$

$$F_s = (1.33 \text{ or } 1.0)(2000 \text{ or } 24000) = \quad 32000 \text{ psi}$$

$$F_b = (SF)(0.33 f'_m) = \quad 330 \text{ psi}$$

$$M_{allowable} = MIN\left[\frac{1}{2} b_w kd F_b\left(d - \frac{kd}{3}\right) \;,\; A_s F_s\left(d - \frac{kd}{3}\right)\right] = \quad 437 \text{ ft-lbs/ft, (MDG-3, page 11-3)}$$

$$M_{max} = \frac{w(L - c_1)^2}{8} = \quad 360 \text{ ft-lbs/ft} \quad < \quad M_{allowable}$$

[Satisfactory]

CHECK SHEAR CAPACITY (TMS 402 2.3.5.2.2)

$$F_v = (SF)\, MIN\left(\sqrt{f'_m} \;,\; 50\right) = \quad 26 \text{ psi} \quad > \quad f_v = 0.5\,(L - c_1)\, w / (b_w d) = \quad 4 \text{ psi}$$

[Satisfactory]

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

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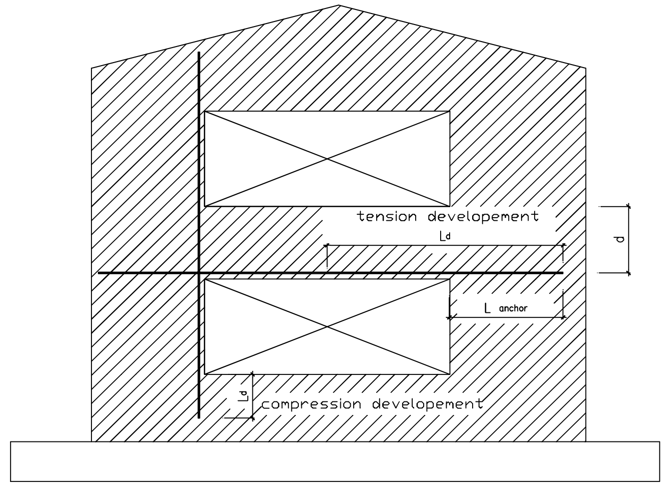
Development & Splice of Reinforcement in Masonry Based on 2010 CBC Chapter A, ASD



LINTEL & JAMB REINFORCEMENT

TENSION & COMPRESSION DEVELOPMENT

$L_d = \psi_e$ MAX[$0.13\ d_b^2\ f_y\ \gamma\ /\ (K\ f_m'^{\ 0.5})$, 12]

= 57 d_b = 43 in

ACI 530-05, 2.1.10.3

where Bar size # 6

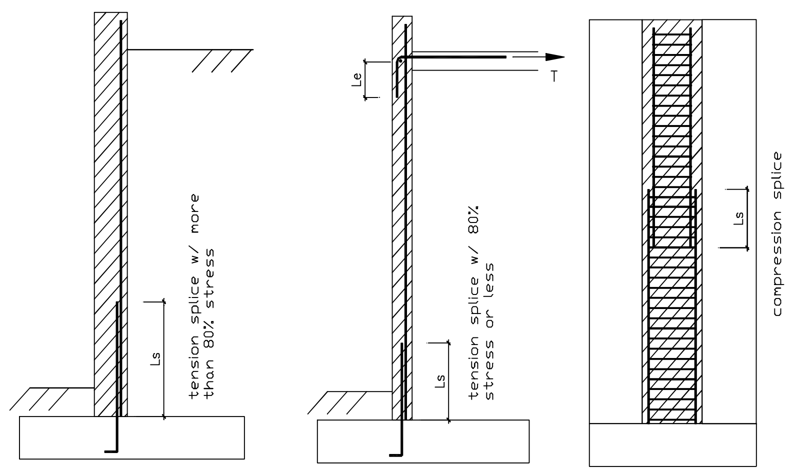
d_b	=	0.75	in
f_y	=	60	ksi
f_m'	=	1.5	ksi
ψ_e	=	1.0	(1.5 for epoxy-coated)
γ	=	1.3	
c	=	3.44	in, masonry cover
s	=	7.25	in, adjacent bars clear spacing
K	=	MIN(c , s , 5d_b) = 3.44	in

ANCHORAGE OF FLEXURAL REINFORCEMENT

L_{anchor} = MAX (d , 12 d_b) = 53 d_b = 40 in, ACI 530-05, 2.1.10.4.1.3

where Bar size # 6

d_b	=	0.75	in
d	=	40	in



RETAINING WALL EXTERIOR WALL PILASTER

TENSION SPLICE WITH MORE THAN 80% STRESS

L_s = 1.5 MAX(0.002 d_b F_s , 40 d_b , 12) = 54 in

CBC 2107A.3

where Bar size # 6

d_b	=	0.75	in
f_y	=	60	ksi
F_s	=	24	ksi

TENSION HOOKS

L_e = 11.25 d_b = 8 in,

ACI 530, 2.1.10.5 & Fig 1.13-1

T = A_s F_s = 3.30 kips

where Bar size # 6

d_b	=	0.75	in
A_s	=	0.44	in^2
F_s	=	7.5	ksi, ACI 530, C 2.1.10.5

TENSION SPLICE WITH 80% STRESS OR LESS, OR COMPRESSION SPLICE

L_s = MAX(0.002 d_b F_s , 40 d_b , 12) = 36 in

CBC 2107A.3

where Bar size # 6

d_b	=	0.75	in
f_y	=	60	ksi
F_s	=	24	ksi, TMS 402-08 2.3.2.1

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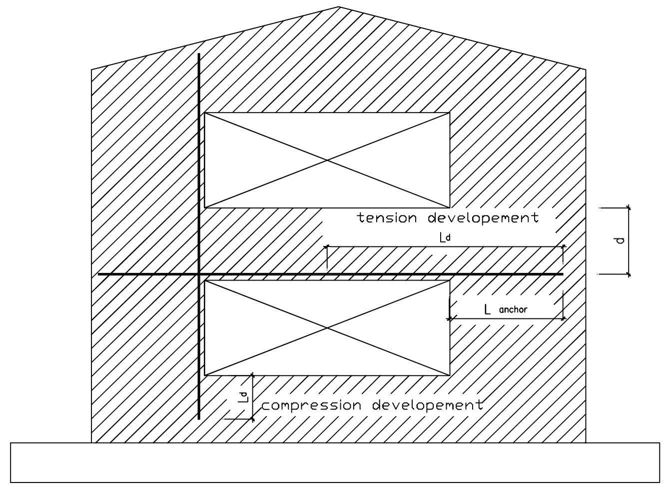
Development & Splice of Reinforcement in Masonry Based on 2010 CBC Chapter A, SD

TENSION & COMPRESSION DEVELOPMENT

$L_d = \psi_e$ MIN{ MAX[0.13 $d_b^2 f_y \gamma$ / (K $f_m'^{0.5}$) , 12] , 72 d_b }
= 40 d_b = 25 in
CBC 10, 2108A.2 & TMS 402-08, 3.3.3.3

where	Bar size	#	5	
	d_b	=	0.625	in
	f_y	=	60	ksi
	f_m'	=	1.5	ksi
	ψ_e	=	1.0	(1.5 for epoxy-coated)
	γ	=	1.0	
	c	=	3.5025	in, masonry cover
	s	=	7.375	in, adjacent bars clear spacing
	K	=	MIN(c , s , 5d_b) = 3.125	in



LINTEL & JAMB REINFORCEMENT

ANCHORAGE OF FLEXURAL REINFORCEMENT

L_{anchor} = MAX (d , 12 d_b) = 53 d_b = 40 in, ACI 530-05, 2.1.10.4.1.3

where	Bar size	#	6	
	d_b	=	0.75	in
	d	=	40	in

TENSION HOOKS

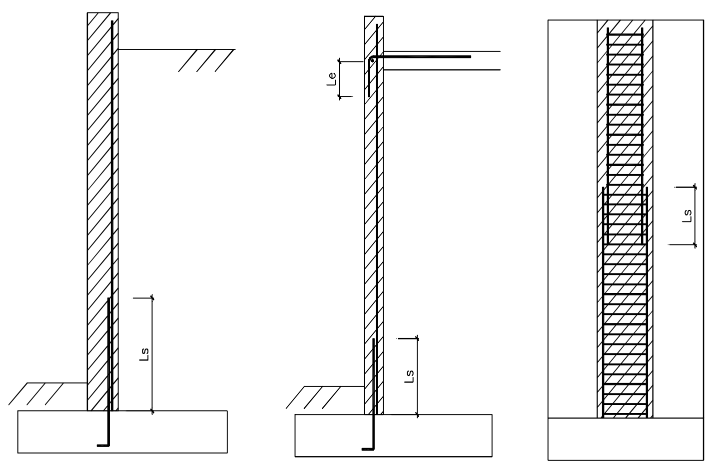
$L_e = 0.5\ L_d$ = 13 in, ACI 530, 3.3.3.3.2.1 & Fig 1.13-1

where Bar size # 5 , from L_d

TENSION OR COMPRESSION SPLICE

$L_d = \psi_e$ MAX[0.13 $d_b^2 f_y \gamma$ / (K $f_m'^{0.5}$) , 12]
= 57 d_b = 43 in
ACI 530-05, 2.1.10.7.1.1

where	Bar size	#	6	
	d_b	=	0.75	in
	f_y	=	60	ksi
	f_m'	=	1.5	ksi
	ψ_e	=	1.0	(1.5 for epoxy-coated)
	γ	=	1.3	
	c	=	3.44	in, masonry cover
	s	=	7.25	in, bars clear spacing
	K	=	MIN(c , s , 5d_b) = 3.44	in



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Development & Splice of Reinforcement in Masonry Based on TMS 402-11 / ACI 530-11 & 2012 IBC

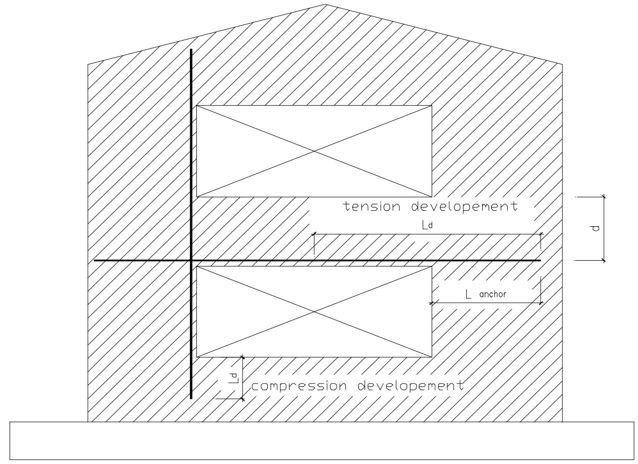
TENSION & COMPRESSION DEVELOPMENT

$L_d = \psi_e$ MAX[$0.13\, d_b^2\, f_y\, \gamma\, / \,(K\, f_m'^{\,0.5})$, 12]

= 57 d_b = 43 in

TMS 402-11, 2.1.7.3

where	Bar size	#	6	
	d_b	=	0.75	in
	f_y	=	60	ksi
	f_m'	=	1.5	ksi
	ψ_e	=	1.0	(1.5 for epoxy-coated)
	γ	=	1.3	
	c	=	3.44	in, masonry cover
	s	=	7.25	in, adjacent bars clear spacing
	K	=	MIN(c , s , $5d_b$) =	3.44 in



LINTEL & JAMB REINFORCEMENT

ANCHORAGE OF FLEXURAL REINFORCEMENT

L_{anchor} = MAX (d , 12 d_b) = 53 d_b = 40 in, TMS 402-11, 2.1.7.4.1.3

where	Bar size	#	6	
	d_b	=	0.75	in
	d	=	40	in

TENSION HOOKS

$L_e = 13\ d_b$ = 10 in, TMS 402, 2.1.7.5

$T = A_s\, F_s$ = 3.30 kips

where	Bar size	#	6	
	d_b	=	0.75	in
	A_s	=	0.44	in^2
	F_s	=	7.5	ksi, TMS 402, C 2.1.7.5

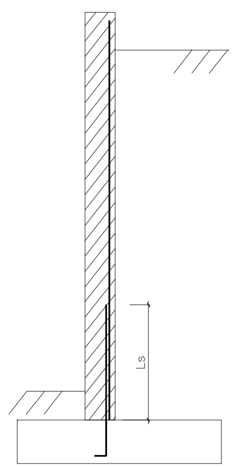
TENSION OR COMPRESSION SPLICE

$L_d = \psi_e$ MAX[$\zeta\, 0.13\, d_b^2\, f_y\, \gamma\, / \,(K\, f_m'^{\,0.5})$, 12, 36 d_b]

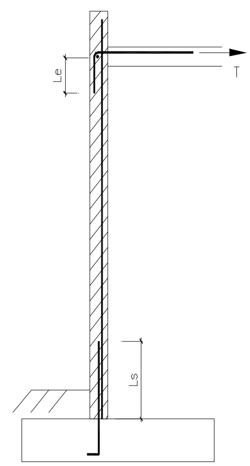
= 57 d_b = 43 in

TMS 402-11, 2.1.7.7.1

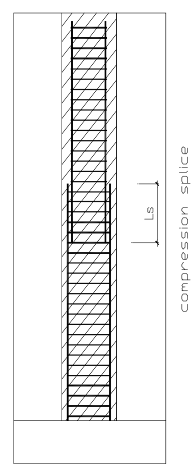
	A_{sc}	=	0	in^2
where	ζ	=	1	, TMS 402-11, 2.1.7.7.2
	Bar size	#	6	
	d_b	=	0.75	in
	f_y	=	60	ksi
	f_m'	=	1.5	ksi
	ψ_e	=	1.0	(1.5 for epoxy-coated)
	γ	=	1.3	
	c	=	3.44	in, masonry cover
	s	=	7.25	in, bars clear spacing
	K	=	MIN(c , s , $5d_b$) =	3.44 in



RETAINING WALL



EXTERIOR WALL



PILASTER

(2012 IBC 2107.2 : L_d = 1.5 MAX(0.002 $d_b\, f_s$, 12 , 40 d_b) = 54 in

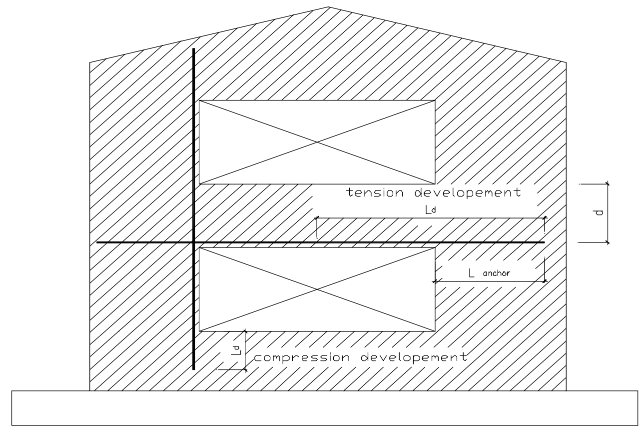
where f_s = 24 ksi)

Tables for Development & Splice of Reinforcement Based on IBC 06 / CBC 07 / ACI 530

TENSION & COMPRESSION DEVELOPMENT

Table 1: L_d Values (inch)

Bar	f_m' (psi)					
Size	1000	1500	2000	2500	3000	3500
# 3	25	24	24	24	24	24
# 4	25	20	17	16	14	13
# 5	31	25	22	20	18	16
# 6	52	43	37	33	30	28
# 7	73			46	42	39
# 8	112			71	64	60
# 9	145	118	102	92	84	77
# 10	188	153	133	119	108	100
# 11	237	193	167	150	137	126
# 14	357	292	253	226	206	191
# 18	702	573	496	444	405	375



LINTEL & JAMB REINFORCEMENT

ANCHORAGE OF FLEXURAL REINFORCEMENT

Table 1: L_{anchor} Values (inch)

Bar	f_m' (psi)					
Size	1000	1500	2000	2500	3000	3500
# 3	40	40	40	40	40	40
# 4	40	40	40	40	40	40
# 5	40	40	40	40	40	40
# 6	40	40	40	40	40	40
# 7	40	40	40	40	40	40
# 8	40	40	40	40	40	40
# 9	40			40	40	40
# 10	40			40	40	40
# 11	40	40	40	40	40	40
# 14	40	40	40	40	40	40
# 18	40	40	40	40	40	40

TENSION SPLICE WITH MORE THAN 80% STRESS

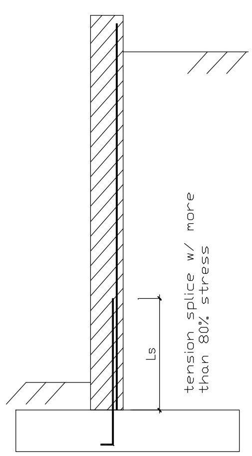
Table 1: L_s Values (inch)

Bar	f_m' (psi)					
Size	1000	1500	2000	2500	3000	3500
# 3	25	24	24	24	24	24
# 4	36	36	36	36	36	36
# 5	45	45	45	45	45	45
# 6	54	54	54	54	54	54
# 7	73	63	63			63
# 8	112	91	79			72
# 9	145	118	102	92	84	81
# 10	188	153	133	119	108	100
# 11	237	193	167	150	137	126
# 14	357	292	253	226	206	191
# 18	702	573	496	444	405	375

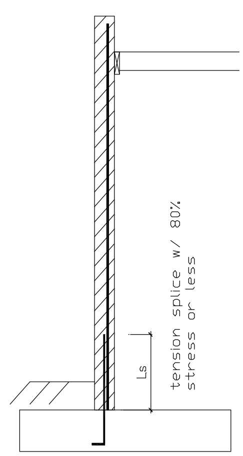
TENSION SPLICE WITH 80% STRESS OR LESS, OR COMPRESSION SPLICE

Table 1: L_s Values (inch)

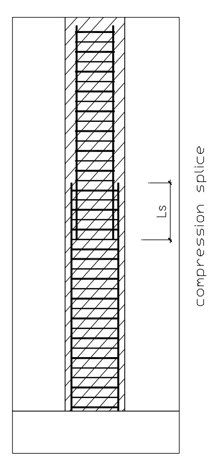
Bar	f_m' (psi)					
Size	1000	1500	2000	2500	3000	3500
# 3	25	24	24	24	24	24
# 4	25	24	24	24	24	24
# 5	31	30	30	30	30	30
# 6	52	43	37	36	36	36
# 7	73			46	42	42
# 8	112			71	64	60
# 9	145	118	102	92	84	77
# 10	188	153	133	119	108	100
# 11	237	193	167	150	137	126
# 14	357	292	253	226	206	191
# 18	702	573	496	444	405	375



RETAINING WALL



EXTERIOR WALL

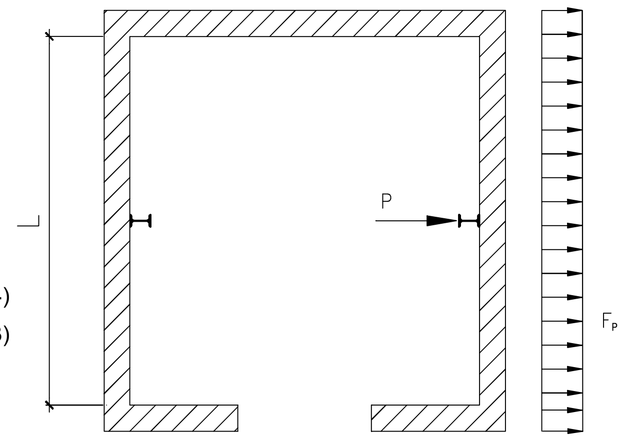


PILASTER

Elevator Masonry Wall Design Based on CBC 10 / CBC 07, Chapter A

INPUT DATA & DESIGN SUMMARY

MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
THICKNESS OF WALL	t	=	16	in
HORIZ. BENDING SPAN	L	=	10.17	ft
SEISMIC COEFFICIENT	S_{DS}	=	0.96	(ASCE 7-05 11.4.4)
IMPORTANCE FACTOR	I_P	=	1.5	(ASCE 7-05 13.1.3)
ELEVATOR CAR WEIGHT	W_{car}	=	2.5	kips
ELEVATOR RATED LOAD	W_{rat}	=	3	kips
WALL HORIZ. REINF. (A_s)	1	#	6	@ 8 in o.c. (at middle)



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

DESIGN LOADS (ASCE 7-05, Sec. 13.3.1)

$$F_p = \frac{0.4 a_p S_{DS} I_p W_p}{1.4 R_p}\left(1+2\frac{z}{h}\right) = 0.411\ W_p \quad = \quad 63 \quad \text{psf , ASD}$$

Where : a_p = 1.0 (ASCE Tab. 13.5-1) R_p = 3.0 (ASCE Tab. 13.5-1) W_p = 153.3 psf z = h (attachment height per ASCE 7-05 13.3.1, horizontal bending, z dynamic)

$$P = \frac{2}{3}\left[0.5\left(W_{car} + 40\% W_{rat}\right)\right]\frac{12"}{b} = \quad 1.85 \quad \text{kip / ft , ASD}$$

Where : Factor 2/3 , 0.5 , & 40% from CBC 07 1614A.1.15 or CBC 10 1615A.1.23
b = 8.0 in, TMS 402-08 1.9.6

$$M_{max} = \frac{F_p L^2}{8} + \frac{PL}{4} = \quad 5.52 \quad \text{ft-kip / ft , ASD}$$

(simple beam since wall cracked)

CHECK HORIZONTAL BENDING CAPACITY (ACI 530-05)

$$M_{allowable} = MIN\left[\frac{1}{2} b_w k d F_b\left(d - \frac{kd}{3}\right) \ , \ A_s F_s\left(d - \frac{kd}{3}\right)\right] = \quad 7.27 \quad \text{ft-kip / ft , ASD}$$

> M_{max} [Satisfactory]

Where : b_w =	12	in,	t_e =	16	in,	d =	8	in
A_s =	0.66	in^2 / ft	ρ =	0.007		n =	21.5	
k =	0.419		F_b =	660	psi	F_s =	32000	psi

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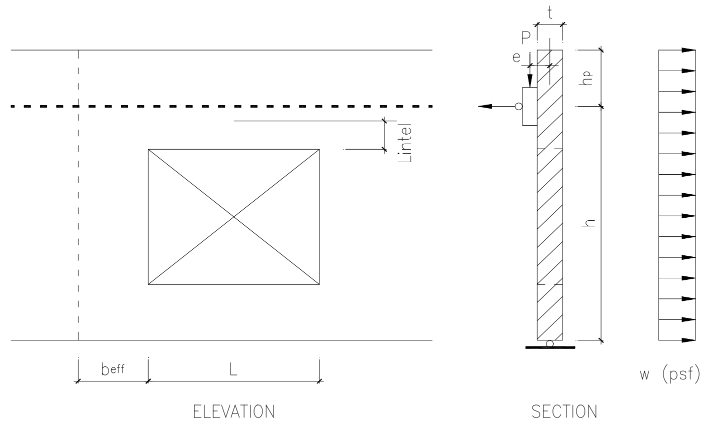
Design of Masonry Bearing Wall with Opening Based on TMS 402-08

INPUT DATA & DESIGN SUMMARY

TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU				
MASONRY STRENGTH	f_m'	=	1.5	ksi				
REBAR YIELD STRESS	f_y	=	60	ksi				
GRAVITY LOAD, ASD	P	=	640	lbs / ft				
ECCENTRICITY	e	=	6	in				
LATERAL LOAD, ASD	w	=	26.7	psf				
OPENING WIDTH	L	=	4.33	ft				
LINTEL HEIGHT	Lintel	=	4	ft				
THICKNESS OF WALL	t	=	8	in				
PARAPET HEIGHT	h_p	=	4	ft				
WALL HEIGHT	h	=	16	ft				
WALL VERT. REINF.	1	layer	#	5	@	16	in o.c. (at middle)	
VERT. JAMB BARS	2	#	5					
LINTEL VERT. REINF	1	leg	#	4	@	8	in o.c. (at middle)	
LINTEL BOT/TOP BARS	2	#	6					

DIAPHRAGM SUPPORT LINTEL LATERAL LOAD (Yes, No) ? No



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

CHECK JAMB CAPACITY UNDER LATERAL & AXIAL LOADS (TMS 402-08 2.3.3)

b_{eff} = 6 t = 48 in, (TMS 402-08 1.9.4.2.3)

P_{Jamb} = P (b_{eff} + 0.5 L) / b_{eff} = 986 lbs / ft, at ledger

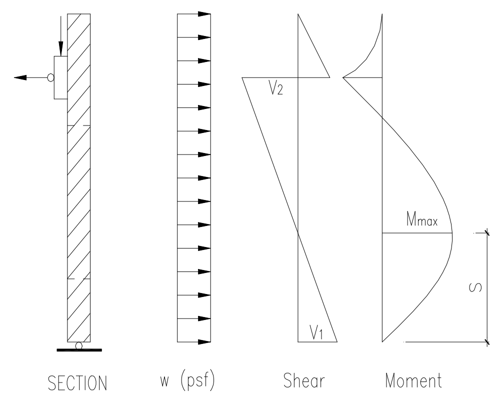
w_{Jamb} = w (b_{eff} + 0.5 L) / b_{eff} = 41 psf

S = 8.25 ft **(Assuming that opening has same wall loads.)**

M_{max} = 1470 ft-lbs / ft

P_M = 2556 lbs / ft, at max M section

A_s = (wall + Jamb) = 0.31 in^2 / ft



M_{allow} = 1711 ft-lbs / ft > M_{max} [Satisfactory]

(See ***MasonryBearingWall-IBC.xls*** on www.engineering-international.com for the M_{allow} at P_M calculation,
or see Masonry Designers' Guide, Third Edition, The Masonry Society, 2001. Page 12-25.)

V_{max} = MAX (V_1 , V_2) = 339 lbs / ft < V_{allow} = 4728 lbs / ft [Satisfactory]

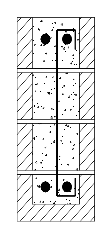
CHECK LINTEL CAPACITY UNDER GRAVITY & LATERAL LOADS (TMS 402-08 2.3)

f_b = Gravity + Lateral = 11 psi + 484 psi = 495 psi

< F_b = 495 psi [Satisfactory]

f_s = Gravity + Lateral = 629 psi + 308 psi = 937 psi

< F_s = 24000 psi [Satisfactory]



where	M_{Gav} =	1940	ft-lbs	ρ =	0.002487	,for gravity
	$A_{s,Gav}$ =	0.88	in^2	k =	0.278	
	M_{Lat} =	135	ft-lbs	ρ =	0.003053	,for lateral
	$A_{s,Lat}$ =	0.88	in^2	k =	0.004	

f_v = Gravity + Lateral = (5.06^2 + 0.43^2)$^{0.5}$ = 5.08 psi

< F_v = 39 psi [Satisfactory]

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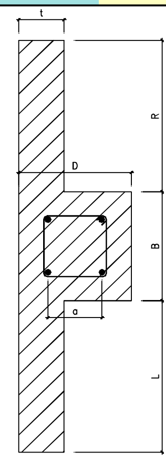
Flush Wall Pilaster Design Based on CBC 10 Chapter A

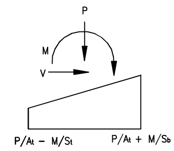
INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SEISMIC DESIGN CATEGORY (5=Gravity)			3	D
SERVICE AXIAL LOAD	P	=	50	k, at pilaster center
SERVICE SHEAR LOAD	V	=	9.5	k, (may be negative)
SERVICE MOMENT	M	=	8.2	ft-k, (posite or negative)

SECTION DIMENSIONS (in)

input		effective	
t =	8	t_e =	7.63
D =	24	d =	23.63
L =	48	L_e =	48.00
B =	24	b =	23.63
R =	80	R_e =	48.00





WALL PILASTER HEIGHT h = 15 ft

VERTICAL REINF. (EACH SIDE) 2 # 8

HORIZ. TIES 2 legs # 4 @ 8 in o.c.

DISTANCE PILASTER REINF. a = 19.13 in (ACI 530, 1.13.3.5)

WALL GROUTED ? (Yes, No) No , (only pilaster grouted)

[THE WALL PILASTER DESIGN IS ADEQUATE.]

ANALYSIS

TOTAL REINFORCEMENT AREA	A_s	=	3.16	in^2
TOTAL EFFECTIVE AREA	A_n	=	798	in^2
NEUTRAL AXIS FROM WALL FACE	y_t	=	9	in
NET MOMENT OF INERTIA	I_n	=	39198	in^4
RADIUS OF GYRATION	r	=	7.01	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48	
PILASTER REINF. RATIO	ρ	=	0.006	
ALLOWABLE STRESS FACTOR	SF	=	1.333	
MAX. TIES SPACING (2106A.5.3.2)	S_{max}	=	8	in

TRANSFORMED WALL PILASTER AREA

$$A_t = A_n + bd\left(2n-1\right)\rho = \quad 931 \ in^2$$

CHECK VERTICAL REINFORCEMENT LIMITATION OF PILASTER (ACI 530, 2.1.6.4)

A_s = 3.16 in^2 > 0.0025 bd = 1.40 in^2 **[Satisfactory]**

< 0.04 bd = 22.34 in^2 **[Satisfactory]**

ALLOWABLE STRESS DUE TO AXIAL LOAD ONLY

$$F_a = \left(SF\right)\left(0.25 f_m'\right)\left(1.0 - \left(\frac{h}{140r}\right)^2\right) = \quad 0.483 \text{ ksi}$$

[for h/r < 99]

AXIAL STRESS AT MIDHEIGHT OF THE PILASTER

$$f_a = \frac{P + \left(half \quad wall \quad weight\right)}{A_t} = \quad 0.060 \text{ ksi}$$

< Fa, [Satisfactory]

ALLOWABLE STRESS DUE TO FLEXURE

$$F_b = \left(SF\right)\left(0.33 f_m'\right) = \quad 0.660 \text{ ksi}$$

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = \left(1.33 \quad or \quad 1.0\right)\left(20 \quad or \quad 24\right) = \quad 32.0 \text{ ksi}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_T = M + \left(0.1\right)\left(\frac{Pd}{2}\right) = \quad 13.1 \text{ ft-kips}$$

TRANSFORMED MOMENT OF INERTIA

$$I_t = I_n + \left(2n-1\right)A_s\left(\frac{a}{2}\right)^2 = \quad 51330 \ in^4$$

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = \frac{M_T y}{I_t} = \quad 0.029 \text{ ksi}$$

< fa, [Satisfactory, the section is uncracked]

MAX.STRESS COMBINED AXIAL & FLEXURE

$$f_m = f_a + f_b = \quad 0.089 \text{ ksi}$$

< Fb, [Satisfactory]

MAX. REINF. STRESS COMBINED AXIAL & FLEXURE

$$f_S = 2n\left(f_a + \frac{a}{d} f_b\right) = \quad 3.6 \text{ ksi}$$

< Fs, [Satisfactory]

AXIAL LOAD AT BASE OF THE WALL PILASTER

$$P_t = P + \left(full \quad col. \quad weight\right) = \quad 61.227 \text{ k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(\left(SF\right)0.25 f_m' A_n + 0.65 F_S A_S\right)\left(1.0 - \left(\frac{h}{140r}\right)^2\right) = \quad 449.26 \text{ k}$$

[for h/r < 99] **> Pt, [Satisfactory]**

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$$f_v = \frac{V}{bd} = \quad 17 \text{ psi} \quad \begin{cases} < \ F_v = \left(SF\right)MIN\left(\sqrt{f_m'}\ ,\ 50\right) = \quad 51.64 \text{ psi} \\ < \ F_v = \left(SF\right)MIN\left(3\sqrt{f_m'}\ ,\ 150\right) = \quad 154.92 \text{ psi} \end{cases}$$

[Satisfactory]

(Sec. 2.3.5.2.1) **[Satisfactory]**

$$\frac{V}{F_s d} = \quad 0.15 \ in^2/ft \quad < \quad \frac{A_v}{s} = \quad 0.60 \ in^2/ft$$

(No shear reinf. Reqd)

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Engineering International

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Flush Wall Pilaster Design Based on CBC 10 Chapter A

INPUT DATA & DESIGN SUMMARY

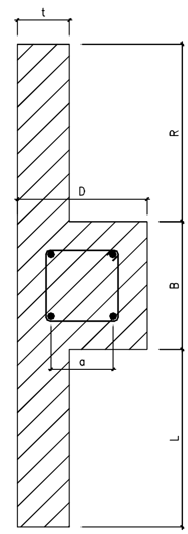
SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SEISMIC DESIGN CATEGORY			4	E or F

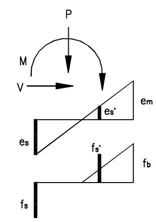
(1 = B, 2 = C, 3 = D, 4 = E or F, 0 = WIND, 5 = GRAVITY)

SERVICE AXIAL LOAD	P	=	11.5	k, @ center top of pilaster
MAX SHEAR LOAD	V	=	20	k
MOMENT AT MIDHEIGHT	M	=	106	ft-k, (posite or negative)

SECTION DIMENSIONS (in)

input		effective	
t =	8	t_e =	7.63
D =	24	c =	23.63
L =	48	L_e =	48.00
B =	24	b =	23.63
R =	80	R_e =	48.00





WALL PILASTER HEIGHT   h   =   29   ft
VERTICAL REINF. (EACH SIDE)   3   #   8
HORIZ. TIES   2   legs #   4   @   8   in o.c.

DISTANCE PILASTER REINF.   a =   19.13   in
(ACI 530, 1.13.3.5)
[THE WALL PILASTER DESIGN IS ADEQUATE.]

WALL GROUTED ? (Yes, No)   No   , (only pilaster grouted)

ANALYSIS

REINFORCEMENT AREA AT ONE SIDE	A_s	=	2.37	in^2
TOTAL EFFECTIVE AREA	A_n	=	798	in^2
NEUTRAL AXIS FROM WALL FACE	y_t	=	9	in
NET EFFECTIVE MOMENT OF INERTIA	I_n	=	39198	in^4
RADIUS OF GYRATION	r	=	7.01	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi
MODULAR RATIO	n	=	21.48	

TRANSFORMED AREA $A_t = A_n+(-\rho+n\rho+n\rho')\,c\,b =$ 908 in^2

PILASTER REINF. RATIO $\rho = \rho'$ = 0.005
DISTANCE d' = 2.25 , d = 21.38 in
ALLOWABLE STRESS FACTOR   SF = 1.333
MAX. TIES SPACING (2106A.5.3.2) S_{max} = 8 in
NEUTRAL AXIS DEPTH FACTOR
$k = \{[n\rho+(2n-1)\rho']^2+2[n\rho+(2n-1)\rho(d'/d)]\}^2 - [n\rho+(2n-1)\rho']$
= 0.278
LEVER-ARM FACTOR $j = 1-k/3$ = 0.907

CHECK VERTICAL REINFORCEMENT LIMITATION (ACI 530, 2.1.6.4)

$A_{s,total}$ = 4.74 in^2 > $0.005A_e = 0.005bd$ = 2.53 in^2 **[Satisfactory]**
< $0.04A_e = 0.04bd$ = 20.21 in^2 **[Satisfactory]**

AXIAL LOAD AT MIDDLE OF THE PILASTER

$$P_{Mid} = P + (half\quad col.\quad weight) = \quad 22.353\ \text{k}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_{Mid} = M + (0.1)\left(\frac{Pd}{2}\right) = \quad 107.1\ \text{ft-kips}$$

CHECK IF THERE IS TENSILE STRESS IN CROSS SECTION

P_{Mid} / A = 28 psi **<** $M_{Mid} / (I_n / y)$ = 309 psi
(tensile exist)

AXIAL LOAD AT BASE OF THE PILASTER

$$P_t = P + (full\quad wall\quad weight) = \quad 26.681\ \text{k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(0.25 f'_m A_n + 0.65 F_S A_{st}\right)\left(1.0 - \left(\frac{h}{140r}\right)^2\right) = \quad 326.35\ \text{k}$$

[for h/r < 99]   **> Pt, [Satisfactory]**

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = \left(1.33_{wind\ \&\ seismic\ only}\right)(20\quad or\quad 24) = \quad 32.0\ \text{ksi}$$

ALLOWABLE STRESS DUE TO FLEXURE

$$F'_b = \left(SF - \frac{P_{Mid}}{P_a}\right)\left(0.33 f'_m\right) = \quad 0.626\ \text{ksi}$$

THE CORRESPONDING STRAIN IN THE TENSILE BARS IS

$$e_S = \frac{F_S}{} = \quad 0.0011$$

$$f_b = e_m E_m =$$

THE STRAIN IN THE EXTREME COMPRESSION FIBER IS

$$e_m = MIN\left[\left(\frac{kd}{d-kd}\right)e_s\ ,\ \frac{F'_b}{E_m}\right] = \quad 0.0004253$$

(steel governs)

(cont'd)

STRESS IN THE EXTREME FIBER DUE TO M_T

$f_b = e_m E_m =$ 0.574 ksi **< Fb', [Satisfactory]**

MOMENT DUE TO THE MASONRY

$M_m = \frac{1}{2} bkd\, f_b (d - \frac{kd}{3}) =$ 65.21 ft-kips

STRAIN IN THE COMPRESSION BARS

$e_S' = \left(\frac{kd - d'}{kd} \right) e_m =$ 0.0002644

STRESS IN THE COMPRESSION BARS

$f_S' = 2 E_S e_S' =$ 15.334 ksi

MOMENT DUE TO THE COMPRESSION BARS

$M_S' = f_S' A_S (d - d') =$ 57.94 ft-kips

ALLOWABLE BENDING MOMENT

$M = M_S' + M_m =$ 123.15 ft-kips

> Mt, [Satisfactory]

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$f_v = \frac{V}{jbd}$ = 39 psi

(Sec. 2107.2.17)

< $F_v = (SF) MIN(\sqrt{f_m'} \;,\; 50) =$ 51.64 psi **[Satisfactory]**

< $F_v = (SF) MIN(3\sqrt{f_m'} \;,\; 150) =$ 154.92 psi **[Satisfactory]**

$\frac{V}{F_s d} =$ 0.32 in^2 / ft < $\frac{A_v}{s} =$ 0.60 in^2 / ft **(No shear reinf. Reqd)**

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

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PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

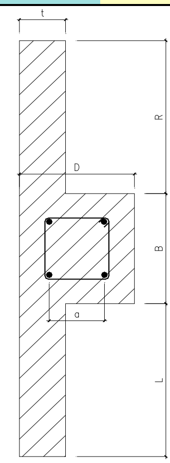
Flush Wall Pilaster Design Based on TMS 402-08 & IBC 09

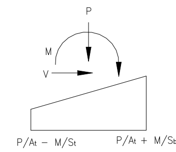
INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SEISMIC DESIGN CATEGORY (5=Gravity)			3	D
SERVICE AXIAL LOAD	P	=	50	k, at pilaster center
SERVICE SHEAR LOAD	V	=	9.5	k, (may be negative)
SERVICE MOMENT	M	=	8.2	ft-k, (posite or negative)

SECTION DIMENSIONS (in)

	input		*effective*
t =	8	t_e =	7.63
D =	24	d =	23.63
L =	48	L_e =	48.00
B =	24	b =	23.63
R =	80	R_e =	48.00





WALL PILASTER HEIGHT h = 15 ft

VERTICAL REINF. (EACH SIDE) 2 # 8

HORIZ. TIES 2 legs # 4 @ 8 in o.c.

DISTANCE PILASTER REINF. a = 19.13 in (TMS 402, 1.15.3.5)

WALL GROUTED ? (Yes, No) No , (only pilaster grouted)

[THE WALL PILASTER DESIGN IS ADEQUATE.]

ANALYSIS

TOTAL REINFORCEMENT AREA	A_s	=	3.16	in^2
TOTAL EFFECTIVE AREA	A_n	=	798	in^2
NEUTRAL AXIS FROM WALL FACE	y_t	=	9	in
NET MOMENT OF INERTIA	I_n	=	39198	in^4
RADIUS OF GYRATION	r	=	7.01	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48	
PILASTER REINF. RATIO	ρ	=	0.006	
ALLOWABLE STRESS FACTOR	SF	=	1.333	
MAX. TIES SPACING (TMS 1.17.4)	S_{max}	=	8	in

TRANSFORMED WALL PILASTER AREA

$$A_t = A_n + bd\left(2n-1\right)\rho = \quad 931 \ in^2$$

CHECK VERTICAL REINFORCEMENT LIMITATION OF PILASTER (TMS 402 1.14.1.2)

A_s = 3.16 in^2 > 0.0025 bd = 1.40 in^2 **[Satisfactory]**

< 0.04 bd = 22.34 in^2 **[Satisfactory]**

ALLOWABLE STRESS DUE TO AXIAL LOAD ONLY

$$F_a = \left(SF\right)\left(0.25 f'_m\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 0.483 \text{ ksi}$$

[for h/r < 99]

AXIAL STRESS AT MIDHEIGHT OF THE PILASTER

$$f_a = \frac{P+\left(half \quad wall \quad weight\right)}{A_t} = \quad 0.060 \text{ ksi}$$

< Fa, [Satisfactory]

ALLOWABLE STRESS DUE TO FLEXURE

$$F_b = \left(SF\right)\left(0.33 f'_m\right) = \quad 0.660 \text{ ksi}$$

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = \left(1.33 \ \ or \ \ 1.0\right)\left(20 \ \ or \ \ 24\right) = \quad 32.0 \text{ ksi}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_T = M + \left(0.1\right)\left(\frac{Pd}{2}\right) = \quad 13.1 \text{ ft-kips}$$

TRANSFORMED MOMENT OF INERTIA

$$I_t = I_n + \left(2n-1\right)A_s\left(\frac{a}{2}\right)^2 = \quad 51330 \ in^4$$

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = \frac{M_T y}{I_t} = \quad 0.029 \text{ ksi}$$

< fa, [Satisfactory, the section is uncracked]

MAX.STRESS COMBINED AXIAL & FLEXURE

$$f_m = f_a + f_b = \quad 0.089 \text{ ksi}$$

< Fb, [Satisfactory]

MAX. REINF. STRESS COMBINED AXIAL & FLEXURE

$$f_S = 2n\left(f_a + \frac{a}{d} f_b\right) = \quad 3.6 \text{ ksi}$$

< Fs, [Satisfactory]

AXIAL LOAD AT BASE OF THE WALL PILASTER

$$P_t = P + \left(full \quad col. \quad weight\right) = \quad 61.227 \text{ k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(\left(SF\right)0.25 f'_m A_n + 0.65 F_S A_S\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 449.26 \text{ k}$$

[for h/r < 99] **> Pt, [Satisfactory]**

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$$f_v = \frac{V}{bd} = \quad 17 \text{ psi}$$

(TMS 402 2.3.5.2.1)

$< \quad F_v = \left(SF\right)MIN\left(\sqrt{f'_m} \ , \ 50\right) = \quad$ 51.64 psi **[Satisfactory]**

$< \quad F_v = \left(SF\right)MIN\left(3\sqrt{f'_m} \ , \ 150\right) = \quad$ 154.92 psi **[Satisfactory]**

$$\frac{V}{F_s d} = \quad 0.15 \ in^2/ft \quad < \quad \frac{A_v}{s} = \quad 0.60 \ in^2/ft$$

(No shear reinf. Reqd)

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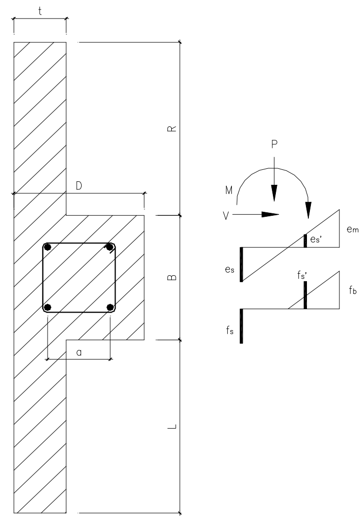
Flush Wall Pilaster Design Based on TMS 402-08 & IBC 09

INPUT DATA & DESIGN SUMMARY



SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
SEISMIC DESIGN CATEGORY			4	E or F

(1 = B, 2 = C, 3 = D, 4 = E or F, 0 = WIND, 5 = GRAVITY)

SERVICE AXIAL LOAD	P	=	11.5	k, @ center top of pilaster
MAX SHEAR LOAD	V	=	20	k
MOMENT AT MIDHEIGHT	M	=	106	ft-k, (posite or negative)

SECTION DIMENSIONS (in)

	input		*effective*
t =	8	t_e =	7.63
D =	24	c =	23.63
L =	48	L_e =	48.00
B =	24	b =	23.63
R =	80	R_e =	48.00

WALL PILASTER HEIGHT h = 29 ft

VERTICAL REINF. (EACH SIDE) 3 # 8

HORIZ. TIES 2 legs # 4 @ 8 in o.c.

DISTANCE PILASTER REINF. a = 19.13 in

(TMS 402, 1.15.3.5)

[THE WALL PILASTER DESIGN IS ADEQUATE.]

WALL GROUTED ? (Yes, No) No , (only pilaster grouted)

ANALYSIS

REINFORCEMENT AREA AT ONE SIDE	A_s	=	2.37	in^2
TOTAL EFFECTIVE AREA	A_n	=	798	in^2
NEUTRAL AXIS FROM WALL FACE	y_t	=	9	in
NET EFFECTIVE MOMENT OF INERTIA	I_n	=	39198	in^4
RADIUS OF GYRATION	r	=	7.01	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi
MODULAR RATIO	n	=	21.48	

TRANSFORMED AREA $A_t = A_n+(-\rho+n\rho+n\rho')\,c\,b$ = 908 in^2

PILASTER REINF. RATIO $\rho = \rho'$ = 0.005

DISTANCE d' = 2.25 , d = 21.38 in

ALLOWABLE STRESS FACTOR SF = 1.333

MAX. TIES SPACING (ACI 1.1.4.6) S_{max} = 8 in

NEUTRAL AXIS DEPTH FACTOR

$k = \{[n\rho+(2n-1)\rho']^2+2[n\rho+(2n-1)\rho(d'/d)]\}^2 - [n\rho+(2n-1)\rho']$ = 0.278

LEVER-ARM FACTOR $j = 1-k/3$ = 0.907

CHECK VERTICAL REINFORCEMENT LIMITATION (TMS 402 1.14.1.2)

$A_{s,total}$ = 4.74 in^2 > $0.005A_e = 0.005bd$ = 2.53 in^2 **[Satisfactory]**

< $0.04A_e = 0.04bd$ = 20.21 in^2 **[Satisfactory]**

AXIAL LOAD AT MIDDLE OF THE PILASTER

$$P_{Mid} = P + (half\ \ col.\ \ weight) =$$ 22.353 k

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_{Mid} = M + (0.1)\left(\frac{Pd}{2}\right) =$$ 107.1 ft-kips

CHECK IF THERE IS TENSILE STRESS IN CROSS SECTION

P_{Mid} / A = 28 psi **<** $M_{Mid} / (I_n / y)$ = 309 psi

(tensile exist)

AXIAL LOAD AT BASE OF THE PILASTER

$$P_t = P + (\ full\ \ wall\ \ weight) =$$ 26.681 k

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(0.25 f'_m A_n + 0.65 F_S A_{st}\right)\left(1.0 - \left(\frac{h}{140r}\right)^2\right) =$$ 326.35 k **> Pt, [Satisfactory]**

[for h/r < 99]

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = (1.33_{wind\ \&\ seismic\ only})(20\ \ or\ \ 24) =$$ 32.0 ksi

ALLOWABLE STRESS DUE TO FLEXURE

$$F'_b = \left(SF - \frac{P_{Mid}}{P_a}\right)\left(0.33 f'_m\right) =$$ 0.626 ksi

THE CORRESPONDING STRAIN IN THE TENSILE BARS IS

$$e_S = \frac{F_S}{} =$$ 0.0011

$$f_b = e_m E_m =$$

THE STRAIN IN THE EXTREME COMPRESSION FIBER IS

$$e_m = MIN\left[\left(\frac{kd}{d-kd}\right)e_s\ \ ,\ \ \frac{F'_b}{E_m}\right] =$$ 0.0004253

(steel governs)

(cont'd)

STRESS IN THE EXTREME FIBER DUE TO M_T

$f_b = e_m E_m =$ 0.574 ksi **< Fb', [Satisfactory]**

MOMENT DUE TO THE MASONRY

$M_m = \frac{1}{2} bkd\, f_b (d - \frac{kd}{3}) =$ 65.21 ft-kips

STRAIN IN THE COMPRESSION BARS

$e'_S = \left(\frac{kd - d'}{kd}\right) e_m =$ 0.0002644

STRESS IN THE COMPRESSION BARS

$f'_S = 2 E_S e'_S =$ 15.334 ksi

MOMENT DUE TO THE COMPRESSION BARS

$M'_S = f'_S A_S (d - d') =$ 57.94 ft-kips

ALLOWABLE BENDING MOMENT

$M = M'_S + M_m =$ 123.15 ft-kips

> Mt, [Satisfactory]

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$f_v = \frac{V}{jbd}$ = 39 psi

(TMS 402 2.3.5.2.1)

< $F_v = (SF) MIN(\sqrt{f'_m}\ ,\ 50) =$ 51.64 psi **[Satisfactory]**

< $F_v = (SF) MIN(3\sqrt{f'_m}\ ,\ 150) =$ 154.92 psi **[Satisfactory]**

$\frac{V}{F_s d} =$ 0.32 in^2 / ft < $\frac{A_v}{s} =$ 0.60 in^2 / ft **(No shear reinf. Reqd)**

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

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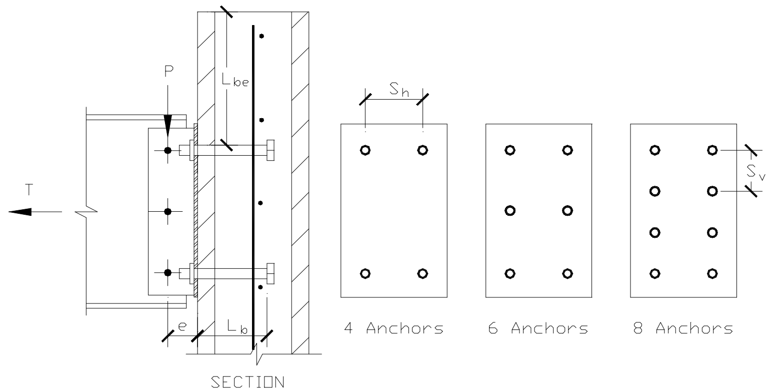
Beam to Wall Anchorage Design Based on TMS 402 / IBC 09

INPUT DATA & DESIGN SUMMARY



MASONRY STRENGTH

f_m' = 1.5 ksi

FASTENER YIELD STRESS

f_y = 60 ksi

SERVICE VERTICAL LOAD

P = 3.2 kips

VERTICAL LOAD TO FACE DIMENSION

e = 6 in

SERVICE HORIZONTAL TENSION LOAD

T = 1.12 kips

WALL THICKNESS

b = 8 in

FASTENER DIAMETER	ϕ	=	3/4	in
EFFECTIVE EMBEDMENT	L_b	=	5	in
DISTANCE TO WALL TOP	L_{be}	=	16	in
NO. ANCHORS (4,6,or 8)	n	=	4	
SPACING	S_h	=	8	in, o.c.
VERTICAL SPACING	S_v	=	11	in, o.c.
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	

[THE BEAM ANCHORAGE DESIGN IS ADEQUATE.]

MIN. FACE PLATE SIZE - 11 in x 14 in

ANALYSIS

MAX TENSION STRESS, ASD

b_a = 1.2 kips / fastener

MAX SHEAR STRESS, VERTICAL

b_v = 0.8 kips / fastener

GOVERNING SPACING

S = 8 in, o.c.

ANCHOR LOCATION & FORCE

Anchor	X (in)	Y (in)	X_{CG}^2	Y_{CG}^2	N_a	V_a
1	-4.00	-5.50	16.00	30.25	1.2	0.8
2	4.00	-5.50	16.00	30.25	1.2	0.8
3	-4.00	5.50	16.00	30.25	-0.6	0.8
4	4.00	5.50	16.00	30.25	-0.6	0.8

CHECK MIN. EMBEDMENT (TMS 402 1.16.6)

$L_{b,min}$ = MIN[4ϕ , 2] = 2.00 in < L_b [SATISFACTORY]

CHECK TENSION CAPACITY FOR A FASTENER (TMS 402 2.1.4.3.1.1)

$B_a = \text{MIN}[\ 1.25A_{pt}(f_m')^{0.5}\ ,\ 0.6A_bf_y] =$ 3.41 kips / fastener

> $k\ b_a =$ 0.86 kips / fastener [SATISFACTORY]

Where $L = \text{MIN}[\ L_b\ ,\ L_{be}] =$ 5.00 in, conservative value

$\theta = \cos^{-1}(0.5S_h / L) =$ 0.64 rad

$A_{seg} = L^2 [\ \theta - 0.5\ \sin(2\theta)] =$ 4.09 in^2

$A_{pt} = \pi\ L^2 - 2A_{seg} =$ 70.36 in^2 (TMS 402 1.16.2)

$A_b = \pi\ \phi^2 / 4 =$ 0.44 in^2

k = 3/4

CHECK SHEAR CAPACITY (TMS 402 2.1.4.3.2)

$B_v = \text{MIN}[1.25A_{pv}(f_m')^{0.5}\ ,\ 350(A_bf_m')^{1/4}\ ,\ 2.5A_{pt}(f_m')^{0.5}\ ,\ 0.36A_bf_y] =$ 1.78 kips / fastener

> $b_v =$ 0.80 kips / fastener , Gravity only [SATISFACTORY]

Where $\theta = \cos^{-1}(0.5S / L_{be}) =$ 1.32 rad

$A_{seg} = L_{be}^2 [\ \theta - 0.5\ \sin(2\theta)] =$ 275.47 in^2

$A_{pv} = 0.5(\pi\ L_{be}^2 - 2A_{seg}) =$ 126.65 in^2 (TMS 402 1.16.3)

CHECK COMBINED SHEAR AND TENSION CAPACITY (TMS 402 2.1.4.3.3)

$b_a / B_a + b_v / B_v =$ 0.79 < 1.33 [SATISFACTORY]

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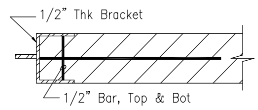
Collector to Wall Connection Design Based on TMS 402-08 / IBC 09

INPUT DATA & DESIGN SUMMARY

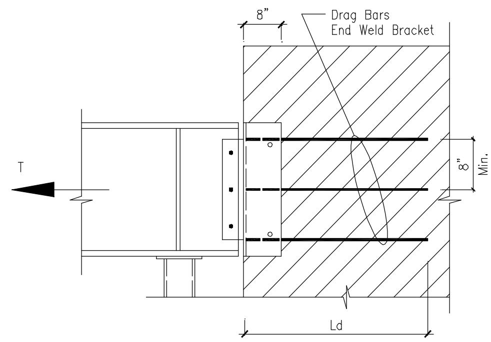


MASONRY STRENGTH		f_m' =	1.5	ksi	
DRAG BARS	3	#	7	x	60 in, Long (A 706)
BAR YIELD STRESS		f_y =	60	ksi	
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)				Yes	

SERVICE COLLECTOR FORCE (ASCE 7-05, 12.10.2.1 & 12.4.3.2)

T = 55 kips, ASD level

[THE COLLECTOR TO WALL DESIGN IS ADEQUATE.]



ANALYSIS

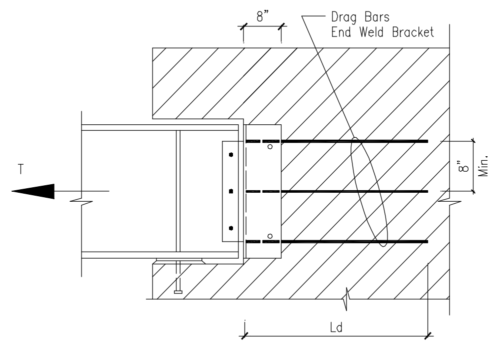
CHECK DRAG BAR LENGTH (TMS 402-08, 2.1.9.3)

$L_d = \psi_e$ MAX[$0.13\ d_b^2\ f_y\ \gamma\ /\ (K\ f_m'^{\ 0.5})$, 12]

= 68 d_b = 59 in

< 60 in [SATISFACTORY]

where	d_b	=	0.875	in
	ψ_e	=	1.0	(1.5 for epoxy-coated)
	γ	=	1.3	
	c	=	3.3775	in, masonry cover
	s	=	7.125	in, adjacent bars clear spacing
	K	=	MIN(c , s , $5d_b$) =	3.3775 in



CHECK DRAG BAR STRENGTH (TMS 402-08, 2.3.2.1)

$T_{allow} = F_s\ A_s$ = 57.60 kips

> T [SATISFACTORY]

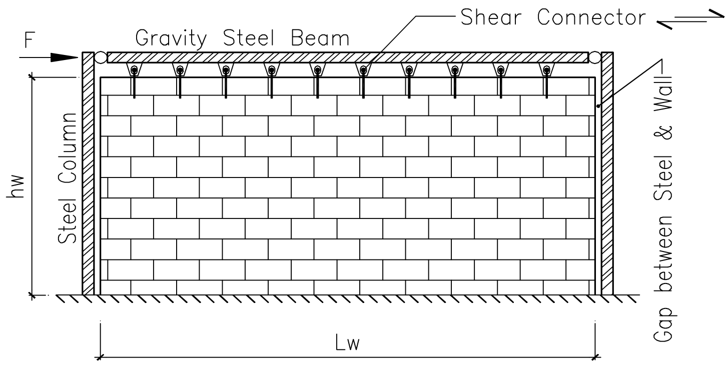
where	F_s	=	32	ksi, (IBC/CBC 1605.3.2 & TMS 402, 2.3.2.1)
	A_s	=	1.8	in^2

Hybrid Masonry Wall Design Based on TMS 402-11

DESIGN CRITERIA

1. Masonry wall is within steel frame. There are gaps between steel column/beam and the wall.
2. All gravity loads are supported by steel beam and steel column.
3. Lateral load, F from diaphragm to beam, is transferred to the masonry wall by shear connectors.
4. Shear connector (fuse detail) can only have horizontal shear capacity, without vertical support.

INPUT DATA & DESIGN SUMMARY



THE WALL DESIGN IS ADEQUATE.
(10 shear connectors at 22 in. o.c.)

HEIGHT OF WALL	h_w =	14	ft
LENGTH OF WALL	L_w =	20	ft
MASONRY STRENGTH	f_m' =	3	ksi
REBAR YIELD STRESS	f_y =	60	ksi

THICKNESS OF WALL t_w = 8 in

REINFORCING OF WALL 2 # 6 at each ends, with 4 in center to edge.

A_{sh}, Horizontal 1 # 6 @ 16 in o.c.

A_{sv}, Vertical 1 # 6 @ 24 in o.c.

SERVICE LATERAL LOAD F = 30 kips, ASD level

SHEAR CONNECTOR CAPACITY v = 3 kips/connector, ASD level

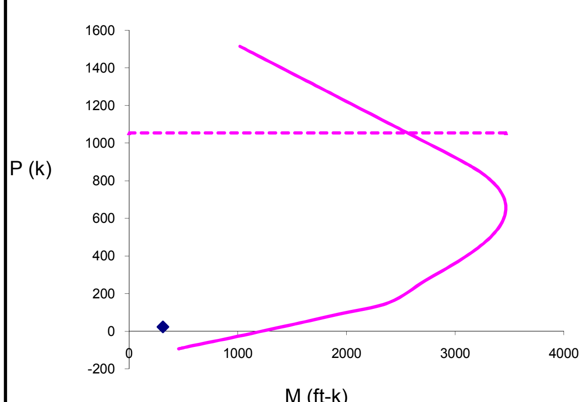
ANALYSIS

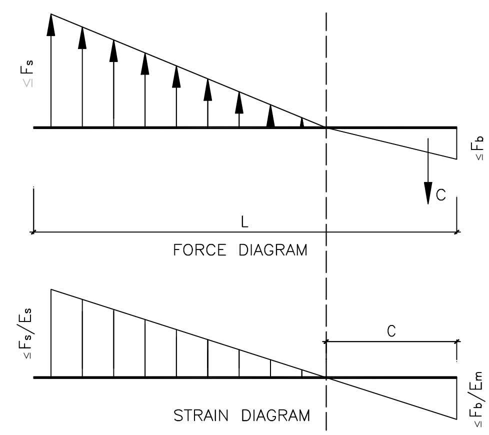
CHECK SPACING OF SHEAR CONNECTORS

n = F / v = 10 , number of connectors

s = L_w / (n + 1) = 22 in, shear connector spacing > 8 in [Satisfactory]

CHECK FLEXURAL & AXIAL CAPACITY BY ALLOWABLE STRESS DESIGN (ASD)





P = 22 kips, wall self weight, M = 312 ft-kips, at wall bottom. [Satisfactory]

CHECK SHEAR CAPACITY (ASD), (TMS 2.3.6)

$$F_v = MAX\left\{(SF)\left[\frac{1}{4}\left(4-1.75MIN\left(1\ ,\ \frac{M_T}{Vd}\right)\right)\sqrt{f'_m}+0.25\frac{P}{A_n}\right]+0.5\frac{A_vF_sd}{A_ns}\ ,\ (SF)\left[\frac{1}{2}\left(4-1.75MIN\left(1\ ,\ \frac{M_T}{Vd}\right)\right)\sqrt{f'_m}+0.25\frac{P}{A_n}\right]\right\}$$

= 103 psi > 1.5 f_v = 25 psi [Satisfactory]

(factor 1.5 from TMS 402 1.18.3.2.6.1.2)

$F_{v,\ Maximum}$ $= (SF)\,MIN\left[3\ ,\ MAX\left(2\ ,\ 2+\frac{4}{3}\left(1-\frac{M_T}{Vd}\right)\right)\right]\sqrt{f'_m} =$ 144 psi > 1.5 f_v

[Satisfactory]

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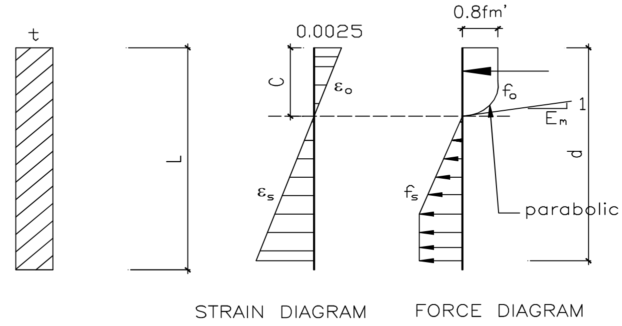
Post-Tensioned Masonry Shear Wall Design Based on TMS 402-11 (SD Method)

DESIGN CRITERIA

1. The Post-Tensioned Masonry Shear Wall can be as the Self-Centering Lateral Frame, which is good against to seismic/wind loads.
2. The PT walls use smooth steel rods as the post-tensioned rods, in lieu of soft tendons, so easy construction.
3. The PT walls can be the exterior walls with the energy savings of LEED Gold, because no grouted all cells were required.

INPUT DATA & DESIGN SUMMARY

REBAR YIELD STRESS	f_y =	60	ksi
MASONRY (CMU) STRENGTH	f_m' =	3	ksi
LENGTH OF SHEAR WALL (within vertical control joints)	L =	12	ft
EQUIVALENT THICKNESS	t =	7.25	in
EFFECTIVE HEIGHT OF WALL	h =	16	ft



SD LEVEL SECTION LOADS

P_u = 280 kips, (not including PT)

M_u = 900 ft-kips

V_u = 193 kips

THE WALL DESIGN IS ADEQUATE.

VERTICAL REINFORCING

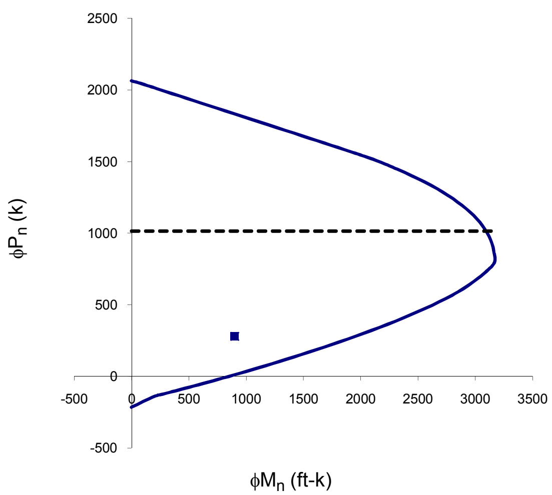
1 # 5 @ 32 in o.c.

ϕ 7/16 in PT (smooth rods) @ 16 in o.c., with effective 5000 lbs / rod

(f_{se} = 14.5513 ksi , 0.24 f_y)

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY



ε_{mu} = 0.0025 , (TMS 3.3.2.c)

ϕ = 0.8 , (TMS 402 4.4.3.3)

d = 140 in

ε_o = 0.00178

c_b = 77 in, (balance point between Tension Controlled and Compression Controlled.)

P_u = 280 kips

< ϕP_n = 1013 kips, (TMS 402 3.3.4.1.1)

M_u = 900 ft-kips

< ϕM_n = 1946 ft-kips, at P_u level.

[Satisfactory]

CHECK SHEAR CAPACITY

V_u = 193 kips < ϕV_{nm} = 216 kips, including PT, (TMS 402 3.3.4.1.2)

[Satisfactory]

Purchaser's LOGO	PROJECT :		PAGE :	
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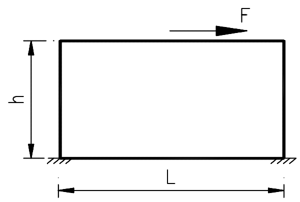
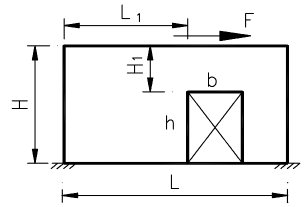
Masonry Shear Wall with Opening Design Using Finite Element Method

INPUT DATA & DESIGN SUMMARY

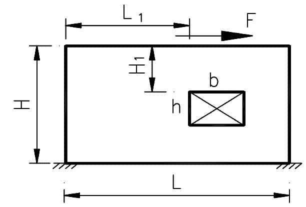
DIMENSIONS	L =	30	ft
	H =	20	ft
	thk =	7.63	in
OPENING	b =	5	ft
	h =	10	ft
LOCATION	L_1 =	10	ft
	H_1 =	5	ft
ONE STORY ?		Yes	



R = 4199 kips (Rigidity)
Def. = 0.09 in (Deflection)



Not Apply



R = 3612 kips
Def. = 0.11 in

(Cantilever Wall with Fixed Base Only) `

The opening reduced the lateral load capacity by 14%.

MASONRY (1=CMU, 2=BRICK) 1 CMU f_m' = 1.5 ksi

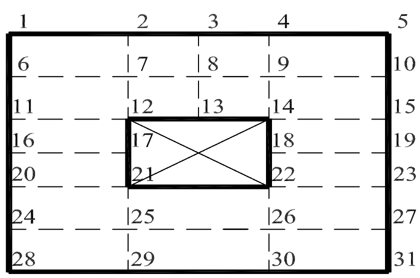
f_y = 60 ksi

JAMB BARS 2 # 6 , four sides opening w/ embed 48 bar dia. min.

LATERAL LOAD F = 345 kips, SD level

ANALYSIS

DETERMINE SHEAR WALL LATERAL RIGIDITY



Element		Dimension (ft)		Thick.
No.	Joints	X	Y	(in)
1	1, 2, 6, 7	*10.0*	*2.5*	*7.63*
2	2, 3, 7, 8	*2.5*	*2.5*	*7.63*
3	3, 4, 8, 9	*2.5*	*2.5*	*7.63*
4	4, 5, 9, 10	*15.0*	*2.5*	*7.63*
5	6, 7, 11, 12	*10.0*	*2.5*	*7.63*
6	7, 8, 12, 13	*2.5*	*2.5*	*7.63*
7	8, 9, 13, 14	*2.5*	*2.5*	*7.63*
8	9, 10, 14, 15	*15.0*	*2.5*	*7.63*
9	11, 12, 16, 17	*10.0*	*5.0*	*7.63*
10	14, 15, 18, 19	*15.0*	*5.0*	*7.63*
11	16, 17, 20, 21	*10.0*	*5.0*	*7.63*
12	18, 19, 22, 23	*15.0*	*5.0*	*7.63*
13	20, 21, 24, 25	*10.0*	*2.5*	*7.63*
14	21, 22, 25, 26	*5.0*	*2.5*	*7.63*
15	22, 23, 26, 27	*15.0*	*2.5*	*7.63*
16	24, 25, 28, 29	*10.0*	*2.5*	*7.63*
17	25, 26, 29, 30	*5.0*	*2.5*	*7.63*
18	26, 27, 30, 31	*15.0*	*2.5*	*7.63*

	Solid Wall with Fixed at Top & Base				Solid Wall with Fixed at Base only			
	Reaction (kips)		Deflection (in)		Reaction (kips)		Deflection (in)	
Joint	X	Y	X	Y	X	Y	X	Y
1	937	-1738	*1*	0.00	577	0	*1*	0.39
2	1104	-100	*1*	0.00	804	0	*1*	0.08
3	435	-40	*1*	0.00	402	0	*1*	0.03
4	1466	56	*1*	0.00	1425	0	*1*	-0.02
5	1579	1890	*1*	0.00	990	0	*1*	-0.33
6	0	0	0.87	0.06	0	0	0.85	0.39
7	0	0	0.89	0.00	0	0	0.85	0.08
8	0	0	0.89	0.00	0	0	0.85	0.03
9	0	0	0.89	0.00	0	0	0.85	-0.02
10	0	0	0.88	-0.04	0	0	0.87	-0.33
11	0	0	0.75	0.11	0	0	0.71	0.37
12	0	0	0.76	0.01	0	0	0.70	0.08
13	0	0	0.80	-0.01	0	0	0.73	0.03
14	0	0	0.76	0.00	0	0	0.70	-0.02
15	0	0	0.76	-0.08	0	0	0.73	-0.31
16	0	0	0.49	0.14	0	0	0.45	0.29
17	0	0	0.50	0.02	0	0	0.42	0.06
18	0	0	0.50	0.00	0	0	0.42	-0.01
19	0	0	0.51	-0.10	0	0	0.47	-0.25
20	0	0	0.24	0.10	0	0	0.22	0.17
21	0	0	0.23	0.01	0	0	0.18	0.04
22	0	0	0.24	0.00	0	0	0.18	0.00
23	0	0	0.25	-0.08	0	0	0.22	-0.14
24	0	0	0.12	0.06	0	0	0.11	0.09
25	0	0	0.11	0.01	0	0	0.08	0.02
26	0	0	0.11	0.00	0	0	0.08	0.00
27	0	0	0.13	-0.05	0	0	0.11	-0.08
28	*-1096*	-1727	0.00	0.00	*-1071*	-2449	0.00	0.00
29	*-1147*	-249	0.00	0.00	*-689*	-545	0.00	0.00
30	*-1808*	27	0.00	0.00	*-1249*	26	0.00	0.00
31	*-1470*	1882	0.00	0.00	*-1191*	2968	0.00	0.00
R =	*-5520*	kips			R = *-4199*	kips		

(cont'd)

Joint	Opening Wall with Fixed at Top & Base				Opening Wall with Fixed at Base only			
	Reaction (kips)		Deflection (in)		Reaction (kips)		Deflection (in)	
	X	Y	X	Y	X	Y	X	Y
1	633	-1534	*1*	0.00	375	0	*1*	0.33
2	992	675	*1*	0.00	659	0	*1*	0.02
3	461	-30	*1*	0.00	567	0	*1*	0.03
4	1168	-792	*1*	0.00	1160	0	*1*	0.04
5	1434	1801	*1*	0.00	851	0	*1*	-0.30
6	0	0	0.90	0.06	0	0	0.87	0.33
7	0	0	0.91	-0.02	0	0	0.88	0.01
8	0	0	0.90	0.00	0	0	0.88	0.03
9	0	0	0.90	0.02	0	0	0.87	0.04
10	0	0	0.89	-0.04	0	0	0.87	-0.30
11	0	0	0.78	0.10	0	0	0.74	0.32
12	0	0	0.82	-0.05	0	0	0.76	0.00
13	0	0	0.84	-0.01	0	0	0.79	0.02
14	0	0	0.79	0.04	0	0	0.74	0.05
15	0	0	0.77	-0.07	0	0	0.75	-0.29
16	0	0	0.48	0.14	0	0	0.44	0.27
17	0	0	0.51	-0.07	0	0	0.44	-0.03
18	0	0	0.48	0.06	0	0	0.42	0.07
19	0	0	0.51	-0.11	0	0	0.47	-0.24
20	0	0	0.21	0.10	0	0	0.19	0.15
21	0	0	0.19	-0.03	0	0	0.14	-0.02
22	0	0	0.19	0.04	0	0	0.15	0.04
23	0	0	0.23	-0.08	0	0	0.20	-0.14
24	0	0	0.10	0.06	0	0	0.09	0.08
25	0	0	0.09	-0.02	0	0	0.07	-0.01
26	0	0	0.09	0.02	0	0	0.07	0.02
27	0	0	0.11	-0.04	0	0	0.10	-0.07
28	*-929*	-1567	0.00	0.00	*-900*	-2163	0.00	0.00
29	*-842*	608	0.00	0.00	*-469*	347	0.00	0.00
30	*-1684*	-933	0.00	0.00	*-1267*	-952	0.00	0.00
31	*-1232*	1771	0.00	0.00	*-976*	2769	0.00	0.00
R =	*-4688*	kips		R =	*-3612*	kips		

CHECK MASONRY SHEAR WALL OPENING

E = 1350 ksi υ = 0.25 (Poisson's ratio)

Min (L_1 , L - L_1 - b , H_1) = 60 in > Max (3 thk , 8 in) = 23 in [Satisfactory]

$\phi\, f_y\, A_g$ = 48 kips > Max Corner Force = 32 kips [Satisfactory]

2 # 6 > 2 # 5 in [Satisfactory]

(Jamb Bars)

Purchaser's LOGO

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Anticipated Cracking Design of Masonry Wall Based on TMS 402-16

DESIGN CRITERIA

1. The anticipated cracking may be from Shrinkage, Thermal Variation, and Carbonation.
2. Typical drying shrinkage will result in a reduction of 2 x 10^{-4} to 4.5 x 10^{-4} inch per inch of wall.
3. Typical expansion and construction will result in a change of 2.5 x 10^{-6} to 5.5 x 10^{-6} inch per inch of wall per ^{o}F.
4. Typical carbonation will result in a reduction of 2.5 x 10^{-4} inch per inch of wall.

INPUT DATA & DESIGN SUMMARY

LENGTH OF WALL	L =	60	*ft, (18.29 m)*
TEMPERATURE REDUCTION	^{o}t =	85	*degree F, (29.4 degree C)*
MASONRY MATERIAL ***(1 = Clay, 2 = Concrete, or 3 = AAC)***		2	*Concrete Masonry*

The Total Anticipated Cracking:	***0.60***	***to***	***0.78***	***inch***
	(15 mm)		*(20 mm)*	

ANALYSIS

Shrinkage:	0.14	to	0.32	inch	
Thermal:	0.15	to	0.34	inch	***, (Use 0.2754 inch, TMS 402-16 4.2.3)***
Carbonation:	0.18	to	0.18	inch	
Sum:	0.60	to	0.78	inch	

Reference: William C. Bracken, PE, SI: ***Cracking Within Existing Concrete Masonry Walls, When are Calculations Required?***
2017 Structural Engineering Summit – Washington, D.C.

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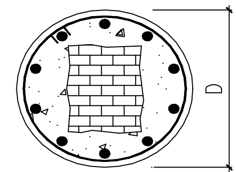
Existing Column Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-14/TMS 402-16

DESIGN CRITERIA

1. The existing column can be either concrete or masonry. The rebars capacity in the existing column ignored conservatively.
2. Enhanced column to circular section, using higher strength concrete, is good for bi-axial bending from eccentricity of axial load. Before place new concrete, to rough, clear and wet the existing surface is required.

INPUT DATA & DESIGN SUMMARY



COLUMN SECTION

EXISTING COLUMN SECTION SIZE	*Width* =	**30**	*in, (762 mm).*
	Height =	**36**	*in, (914 mm).*
EXISTING COLUMN STRENGTH	f_m'/f_c' =	**2.79**	*ksi, (19 MPa)*
COLUMN OUTSIDE DIAMETER	D =	**48**	*in, (1219 mm).*
NEW STEEL YIELD STRESS	F_y =	**60**	*ksi, (414 MPa)*
NEW CONCRETE STRENGTH	f_c' =	**5**	*ksi, (34 MPa)*
THE MAX CONCRETE STRESS	β =	**0.85**	*f_c' (0.85 on ACI, 1.0 on AISC)*
THE MAX CONCRETE STRAINS	ε_{max} =	**0.003**	*, (ASCE 41-17 10.3.3.1)*

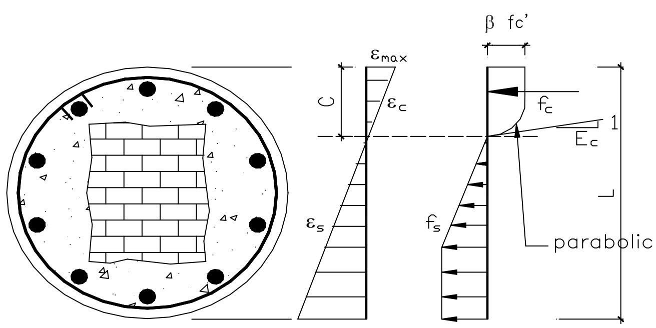
THE DESIGN IS ADEQUATE.

SECTION FORCES	V_u =	**202**	*kips, (899 kN)*			
(Strength Level, LRFD)	P_u =	**980**	*kips, (4359 kN)*	*<== For this P_u, the max ϕM_n =*	1401.9	*ft-kips, (1901 kN-m).*
	M_u =	**1300**	*ft-kips, (1763 kN-m)*	*<== For this M_u, the max ϕP_n =*	3955.4	*kips, (17594 kN),*
				P_u / ϕP_n =	0.248	*, (0.3 / ϕ, ACI 318-14 18.7.5)*

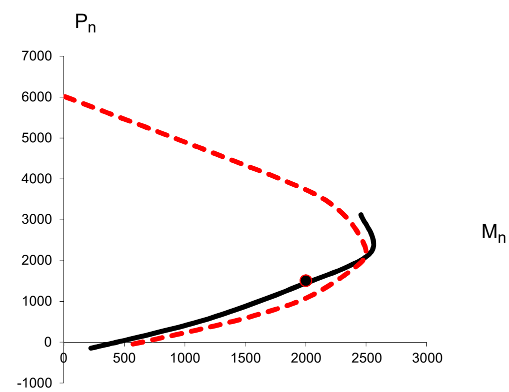
NEW VERTICAL REINFORCEMENT	**10**	#	**7**	Cover	**0.75**	*in, (19 mm)*	
NEW LATERAL REINFORCEMENT	#	**4**	@	**8**	*in. (203 mm), o.c.*		

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY



STRAIN DIAGRAM FORCE DIAGRAM



Solid Line - Tension Controlled

Dash Line - Compression Controlled

P_u / ϕ = 1507.7 *kips, (6707 kN)*

M_{nc} @ P_u / ϕ = 2156.7 *ft-kips, (2924 kN-m)*

> M_u / ϕ = 2000.0 *ft-kips, (2712 kN-m)*

[Satisfactory]

where ϕ = 0.650 , (TMS 402-16 9.1.4.1, ACI 318-14 21.2 & AISC 341-16 B3.2)

CHECK SHEAR CAPACITY (ACI 318-14 10 & 22.5)

V_u / ϕ = 269.3 *kips, (1198 kN)* **<** $V_c + V_s$ = 278.1 *kips, (1237 kN)*

where ϕ = 0.75 , (ACI 318-11 21.2) **[Satisfactory]**

V_c = 138.6 *kips, (617 kN)*

V_s = 139.5 *kips, (621 kN)*

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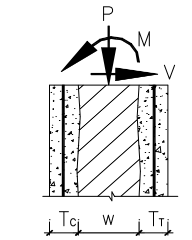
Existing Wall Enhancement Based on 2015 IEBC, ASCE 41-17 & ACI 318-14/TMS 402-16

DESIGN CRITERIA

1. This software is to enhance wall capacity at perpendicular bending and vertical loads. The existing wall can be either concrete wall or masonry wall.
2. The new concrete and reinforcement can be on both sides, or only one side. Before new concrete/shotcrete, to rough, clear and wet the existing surface is required.

INPUT DATA & DESIGN SUMMARY



WALL SECTION

EXISTING WALL SECTION THICKNESS	w =	8	*in, (203 mm).*	
EXISTING WALL STRENGTH	*old* f_m' / f_c' =	1.95	*ksi, (13 MPa)*	
EXISTING WALL VERT. REINF.	1	# 5	@ 16	in o.c. (at middle)
NEW COMPRESSION THICKNESS	T_C =	5	*in, (127 mm).*	
# 5	@ 18	*in. (457 mm), o.c.*	Cover 1	*in, (25 mm)*
NEW TENSION THICKNESS	T_T =	4.75	*in, (121 mm).*	
# 5	@ 12	*in. (305 mm), o.c.*	Cover 0.75	*in, (19 mm)*

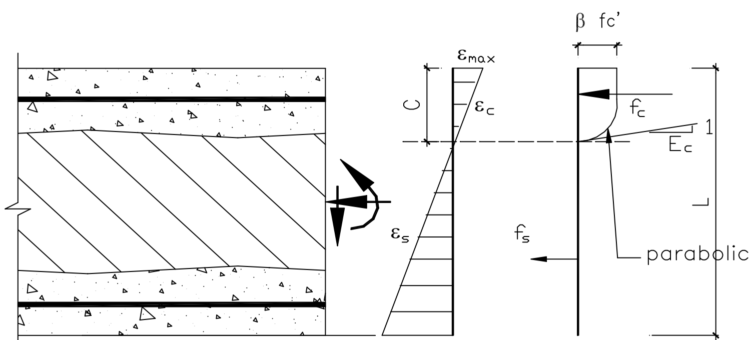
NEW STEEL YIELD STRESS	F_y =	60	*ksi, (414 MPa)*
NEW CONCRETE STRENGTH	f_c' =	5	*ksi, (34 MPa)*
THE MAX CONCRETE STRESS	β =	0.85	f_c' *(0.85 on ACI, 1.0 on AISC)*
THE MAX CONCRETE STRAINS	ε_{max} =	0.003	*, (ASCE 41-17 10.3.3.1)*

THE DESIGN IS ADEQUATE.

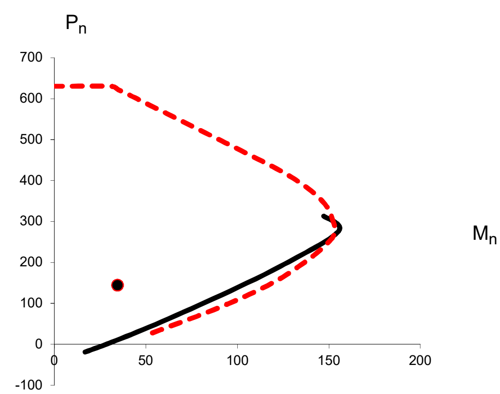
SECTION FORCES	V_u =	9.8	*kips, (44 kN), per foot*
(Strength Level, LRFD)	P_u =	130	*kips, (578 kN), per foot*
	M_u =	31	*ft-kips, (42 kN-m), per foot*

ANALYSIS

CHECK FLEXURAL & AXIAL CAPACITY



STRAIN DIAGRAM FORCE DIAGRAM



Solid Line - Tension Controlled

Dash Line - Compression Controlled

P_u / ϕ = 144.4 *kips, (643 kN)*

M_{nc} @ P_u / ϕ = 109.6 *ft-kips, (149 kN-m)*

> M_u / ϕ = 34.4 *ft-kips, (47 kN-m)*

[Satisfactory]

where ϕ = 0.900 , (ACI 318-14 21.2 & AISC 341-16 B3.2)

CHECK SHEAR CAPACITY (ACI 318-14 10 & 22.5)

V_u / ϕ = 13.1 *kips, (58 kN)* **<** $V_c + V_s$ = 17.7 *kips, (79 kN)*

[Satisfactory]

where ϕ = 0.75 , (ACI 318-11 21.2)

V_c = 17.7 *kips, (79 kN)*

V_s = 0.0 *kips, (0 kN)*

Purchaser's LOGO

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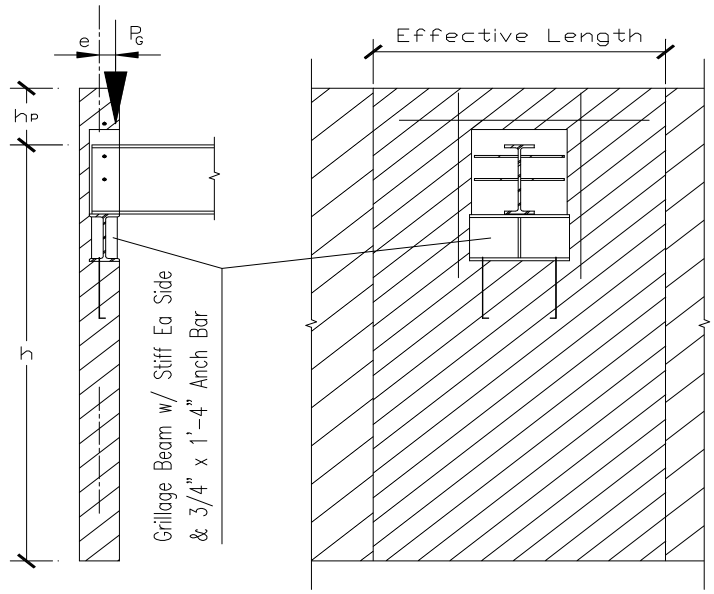
Design for Grillage Beam Masonry Wall Based on TMS 402-16

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			No	
GIRDER SERVICE LOAD	P_G	=	40	kips
ECCENTRICITY	e	=	3	in
GRILLAGE BEAM (W8 Min) LENGTH	L_{br}	=	42	in
SERVICE LATERAL LOAD	w_1	=	25	psf
SERVICE PARAPET LOAD	w_2	=	45	psf
THICKNESS OF WALL	t	=	8	in
PARAPET HEIGHT	h_p	=	2	ft
WALL HEIGHT	h	=	15	ft

WALL VERT. REINF. (A_{sv}) 2 # 7
@ 16 in o.c. (at each face)



[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

EFFECTIVE THICKNESS t_e = 7.63 in

EFFECTIVE LENGTH (TMS 402 4.3.4)

$L_e = 2\ L_{br}$ = 84.00 in

LOAD DISTRIBUTION (TMS 402 5.1.3)

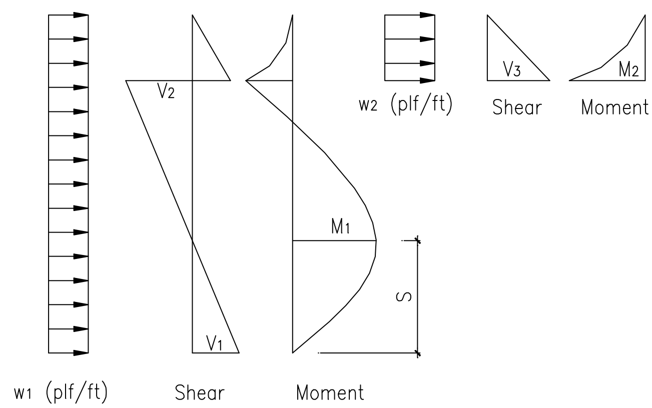
$P = P_G / (0.25\ h + L_{br})$ = 5517 lbs / ft

CHECK BEARING CAPACITY (TMS 402 8.1.5)

$f_{br} = P_G / (t_e\ L_{br})$ = 125 psi

< $0.33\ f_m'$ = 495 psi

[Satisfactory]



REINF. AREA AT EACH SIDE	A_s	=	0.45	in^2
EFFECTIVE DEPTH (TMS 402 6.1.3.5)	d	=	5.44	in
WIDTH OF SECTION	b_w	=	12.00	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi
MODULAR RATIO	n	=	21.48	

REINFORCEMENT RATIO ρ = 0.0069

ALLOWABLE STRESS FACTOR SF = 1.000

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \quad = 0.41584$$

THE ALLOWABLE STRESS DUE TO FLEXURE IS

$$F_b = (SF)(0.33 f'_m) = \quad 495 \text{ psi}$$

THE ALLOWABLE REINF. STRESS DUE TO FLEXURE IS

$$F_S = (1.33 \ \ or \ \ 1.0)(20) \ \ or \ \ 32 = \quad 32000 \text{ psi}$$

THE DISTANCE FROM BOTTOM TO M_1 IS

$$S = h + h_p - \left[\frac{(h+h_p)^2}{2h} - \frac{Pe}{hw_1}\right] \quad = 11.0 \text{ ft}$$

THE GOVERNING MOMENTS AND AXIAL FORCES ARE

$$M_1 = \frac{1}{2w_1h^2}\left[Pe + \frac{w_1}{2}(h^2 - h_p^2)\right]^2 \quad = 1525 \text{ ft-lbs/ft}$$

$$P_1 = P + (wall \quad weight) \quad = 6033 \text{ lbs / ft}$$

(cont'd)

THE GOVERNING SHEAR FORCES ARE

$$V_1 = (h + h_p) w_1 - \frac{(h+h_p)^2 w_1}{2h} + \frac{Pe}{h}$$ = 276 lbs / ft

$$V_2 = h w_1 - V_1$$ = 99 lbs / ft

$$V_3 = h_p w_2$$ = 90 lbs / ft

$$M_2 = \frac{w_2 h_p^2}{2}$$ = 90 ft-lbs/ft

$$P_2 = P + (wall \quad weight)$$ = 5691 lbs / ft

THE GOVERNING SHEAR STRESS IN MASONRY IS

$$f_v = \frac{MAX(V_1 \;,\; V_2 \;,\; V_3)}{t_e b_w}$$ = 3.02 psi

DETERMINE THE REGION FOR FLEXURE AND AXIAL LOAD (MDG-3 Tab 12.2.1, Fig 12.2-12 & 13, page 12-25).

$$\frac{M}{Pd} \le \frac{t_e}{6d}$$

1. Wall is in compression and not cracked.

$$\frac{M}{Pd} \le \left(\frac{t_e}{2d} - \frac{1}{3}\right)$$

2. Wall is cracked but steel is in compression.

$$\frac{M}{Pd} > \left(\frac{t_e}{2d} - \frac{1}{3}\right)$$

3. Wall is cracked and steel is in tension.

REGION 3 APPLICABLE FOR (M1, P1)

REGION 1 APPLICABLE FOR (M2, P2)

CHECK REGION 1 CAPACITY

$$M_m = \frac{b_w t_e^2}{6} F_b - P \frac{t_e}{6} = \begin{cases} 4164 \text{ ft-lbs / ft} > M_1 \\ 4200 \text{ ft-lbs / ft} > M_2 \end{cases}$$

4164 ft-lbs / ft > M_1 **[Not applicable]**

4200 ft-lbs / ft > M_2 **[Satisfactory]**

CHECK REGION 2 CAPACITY

$$M_m = P \frac{t_e}{2} - \frac{2P^2}{3 b_w F_b} = \begin{cases} 1578 \text{ ft-lbs / ft} > M1 \\ 1506 \text{ ft-lbs / ft} > M2 \end{cases}$$

1578 ft-lbs / ft > M1 **[Not applicable]**

1506 ft-lbs / ft > M2 **[Not applicable]**

CHECK REGION 3 CAPACITY (The moment maybe limited by either the masonry compression or steel tension, MDG-3 page 12-25).

$$M_m = MIN\left[\frac{1}{2} b_w kd\, F_b \left(d - \frac{kd}{3}\right) - P\left(d - \frac{t_e}{2}\right) \;,\; A_s F_s \left(d - \frac{kd}{3}\right) + P\left(\frac{t_e}{2} - \frac{kd}{3}\right)\right]$$

$$= \begin{cases} 1808 \text{ ft-lbs / ft} > M_1 \\ 1854 \text{ ft-lbs / ft} > M_2 \end{cases}$$

1808 ft-lbs / ft > M_1 **[Satisfactory]**

1854 ft-lbs / ft > M_2 **[Not applicable]**

CHECK ALLOWABLE SHEAR STRESS (TMS 402 8.2.6)

$$F_v = (SF) 1.125 \left(\sqrt{f'_m}\right)$$ = 43.571 psi > f_v **[Satisfactory]**

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

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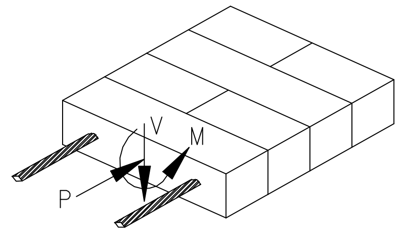
Masonry Plate/Shell Element Design (ASD) Based on 2018 IBC & TMS 402-16

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			1	*Yes*
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	*CMU*
MASONRY STRENGTH	f_m'	=	1.5	*ksi, (10 MPa)*
REBAR YIELD STRESS	f_y	=	60	*ksi, (414 MPa)*
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SERVICE AXIAL LOAD (ASD)	P	=	0.625	*kips/ft, (9.1 kN/m)*
SERVICE MOMENT LOAD (ASD)	M	=	0.11	*ft-kips / ft, (0.5 kN-m / m)*
SERVICE SHEAR LOAD (ASD)	V	=	3.6	*kips/ft, (52.5 kN/m)*
THICKNESS OF ELEMENT	t	=	8	*in, (203 mm)*
ELEMENT REINFORCEMENT	1	#	5	@ 16 in o.c. (at middle)



[THE ELEMENT DESIGN IS ADEQUATE.]

ANALYSIS

VERT. REINF. AREA AT EACH SIDE	A_s	=	0.23	in^2
EFFECTIVE DEPTH (TMS 402 6.1.3.5)	d	=	3.82	in
WIDTH OF SECTION	b_w	=	12.00	in
EFFECTIVE THICKNESS	t_e	=	7.63	in
MASONRY ELASTICITY MODULUS	E_m	=	1350	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	21.48
REINFORCEMENT RATIO	ρ	=	0.0051
ALLOWABLE STRESS FACTOR	SF	=	1.333

THE NEUTRAL AXIS DEPTH FACTOR IS

$$k=\sqrt{2\rho n+(\rho n)^2}-\rho n \quad = \quad 0.37059$$

THE ALLOWABLE STRESS DUE TO FLEXURE IS

$$F_b=(SF)\left(0.33f_m'\right)= \quad 660 \text{ psi}$$

THE ALLOWABLE REINF. STRESS DUE TO FLEXURE IS

$$F_S=(1.33 \;\; or \;\; 1.0)(20) \;\; or \;\; 32= \quad 32000 \text{ psi}$$

DETERMINE THE REGION FOR FLEXURE AND AXIAL LOAD (MDG-3 Tab 12.2.1, Fig 12.2-12 & 13, page 12-25).

$$\frac{M}{Pd}\le\frac{t_e}{6d}$$

1. Element is in compression and not cracked.

$$\frac{M}{Pd}\le\left(\frac{t_e}{2d}-\frac{1}{3}\right)$$

2. Element is cracked but steel is in compression.

$$\frac{M}{Pd}>\left(\frac{t_e}{2d}-\frac{1}{3}\right)$$

3. Element is cracked and steel is in tension.

REGION 2 APPLICABLE FOR (M, P)

CHECK REGION 1 CAPACITY

$$M_m=\frac{b_w t_e^2}{6}F_b-P\frac{t_e}{6}= \quad 6.338 \text{ ft-kips / ft} \quad > \quad M$$

[Not applicable]

CHECK REGION 2 CAPACITY

$$M_m=P\frac{t_e}{2}-\frac{2P^2}{3b_wF_b}= \quad 0.196 \text{ ft-kips / ft} \quad > \quad M$$

[Satisfactory]

CHECK REGION 3 CAPACITY (The moment maybe limited by either the masonry compression or steel tension, MDG-3 page 12-25).

$$M_m=MIN\left[\frac{1}{2}b_w kd F_b\left(d-\frac{kd}{3}\right)-P\left(d-\frac{t_e}{2}\right) \;\; , \;\; A_sF_s\left(d-\frac{kd}{3}\right)+P\left(\frac{t_e}{2}-\frac{kd}{3}\right)\right]$$

= 1.56002 ft-kips / ft > M [Not applicable]

THE ALLOWABLE SHEAR STRESS IS GIVEN BY (TMS 402 8.2.6)

$$F_v=(SF)1.125\left(\sqrt{f_m'}\right)= \quad 58.09 \text{ psi} \quad > \quad f_v=\frac{V}{t_e b_w}= \quad 39.32 \text{ psi}$$

[Satisfactory]

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

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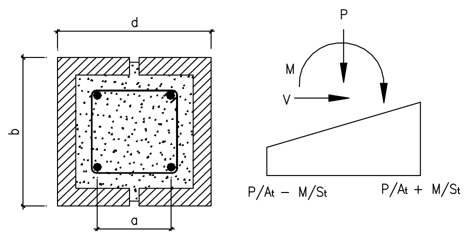
Lightly Loaded Column Design Based on TMS 402-16

INPUT DATA & DESIGN SUMMARY

SPECIAL INSPECTION (0=NO, 1=YES)			0	No, (reduced fm' by 0.5)
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.35	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SEISMIC DESIGN CATEGORY (0, 1, 2 or 5=Gravity)			2	C
SERVICE AXIAL LOAD	P	=	2	k, @ top of col.
SERVICE SHEAR LOAD	V	=	0	k
MOMENT AT MIDHEIGHT	M	=	0.4	ft-k
EFFECTIVE WIDTH	b	=	15.63	ft
EFFECTIVE DEPTH	d	=	15.63	ft
EFFECTIVE HEIGHT	h	=	12	ft



DISTANCE BETWEEN COL. REINF. a = 11.13 in
(TMS 402 6.1.4.1)

VERTICAL REINF. (EACH SIDE) 1 # 4

HORIZ. TIES 2 legs # 3 @ 8 in o.c.

(0 leg for no tie)

[THE COLUMN DESIGN IS ADEQUATE.]

ANALYSIS

TOTAL REINFORCEMENT AREA	A_s	=	0.40	in^2
EFFECTIVE COLUMN AREA	A_n	=	244	in^2
NET EFFECTIVE MOMENT OF INERTIA	I_n	=	4973	in^4
RADIUS OF GYRATION	r	=	4.51	in
MASONRY ELASTICITY MODULUS	E_m	=	1215	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi

MODULAR RATIO	n	=	23.87	
REINFORCEMENT RATIO	ρ	=	0.002	
ALLOWABLE STRESS FACTOR	SF	=	0.667	
MAX. TIES SPACING (TMS-13 7.4.1)	S_{max}	=	16	in

TRANSFORMED COLUMN AREA

$$A_t = A_n\left(1+(2n-1)\rho\right) = \quad 263 \text{ in}^2$$

CHECK VERTICAL REINFORCEMENT LIMITATION (TMS 402 5.3.2)

A_s = 0.40 in^2 > 0.20 in^2 **[Satisfactory]**

< $0.04A_n$ = 9.77 in^2 **[Satisfactory]**

ALLOWABLE STRESS DUE TO AXIAL LOAD ONLY

$$F_a = (SF)\left(0.25f'_m\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 0.213 \text{ ksi}$$

[for h/r < 99]

AXIAL STRESS AT MIDHEIGHT OF THE COLUMN

$$f_a = \frac{P+(half \quad col. \quad weight)}{A_t} = \quad 0.013 \text{ ksi}$$

< Fa, [Satisfactory]

ALLOWABLE STRESS DUE TO FLEXURE

$$F_b = (SF)\left(0.33f'_m\right) = \quad 0.297 \text{ ksi}$$

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = (1.33 \quad or \quad 1.0)(20) \quad or \quad 32 = \quad 32.0 \text{ ksi}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_T = M + (0.1)\left(\frac{Pd}{2}\right) = \quad 0.5 \text{ ft-kips}$$

TRANSFORMED MOMENT OF INERTIA

$$I_t = I_n + (2n-1)A_s\left(\frac{a}{2}\right)^2 = \quad 5552 \text{ in}^4$$

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = \frac{M_T d}{2I_t} = \quad 0.009 \text{ ksi}$$

< fa, [Satisfactory, the section is uncracked]

MAX.STRESS COMBINED AXIAL & FLEXURE

$$f_m = f_a + f_b = \quad 0.022 \text{ ksi}$$

< Fb, [Satisfactory]

MAX. REINF. STRESS COMBINED AXIAL & FLEXURE

$$f_S = 2n\left(f_a + \frac{af_b}{d}\right) = \quad 0.9 \text{ ksi}$$

< Fs, [Satisfactory]

AXIAL LOAD AT BASE OF THE COLUMN

$$P_t = P + (full \quad col. \quad weight) = \quad 4.748 \text{ k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left((SF)0.25f'_m A_n + 0.65F_S A_S\right)\left(1.0-\left(\frac{h}{140r}\right)^2\right) = \quad 60.00 \text{ k}$$

[for h/r < 99] **> Pt, [Satisfactory]**

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$$f_v = \frac{V}{bd} = \quad 0 \text{ psi} \quad < \quad F_v = MIN\left[(SF)1.125\sqrt{f'_m} + 0.5\left(\frac{A_v F_s d}{A_n S}\right) \ , \ (SF)2\sqrt{f'_m}\right]$$

(TMS 402 8.3.6) = 48.98979 psi **[Satisfactory]**

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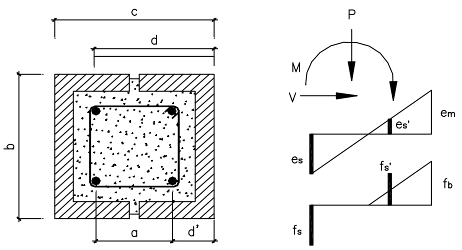
Lightly Loaded Column Design Based on TMS 402-16

INPUT DATA & DESIGN SUMMARY



SPECIAL INSPECTION (0=NO, 1=YES)			1	Yes
TYPE OF MASONRY (1=CMU, 2=BRICK)			1	CMU
MASONRY STRENGTH	f_m'	=	1.35	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)			Yes	
SEISMIC DESIGN CATEGORY (0 = WIND, 1 = B, 2 = C, 5 = GRAVITY)			2	C
SERVICE AXIAL LOAD	P	=	2	k, @ top of col.
MAX SHEAR LOAD	V	=	0	k
MOMENT AT MIDHEIGHT	M	=	1.5	ft-k, @ mid of col
EFFECTIVE WIDTH	b	=	7.63	ft
EFFECTIVE DEPTH	c	=	7.63	ft
EFFECTIVE HEIGHT	h	=	12	ft

DISTANCE BETWEEN COL. REINF. a = 3.13 in (TMS 402 6.1.4.1)

VERTICAL REINF. (EACH SIDE) 1 # 4

HORIZ. TIES 0 legs # 3 @ 16 in o.c. (0 leg for no tie)

[THE COLUMN DESIGN IS ADEQUATE.]

ANALYSIS

REINFORCEMENT AREA AT ONE SIDE	A_s	=	0.20	in^2
EFFECTIVE COLUMN AREA	A_n	=	58	in^2
NET EFFECTIVE MOMENT OF INERTIA	I_n	=	282	in^4
RADIUS OF GYRATION	r	=	2.20	in
MASONRY ELASTICITY MODULUS	E_m	=	1215	ksi
STEEL ELASTICITY MODULUS	E_s	=	29000	ksi
MODULAR RATIO	n	=	23.87	
TRANSFORMED AREA	$A_t = A_n(1-\rho+n\rho+n\rho')$	=	71	in^2

REINFORCEMENT RATIO	$\rho = \rho'$	=	0.005	
DISTANCE	d' = 2.25 , d =		5.38	in
ALLOWABLE STRESS FACTOR	SF	=	1.333	
MAX. TIES SPACING (TMS-13 7.4.1)	S_{max}	=	16	in

NEUTRAL AXIS DEPTH FACTOR

$k = \{[n\rho+(2n-1)\rho']^2+2[n\rho+(2n-1)\rho(d'/d)]\}^{0.5} - [n\rho+(2n-1)\rho']$ = 0.392

LEVER-ARM FACTOR $j = 1-k/3$ = 0.869

CHECK VERTICAL REINFORCEMENT LIMITATION (TMS 402 5.3.2)

$A_{s,total}$ = 0.40 in^2 > 0.20 in^2 **[Satisfactory]**

< $0.04A_e = 0.04bd$ = 1.64 in^2 **[Satisfactory]**

AXIAL LOAD AT MIDDLE OF THE COLUMN

$$P_{Mid} = P + (half \quad col. \quad weight) = \quad 2.327 \text{ k}$$

TOTAL MOMENT ACTING AT MIDHEIGHT

$$M_{Mid} = M + (0.1)\left(\frac{Pd}{2}\right) = \quad 1.6 \text{ ft-kips}$$

CHECK IF THERE IS TENSILE STRESS IN CROSS SECTION

P_{Mid} / A = 40 psi < $M_{Mid} / (bc^2/6)$ = 253 psi

(tensile exist)

AXIAL LOAD AT BASE OF THE COLUMN

$$P_t = P + (full \quad col. \quad weight) = \quad 2.655 \text{ k}$$

ALLOWABLE AXIAL LOAD FOR AXIAL COMPRESSION ONLY

$$P_a = \left(0.25 f'_m A_n + 0.65 F_S A_{st}\right)\left(1.0 - \left(\frac{h}{140r}\right)^2\right) = \quad 20.24 \text{ k}$$

> Pt, [Satisfactory]

[for h/r < 99]

ALLOWABLE REINF. STRESS DUE TO FLEXURE

$$F_S = (1.33 \quad or \quad 1.0)(20) \quad or \quad 32 = \quad 32.0 \text{ ksi}$$

ALLOWABLE STRESS DUE TO FLEXURE

$$F'_b = \left(SF - \frac{P_{Mid}}{P_a}\right)\left(0.33 f'_m\right) = \quad 0.543 \text{ ksi}$$

THE CORRESPONDING STRAIN IN THE TENSILE BARS IS

$$e_S = \frac{F_S}{E_S} = \quad 0.0011$$

THE STRAIN IN THE EXTREME COMPRESSION FIBER IS

$$e_m = MIN\left[\left(\frac{kd}{d-kd}\right)e_s \;,\; \frac{F'_b}{E_m}\right] = \quad 0.0004467$$

(masonry governs)

STRESS IN THE EXTREME FIBER DUE TO M_T

$$f_b = e_m E_m = \quad 0.543 \text{ ksi}$$

< Fb', [Satisfactory]

MOMENT DUE TO THE MASONRY

$$M_m = \frac{1}{2} bkd f_b \left(d - \frac{kd}{3}\right) = \quad 1.70 \text{ ft-kips}$$

STRAIN IN THE COMPRESSION BARS

$$e'_S = \left(\frac{kd - d'}{kd}\right) e_m = \quad -0.0000301$$

STRESS IN THE COMPRESSION BARS

$$f'_S = 2 E_S e'_S = \quad -1.748 \text{ ksi}$$

(cont'd)

MOMENT DUE TO THE COMPRESSION BARS

$$M'_S = f'_S A_S (d - d') = \quad -0.09 \quad \text{ft-kips}$$

ALLOWABLE BENDING MOMENT

$$M = M'_S + M_m = \quad 1.61 \quad \text{ft-kips}$$

> Mt, [Satisfactory]

SHEAR DESIGN DETERMINED FROM THE FOLLOWING EXPRESSION

$$f_v = \frac{V}{jbd} \quad = \quad 0 \quad \text{psi} \quad < \quad F_v = MIN\left[(SF)1.125\sqrt{f'_m} + 0.5\left(\frac{A_v F_s d}{A_n S}\right) \;,\; (SF)2\sqrt{f'_m} \right]$$

(TMS 402 8.3.6)

$= \quad 55.11352 \quad \text{psi}$ **[Satisfactory]**

Technical References:

1. "Masonry Designers' Guide, Third Edition" (MDG-3), The Masonry Society, 2001.

Daniel T. Li
Engineering International

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Wood Joist Design Based on NDS 05 / NDS 01, ICC PFC-4354 & PFC-5803

INPUT DATA & DESIGN SUMMARY

JOIST SPAN	L =	**35**	ft
DEAD LOAD	DL =	**22**	psf, (w/o self Wt)
LIVE LOAD / SNOW	LL =	**16**	psf
JOIST SPACING	S =	**24**	in o.c.
DURATION FACTOR	C_D =	**1.25**	(NDS Tab. 2.3.2)
REPETITIVE FACTOI	C_r =	**1.15**	(NDS 4.3.9. For DSA, 1.0)
DEFLECTION LIMIT OF LIVE LOAD	Δ_{LL} = L /	**360**	(L / 360 , 1.2 in)
DEFLECTION LIMIT OF LONG-TERM LOAD	$\Delta_{1.5(DL+0.33LL)}$ = L /	**480**	(L / 480 , 0.9 in)
DEFLECTION LIMIT OF TOTAL LOAD	$\Delta_{(DL+LL)}$ = L /	**240**	(L / 240 , 1.8 in)

AVAILABLE MINIMUM Douglas Fir-Larch SIZES

AVAILABLE MINIMUM TJI SIZES

30" TJI/L65 **28" TJI/L90** **26" TJI/H90**

AVAILABLE MINIMUM SSI SIZES

28" SSI 42MX **26" SSI 43LX**

ANALYSIS

JOIST PROPERTIES & ALLOWABLE MOMENT & SHEAR

2x No. 2, Douglas Fir-Larch (ASD Supplements, Tab. 5.4a)

Deep (in)	Wt (lbs/ft)	M (ft-lbs) (C_F included)	V (lbs)	EI x 10^6 (in^2-lbs)
4	1.00	344	630	9
6	2.00	738	990	33
8	2.00	1183	1310	76
10	3.00	1767	1670	158
12	4.00	2375	2030	285

2x No. 1, Douglas Fir-Larch (from WoodBeam.xls)

Deep (in)	Wt (lbs/ft)	M (ft-lbs) (C_F included)	V (lbs)	EI x 10^6 (in^2-lbs)
4	1.00	383	630	9
6	2.00	819	990	35
8	2.00	1314	1305	81
10	3.00	1961	1665	168
12	4.00	2637	2025	303

2x Structural, Douglas Fir-Larch (ASD Supplements, Tab. 5.4a)

Deep (in)	Wt (lbs/ft)	M (ft-lbs) (C_F included)	V (lbs)	EI x 10^6 (in^2-lbs)
4	1.00	574	630	10
6	2.00	1225	990	40
8	2.00	1975	1310	91
10	3.00	2942	1670	188
12	4.00	3958	2030	338

Where:
1. ASD Supplements, Tab. 5.4a is from American Wood Council, 2001.
2. Assume that the joist top is fully lateral supported by diaphragm. (C_L = 1.0)
3. WoodBeam.xls is at www.engineering-international.com

TJI/L65 (from Trusjoist # 1062, page 5)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)
11 7/8	3.30	6750	1925	450
14	3.60	8030	2125	666
16	3.90	9210	2330	913
18	4.20	10380	2535	1205
20	4.40	11540	2740	1545
22	4.70	12690	2935	1934
24	5.00	13830	3060	2374
26	5.30	14960	2900	2868
28	5.50	16085	2900	3417
30	5.80	17205	2900	4025

SSI 32MX (from ICC PFC-5803, page 5 & 6)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)	C x 10^6 (in^2-lbs)
11 7/8	3.10	5391	2115	460	9.39
14	3.30	6570	2330	667	10.99
16	3.60	7684	2530	900	12.50
18	3.90	8800	2735	1170	14.02
20	4.10	9918	2935	1478	15.55
22	4.40	11038	3135	1824	17.08
24	4.70	12159	3335	2211	18.62
26	5.00	13279	3540	2638	20.15
28	5.20	14401	3740	3106	21.68
30	5.50	15524	3940	3616	23.21

TJI/L90 (from Trusjoist # 1062, page 5)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)
11 7/8	4.20	9605	1925	621
14	4.50	11430	2125	913
16	4.70	13115	2330	1246
18	5.00	14785	2535	1635
20	5.30	16435	2740	2085
22	5.60	18075	2935	2597
24	5.80	19700	3060	3172
26	6.10	21315	2900	3814
28	6.40	22915	2900	4525
30	6.60	24510	2900	5306

SSI 42MX (from ICC PFC-5803, page 5 & 6)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)	C x 10^6 (in^2-lbs)
11 7/8	3.80	7592	2060	637	9.54
14	4.10	9274	2350	924	11.15
16	4.30	10863	2620	1246	12.68
18	4.60	12456	2895	1617	14.22
20	4.90	14051	3165	2040	15.77
22	5.10	15649	3440	2514	17.32
24	5.40	17248	3710	3042	18.87
26	5.70	18849	3985	3622	20.42
28	6.00	20450	4255	4257	21.97
30	6.20	22052	4530	4948	23.53

TJI/H90 (from Trusjoist # 1062, page 5)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)
11 7/8	4.60	10960	1925	687
14	4.90	13090	2125	1015
16	5.20	15065	2330	1389
18	5.40	17010	2535	1827
20	5.70	18945	2740	2331
22	6.00	20855	2935	2904
24	6.30	22755	3060	3549
26	6.50	24645	2900	4266
28	6.80	26520	2900	5059
30	7.10	28380	2900	5930

SSI 43L (from ICC PFC-5803, page 5 & 6)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)	C x 10^6 (in^2-lbs)
11 7/8	4.60	9789	2080	707	6.81
14	4.90	12081	2260	1031	7.91
16	5.20	14251	2425	1394	8.97
18	5.40	16269	2590	1944	10.05
20	5.70	18419	2755	2454	11.13
22	5.90	20573	2920	3026	12.21
24	6.20	22730	3090	3661	13.30
26	6.40	24889	3255	4358	14.39
28	6.70	27050	3420	5119	15.47
30	7.00	29212	3585	5944	16.56

(cont'd)

DESIGN EQUATIONS

$$M = \frac{wL^2}{8C_D C_r} \qquad V = \frac{wL}{2C_D C_r} \qquad \Delta_{DFL} = \frac{5wL^4}{384EI}$$

$$\Delta_{TJI} = \frac{22.5wL^4}{EI} + \frac{2.26wL^2}{d \times 10^5}$$ (from Trusjoist # 1062, page 21)

$$\Delta_{SSI} = \frac{5wL^4}{384EI} + \frac{wL^2}{C}$$ (from ICC PFC-5803, page 2)

CHECK JOIST CAPACITIES & DEFLECTIONS

2x No. 2, Douglas Fir-Larch

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
4	8202	937	120.05	312.66	288.87	N.G.
6	8309	950	32.74	86.80	79.81	N.G.
8	8309	950	14.22	37.69	34.65	N.G.
10	8415	962	6.84	18.45	16.88	N.G.
12	8522	974	3.79	10.41	9.48	N.G.

2x No. 1, Douglas Fir-Larch

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
4	8202	937	120.05	312.66	288.87	N.G.
6	8309	950	30.87	81.84	75.25	N.G.
8	8309	950	13.34	35.36	32.51	N.G.
10	8415	962	6.43	17.35	15.88	N.G.
12	8522	974	3.57	9.79	8.91	N.G.

2x Structural, Douglas Fir-Larch

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
4	8202	937	108.05	281.39	259.98	N.G.
6	8309	950	27.01	71.61	65.84	N.G.
8	8309	950	11.87	31.48	28.94	N.G.
10	8415	962	5.75	15.51	14.19	N.G.
12	8522	974	3.20	8.77	7.99	N.G.

TJI/L65

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
11 7/8	8447	965	2.48	6.71	6.13	N.G.
14	8479	969	1.69	4.60	4.19	N.G.
16	8511	973	1.24	3.39	3.09	N.G.
18	8543	976	0.95	2.61	2.37	N.G.
20	8564	979	0.74	2.06	1.87	N.G.
22	8596	982	0.60	1.66	1.51	N.G.
24	8628	986	0.49	1.37	1.25	N.G.
26	8660	990	0.41	1.15	1.04	N.G.
28	8682	992	0.35	0.98	0.89	N.G.
30	8713	996	0.30	0.84	0.76	o.k.

SSI 32MX

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
11 7/8	8426	963	2.40	6.48	5.93	N.G.
14	8447	965	1.66	4.51	4.12	N.G.
16	8479	969	1.24	3.38	3.08	N.G.
18	8511	973	0.96	2.62	2.39	N.G.
20	8532	975	0.76	2.09	1.91	N.G.
22	8564	979	0.62	1.71	1.56	N.G.
24	8596	982	0.51	1.43	1.30	N.G.
26	8628	986	0.43	1.21	1.10	N.G.
28	8650	989	0.37	1.04	0.94	N.G.
30	8682	992	0.32	0.90	0.81	N.G.

TJI/L90

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
11 7/8	8543	976	1.81	5.00	4.55	N.G.
14	8575	980	1.25	3.45	3.14	N.G.
16	8596	982	0.92	2.56	2.33	N.G.
18	8628	986	0.71	1.98	1.80	N.G.
20	8660	990	0.56	1.58	1.43	N.G.
22	8692	993	0.46	1.29	1.16	N.G.
24	8713	996	0.38	1.07	0.97	N.G.
26	8745	999	0.32	0.90	0.81	N.G.
28	8777	1003	0.27	0.77	0.70	o.k.
30	8799	1006	0.23	0.67	0.60	o.k.

SSI 42MX

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
11 7/8	8500	971	1.75	4.77	4.35	N.G.
14	8532	975	1.21	3.33	3.03	N.G.
16	8554	978	0.90	2.49	2.27	N.G.
18	8586	981	0.70	1.94	1.77	N.G.
20	8618	985	0.56	1.56	1.41	N.G.
22	8639	987	0.46	1.28	1.16	N.G.
24	8671	991	0.38	1.07	0.97	N.G.
26	8703	995	0.32	0.91	0.82	N.G.
28	8735	998	0.28	0.78	0.71	o.k.
30	8756	1001	0.24	0.68	0.61	o.k.

TJI/H90

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
11 7/8	8586	981	1.65	4.57	4.15	N.G.
14	8618	985	1.13	3.14	2.85	N.G.
16	8650	989	0.83	2.33	2.11	N.G.
18	8671	991	0.64	1.80	1.63	N.G.
20	8703	995	0.51	1.43	1.30	N.G.
22	8735	998	0.41	1.17	1.06	N.G.
24	8767	1002	0.34	0.97	0.88	N.G.
26	8788	1004	0.29	0.82	0.74	o.k.
28	8820	1008	0.25	0.71	0.63	o.k.
30	8852	1012	0.21	0.61	0.55	o.k.

SSI 43L

Deep (in)	M (ft-lbs)	V (lbs)	Δ_{LL} (in)	Δ_{LT} (in)	Δ_{D+L} (in)	CHECK
11 7/8	8586	981	1.60	4.43	4.02	N.G.
14	8618	985	1.11	3.09	2.80	N.G.
16	8650	989	0.83	2.32	2.10	N.G.
18	8671	991	0.60	1.69	1.53	N.G.
20	8703	995	0.48	1.36	1.23	N.G.
22	8724	997	0.40	1.12	1.01	N.G.
24	8756	1001	0.33	0.94	0.85	N.G.
26	8777	1003	0.28	0.80	0.72	o.k.
28	8809	1007	0.24	0.69	0.62	o.k.
30	8841	1010	0.21	0.61	0.55	o.k.

Daniel T. Li
Engineering International

PROJECT :
CLIENT :
JOB NO. :
DATE :

PAGE :
DESIGN BY :
REVIEW BY :

Double Joist Design for Mechanical Equipment Based on NDS 05 / NDS 01, ICC PFC-4354 & PFC-5803

INPUT DATA & DESIGN SUMMARY

JOIST SPAN	L =	30	ft
DEAD LOAD	DL =	26	psf, (w/o self Wt)
JOIST SPACING	S =	24	in o.c.
DURATION FACTOR	C_D =	1.33	(NDS Tab. 2.3.2)

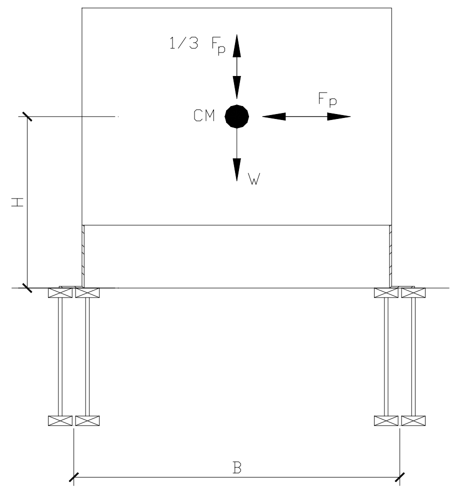
AVAILABLE MINIMUM Douglas Fir-Larch SIZES

AVAILABLE MINIMUM TJI SIZES
24" TJI/L90 **22" TJI/H90**

AVAILABLE MINIMUM SSI SIZES
28" SSI 42MX **22" SSI 43L**

DEFLECTION LIMITATION $\Delta_{(DL+E)}$ = L / 240 (L / 240 , 1.5 in)

EQUIPMENT WEIGHT	W =	3	kips
HEIGHT OF MASS CENTER	H =	3	ft, 2/3 total height
EQUIPMENT LENGTH	D =	4	ft
EQUIPMENT WIDTH	B =	6	ft, double joist spacing



SEISMIC LOADS, (CBC 2007 / IBC 2006)

$F_H = F_p =$ (K_H) MAX{ $0.3S_{DS}I_pW$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p$ W , $1.6S_{DS}I_pW$] }
= 1.3 MAX{ 0.43W , MIN[1.15W , 2.30W]}
= 1.50 W , (SD)
= 1.07 W , (ASD) = 3.21 kips
(ASCE 7-05, Sec. 13.3.1)

$F_V = K_V$ W = 0.18 W , (ASD) = 0.53 kips, up & down

where

S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
I_p =	1.5	(ASCE Sec. 13.1.3)
a_p =	1	(ASCE Tab. 13.6-1)
R_p =	1.5	(ASCE Tab. 13.6-1)
z =	h	ft
h =	36	ft
K_H =	1.3	(ASCE Sec. 13.4.2a)

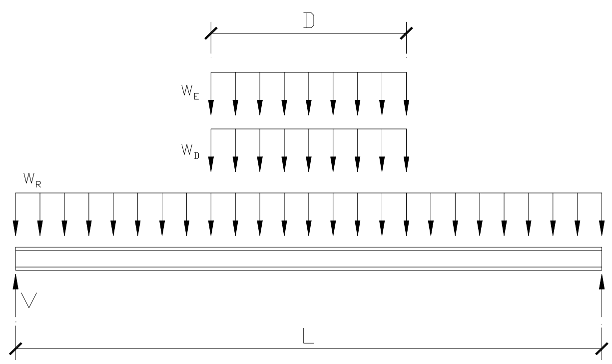
$K_V = K_H\ 0.2\ S_{DS} / 1.4$ = 0.18 (vertical seismic factor)

$w_E = (0.5\ F_H\ H / B + 0.25\ F_V) / L$ = 234 plf / joist, at middle of span

GRAVITY LOADS

w_R = 0.5 (B + S) DL = 104 plf / joist, full span

w_D = 0.25 W / D = 188 plf / joist, at middle of span



ANALYSIS

DESIGN EQUATIONS

$$M = \frac{w_R L^2}{8C_D} + \left(w_D + w_E\right)\left(\frac{LD}{4C_D} + \frac{D^2}{8C_D}\right)$$

$$V = \frac{w_R L + \left(w_D + w_E\right) D}{2C_D}$$

$$\Delta_{(DL+E)} = \frac{5 w_R L^4}{384EI} + \frac{\left(w_D + w_E\right) L^3 D}{48EI}$$

JOIST PROPERTIES & ALLOWABLE MOMENT & SHEAR

2x No. 2, Douglas Fir-Larch (ASD Supplements, Tab. 5.4a)

Deep (in)	Wt (lbs/ft)	M (ft-lbs) (C_F included)	V (lbs)	EI x 10^6 (in^2-lbs)
4	1.00	344	630	9
6	2.00	738	990	33
8	2.00	1183	1310	76
10	3.00	1767	1670	158
12	4.00	2375	2030	285

2x No. 1, Douglas Fir-Larch (from WoodBeam.xls)

Deep (in)	Wt (lbs/ft)	M (ft-lbs) (C_F included)	V (lbs)	EI x 10^6 (in^2-lbs)
4	1.00	383	630	9
6	2.00	819	990	35
8	2.00	1314	1305	81
10	3.00	1961	1665	168
12	4.00	2637	2025	303

2x Structural, Douglas Fir-Larch (ASD Supplements, Tab. 5.4a)

Deep (in)	Wt (lbs/ft)	M (ft-lbs) (C_F included)	V (lbs)	EI x 10^6 (in^2-lbs)
4	1.00	574	630	10
6	2.00	1225	990	40
8	2.00	1975	1310	91
10	3.00	2942	1670	188
12	4.00	3958	2030	338

Where:
1. ASD Supplements, Tab. 5.4a is from American Wood Council, 2001.
2. Assume that the joist top is fully lateral supported by diaphragm. (C_L = 1.0)
3. WoodBeam.xls is at www.engineering-international.com

TJI/L65 (from Trusjoist # 1062, page 5)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)
11 7/8	3.30	6750	1925	450
14	3.60	8030	2125	666
16	3.90	9210	2330	913
18	4.20	10380	2535	1205
20	4.40	11540	2740	1545
22	4.70	12690	2935	1934
24	5.00	13830	3060	2374
26	5.30	14960	2900	2868
28	5.50	16085	2900	3417
30	5.80	17205	2900	4025

SSI 32MX (from ICC PFC-5803, page 5 & 6)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)	C x 10^6 (in^2-lbs)
11 7/8	3.10	5391	2115	460	9.39
14	3.30	6570	2330	667	10.99
16	3.60	7684	2530	900	12.50
18	3.90	8800	2735	1170	14.02
20	4.10	9918	2935	1478	15.55
22	4.40	11038	3135	1824	17.08
24	4.70	12159	3335	2211	18.62
26	5.00	13279	3540	2638	20.15
28	5.20	14401	3740	3106	21.68
30	5.50	15524	3940	3616	23.21

(cont'd)

TJI/L90 (from Trusjoist # 1062, page 5)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)
11 7/8	4.20	9605	1925	621
14	4.50	11430	2125	913
16	4.70	13115	2330	1246
18	5.00	14785	2535	1635
20	5.30	16435	2740	2085
22	5.60	18075	2935	2597
24	5.80	19700	3060	3172
26	6.10	21315	2900	3814
28	6.40	22915	2900	4525
30	6.60	24510	2900	5306

SSI 42MX (from ICC PFC-5803, page 5 & 6)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)	C x 10^6 (in^2-lbs)
11 7/8	3.80	7592	2060	637	9.54
14	4.10	9274	2350	924	11.15
16	4.30	10863	2620	1246	12.68
18	4.60	12456	2895	1617	14.22
20	4.90	14051	3165	2040	15.77
22	5.10	15649	3440	2514	17.32
24	5.40	17248	3710	3042	18.87
26	5.70	18849	3985	3622	20.42
28	6.00	20450	4255	4257	21.97
30	6.20	22052	4530	4948	23.53

TJI/H90 (from Trusjoist # 1062, page 5)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)
11 7/8	4.60	10960	1925	687
14	4.90	13090	2125	1015
16	5.20	15065	2330	1389
18	5.40	17010	2535	1827
20	5.70	18945	2740	2331
22	6.00	20855	2935	2904
24	6.30	22755	3060	3549
26	6.50	24645	2900	4266
28	6.80	26520	2900	5059
30	7.10	28380	2900	5930

SSI 43L (from ICC PFC-5803, page 5 & 6)

Deep (in)	Wt (lbs/ft)	M (ft-lbs)	V (lbs)	EI x 10^6 (in^2-lbs)	C x 10^6 (in^2-lbs)
11 7/8	4.60	9789	2080	707	6.81
14	4.90	12081	2260	1031	7.91
16	5.20	14251	2425	1394	8.97
18	5.40	16269	2590	1944	10.05
20	5.70	18419	2755	2454	11.13
22	5.90	20573	2920	3026	12.21
24	6.20	22730	3090	3661	13.30
26	6.40	24889	3255	4358	14.39
28	6.70	27050	3420	5119	15.47
30	7.00	29212	3585	5944	16.56

CHECK JOIST CAPACITIES & DEFLECTIONS

2x No. 2, Douglas Fir-Larch

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
4	19023	1818	392.69	N.G.
6	19108	1829	107.10	N.G.
8	19108	1829	46.50	N.G.
10	19192	1841	22.37	N.G.
12	19277	1852	12.40	N.G.

2x No. 1, Douglas Fir-Larch

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
4	19023	1807	392.69	N.G.
6	19108	1807	100.98	N.G.
8	19108	1807	43.63	N.G.
10	19192	1807	21.04	N.G.
12	19277	1807	11.66	N.G.

2x Structural, Douglas Fir-Larch

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
4	19023	1818	353.42	N.G.
6	19108	1829	88.35	N.G.
8	19108	1829	38.84	N.G.
10	19192	1841	18.80	N.G.
12	19277	1852	10.46	N.G.

TJI/L65

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
11 7/8	19217	1844	7.85	N.G.
14	19243	1847	5.31	N.G.
16	19268	1851	3.87	N.G.
18	19294	1854	2.93	N.G.
20	19311	1856	2.29	N.G.
22	19336	1860	1.83	N.G.
24	19361	1863	1.49	N.G.
26	19387	1867	1.23	N.G.
28	19404	1869	1.03	N.G.
30	19429	1872	0.88	N.G.

SSI 32MX

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
11 7/8	19201	1842	7.68	N.G.
14	19217	1844	5.30	N.G.
16	19243	1847	3.93	N.G.
18	19268	1851	3.02	N.G.
20	19285	1853	2.39	N.G.
22	19311	1856	1.94	N.G.
24	19336	1860	1.60	N.G.
26	19361	1863	1.34	N.G.
28	19378	1865	1.14	N.G.
30	19404	1869	0.98	N.G.

TJI/L90

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
11 7/8	19294	1854	5.69	N.G.
14	19319	1858	3.87	N.G.
16	19336	1860	2.84	N.G.
18	19361	1863	2.16	N.G.
20	19387	1867	1.70	N.G.
22	19412	1870	1.36	N.G.
24	19429	1872	1.11	o.k.
26	19454	1876	0.93	o.k.
28	19480	1879	0.78	o.k.
30	19497	1881	0.67	o.k.

SSI 42MX

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
11 7/8	19260	1850	5.55	N.G.
14	19285	1853	3.82	N.G.
16	19302	1855	2.84	N.G.
18	19327	1859	2.19	N.G.
20	19353	1862	1.73	N.G.
22	19370	1864	1.41	N.G.
24	19395	1868	1.16	N.G.
26	19420	1871	0.98	N.G.
28	19446	1874	0.83	o.k.
30	19463	1877	0.71	o.k.

TJI/H90

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
11 7/8	19327	1859	5.14	N.G.
14	19353	1862	3.48	N.G.
16	19378	1865	2.54	N.G.
18	19395	1868	1.93	N.G.
20	19420	1871	1.52	N.G.
22	19446	1874	1.22	o.k.
24	19471	1878	1.00	o.k.
26	19488	1880	0.83	o.k.
28	19514	1883	0.70	o.k.
30	19539	1887	0.60	o.k.

SSI 43L

Deep (in)	M (ft-lbs)	V (lbs)	$\Delta_{(DL+E)}$ (in)	CHECK
11 7/8	19327	1859	5.00	N.G.
14	19353	1862	3.43	N.G.
16	19378	1865	2.54	N.G.
18	19395	1868	1.82	N.G.
20	19420	1871	1.44	N.G.
22	19437	1873	1.17	o.k.
24	19463	1877	0.97	o.k.
26	19480	1879	0.81	o.k.
28	19505	1882	0.69	o.k.
30	19530	1886	0.59	o.k.

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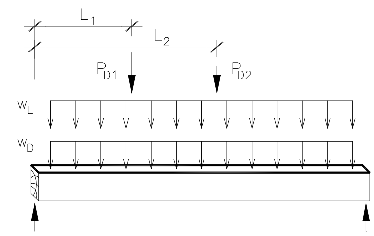
Wood Beam Design Based on NDS 2012

INPUT DATA & DESIGN SUMMARY

MEMBER SIZE	6 x 14		No. 1, Douglas Fir-Larch
MEMBER SPAN	$L =$	10	ft
UNIFORMLY DISTRIBUTED DEAD LOAD	$w_D =$	112	lbs / ft
UNIFORMLY DISTRIBUTED LIVE LOAD	$w_L =$	0	lbs / ft
CONCENTRATED DEAD LOADS	$P_{D1} =$	1500	lbs
(0 for no concentrated load)	$L_1 =$	4	ft
	$P_{D2} =$	3936	lbs
	$L_2 =$	5	ft
DEFLECTION LIMIT OF LIVE LOAD	$\Delta_L = L /$	360	
DEFLECTION LIMIT OF LONG-TERM	$\Delta_{Kcr\ D + L} = L /$	180	



Camber => 0.18 inch

THE BEAM DESIGN IS ADEQUATE.

Does member have continuous lateral support by top diaphragm ?
(1= yes, 0= no) 0 No

Code	Duration Factor, C_D	Condition
1	0.90	Dead Load
2	1.00	Occupancy Live Load
3	1.15	Snow Load
4	1.25	Construction Load
5	1.60	Wind/Earthquake Load
6	2.00	Impact Load
Choice	=> 4	Construction Load

Code	Designation
1	Select Structural, Douglas Fir-Larch
2	No. 1, Douglas Fir-Larch
3	No. 2, Douglas Fir-Larch
4	Select Structural, Southern Pine
5	No. 1, Southern Pine
6	No. 2, Southern Pine
Choice	=> 2

ANALYSIS

DETERMINE REACTIONS, MOMENT, SHEAR

$w_{Self\ Wt} =$ 16 lbs / ft $R_{Left} =$ 3.51 kips $R_{Right} =$ 3.21 kips

$V_{Max} =$ 3.36 kips, at 13.5 inch from left end $M_{Max} =$ 14.44 ft-kips, at 5.00 ft from left end

DETERMINE SECTION PROPERTIES& ALLOWABLE STRESSES

$b =$	5.50	in	$E'_{min} =$	580	ksi	$E = E_x =$	1600	ksi	$F_b^* =$	1665.5596	psi
$d =$	13.50	in	$F_{bE} =$	6605	psi	$F_b =$	1,350	psi	$F = F_{bE} / F_b^* =$	3.97	
$A =$	74.3	in^2	$I =$	1,128	in^4	$F_v =$	170	psi	$F_b' =$	1,639	psi
$S_x =$	167.1	in^3	$R_B =$	10.265	< 50	$E' =$	1,600	ksi	$F_v' =$	213	psi
$l_E =$	19.7	(ft, Tab 3.3.3 footnote 1)									

C_D	C_M	C_t	C_i	C_L	C_F	C_V	C_c	C_r
1.25	1.00	1.00	1.00	0.98	0.99	1.00	1.00	1.00

CHECK BENDING AND SHEAR CAPACITIES

$f_b = M_{Max} / S_x =$ 1037 psi < $F_b =$ 1639 psi **[Satisfactory]**

$f_v' = 1.5\ V_{Max} / A =$ 68 psi < F_v' **[Satisfactory]**

CHECK DEFLECTIONS

$\Delta_{(L,\ Max)} =$ 0.00 in, at 5.000 ft from left end, < $\Delta_L = L / 360$ **[Satisfactory]**

$\Delta_{(Kcr\ D + L\ ,\ Max)} =$ 0.18 in, at 4.900 ft from left end < $\Delta_{Kcr\ D + L} = L / 180$ **[Satisfactory]**

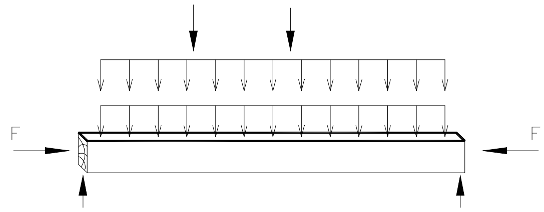
Where $K_{cr} =$ 1.50 , (NDS 3.5.2)

DETERMINE CAMBER AT 1.5 (DEAD + SELF WEIGHT)

$\Delta_{(1.5D,\ Max)} =$ 0.18 in, at 4.900 ft from left end

CHECK THE BEAM CAPACITY WITH AXIAL LOAD

AXIAL LOAD F = 8.1 kips



THE ALLOWABLE COMPRESSIVE STRESS IS

$F_c' = F_c\, C_D\, C_P\, C_F$ = 804 psi

Where F_c = 925 psi

C_D = 1.60

C_F = 0.99 (Lumber only)

$C_P = (1+F) / 2c - [((1+F) / 2c)^2 - F / c]^{0.5}$ = 0.551

$F_c^* = F_c\, C_D\, C_F$ = 1461 psi

$L_e = K_e\, L = 1.0\, L$ = 120 in

b = 5.5 in

SF = slenderness ratio = 21.8 **<** 50 [Satisfies NDS 2012 Sec. 3.7.1.4]

$F_{cE} = 0.822\, E'_{min} / SF^2$ = 1002 psi

E'_{min} = 580 ksi

$F = F_{cE} / F_c^*$ = 0.686

c = 0.8

THE ACTUAL COMPRESSIVE STRESS IS

$f_c = F / A$ = 109 psi **<** F_c' **[Satisfactory]**

THE ALLOWABLE FLEXURAL STRESS IS

F_b' = 2097 psi, [for C_D = 1.6]

THE ACTUAL FLEXURAL STRESS IS

$f_b = (M + Fe) / S$ = 1304 psi **<** F_b' **[Satisfactory]**

CHECK COMBINED STRESS [NDS 2012 Sec. 3.9.2]

$(f_c / F_c')^2 + f_b / [F_b' (1 - f_c / F_{cE})]$ = 0.716 **<** 1 **[Satisfactory]**

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Wood Post, Wall Stud, or King Stud Design Based on NDS 2005

INPUT DATA

HEIGHT H = 14.33 ft
Effective Length (NDS 3.7) Le x-x = 14.33 ft, (strong axis bending)
Le y-y = 14.33 ft, (weak axis bending)

AXIAL LOAD P_{DL} = 800 lbs
P_{LL} = 580 lbs
Total P = 1,380 lbs

LATERAL LOAD w = 20 plf
F = 406 lbs, at 10 ft, from bottom

Max Section M = 1660 ft-lbs, at 10.00 ft from bottom
Max Section V = 427 s, at top end

SPECIES (1 = DFL, 2 = SP, 3 = LSL, 4 = PSL) 1 DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6) 3 No. 1

SECTION 1 pcs, b = 4 in
h = 6 in

WET / DRY ? (1 = DRY, 2 = WET) 1 DRY

DESIGN SUMMARY

USE: 1 - 4" x 6" DOUGLAS FIR-LARCH No. 1

1. CHECK VERTICAL LOADS : $f_c < F_c'$?
72 psi < 208 psi **[Satisfactory]**

2. CHECK BENDING LOADS : $f_b < F_b'$?
1129 psi < 2023 psi **[Satisfactory]**

3. CHECK INTERACTION : $\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1-f_c/F_{cEx}}\right)\frac{f_{bx}}{F_{bx}'} \le 1$?
0.964 < 1 **[Satisfactory]**

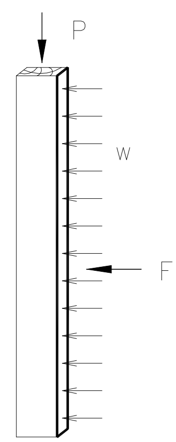
4. CHECK SHEAR LOADS : $f_v < F_v'$?
33 psi < 288 psi **[Satisfactory]**

5. MAXIMUM HORIZONTAL DEFLECTION
Δ = 0.65 in, at 7.73 ft from bottom
(**H / 265**)

ANALYSIS

COLUMN BASIC DESIGN STRESSES:

COMPRESSIVE STRESS	F_c =	1500	psi
MODULUS OF ELASTICITY	E =	1700	ksi
BENDING STRESS (X-Axis)	F_{bx} =	1000	psi
SHEAR STRESS (X-Axis)	F_v =	180	psi



COLUMN PROPERTIES:

STANDARD DRESSED SIZE		dy =	5.50	in
		dx =	3.50	in
AREA		A =	19.25	in^2
SECTION PROPERTIES	Abt. x-x	Sx =	17.65	in^3
		Ix =	48.53	in^4
LENGTH-DEPTH RATIO		Le x-x / dy =	31.3	
		Le y-y / dx =	49.1	

ADJUSTMENT FACTORS:

		F_{bx}'	F_c'	F_v'	E'
DURATION (NDS 2.3.2)	C_D	1.60	1.60	1.60	
MOISTURE FACTOR	C_M	1.00	1.00	1.00	1.00
TEMPERATURE FACTOR	C_t	1.00	1.00	1.00	1.00
INCISING FACTOR	C_i	1.00	1.00	1.00	1.00
SIZE FACTOR	C_F	1.30	1.10		1.00
FLAT USE FACTOR	C_{fu}				
COLUMN STABILITY	C_P		0.079		
REPETITIVE (1.15 or 1.0)	C_r	1.00			
BEAM STABILITY	C_L	0.97			

MODULUS OF ELASTICITY E'_{min} = 620 ksi
COLUMN PARAMETER c = 0.80
BEAM PARAMETER R_B = 11.544 < 50

BUCKLING VALUES

F_{cE} =	211	psi	F_{bE} =	5583	psi
F_c^* =	2640	psi	F_b^* =	2080	psi

ADJUSTED PROPERTIES:

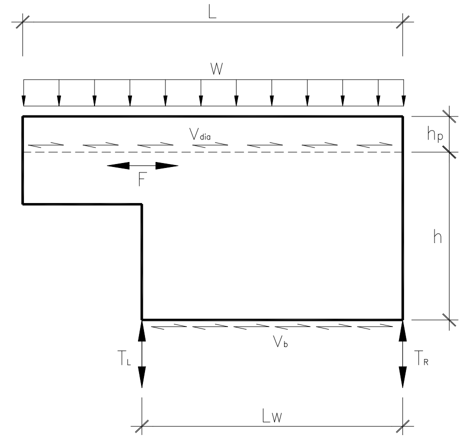
MODULUS OF ELASTICITY	E' =	1700	ksi	AXIAL STRESS	F_c' =	208	psi
BENDING STRESS (X-Axis)	F_{bx}' =	2023	psi	SHEAR STRESS	F_v' =	288	psi

ACTUAL STRESSES:

AXIAL STRESS	f_c =	71.7	psi	SHEAR STRESS	f_v =	33	psi
BENDING STRESSES	f_{bx} =	1128.7	psi				

Shear Wall Design Based on IBC 09 / CBC 10 / NDS 05

INPUT DATA



LATERAL FORCE ON DIAPHRAGM:			$v_{dia, WIND}$ =	240	plf,for wind		
			$v_{dia, SEISMIC}$ =	286	plf,for seismic, ASD		
GRAVITY LOADS ON THE ROOF:			w_{DL} =	262	plf,for dead load		
			w_{LL} =	0	plf,for live load		
DIMENSIONS:	L_w =	9	ft , h =	16	ft		
	L =	11	ft , h_p =	0	ft		
PANEL GRADE (0 or 1) =		1	<= Sheathing and Single-Floor				
MINIMUM NOMINAL PANEL THICKNESS =				15/32	in		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)				1	8d		
SPECIFIC GRAVITY OF FRAMING MEMBERS				0.5			
EDGE STUD SECTION		2	pcs, b =	2	in , h =	6	in
SPECIES (1 = DFL, 2 = SP)			1	DOUGLAS FIR-LARCH			
GRADE (1, 2, 3, 4, 5, or 6)			3	No. 1			
STORY OPTION (1=ground level, 2=upper level)				1	ground level shear wall		

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 30 in O.C.

HOLD-DOWN FORCES: T_L = 3.69 k , T_R = 4.27 k (USE PHD5-SDS3 SIMPSON HOLD-DOWN)
DRAG STRUT FORCES: F = 0.57 k
EDGE STUD: 2 - 2" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.66 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 1.8 < 3.5 **[Satisfactory]**
DETERMINE REQUIRED CAPACITY v_b = 350 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-08 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	8d	1 1/2	15/32	260	380	490	640

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-08 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: F = (L-L_w) MAX($v_{dia, WIND}$, $\Omega_0 v_{dia, SEISMIC}$) = 0.57 k (Ω_0 = 1) (Sec. 1633.2.6)

DETERMINE MAX SPACING OF 5/8" DIA ANCHOR BOLT (NDS 2005, Tab.11E)
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 30 in O.C.
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	286	230	52179	Left	21035	0.9	T_L =	3694	PHD5-SDS3
				Right	15271	0.9	T_R =	4271	
WIND	240		42240	Left	21035	2/3	T_L =	3135	
				Right	15271	2/3	T_R =	3562	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-08 4.3.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w} = 0.665 \text{ in, ASD} < \delta_{xe,allowable,\ ASD} = 0.686 \text{ in}$$

[Satisfactory] (ASCE 7-05 12.8.6)

Where: v_b = 350 plf , ASD; L_w = 9 ft; E = 1.7E+06 psi
A = 16.50 in^2; h = 16 ft; G = 9.0E+04 psi; C_d = 4; I = 1
t = 0.298 in; e_n = 0.012 in; d_a = 0.15 in; ,(ASCE 7-05 Tab 12.2-1 & Tab 11.5-1)
Δ_a = 0.02 h_{sx} , (ASCE 7-05 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 5.42 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1500 psi; C_D = 1.60; C_P = 0.15; A = 16.5 in^2
E = 1700 ksi; C_F = 1.10; F_c' = 404 psi > f_c = 329 psi

[Satisfactory]

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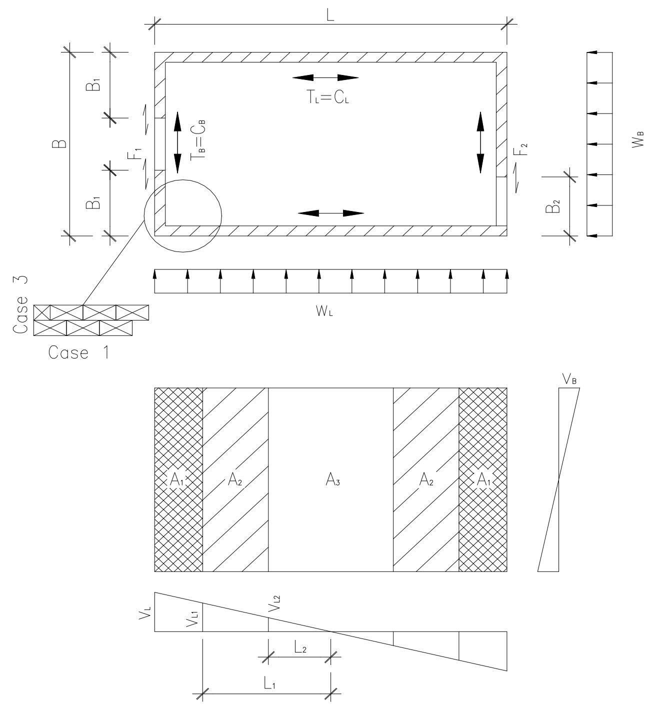
Wood Diaphragm Design Based on NDS 2005

INPUT DATA



LATERAL FORCE ALONG L SIDE:	$w_{L, WIND}$ =	256	plf,for wind
	$w_{L, SEISMIC}$ =	300	plf,for seismic
LATERAL FORCE ALONG B SIDE:	$w_{B, WIND}$ =	256	plf,for wind
	$w_{B, SEISMIC}$ =	343	plf,for seismic

DIMENSIONS: L = 240 ft , B = 110 ft
B_1 = 45 ft , B_2 = 40 ft
PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor
MINIMUM NOMINAL FRAMING WITH (2 or 3) = 3 in
MINIMUM NOMINAL PANEL THICKNESS = 15/32 in
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) = 1 8d
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.43
FRAMING OF ***Douglas-Fir-Larch*** OR ***Southern Pine*** ? Yes

DESIGN SUMMARY

A1: (2) - 10 ft x 110 ft
BLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 4 in O.C. BOUNDARY / 6 in O.C. EDGES / 12"O.C. FIELD.

A2: (2) - 14 ft x 110 ft
BLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 6" O.C. BOUNDARY & EDGES / 12"O.CFIELD.

A3: (1) - 192.00 ft x 110 ft
UNBLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 6" O.C. ALL EDGES / 12"O.CFIELD.

THE CHORD FORCES: $T_L = C_L$ = 19.64 k , $T_B = C_B$ = 2.16 k
THE DRAG STRUT FORCES: F_1 = 9.16 k , F_2 = 36.65 k
THE MAXIMUM DIAPHRAGM DEFLECTION: Δ = 3.30 in

ANALYSIS

THE DIAPHRAGM IS CONSIDER FLEXIBLE IF ITS MAXIMUM LATERAL DEFORMATION IS MORE THAN TWO TIMES THE AVERAGE SHEAR WALL DEFLECTION OF THE ASSOCIATED STORY. WITHOUT FURTHER CALCULATIONS, ASSUME A FLEXIBLE DIAPHRAGM HERE. FROM THE TABLE 3.1 IN ASD MANUAL SUPP 01, PAGE SP-12, THE PANEL BENDING STRENGTH CAPACITY IS 355 in-lbs/ft, THAT IS THE DIAPHRAGM CAN RESISTS 59 psf GRAVITY LOADS (DL+LL) AT 2'-0" o.c. SPACING SUPPORTS.

THE MAX DIAPHRAGM DIMENSION RATIO L / B = 2.2 < 3, [satisfactory]
THE MAX SHEAR FORCE ALONG B SIDE v_L = 327 plf, (Boundary Spacing = 4 in, Edges ReqD = 6 in)
THE MAX SHEAR FORCE ALONG L SIDE v_B = 79 plf, (Required Boundary/Edges Nail Spacing for Case 3 = 6 in)
THE ALLOWABLE SHEAR FORCE FOR CASE 1 @ 6 in NAIL SPACING v1 = 300 plf, L1 = 110.0 ft
THE MAX ALLOWABLE UNBLOCKED SHEAR FORCE FOR CASE 1 v1 = 265 plf, L2 = 97.2 ft

THE SHEAR CAPACITIES PER IBC Table 2306.2.1 / SDPWS-08 Table 4.2A with ASD reduction factor 2.0 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Member Width (in)	Blocked Nail Spacing Boundary / Other Edges 6 / 6	4 / 6	2.5 / 4	2 / 3	Unblocked Case 1	Others
Sheathing and Single-Floor	8d	1 1/2	15/32	3	300	400	600	675	265	200

Note: The indicated shear numbers have reduced by specific gravity factor per SDPWS-08 Table 4.2A note 2.

THE CHORD FORCES: $T_L = C_L = (w_L L^2) / (8B)$ = 19.64 k $T_B = C_B = (w_B B^2) / (8L)$ = 2.16 k
THE DRAG STRUT FORCES: $F_1 = 0.5\,(B-2B_1)\,\text{MAX}(v_{1,\,WIND},\ \Omega_0 v_{1,\,SEISMIC})$ = 9.16 k
$F_2 = B_2\,\text{MAX}(v_{1,\,WIND},\ \Omega_0 v_{1,\,SEISMIC})$ = 36.65 k
Ω_0 = 2.8 (ASCE 7-05 Table 12.2-1)

THE MAXIMUM DIAPHRAGM DEFLECTION: (IBC 2305.2.2 , / SDPWS-08 4.2.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ \ slip} + \Delta_{Chord\ \ splice\ \ slip} = \frac{5 v_L L^3}{8EAB} + \frac{v_L L}{4Gt} + 0.188 L e_n + \frac{\Sigma(D_C x)}{2B} = 3.301 \text{ in}$$

Where: v_L = 327 plf, L = 240 ft, E = 1.7E+06 psi
A = 21.75 in^2, B = 110 ft, G = 9.0E+04 psi,(UBC97 Page3-421)
t = 0.298 in,(UBC97 Page3-420), e_n = 0.037 in,(UBC97 Page3-422), $\Sigma(D_c x)$ = 45.00 in

Note: The deflection, Δ, above is based on completely blocked. For unblocked diaphragm, 2.4Δ should be used.

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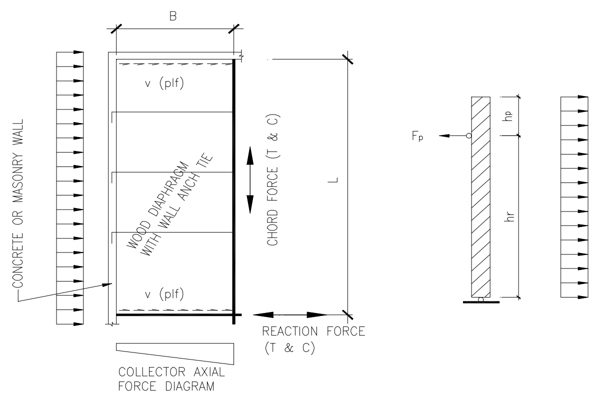
Subdiaphragm Design Based on ASCE 7-05

INPUT DATA

Length	L =	60	ft
Width	B =	46	ft
Roof Height	h_r =	28	ft
Parapet Height	h_P =	4	ft
Wall Weight	W_P =	112	psf
Coefficient	S_{DS} =	0.54	
Importance Factor	I =	1	
Diaphragm Shear Capacity	$v_{allowable}$ =	720	plf



ANALYSIS

The subdiaphragms comply with 2.5:1 of max. length-to-width rario. (ASCE 7-05, 12.11.2.2.1)
The wall anchor force is given by ASCE 7-05, 12.11.2 as

$$F_P = MAX\left[0.8 S_{DS} I W_p \frac{\left(h_r + h_p\right)^2}{2h_r}, 400 S_{DS} I, F_{min}\right] = \quad 885 \text{ plf}$$

Where : F_{min} = 280 plf
(ASCE 7-05, 12.11.2c)

Wood subdiaphragm shear :

$$v = \frac{0.5 F_P L}{1.4B} = \quad \mathbf{412} \text{ plf, for ASD}$$

< $v_{allowable}$

Satisfactory to use diaphragm nailing for subdiaphragm.

Chord force :

$$T = C = \frac{F_P L^2}{8B} = \quad \mathbf{8.66}$$

k, (Indicated force has NOT been reduced for ASD)

Reaction force :

$$R = \frac{F_P L}{2} = \quad \mathbf{26.54} \text{ k}$$

(Indicated force has NOT been reduced for ASD)

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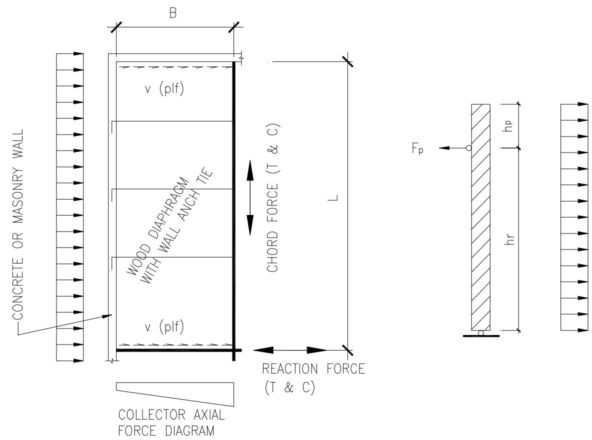
Subdiaphragm Design Based on UBC 97

INPUT DATA

Length	L =	60	ft
Width	B =	46	ft
Roof Height	h_r =	28	ft
Parapet Height	h_P =	4	ft
Wall Weight	W_P =	112	psf
Coefficient	C_a =	0.44	
Seismic Zone (2A, 2B, 3, 4)		4	
Importance Factor	I_P =	1	
Diaphragm Capacity	$v_{allowable}$ =	720	plf (ASD)



ANALYSIS

Rp = 3
ap = 1.5

The subdiaphragms comply with 2.5:1 max. length-to-width rario. (Sec.1633.2.9,UBC 97)

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1+3\frac{h_x}{h_r}\right) w_p = \frac{4 a_p C_a I_p w_p}{R_p} = \quad 99 \quad \text{psf}$$

Check minimum wall-roof anchorage force : (Sec.1633.2.8.1 & 1611.4, UBC97)

$$w = F_p \frac{\left(h_r + h_p\right)^2}{2 h_r} = \quad 1802 \quad \text{plf} \quad > \quad 420 \quad \text{plf}$$

Thus, w = 1802 plf

Wood subdiaphragm shear : (Sec.1633.2.8.1 item 5, UBC97)

$$v = \begin{cases} 0.85\dfrac{0.5wL}{1.4B} \;, & for \;\; zone \;\; 3 \;\; or \;\; 4 \\ \dfrac{0.5wL}{1.4B} \;, & for \;\; zone \;\; 1 \;\; or \;\; 2 \end{cases}$$

= **714** plf, for ASD
< 720 plf

Use diaphragm nailing for subdiaphragm is adequate.

Chord force :

$$T = C = \frac{wL^2}{8B}$$

= **17.63** kips, (for SD) = **12.59** kips, (for ASD)

Steel tie/reaction force : (Sec.1633.2.8.1 item 4, UBC97)

$$R = \begin{cases} 1.4\dfrac{wL}{2} \;, & for \;\; zone \;\; 3 \;\; or \;\; 4 \\ \dfrac{wL}{2} \;, & for \;\; zone \;\; 1 \;\; or \;\; 2 \end{cases}$$

= **75.69** kips, (SD) = **54.07** kips, (for ASD)

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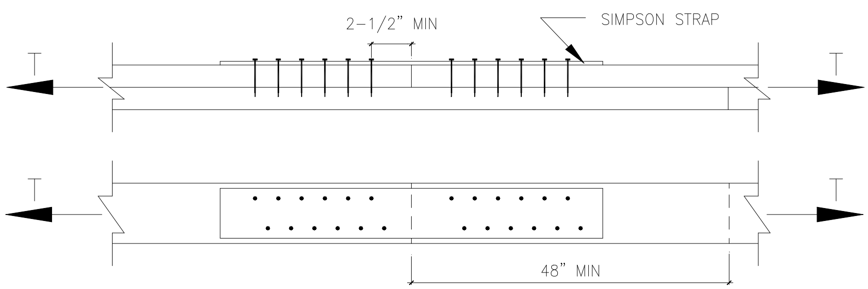
Top Plate Connection Design Based on NDS 2005

INPUT DATA & DESIGN SUMMARY

DIAPHRAGM CHORD FORCE RESISTED BY THE TOP PLATE	T = C =	15	k
NAIL TYPE (0=Common Wire, 1=Box, 2=Sinker)		0	Common Wire Nail
NAIL PENNY-WEIGHT (12d, 16d, 20d)		16d	
LUMBER TYPE (0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir)		0	Douglas Fir-Larch, G=0.5
LUMBER GRADE (0=Select Structural, 1=No.1 & Btr, 2=No.1, 3=No.2, 4=No.3, 5=Stud, 6=Construction, 7=Standard, 8=Utility)		2	No.1
TOP PLATE SIZE Double 2	x	6	No.1, Douglas Fir-Larch, G=0.5
LOAD DURATION FACTOR (Tab 2.3.2, NDS 2005, Page 8)	C_D =	1.6	
WET SERVICE FACTOR (Tab 10.3.3, NDS 2005, Page 58)	C_M =	1.0	
TEMPERATURE FACTOR (Tab 10.3.4, NDS 2005, Page 58)	C_t =	1.0	

Use CMSTC16 with 32-16d sinkers, Each Side.



ANALYSIS

DESIGN VALUE FOR TENSION (Tab 4A, NDS 2005 SUPP, Page 32) F_t = 675 psi

AREA OF CROSS SECTION FOR ONE 2 x 6 MEMBER A = 8.25 in^2

SIZE FACTOR (Tab 4A, NDS 2005 SUPP, Page 30) C_F = 1.30

ALLOWABLE TENSION CAPACITY FOR ONE 2 x 6 ONLY $T' = AF_tC_DC_MC_tC_F$ = 11.58 k

< T, SIMPSON STRAP REQUIRED.

NAIL LENGTH L = 3 1/2 in

SIDE MEMBER THICKNESS t_s = 1 1/2 in

THE PENETRATION OF THE NAIL INTO THE MAIN MEMBER p = 1.99 in

NAIL DIAMETER D = 0.162 in

THE PENETRATION FACTOR (Note 3, Tab 11N, NDS 2005, Page 97) C_d = 1.00

THE NOMINAL DESIGN VALUE FOR SINGLE SHEAR IS TABULATED IN NDS 2005 TABLE 11N, PAGE 97, AS

Z = 141 lbf

THE ALLOWABLE LATERAL DESIGN VALUE FOR THE ONE NAIL IS

$Z' = ZC_DC_MC_tC_d$ = 226 lbf

THE NUMBER OF NAILS REQUIRED IS n = T / Z' = 66.5 = 67 Nails

THE MIN. FORCE RESISTED BY THE SIMPSON STRAP T - T' = 3.42 k

Use CMSTC16 with 32-16d sinkers, Each Side.

Technical References:

1. "National Design Specification, NDS", 2005 Edition, AF&AP, AWC, 2005.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.

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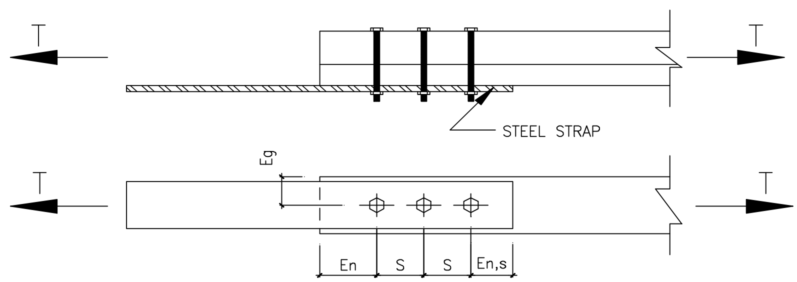
Bolt Connection Design Based on NDS 2012

INPUT DATA & DESIGN SUMMARY

AXIAL TENSILE FORCE (ASD)	T =	5.3	k
NUMBER OF BOLTS	n =	3	
BOLT DIAMETER	ϕ =	3/4	
BOLT SPACING	S =	3	in
END DISTANCE OF WOOD	E_n =	4	in
END DISTANCE OF STEEL	$E_{n,s}$ =	1.5	in
LUMBER TYPE (0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir)		0	Douglas Fir-Larch, G=0.5
LUMBER SIZE (2) - 2 thk. x		6	width
STRAP SIZE 5 width x		1/4	thk.
LOAD DURATION FACTOR (Tab 2.3.2, NDS 2012)	C_D =	1.6	
WET SERVICE FACTOR (Tab 10.3.3, NDS 2012)	C_M =	1.0	
TEMPERATURE FACTOR (Tab 10.3.4, NDS 2012)	C_t =	1.0	

THE CONNECTION DESIGN IS ADEQUATE.



ANALYSIS

CHECK STEEL STRAP CAPACITIES (AISC 360-10, ASD)

A_g = 1.25 in^2, yielding criterion F_y = 36.00 ksi

T_{allow} = $0.6\ F_y\ A_g$ = 27.00 k > T [Satisfactory]

(0.6 from $1/\Omega_t$, Typ.)

A_n = 1.03 in^2, fracture criterion F_u = 58.00 ksi

T_{allow} = $0.5\ F_u\ A_n$ = 29.91 k > T [Satisfactory]

A_v = 1.33 in^2, block shear

T_{allow} = $0.3\ F_u\ A_v + 0.5\ F_u\ (0.5\ A_n)$ = 38.06 k > T [Satisfactory]

r_{min} = $t / (12)^{0.5}$ = 0.072 in L = Max (E_n , S) = 4 in

L / r_{min} = 55 < 300 [Satisfactory] (AISC 360-10 D1)

CHECK EDGE, END, & SPACING DISTANCE REQUIREMENTS (NDS 2012, Table 11.5.1A, Table 11.5.1B, & Table 11.5.1C)

E_g =	2.75	in	>	1.5 D	[Satisfactory]
E_n =	4	in	>	3.5 D	[Satisfactory]
S =	3	in	>	3 D	[Satisfactory]

CHECK WOOD CAPACITY

C_Δ = Min ($C_{\Delta1}$, $C_{\Delta2}$, $C_{\Delta3}$) = 0.762 , (geometry factor, NDS 2012, 11.5.1)

where $C_{\Delta1}$ = (actual end distance) / (min end distance for full design value) = E_n / 7D = 0.762

$C_{\Delta2}$ = (actual shear area) / (min shear area for full design value) = 1.000

$C_{\Delta3}$ = (actual spacing) / (min spacing for full design value) = S / 4D = 1.000

$$C_g = \left[\frac{m\left(1-m^{2n}\right)}{n\left[\left(1+R_{EA}m^n\right)\left(1+m\right)-1+m^{2n}\right]}\right]\left[\frac{1+R_{EA}}{1-m}\right] = \quad 0.988$$

, (group action factor, NDS 2012, 10.3.6)

where n = 3 R_{EA} = Min [(E_sA_s/E_mA_m) , (E_mA_m/E_sA_s)] = 0.616

E_sA_s = 37500000 lbs, (NDS 2012, Table 10.3.6C) $\gamma = 180000\ D^{1.5}$ = 116913.4

t_m = 3 in $u = 1+\gamma\ S/2\ [1 / E_mA_m + 1 / E_sA_s]$ = 1.012

E_mA_m = 23100000 lbs, (NDS 2012, Table 10.3.6C) $m = u - (u^2 - 1)^{0.5}$ = 0.855

$Z'_{//}$ = $n\ Z_{//}\ C_D\ C_M\ C_t\ C_g\ C_\Delta$ = 5.309 kips > T [Satisfactory]

where $Z_{//}$ = 1470 lbs / bolt, (interpolated from NDS 2012, Table 11B)

Technical References:

1. "National Design Specification, NDS", 2012 Edition, AF&AP, AWC, 2012.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.

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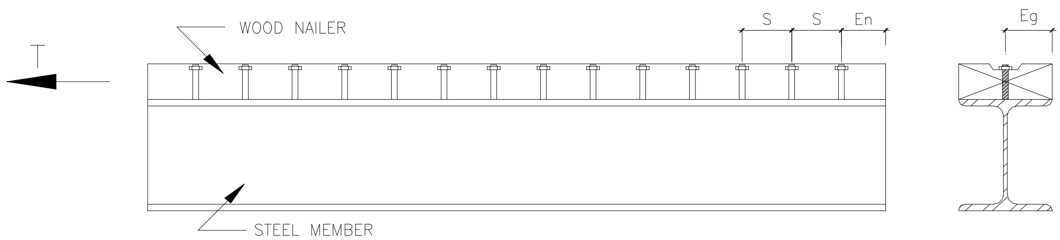
Nailer Connection Design Based on NDS 2005

INPUT DATA & DESIGN SUMMARY

AXIAL TENSILE FORCE (ASD)	T =	8	k
NUMBER OF STUDS	n =	6	
THREAD STUD DIAMETER	ϕ =	3/4	
THREAD STUD SPACING	S =	12	in
END DISTANCE OF WOOD	E_n =	4	in
NAILER TYPE (0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir)		0	Douglas Fir-Larch, G=0.5
NAILER SIZE	4 thk. x	6	width
SECTION AREA OF STEEL MEMBER	A_s =	11.8	in^2
LOAD DURATION FACTOR (Tab 2.3.2, NDS 2005, Page 8)	C_D =	1.6	
WET SERVICE FACTOR (Tab 10.3.3, NDS 2005, Page 58)	C_M =	1.0	
TEMPERATURE FACTOR (Tab 10.3.4, NDS 2005, Page 58)	C_t =	1.0	
GROUP ACTION FACTOR APPLY? (0 = No, 1 = Yes)		1	Yes, (C_g = 0.719 , NDS 2005, 10.3.6)

(If T is drag/collector force or stud spacing less than 12" o.c., the group action factor C_g must apply.)

THE CONNECTION DESIGN IS ADEQUATE.



ANALYSIS

CHECK EDGE, END, & SPACING DISTANCE REQUIREMENTS (NDS 2005, Table 11.5.1A, Table 11.5.1B, & Table 11.5.1C)

E_g =	2.75	in	>	1.5 D	[Satisfactory]
E_n =	4	in	>	3.5 D	[Satisfactory]
S =	12	in	>	3 D	[Satisfactory]

CHECK WOOD CAPACITY

$C_\Delta = \text{Min}(C_{\Delta 1}, C_{\Delta 2}, C_{\Delta 3})$ = 0.762 , (geometry factor, NDS 2005, 11.5.1, page 76)

where $C_{\Delta 1}$ = (actual end distance) / (min end distance for full design value) = E_n / 7D = 0.762

$C_{\Delta 2}$ = (actual shear area) / (min shear area for full design value) = 1.000

$C_{\Delta 3}$ = (actual spacing) / (min spacing for full design value) = S / 4D = 4.000

$$C_g = \left[\frac{m\left(1-m^{2n}\right)}{n\left[\left(1+R_{EA}m^{n}\right)(1+m)-1+m^{2n}\right]}\right]\left[\frac{1+R_{EA}}{1-m}\right] =$$ 0.719 , (group action factor, NDS 2005, 10.3.6, page 60)

where n = 6

E_sA_s = 3.4E+08 lbs

t_m = 3.5 in

E_mA_m = 2.7E+07 lbs, (E_m fr NDS, Tab.10.3.6C)

$R_{EA} = \text{Min}[(E_sA_s/E_mA_m), (E_mA_m/E_sA_s)]$ = 0.079

$\gamma = 180000\ D^{1.5}$ = 116913

$u = 1+\gamma\ S/2\ [1 / E_mA_m + 1 / E_sA_s]$ = 1.028

$m = u - (u^2 - 1)^{0.5}$ = 0.789

$Z'_{//}$ = $n\ Z_{//}\ C_D\ C_M\ C_t\ C_g\ C_\Delta$ = 9.050 kips > T [Satisfactory]

where $Z_{//}$ = 1720 lbs / stud, (interpolated from Tab 11E, NDS 2005, Page 85 or Tab 11B, NDS 2005, Page 82)

Technical References:

1. "National Design Specification, NDS", 2005 Edition, AF&AP, AWC, 2005.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.

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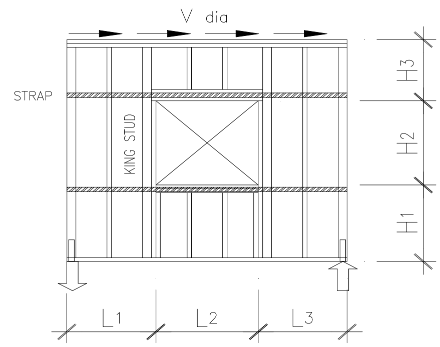
Wood Shear Wall with an Opening Based on IBC 09 / CBC 10 / NDS 05

INPUT DATA



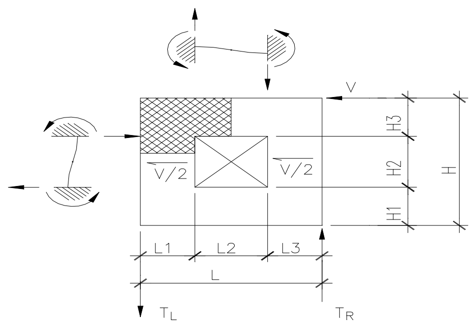
LATERAL FORCE ON DIAPHRAGM:	$V_{dia,\ WIND}$ =	151	plf,for wind					
(SERVICE LOADS)	$V_{dia,\ SEISMIC}$ =	151	plf,for seismic, ASD					
DIMENSIONS:	L_1 =	3	ft , L_2 =	4	ft , L_3 =	3	ft	
	H_1 =	2.67	ft , H_2 =	4	ft , H_3 =	2.33	ft	
KING STUD SECTION		2	pcs, b =	2	in , h =	6	in	
SPECIES (1 = DFL, 2 = SP)		1	DOUGLAS FIR-LARCH					
GRADE (1, 2, 3, 4, 5, or 6)		3	No. 1					
EDGE STUD SECTION		1	pcs, b =	4	in , h =	6	in	
SPECIES (1 = DFL, 2 = SP)		1	DOUGLAS FIR-LARCH					
GRADE (1, 2, 3, 4, 5, or 6)		3	No. 1					
PANEL GRADE (0 or 1)	=	1	<= Sheathing and Single-Floor					
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in					
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d					
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						
STORY OPTION (1=ground level, 2=upper level)		2	upper level shear wall					

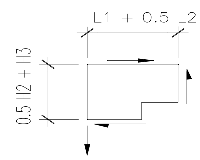
THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

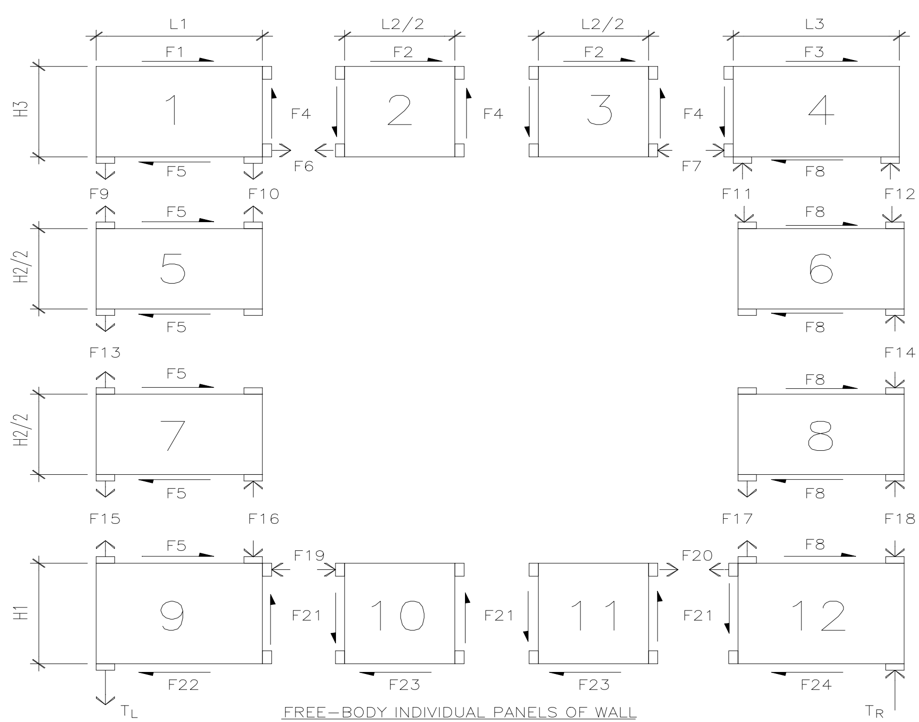
BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 6" O.C.

HOLD-DOWN FORCES: T_L = 1.42 k , T_R = 1.42 k (USE CS16 SIMPSON HOLD-DOWN)
MAX STRAP FORCE: F = 0.56 k (USE SIMPSON CS22 OVER WALL SHEATHING WITH FLAT BLOCKING)
KING STUD: 2 - 2" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
EDGE STUD: 1 - 4" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.24 in





ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 3.5 **[Satisfactory]**

DETERMINE FORCES & SHEAR STRESS OF FREE-BODY INDIVIDUAL PANELS OF WALL

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	3.00	2.33	65	F1	194	F13	654
2	2.00	2.33	281	F2	561	F14	654
3	2.00	2.33	281	F3	194	F15	1157
4	3.00	2.33	65	F4	654	F16	503
5	3.00	2.00	252	F5	755	F17	503
6	3.00	2.00	252	F6	561	F18	1157
7	3.00	2.00	252	F7	561	F19	455
8	3.00	2.00	252	F8	755	F20	455
9	3.00	2.67	100	F9	150	F21	770
10	2.00	2.67	288	F10	503	F22	300
11	2.00	2.67	288	F11	503	F23	455
12	3.00	2.67	100	F12	150	F24	300

DETERMINE REQUIRED CAPACITY v_b = 288 plf, (1 Side Panel Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-08 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Note: The indicated shear numbers have reduced by specific gravity factor per IBC note a.

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2005, Table 11Q & Table 11L)

SILL PLATE ATTACHMENT 16d AT 6" O.C.

THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	151	144	14238	Left	0	0.9	T_L =	1424	CS16
				Right	0	0.9	T_R =	1424	
WIND	151		13590	Left	0	2/3	T_L =	1359	
				Right	0	2/3	T_R =	1359	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-08 4.3.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w}$$

= 0.238 in, ASD < $\delta_{xe,allowable,\ ASD}$ = 0.386 in **[Satisfactory]** (ASCE 7-05 12.8.6)

Where: v_b = 288 plf , ASD L_w = 10 ft E = 1.7E+06 psi

A = 16.50 in² h = 9 ft G = 9.0E+04 psi C_d = 4 I = 1

t = 0.298 in e_n = 0.000 in d_a = 0.15 in ,(ASCE 7-05 Tab 12.2-1 & Tab 11.5-1)

Δ_a = 0.02 h_{sx} , (ASCE 7-05 Tab 12.12-1)

CHECK KING STUD CAPACITY

P_{max} = 0.50 kips

F_c = 1500 psi C_D = 1.60 C_P = 0.43 A = 16.50 in²

E = 1700 ksi C_F = 1.10 F_c' = 1146 psi > f_c = 31 psi

[Satisfactory]

CHECK EDGE STUD CAPACITY

P_{max} = 1.42 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1500 psi C_D = 1.60 C_P = 0.43 A = 19.25 in²

E = 1700 ksi C_F = 1.10 F_c' = 1146 psi > f_c = 74 psi

[Satisfactory]

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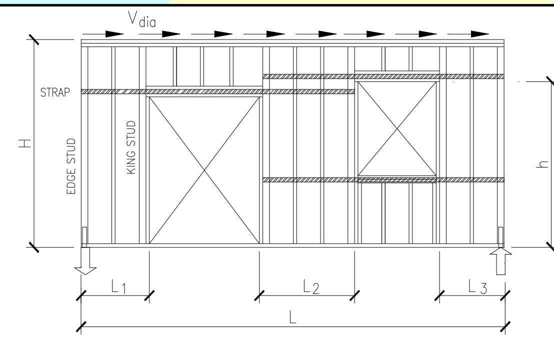
Perforated Shear Wall Design Based on IBC 09 / CBC 10 / NDS 05



INPUT DATA

LATERAL FORCE ON DIAPHRAGM (SERVICE LOADS):

$V_{dia, WIND}$ = 151 plf,for wind

$V_{dia, SEISMIC}$ = 130 plf,for seismic, ASD

NUMBER OF OPENINGS n = 5

DIMENSIONS: L = 72 ft

H = 18 ft

h = 10 ft, (the highest opening)

i	L_i		H to L_i ratio	
1	6	ft	3.00	ok
2	6.5	ft	2.77	ok
3	5.5	ft	3.27	ok
4	7.5	ft	2.40	ok
5	6.33	ft	2.84	ok
6	6.67	ft	2.70	ok
Σ	38.5	ft		

KING STUD SECTION

2 pcs, b = 3 in , h = 6 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 3 No. 1

EDGE STUD SECTION

1 pcs, b = 6 in , h = 6 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 3 Dense No.1

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

STORY OPTION (1=ground level, 2=upper level) 1 ground

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 28 in O.C.

HOLD-DOWN FORCES: T_L = 6.62 k , T_R = 6.62 k (USE HDQ8-SDS3 SIMPSON HOLD-DOWN)
MAX STRAP FORCE: F = 2.76 k (USE SIMPSON CMSTC16 OVER WALL SHEATHING WITH FLAT BLOCKING)
KING STUD: 2 - 3" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
EDGE STUD: 1 - 6" x 6" DOUGLAS FIR-LARCH Dense No.1, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.43 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 3.3 < 3.5 **[Satisfactory]** (allow reduced 2w/h, SDPWS-08 4.3.4)

DETERMINE SHEAR RESISTANCE ADJUSTMENT FACTOR, C_o

Percentage of Full-Height Sheathing = 53% , (SDPWS-08 4.3.3.5)

Maximum Opening Height = 0.56 H

C_o = 0.768 , (SDPWS-08 Table 4.3.3.5)

DETERMINE REQUIRED CAPACITY (SDPWS-08 4.3.3.5)

$v_b = V / (C_o \ \Sigma L_i) =$ 368 plf, (1 Side Panel Required, the Max. Nail Spacing = 4 in)

< 870 plf, (SDPWS-08 4.3.5.3.3) **[Satisfactory]**

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-08 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Note: The indicated shear numbers have reduced by specific gravity factor per IBC note a.

DETERMINE MAX SPACING OF 5/8" DIA ANCHOR BOLT (NDS 2005, Tab.11E)

5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 28 in O.C.

THE HOLD-DOWN FORCES:

	V_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	130	2074	168480	Left	0	0.9	T_L =	5696	HDQ8-SDS3
				Right	0	0.9	T_R =	5696	
WIND	151		195696	Left	0	2/3	T_L =	6616	
				Right	0	2/3	T_R =	6616	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

(cont'd)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-08 4.3.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8v_b h^3}{EAL_w} + \frac{v_b h}{Gt} + 0.75he_n + \frac{hd_a}{L_w} = 0.433 \text{ in, ASD} < \delta_{xe,allowable,\ ASD} = 0.771 \text{ in}$$

[Satisfactory]

Where: v_b = 368 plf , ASD; L_w = 38.5 ft; E = 1.7E+06 psi; (ASCE 7-05 12.8.6)

A = 16.50 in^2; h = 18 ft; G = 9.0E+04 psi; C_d = 4; I = 1

t = 0.298 in; e_n = 0.00E+00 in; d_a = 0.15 in; ,(ASCE 7-05 Tab 12.2-1 & Tab 11.5-1)

Δ_a = 0.02 h_{sx} , (ASCE 7-05 Tab 12.12-1)

CHECK KING STUD CAPACITY (SDPWS-08 4.3.6.1.2)

P_{max} = 5.16 kips

F_c = 1500 psi; C_D = 1.60; C_P = 0.12; A = 27.50 in^2

E = 1700 ksi; C_F = 1.10; F_c' = 322 psi > f_c = 187 psi

[Satisfactory]

CHECK EDGE STUD CAPACITY (SDPWS-08 4.3.6.1.2)

P_{max} = 6.62 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1200 psi; C_D = 1.60; C_P = 0.17; A = 30.25 in^2

E = 1700 ksi; C_F = 1.00; F_c' = 318 psi > f_c = 219 psi

[Satisfactory]

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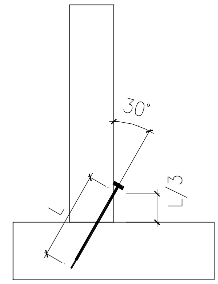
Toe-Nail Connection Design Based on NDS 2005

INPUT DATA & DESIGN SUMMARY



NAIL TYPE (0=Common Wire, 1=Box, 2=Sinker) 0 Common Wire Nail

NAIL PENNY-WEIGHT (6d, 7d, 8d, 10d, 12d, 16d, 20d, 30d, 40d, 50d, 60d) 10d

LUMBER SPECIES (0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir) 0 Douglas Fir-Larch, G=0.5

LOAD DURATION FACTOR (Tab 2.3.2, NDS 2005, Page 8) C_D = 1.6

WET SERVICE FACTOR (Tab 10.3.3, NDS 2005, Page 58) C_M = 1.0

TEMPERATURE FACTOR (Tab 10.3.4, NDS 2005, Page 58) C_t = 1.0

THE ALLOWABLE LATERAL DESIGN VALUE FOR THE TOE-NAIL = 157 lbf

ANALYSIS

TOE-NAIL FACTOR (Sec 11.5.4.2, NDS 05, Page 78) C_{tn} = 0.83

NAIL LENGTH L = 3 in

THE SIDE MEMBER THICKNESS IS TAKEN TO BE EQUAL TO $t_s = L / 3 =$ 1 in

THE PENETRATION OF THE NAIL INTO THE MAIN MEMBER $p = L(\cos 30^o) - t_s =$ 1.60 in

NAIL DIAMETER D = 0.148 in

THE PENETRATION FACTOR (Note 3, Tab 11N, NDS 2005, Page 97) C_d = 1.00

THE NOMINAL DESIGN VALUE FOR SINGLE SHEAR IS TABULATED IN NDS 2005 TABLE 11N, PAGE 97, AS

Z = 118 lbf

THE ALLOWABLE LATERAL DESIGN VALUE FOR THE TOE-NAIL IS

$Z' = ZC_DC_MC_tC_dC_{tn} =$ 157 lbf

Typical Nail Dimensions (Appendix L4, NDS 2005, Page 167)

Type		Penny-Weight										
		6d	7d	8d	10d	12d	16d	20d	30d	40d	50d	60d
Common	Length	2	2 1/4	2 1/2	3	3 1/4	3 1/2	4	4 1/2	5	5 1/2	6
	Diameter	0.113	0.113	0.131	0.148	0.148	0.162	0.192	0.207	0.225	0.244	0.263
	Head	0.266	0.266	0.281	0.312	0.312	0.344	0.406	0.438	0.469	0.500	0.531
Box	Length	2	2 1/4	2 1/2	3	3 1/4	3 1/2	4	4 1/2	5		
	Diameter	0.099	0.099	0.113	0.128	0.128	0.135	0.148	0.148	0.162		
	Head	0.266	0.266	0.297	0.312	0.312	0.344	0.375	0.375	0.406		
Sinker	Length	1 7/8	8 1/8	2 3/8	2 7/8	3 1/8	3 1/4	3 3/4	4 1/4	4 3/4		5 3/4
	Diameter	0.092	0.099	0.113	0.120	0.135	0.148	0.177	0.192	0.207		0.244
	Head	0.234	0.250	0.266	0.281	0.312	0.344	0.375	0.406	0.438		0.500

Technical References:

1. "National Design Specification, NDS", 2005 Edition, AF&AP, AWC, 2005.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.

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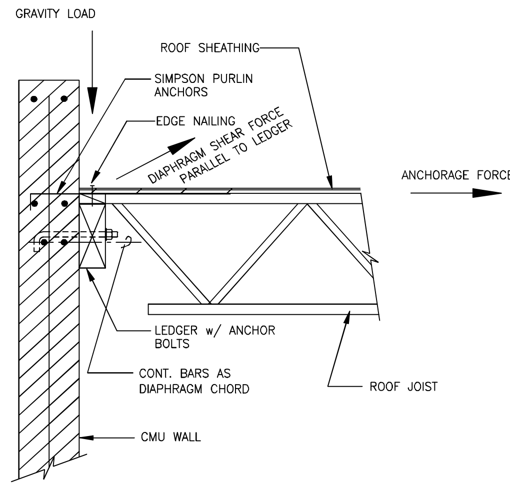
Connection Design for Wall & Diaphragm Based on IBC 09 / CBC 10



INPUT DATA

GRAVITY LOAD	D+L =	250	plf
SERVICE ANCHORAGE FORCE	F_{anch} =	420	plf
SERVICE DIAPHRAGM SHEAR FORCE	V =	640	plf
LEDGER SPECIES (0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir)		0	Douglas Fir-Larch, G=0.5
LEDGER GRADE (0=Select Structural, 1=No.1 & Btr, 2=No.1, 3=No.2, 4=No.3, 5=Stud, 6=Construction, 7=Standard, 8=Utility)		2	No.1
LEDGER SIZE	4 x 12	(No.1, Douglas Fir-Larch, G=0.5)	
ANCHOR BOLT DIAMETER	ϕ =	3/4	in
WET SERVICE FACTOR (Tab 10.3.3, NDS 2005, Page 58)	C_M =	1.0	
TEMPERATURE FACTOR (Tab 10.3.4, NDS 2005, Page 58	C_t =	1.0	
PURLIN ANCHORS SPACING	S =	36	in
TYPE OF MASONRY (1=CMU, 2=BRICK)		1	CMU
THICKNESS OF WALL	t =	8	in
CMU SPECIAL INSPECTION (0=NO, 1=YES)		1	Yes
MASONRY STRENGTH	f_M' =	1.5	ksi
REBAR YIELD STRESS	f_y =	60	ksi
WALL HORIZ. REINF.	1 # 5 @ 16		in. o. c.
ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2)		Yes	

DESIGN SUMMARY

4 x 12 LEDGER with 3/4 in DIA. A.B.'s @ 24 in o.c.
SIMPSON PLURLIN ANCHOR PAI23 w/ 11-10d x 1 1/2 Nails @ 36 in o.c.

ANALYSIS

THE DIAPHRAGM SHEAR TRANSFERS INTO THE TOP OF THE LEDGER BY NAILS THOUGH PLY WOOD AND THE LEDGER BOLTS WOULD RESIST BOTH THE DIAPHRAGM SHEAR BY PARALLEL TO GRAIN BEARING AND THE GRAVITY LOAD BY PERPENDICULAR TO GRAIN BEARING. LATER FORCES IN THE OTHER DIRECTION WOULD BE RESIST BY SIMPSON PLURLIN ANCHORS.

CHECK LEDGER CAPACITY

LOAD DURATION FACTOR (Tab 2.3.2, NDS 2005, Page 8) $(C_D)_{Lat}$ = 1.6 (for wind/seismic loads)
$(C_D)_{D+L}$ = 1.0 (for gravity loads)
GROUP ACTION FACTOR (Sec 10.3.6, NDS 2005, Page 60) C_g = 1.0
GEOMETRY FACTOR (Sec 11.5.1, NDS 2005, Page 76) C_Δ = 1.0 (All dimensions conform to the specified minimums.)
DESIGN VALUE FOR THE BOLT BEARING PERPENDICULAR TO GRAIN (Tab 11E, NDS 2005, Page 85) $Z_\perp$ = 880 lbf
DESIGN VALUE FOR THE BOLT BEARING PARALLEL TO GRAIN (Tab 11E, NDS 2005, Page 85) $Z_{||}$ = 1640 lbf
THE ALLOWABLE DESIGN VALUE FOR THE BOLT BEARING PERPENDICULAR TO GRA $Z_\perp' = Z_\perp (C_D)_{D+L} C_M C_t C_g C_\Delta$ = 880 lbf
THE ALLOWABLE DESIGN VALUE FOR THE BOLT BEARING PARALLEL TO GRAIN $Z_{||}' = Z_{||} (C_D)_{LAT} C_M C_t C_g C_\Delta$ = 2624 lbf
THE ANGLE BETWEEN DIRECTION OF COMBINED LOAD AND DIRECTION OF GRAIN θ = 21.34 °
THE ALLOWABLE DESIGN VALUE FOR THE BOLT BEARING AT THE ANGLE TO GRAIN $Z_\theta' = Z_\perp' Z_{||}' / (Z_\perp' \cos^2\theta + Z_{||}' \sin^2\theta)$ = 2079 lbf
THE MAXIMUM ALLOWABLE BOLT SPACING SHALL BE CALCULATED AS FOLLOWS :

$$S_1 = MIN\left[\frac{Z'_\perp}{D+L}\ ,\ \frac{Z'_\theta}{\sqrt{(D+L)^2+V^2}}\right] = 36.30 \text{ in}$$

(Use 24 in)

BASIC DESIGN VALUE FOR SHEAR (Tab 4A, NDS 2005 SUPP, Page 31) F_v = 180 psi
THE ALLOWABLE DESIGN VALUE FOR SHEAR $F_v' = F_v (C_D)_{D+L} C_M C_t$ = 180 psi
DEPTH FROM THE UNLOADED EDGE OF THE LEDGER TO THE CENTER OF THE BOLT d_e = 5.63 in
THE ALLOWABLE DESIGN SHEAR IS GIVEN BY NDS 2005 Eq. 3.4-6, Page 17, AS

$$V' = \frac{2}{3} F'_v b d_e \left(\frac{d_e}{d}\right)^2 = 591 \text{ lbf} > (D+L)S = 500 \text{ lbf}$$

[Satisfactory]

DESIGN PURLIN ANCHORS ***cont'd***

SELECT SIMPSON PAI23 w/ 11-10d x 1 1/2 Nails @ 36 in o.c.

DESIGN FORCE FOR STEEL ANCHOR

$F_{anch}S$ = 1260 lbf **<** 1880 lbf, allowable

DESIGN FORCE FOR WOOD MEMBER

$F_{anch}S$ = 1260 lbf **<** 1379 lbf, allowable **[Satisfactory]**

CHECK WALL CAPACITY TO SPAN LATERALLY S SPACING

ALLOWABLE MASONRY STRESS FACTOR : SF = 1.333

Allowable reinf. stress	F_s	=	32000	psi	Modular ratio	n	=	21.48	
Allowable stress	F_b=(SF)(0.33f_m')	=	660	psi	Wall reinf. area	A_s	=	0.23	in / ft
Masonry elasticity modulus	E_m	=	1350	ksi	Tension reinf. ratio	ρ	=	0.005	
Steel elasticity modulus	E_s	=	29000	ksi	The neutral axis depth factor	k	=	0.36	
Effective width	b_w	=	12	in	The lever-arm factor	j	=	0.88	

The tensile stress in reinforcement is

$$f_s = \frac{F_{anch}S^2}{8A_s jd} = 6938 \text{ psi} < F_s$$

[Satisfactory]

The compressive stress in the extreme fiber is

$$f_b = \frac{F_{anch}S^2}{4b_w jkd^2} = 185 \text{ psi} < F_b$$

[Satisfactory]

Technical References:

1. "National Design Specification, NDS", 2005 Edition, AF&AP, AWC, 2005.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.

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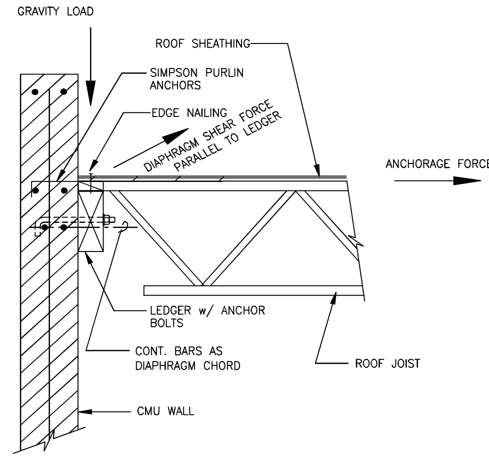
Connection Design for Wall & Diaphragm Based on IBC 09 / CBC 10



INPUT DATA

GRAVITY LOAD	D+L =	250	plf
SERVICE ANCHORAGE FORCE	F_{anch} =	300	plf
SERVICE DIAPHRAGM SHEAR FORCE	V =	640	plf
LEDGER SPECIES (0=Southern Pine with 2"-4" Wide, 1=Southern Pine with 5"-6" Wide)		0	Southern Pine
LEDGER GRADE		2	Non-Dense Select Structural

(**0**=Dense Select Structural, **1**=Select Structural, **2**=Non-Dense Select Structural, **3**=No.1 Dense, **4**=No.1, **5**=No.1 Non-Dense, **6**=No.2 Dense, **7**=No.2, **8**=No.2 Non-Dense)

LEDGER SIZE 4 x 12

(Non-Dense Select Structural, Southern Pine)

ANCHOR BOLT DIAMETER	ϕ =	3/4	in
WET SERVICE FACTOR (Tab 10.3.3, NDS 2005, Page 58)	C_M =	1.0	
TEMPERATURE FACTOR (Tab 10.3.4, NDS 2005, Page 58	C_t =	1.0	
PURLIN ANCHORS SPACING	S =	36	in
TYPE OF MASONRY (1=CMU, 2=BRICK)		1	CMU
THICKNESS OF WALL	t =	8	in
CMU SPECIAL INSPECTION (0=NO, 1=YES)		1	Yes
MASONRY STRENGTH	f_M' =	1.5	ksi
REBAR YIELD STRESS	f_y =	60	ksi

WALL HORIZ. REINF. 1 # 5 @ 16 in. o. c.

ALLOWABLE INCREASING ? (IBC/CBC 1605.3.2) Yes

DESIGN SUMMARY

4 x 12 LEDGER with 3/4 in DIA. A.B.'s @ 24 in o.c.
SIMPSON PLURLIN ANCHOR PAI18 w/ 8-10d x 1 1/2 Nails @ 36 in o.c.

ANALYSIS

THE DIAPHRAGM SHEAR TRANSFERS INTO THE TOP OF THE LEDGER BY NAILS THOUGH PLY WOOD AND THE LEDGER BOLTS WOULD RESIST BOTH THE DIAPHRAGM SHEAR BY PARALLEL TO GRAIN BEARING AND THE GRAVITY LOAD BY PERPENDICULAR TO GRAIN BEARING. LATER FORCES IN THE OTHER DIRECTION WOULD BE RESIST BY SIMPSON PLURLIN ANCHORS.

CHECK LEDGER CAPACITY

LOAD DURATION FACTOR (Tab 2.3.2, NDS 2005, Page 8) $(C_D)_{Lat}$ = 1.6 (for wind/seismic loads)
$(C_D)_{D+L}$ = 1.0 (for gravity loads)
GROUP ACTION FACTOR (Sec 10.3.6, NDS 2005, Page 60) C_g = 1.0
GEOMETRY FACTOR (Sec 11.5.1, NDS 2005, Page 76) C_Δ = 1.0 (All dimensions conform to the specified minimums.)
DESIGN VALUE FOR THE BOLT BEARING PERPENDICULAR TO GRAIN (Tab 11E, NDS 2005, Page 85) $Z_\perp$ = 950 lbf
DESIGN VALUE FOR THE BOLT BEARING PARALLEL TO GRAIN (Tab 11E, NDS 2005, Page 85) $Z_{||}$ = 1680 lbf
THE ALLOWABLE DESIGN VALUE FOR THE BOLT BEARING PERPENDICULAR TO GR/ $Z_\perp' = Z_\perp (C_D)_{D+L} C_M C_t C_g C_\Delta$ = 950 lbf
THE ALLOWABLE DESIGN VALUE FOR THE BOLT BEARING PARALLEL TO GRAIN $Z_{||}' = Z_{||}(C_D)_{LAT} C_M C_t C_g C_\Delta$ = 2688 lbf
THE ANGLE BETWEEN DIRECTION OF COMBINED LOAD AND DIRECTION OF GRAIN θ = 21.34 ^{o}
THE ALLOWABLE DESIGN VALUE FOR THE BOLT BEARING AT THE ANGLE TO GRAIN $Z_\theta' = Z_\perp' Z_{||}' / (Z_\perp' \cos^2\theta + Z_{||}' \sin^2\theta)$ = 2164 lbf
THE MAXIMUM ALLOWABLE BOLT SPACING SHALL BE CALCULATED AS FOLLOWS :

$$S_1 = MIN\left[\frac{Z'_\perp}{D+L} \ , \ \frac{Z'_\theta}{\sqrt{(D+L)^2+V^2}}\right] = 37.79 \text{ in}$$

(Use 24 in)

BASIC DESIGN VALUE FOR SHEAR (Tab 4B, NDS 2005 SUPP, Page 37) F_v = 175 psi
THE ALLOWABLE DESIGN VALUE FOR SHEAR $F_v' = F_v (C_D)_{D+L} C_M C_t$ = 175 psi
DEPTH FROM THE UNLOADED EDGE OF THE LEDGER TO THE CENTER OF THE BOL d_e = 5.63 in
THE ALLOWABLE DESIGN SHEAR IS GIVEN BY NDS 2005 Eq. 3.4-6, Page 17, AS

$$V' = \frac{2}{3} F'_v b d_e \left(\frac{d_e}{d}\right)^2 = 574 \text{ lbf} > (D+L)S = 500 \text{ lbf}$$

[Satisfactory]

DESIGN PURLIN ANCHORS

SELECT SIMPSON PAI18 w/ 8-10d x 1 1/2 Nails @ 36 in o.c.
DESIGN FORCE FOR STEEL ANCHOR $F_{anch}S$ = 900 lbf < 1255 lbf, allowable
DESIGN FORCE FOR WOOD MEMBER $F_{anch}S$ = 900 lbf < 1004 lbf, allowable
[Satisfactory]

CHECK WALL CAPACITY TO SPAN LATERALLY S SPACING ***cont'd***

ALLOWABLE MASONRY STRESS FACTOR :	SF	=	1.333							
Allowable reinf. stress	F_s	=	32000	psi	Modular ratio	n	=	21.48		
Allowable stress	F_b=(SF)(0.33f_m')	=	660	psi	Wall reinf. area	A_s	=	0.23	in / ft	
Masonry elasticity modulus	E_m	=	1350	ksi	Tension reinf. ratio	ρ	=	0.005		
Steel elasticity modulus	E_s	=	29000	ksi	The neutral axis depth factor	k	=	0.36		
Effective width	b_w	=	12	in	The lever-arm factor	j	=	0.88		

The tensile stress in reinforcement is

$$f_s = \frac{F_{anch} S^2}{8 A_s j d} = \quad 4956 \text{ psi} \quad < \quad F_s$$

[Satisfactory]

The compressive stress in the extreme fiber is

$$f_b = \frac{F_{anch} S^2}{4 b_w j k d^2} = \quad 132 \text{ psi} \quad < \quad F_b$$

[Satisfactory]

Technical References:

1. "National Design Specification, NDS", 2005 Edition, AF&AP, AWC, 2005.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.

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Drag / Collector Force Diagram Generator

INPUT DATA

TOTAL SHEAR FORCE (ASD) F_p = 15.16 kips

NUMBER OF SEGMENTS n = 5

Segment	1	2	3	4	5
Length, ft	17.5	22.67	13.83	35.5	10
Shear Wall ?	Yes	No	Yes	No	Yes

ANALYSIS

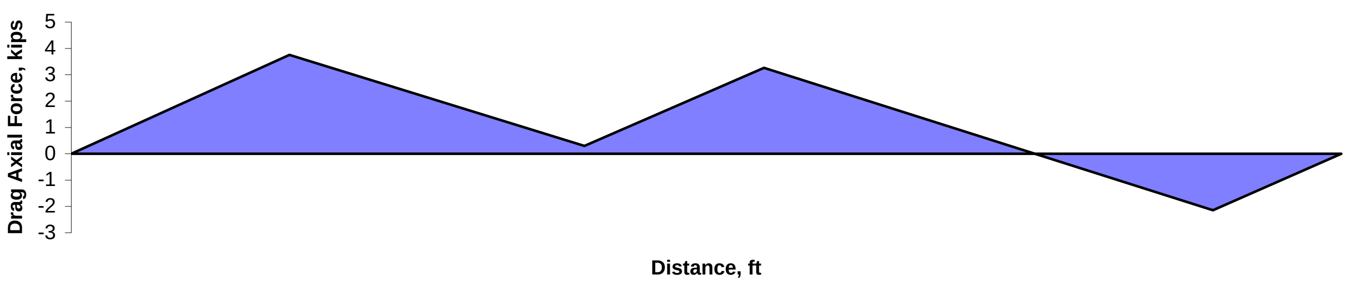
TOTAL DRAG LENGTH L_{drag} = 99.5 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 41.33 ft

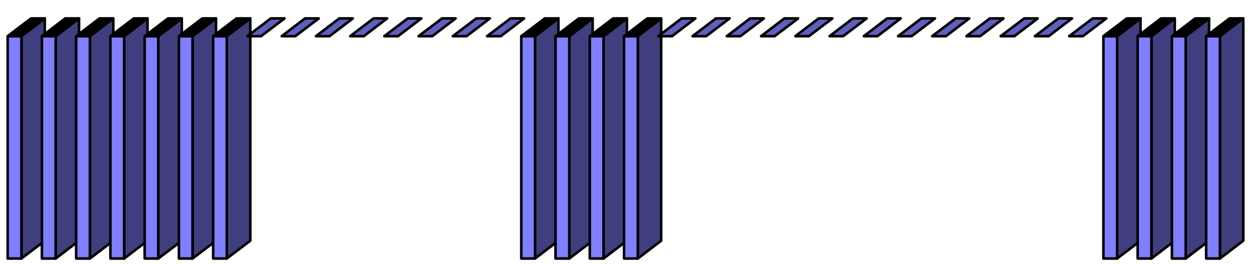
DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 152 plf

SHEAR WALL SHEAR STRESS $v_{shear\ wall} = F_p / v_{wall}$ = 367 plf

Section Point	0	1	2	3	4	5
Distance, ft	0	17.5	40.17	54	89.5	99.5
Axial Force	0	3.75	0.30	3.26	-2.14	0.00



DRAG / COLLECTOR FORCE DIAGRAM



SHEAR WALL & DRAG ELEVATION

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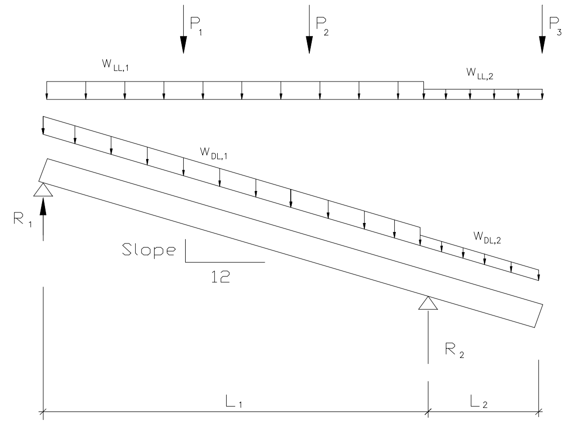
Wood Beam Design Based on NDS 2012

INPUT DATA & DESIGN SUMMARY



BEAM SECTION **LSL 1 3/4 x 14** Timberstrand LSL 2.1E

BEAM SPAN $L_1 =$ 12 ft

CANTILEVER $L_2 =$ 3 ft, (0 for no cantilever)

SLOPED DEAD LOADS $w_{DL,1} =$ 0.35 kips / ft

$w_{DL,2} =$ 0.25 kips / ft

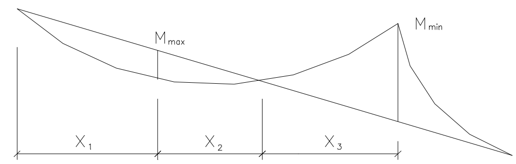
PROJECTED LIVE LOADS $w_{LL,1} =$ 0.14 kips / ft

$w_{LL,2} =$ 0.2 kips / ft

CONCENTRATED LOADS

	P_1	P_2	P_3
Dead (kips)	0.65	0.6	1.2
Live (kips)	0.26	0.4	1
Location from Left End (ft)	4	8	15

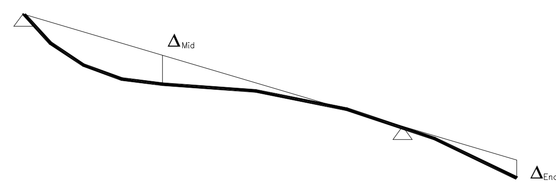
SLOPE 4 **: 12** (θ = 18.43 ^{0})



DEFLECTION LIMIT OF LIVE LOAD

$\Delta_{LL} = L /$ 360

Code	Duration Factor, C_D	Condition
1	0.90	Dead Load
2	1.00	Occupancy Live Load
3	1.15	Snow Load
4	1.25	Construction Load
5	1.60	Wind/Earthquake Load
6	2.00	Impact Load
Choice =>	4	Construction Load



THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE REACTIONS, MOMENTS & SHEARS

$$R_2 = 0.5\left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)L_1+\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)(L_1+0.5L_2)\frac{L_2}{L_1}+\sum P\frac{d}{L_1} =$$ 8.34 kips

$X_1 =$ 4.80 ft

$X_2 =$ 5.20 ft

$X_3 =$ 2.00 ft

$$R_1 = \left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)L_1+\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)L_2+\sum P-R_2 =$$ 3.27 kips

$M_{max} =$ 9.1 ft-kips

$V_{max} =$ 6.95 kips, at R2 left.

$$M_{Min} = 0.5\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)L_2^2+P_3L_2 =$$ 8.7 ft-kips

DETERMINE SECTION PROPERTIES AND DESIGN FACTORS

$L_u = Max(X_3\ ,\ L_2) =$ 3.0 ft, (NDS 2012 Table 3.3.3)

LSL 1 3/4 x 14 Properties

b =	1.75	in	F_b =	3,451	psi	l_E =	6.2	ft, (Tab 3.3.3 footnote 1)
d =	14.00	in	F_v =	400	psi, (NDS 97 C_H included)	R_B =	18.4	< 50
A =	24.5	in^2	E' =	2,100	ksi	E'_y =	2,100	ksi

(cont'd)

S_x	=	57.2	in^3	F_b'	=	3,302	psi	F_{bE} =	3841	psi	
I	=	400	in^4	F_v'	=	500	psi	F_b^* =	4313.39	psi	
$E = E_x$	=	2100	ksi	E'_{min}	=	1,085	ksi	$F = F_{bE} / F_b^*$ =		0.89	

C_D	C_M	C_t	C_i	C_L	C_F	C_V	C_c	C_r
1.25	1.00	1.00	1.00	0.77	1.00	1.00	1.00	1.00

CHECK BENDING AND SHEAR CAPACITIES

Cantilever: $f_b' = M_{Min} / S_x =$ 1823 psi < $F_b' =$ 3302 psi **[Satisfactory]**

Middle Span: $f_b = M_{Max} / S_x =$ 1911 psi < $F_b =$ 4313 psi **[Satisfactory]**

Shear: $f_v' = 1.5\, V_{Max} / A =$ 425 psi < F_v' **[Satisfactory]**
(neglected ***d*** offset conservatively)

CHECK DEFLECTION AT LIVE LOAD CONDITION

$L = L_1 / \cos\,\theta =$ 12.65 ft, beam sloped span

	P_{LL1}	P_{LL2}	P_{LL3}
a or b, (ft)	8.43	4.22	3.16
P, (k)	0.25	0.38	0.95

$w_1 = w_{LL,1}\cos^2\theta =$ 0.13 klf, perpendicular to beam

$w_2 = w_{LL,2}\cos^2\theta =$ 0.18 klf, perpendicular to beam

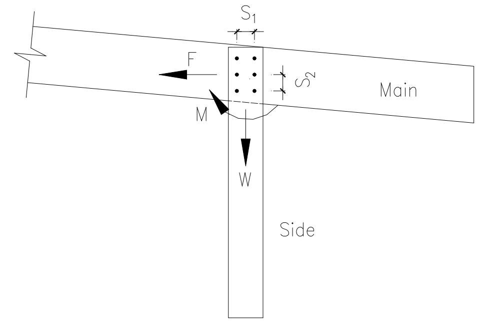
$$\Delta_{End} = \left[\frac{P_3 a_3^2 (L + a_3)}{3EI} + \frac{w_2 a_3^3 (4L + 3a_3)}{24EI}\right]\cos\theta =$$ 0.13 in, downward to vertical direction.

< $2\,L_2 / 360$ = 0.20 in **[Satisfactory]**

$$\Delta_{Mid} = \left[\sum \frac{0.06415 Pb}{EIL}\left(L^2 - b^2\right)^{1.5} + \frac{5 w_1 L^4}{384EI}\right]\cos\theta =$$ 0.13 in, downward to vertical direction.

< $L_1 / 360$ = 0.40 in **[Satisfactory]**

Lag Screw Connection Design Based on NDS 2005



INPUT DATA & DESIGN SUMMARY

LOADS (ASD) W = 3 lbs
F = 15 lbs
M = 14 ft-lbs

NUMBER OF LAG SCREWS n = 4
LAG SCREW DIAMETER ϕ = 1/4 in
LAG SCREW SPACING S_1 = 1 in, (if one row input 0)
S_2 = 1 in

MAIN MEMBER 2 thk. x 4 width
SIDE MEMBER 2 thk. x 4 width

LOAD DURATION FACTOR (Tab 2.3.2, NDS 2005, Page 8) C_D = 1.6
WET SERVICE FACTOR (Tab 10.3.3, NDS 2005, Page 58) C_M = 1.0
TEMPERATURE FACTOR (Tab 10.3.4, NDS 2005, Page 58) C_t = 1.0
GROUP ACTION FACTOR APPLY? (0 = No, 1 = Yes) 1 Yes, (C_g = 0.997 , NDS 2005, 10.3.6)

THE CONNECTION, (4) - 0.25 Dia. x 3", IS ADEQUATE.

ANALYSIS

CHECK EDGE, END, & SPACING DISTANCE REQUIREMENTS (NDS 2005, Table 11.5.1A, Table 11.5.1B, & Table 11.5.1C)

E_g = 1.25 in > 1.5 D [Satisfactory]
E_n = 1.25 in > 3.5 D [Satisfactory]
S = 1 in > 3 D [Satisfactory]

CHECK WOOD CAPACITY

$F = [(F_F + F_{MF})^2 + (F_W + F_{MW})^2]^{0.5}$ = 63 lbs / screw, actual load at corner screw

where F_F = 4 lbs / screw F_{MF} = 42 lbs / screw, at corner screw
F_W = 1 lbs / screw F_{MW} = 42 lbs / screw, at corner screw
J = 2 (in^2 Screw Area)

$C_\Delta = \text{Min}(C_{\Delta 1}, C_{\Delta 2}, C_{\Delta 3})$ = 0.714 , (geometry factor, NDS 2005, 11.5.1, page 76)

where $C_{\Delta 1}$ = (actual end distance) / (min end distance for full design value) = E_n / 7D = 0.714
$C_{\Delta 2}$ = (actual shear area) / (min shear area for full design value) = 1.000
$C_{\Delta 3}$ = (actual spacing) / (min spacing for full design value) = S / 4D = 1.000

$$C_g = \left[\frac{m(1-m^{2n})}{n\left[(1+R_{EA}m^n)(1+m)-1+m^{2n}\right]}\right]\left[\frac{1+R_{EA}}{1-m}\right] =$$ 0.997 , (group action factor, NDS 2005, 10.3.6, page 60)

where n = 4 $R_{EA} = \text{Min}[(E_sA_s/E_mA_m), (E_mA_m/E_sA_s)]$ = 1.000
E_sA_s = 7350000 lbs, (E_m fr Tab.10.3.6C) $\gamma = 180000\ D^{1.5}$ = 22500
t_m = 1 1/2 in $u = 1+\gamma\ S/2\ [1 / E_mA_m + 1 / E_sA_s]$ = 1.003
E_mA_m = 7350000 lbs, (E_m fr Tab.10.3.6C) $m = u - (u^2 - 1)^{0.5}$ = 0.925

C_d = P / (8 D) = 0.672 , (penetration depth factor, NDS 2005, tab 11J, footnote 3, page 92)

where P = S + (T - E) - t_s = 1 11/32 in, (length of thread penetration in main member)
S = 1 in, (NDS 2005 tab L2, page 166)
T - E = 1 27/32 in, (NDS 2005 tab L2, page 166)

$$Z_{s\theta} = \frac{Z_{//}Z_{s\perp}}{Z_{//}\sin^2\theta + Z_{s\perp}\cos^2\theta} =$$ 132 lbs / screw, side member allowable capacity (NDS 2005, 11.3.3, page 72)

where $Z_{//}$ = 150 lbs / screw, (NDS 2005, tab 11J, page 92)
$Z_{s\perp}$ = 120 lbs / screw, (NDS 2005, tab 11J, page 92)
θ = 46.94 ^{o} , angle between F & grain for side member at corner screw

$$Z_{m\theta} = \frac{Z_{//}Z_{m\perp}}{Z_{//}\sin^2\theta + Z_{m\perp}\cos^2\theta} =$$ 134 lbs / screw, main member allowable capacity (NDS 2005, 11.3.3, page 72)

where $Z_{//}$ = 150 lbs / screw, (NDS 2005, tab 11J, page 92)
$Z_{m\perp}$ = 120 lbs / screw, (NDS 2005, tab 11J, page 92)
θ = 43.06 ^{o} , angle between F & grain for main member at corner screw

$Z_\theta = \text{MIN}(Z_{//}, Z_\perp)\ C_D\ C_g\ C_d$ = 63 lbs / corner screw, connection allowable capacity
> F = 63 lbs / screw [Satisfactory]

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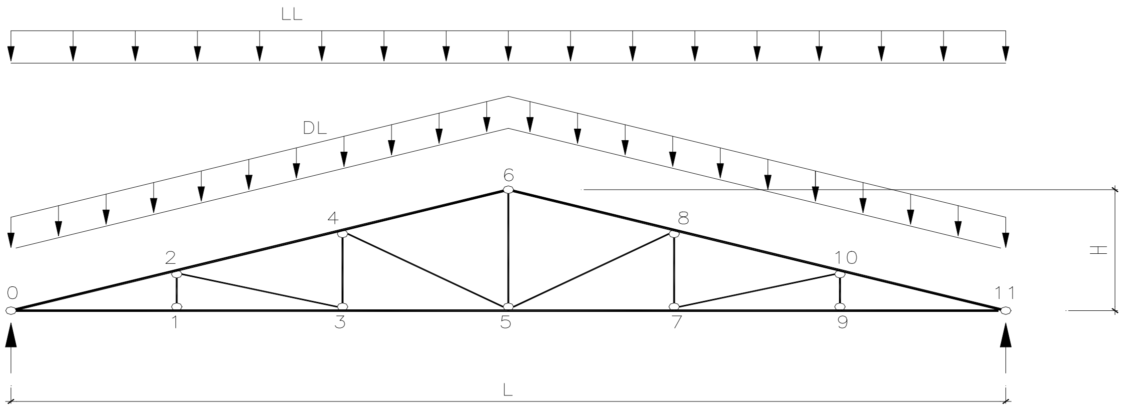
Wood Truss Design Based on NDS 2005

INPUT DATA & DESIGN SUMMARY



CONTINUED TOP CHORD (from 0 to 6 & from 6 to 11)	GLB 5 1/8 x 15	SLOPED DEAD LOADS	DL =	400	lbs / ft
CONTINUED BOTTOM CHORD (from 0 to 11)	GLB 5 1/8 x 10 1/2	PROJECTED LIVE LOADS	LL =	320	lbs / ft
ALL WEB MEMBER SIZE	6 x 6	SPECIE & GRADE		2	No. 1, Douglas Fir-Larch

TRUSS SPAN LENGTH L = 60 ft
TRUSS DEPTH H = 10 ft

THE TRUSS DESIGN IS ADEQUATE.

ANALYSIS

DESIGN CRITERIA

1. The intermediate finite element analysis include shear deformations using G equal to 0.5 E / (1+0.3).
2. There are different rotations at same joint of web to chord, since web pinned to continued chord.
3. The top chord factors, C_P & C_L , are based on two joints length. Other member factors, including C_V, are based on their physical length.

DETERMINE JOINT DEFLECTIONS BY FINITE ELEMENT ANALYSIS

	DL Deflection (in, deg)					LL Deflection (in, deg)				
Joint	X	Y	θ (cont.)	θ (pin-1)	θ (pin-2)	X	Y	θ (cont.)	θ (pin-1)	θ (pin-2)
0	0	0		-0.2602	-0.2914	0	0		-0.1975	-0.2211
1	-0.0690	-0.4614	-0.1391	-0.1237		-0.0523	-0.3502	-0.1056	-0.0939	
2	0.0174	-0.4618	-0.1234	-0.1237	-0.0548	0.0132	-0.3505	-0.0937	-0.0939	-0.0416
3	-0.0319	-0.5729	-0.0039	-0.0548	-0.0426	-0.0242	-0.4348	-0.0029	-0.0416	-0.0323
4	0.0276	-0.5703	-0.0086	-0.0426	-0.0054	0.0209	-0.4328	-0.0065	-0.0323	-0.0041
5	0	-0.5682	0	-0.0054	0.0054	0	-0.4312	5.56E-17	-0.0041	0.0041
6	0	-0.5457		0.0561	-0.0561	0	-0.4141		0.0426	-0.0426
7	0.0319	-0.5729	0.0039	0.0548	0.0426	0.0242	-0.4348	0.0029	0.0416	0.0323
8	-0.0276	-0.5703	0.0086	0.0426	0.0054	-0.0209	-0.4328	0.0065	0.0323	0.0041
9	0.0690	-0.4614	0.1391	0.1237		0.0523	-0.3502	0.1056	0.0939	
10	-0.0174	-0.4618	0.1234	0.1237	0.0548	-0.0132	-0.3505	0.0937	0.0939	0.0416
11	0	0		0.2602	0.2914	0	0		0.1975	0.2211

CHECK MAXIMUM LIVE LOAD DEFLECTION

D_{LL} = 0.43 in < L / 240 = 3.00 in **[Satisfactory]**

DETERMINE REQUIRED CAMBER AT 1.5 TIMES MAXIMUM DEAD LOAD DEFLECTION

1.5 D_{DL} = 0.86 in

DETERMINE MEMBER FORCES UNDER LOAD COMBINATION **(DL + LL)**

	Joints	I end section			J end section		
Member	I - J	N (kips)	V (kips)	M(ft-kips)	N (kips)	V (kips)	M(ft-kips)
1	0 - 1	-52.665	0.460	0.000	52.665	-0.460	4.595
2	0 - 2	56.853	4.017	0.000	-54.508	3.019	5.264
3	1 - 2	0.865	0.000	0.000	-0.865	0.000	0.000
4	1 - 3	-52.665	-0.406	-4.595	52.665	0.406	0.537
5	2 - 3	7.813	0.000	0.000	-7.813	0.000	0.000
6	2 - 4	48.531	2.490	-5.264	-46.186	4.546	-5.570
7	3 - 4	-2.784	0.000	0.000	2.784	0.000	0.000

REACTIONS:
$R_0 = R_{11}$ = 22.25 kips, upward

							(cont'd)
8	3 - 5	-45.253	-0.093	-0.537	45.253	0.093	-0.390
9	4 - 5	14.231	0.000	0.000	-14.231	0.000	0.000
10	4 - 6	36.569	4.046	5.570	-34.224	2.989	0.000
11	5 - 6	-15.973	0.000	0.000	15.973	0.000	0.000
12	5 - 7	-45.253	0.093	0.390	45.253	-0.093	0.537
13	5 - 8	14.231	0.000	0.000	-14.231	0.000	0.000
14	6 - 8	34.224	2.989	0.000	-36.569	4.046	-5.570
15	7 - 8	-2.784	0.000	0.000	2.784	0.000	0.000
16	7 - 9	-52.665	0.406	-0.537	52.665	-0.406	4.595
17	7 - 10	7.813	0.000	0.000	-7.813	0.000	0.000
18	8 - 10	46.186	4.546	5.570	-48.531	2.490	5.264
19	9 - 10	0.865	0.000	0.000	-0.865	0.000	0.000
20	9 - 11	-52.665	-0.460	-4.595	52.665	0.460	0.000
21	10 - 11	54.508	3.019	-5.264	-56.853	4.017	0.000

CHECK WOOD MEMBERS CAPACITIES

Member	Max. Section Force		
	N (kips)	V (kips)	M(ft-kips)
Top Chord	-56.853	4.546	5.570

C_D	C_F	C_P	C_L	C_V	
1.25	1.00	0.51	0.97	0.94	
f_b	F'_b	f_c	F'_c	f_v	F'_v
348	2816	740	1015	89	331

[Satisfactory]

Member	Max. Section Force		
	N (kips)	V (kips)	M(ft-kips)
Bot Chord	52.665	0.460	4.595

C_D	C_F		C_L	C_V	
1.25	1.00		1.00	0.91	
f_b	F'_b	f_t	F'_t	f_v	F'_v
586	2737	979	1375	13	331

[Satisfactory]

Compression Web Member	Max. Section Force		
	N (kips)	V (kips)	M(ft-kips)
9 & 13	-14.231		

C_D	C_F	C_P
1.25	1.00	0.50
f_c	F'_c	
470	578	

[Satisfactory]

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Tables for Wood Post Design Based on NDS 2005

DURATION FACTOR (1.0, 1.15, 1.25, 1.6) C_D = **1.00** , (NDS 2.3.2)
COMMERCIAL GRADE (# 1 or # 2) # **1**

Post Axial Capacity for Douglas Fir-Larch # 1, (kips)

Height	Section Size										
(ft)	4 x 4	4 x 6	4 x 8	4 x 10	4 x 12	6 x 6	6 x 8	6 x 10	6 x 12	8 x 8	8 x 10
6	***11.78***	***18.25***	***23.68***	***29.67***	***36.09***	***27.56***	***37.59***	***47.61***	***57.64***	***53.81***	***68.16***
7	***9.35***	***14.57***	***19.01***	***23.97***	***29.15***	***26.34***	***35.92***	***45.50***	***55.07***	***52.79***	***66.87***
8	7.48	11.68	15.29	19.36	23.55	***24.80***	***33.81***	***42.83***	***51.84***	***51.52***	***65.26***
9	6.07	9.49	12.45					***39.65***	***48.00***	***49.96***	***63.28***
10	5.00	7.83	10.29					***36.13***	***43.74***	***48.09***	***60.91***
11	4.18	6.56	8.62					32.52	39.37	***45.89***	***58.13***
12	3.55	5.56	7.31					29.06	35.18	***43.41***	***54.98***
13	3.04	4.77	6.28					25.88	31.33	***40.69***	***51.55***
14	2.63	4.13	5.44					23.06	27.91	***37.86***	***47.96***
15	2.30	3.62	4.76	6.07	7.38	11.92	16.25	20.58	24.92	35.01	44.35
16	2.03	3.19	4.20	5.35	6.51	10.67	14.55	18.43	22.31	32.25	40.85
17	1.80	2.83	3.73	4.75	5.78	9.59	13.08	16.57	20.06	29.64	37.55
18	1.61	2.53	3.33	4.25	5.17	8.66	11.80	14.95	18.10	27.23	34.49
19	1.45	2.28	3.00	3.82	4.65	7.84	10.70	13.55	16.40	25.01	31.68
20	1.31	2.06	2.71	3.46	4.20	7.13	9.73	12.32	14.92	23.01	29.14
21	1.19	1.87	2.46	3.14	3.82	6.51	8.88	11.25	13.62	21.19	26.84
22	1.09	1.70	2.25	2.86	3.48	5.97	8.14	10.31	12.48	19.56	24.77
23	0.99	1.56	2.06	2.62	3.19	5.48	7.48	9.47	11.47	18.09	22.91
24	0.91	1.44	1.89	2.41	2.93	5.06	6.90	8.73	10.57	16.76	21.23
25	0.84	1.32	1.74	2.22	2.71	4.68	6.38	8.08	9.78	15.57	19.72
26	0.78	1.22	1.61	2.06	2.50	4.34	5.91	7.49	9.07	14.49	18.36
27	0.72	1.14	1.50	1.91	2.32	4.03	5.50	6.96	8.43	13.52	17.12
28	0.67	1.06	1.39	1.78	2.16	3.76	5.12	6.49	7.86	12.63	16.00
29	0.63	0.99	1.30	1.66	2.02	3.51	4.79	6.06	7.34	11.83	14.98
30	0.59	0.92	1.21	1.55	1.88	3.29	4.48	5.68	6.87	11.09	14.05

Post Axial Capacity for Southern Pine # 1, (kips)

Height	Section Size										
(ft)	4 x 4	4 x 6	4 x 8	4 x 10	4 x 12	6 x 6	6 x 8	6 x 10	6 x 12	8 x 8	8 x 10
6	***12.03***	***18.60***	***24.06***	***30.38***	***36.94***	***23.08***	***31.47***	***39.86***	***48.25***	***44.68***	***56.59***
7	***9.48***	***14.74***	***19.20***	***24.33***	***29.59***	***22.24***	***30.33***	***38.41***	***46.50***	***43.97***	***55.70***
8	7.55	11.77	15.40	19.56	23.79	***21.17***	***28.87***	***36.57***	***44.27***	***43.09***	***54.59***
9	6.11	9.55	12.51	15.92	19.36	***19.88***	***27.11***	***34.34***	***41.57***	***42.02***	***53.23***
10	5.02	7.87	10.33	13.14	15.98	18.40	25.09	31.78	38.47	***40.74***	***51.61***
11	4.20	6.58	8.64	11.01					35.14	***39.23***	***49.69***
12	3.56	5.57	7.33	9.34					31.80	***37.48***	***47.48***
13	3.05	4.78	6.29	8.01					28.60	***35.54***	***45.02***
14	2.64	4.14	5.45	6.95					25.67	33.44	42.36
15	2.31	3.62	4.77	6.08					23.05	31.26	39.59
16	2.03	3.19	4.20	5.36					20.73	29.06	36.81
17	1.81	2.84	3.73	4.76					18.70	26.93	34.11
18	1.61	2.53	3.34	4.26					16.92	24.90	31.54
19	1.45	2.28	3.00	3.83					15.36	23.00	29.13
20	1.31	2.06	2.71	3.46					13.99	21.25	26.91
21	1.19	1.87	2.46	3.14					12.79	19.64	24.88
22	1.09	1.71	2.25	2.87	3.49	5.61	7.65	9.69	11.73	18.18	23.03
23	0.99	1.56	2.06	2.63	3.19	5.16	7.04	8.92	10.79	16.86	21.35
24	0.91	1.44	1.89	2.41	2.94	4.76	6.50	8.23	9.96	15.65	19.83
25	0.84	1.32	1.75	2.23	2.71	4.41	6.01	7.61	9.22	14.56	18.45
26	0.78	1.23	1.61	2.06	2.50	4.09	5.58	7.06	8.55	13.57	17.19
27	0.72	1.14	1.50	1.91	2.32	3.80	5.19	6.57	7.95	12.67	16.05
28	0.67	1.06	1.39	1.78	2.16	3.55	4.84	6.13	7.42	11.86	15.02
29	0.63	0.99	1.30	1.66	2.02	3.31	4.52	5.73	6.93	11.11	14.08
30	0.59	0.92	1.22	1.55	1.89	3.10	4.23	5.36	6.49	10.43	13.21

Note:

1. The ***bold values*** require steel bearing plate based on, $F_{c\perp}$, 625 psi.
2. The table values are from Wood Column software at ***www.Engineering-International.com*** .

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Tables for Wood Beam Design Based on NDS 2005

DURATION FACTOR (0.9, 1.0, 1.15, 1.25)	C_D =	1.15	, (NDS 2.3.2)
COMMERCIAL GRADE (# 1 or # 2)	#	1	
DEFLECTION LIMITATION	Δ = L /	240	

Beam Allowable Uniform Load for Douglas Fir-Larch # 1, (plf)

Span	Section Size												
(ft)	2 x 6	2 x 8	2 x 10	2 x 12	4 x 6	4 x 8	4 x 10	4 x 12	6 x 8	6 x 10	6 x 12	GLB 5 1/8 x 15	GLB 5 1/8 x 18
6	209	336	501	674	489	849	1276	1729	1482	2378	3485	9823	14145
7	154	247	368	495	359	624	937	1271	1089	1747	2561	7217	10392
8	118	189	282	379	275	477	717	973	834	1338	1961	5525	7957
9	93	149	223	299	217	377	567	769	659	1057	1549	4366	6287
10	75	121	180	243	176	306	459	623	534	856	1255	3536	5092
11	***59***	100	149	200	***138***	253	379	515	441	708	1037	2923	4208
12	***45***	84	125	168	***106***	212	319	432	371	595	871	2456	3536
13	***36***	72	107				272	368	***313***	507	742	2092	3013
14	***29***	62	92				234	318	***251***	437	640	1804	2598
15	***23***	***53***	80				204	277	***204***	381	558	1572	2248
16	***19***	***44***	70				179	243	***168***	334	490	1381	1963
17	***16***	***37***	62				159	215	***140***	***284***	434	***1174***	1728
18	***13***	***31***	56				142	192	***118***	***240***	387	***989***	1533
19	***11***	***26***	50	67	***27***	***61***	***127***	172	***100***	***204***	348	***841***	1368
20	***10***	***22***	45	61	***23***	***52***	***109***	156	***86***	***175***	***310***	***721***	1228
21	***8***	***19***	***40***	55	***20***	***45***	***94***	141	***74***	***151***	***268***	***623***	***1076***
22	***7***	***17***	***35***	50	***17***	***39***	***82***	129	***65***	***131***	***233***	***541***	***936***
23	***6***	***15***	***31***	46	***15***	***35***	***72***	118	***57***	***115***	***204***	***474***	***819***
24	***6***	***13***	***27***	42	***13***	***30***	***63***	108	***50***	***101***	***179***	***417***	***721***
25	***5***	***12***	***24***	39	***12***	***27***	***56***	100	***44***	***89***	***159***	***369***	***638***
26	***4***	***10***	***21***	36	***10***	***24***	***50***	***89***	***39***	***79***	***141***	***328***	***567***
27	***4***	***9***	***19***	33	***9***	***21***	***44***	***80***	***35***	***71***	***126***	***293***	***506***
28	***4***	***8***	***17***	***31***	***8***	***19***	***40***	***71***	***31***	***64***	***113***	***263***	***454***
29	***3***	***7***	***15***	***28***	***8***	***17***	***36***	***64***	***28***	***57***	***102***	***236***	***409***
30	***3***	***7***	***14***	***25***	***7***	***16***	***32***	***58***	***25***	***52***	***92***	***214***	***369***

Beam Allowable Uniform Load for Southern Pine # 1, (plf)

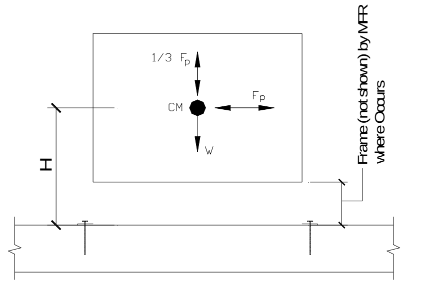
Span	Section Size												
(ft)	2 x 6	2 x 8	2 x 10	2 x 12	4 x 6	4 x 8	4 x 10	4 x 12	6 x 8	6 x 10	6 x 12	GLB 5 1/8 x 15	GLB 5 1/8 x 18
6	266	420	592	842	620	1077	1520	2162	1812	2907	4260	9823	14145
7	195	308	435	619	456	792	1117	1588	1331	2136	3130	7217	10392
8	149	236	333	474	349	606	855	1216	1019	1635	2396	5525	7957
9	***108***	187	263	374	***251***	479	676	961	805	1292	1893	4366	6287
10	***79***	151	213	303	***183***	388	547	778	652	1047	1534	3536	5092
11	***59***	125	176	251	***138***	***315***	452	643	539	865	1267	2923	4208
12	***45***	***104***	148	211	***106***	***243***	380	540	***423***	727	1065	2456	3536
13	***36***	***82***	126	179	***83***	***191***	324	461	***332***	619	907	2092	3013
14	***29***	***66***	109	155	***67***	***153***	279	397	***266***	534	782	1804	2598
15	***23***	***53***	95	135	***54***	***124***	243	346	***216***	***440***	682	1572	2255
16	***19***	***44***	83	118	***45***	***103***	***213***	304	***178***	***362***	599	1381	1976
17	***16***	***37***	74	105	***37***	***85***					531	***1174***	1745
18	***13***	***31***	***64***	94	***31***	***72***					***452***	***989***	1552
19	***11***	***26***	***54***	84	***27***	***61***					***384***	***841***	1389
20	***10***	***22***	***47***	76	***23***	***52***					***329***	***721***	***1245***
21	***8***	***19***	***40***	69	***20***	***45***					***284***	***623***	***1076***
22	***7***	***17***	***35***	63	***17***	***39***					***247***	***541***	***936***
23	***6***	***15***	***31***	***55***	***15***	***35***					***216***	***474***	***819***
24	***6***	***13***	***27***	***49***	***13***	***30***	***63***	***113***	***53***	***107***	***190***	***417***	***721***
25	***5***	***12***	***24***	***43***	***12***	***27***	***56***	***100***	***47***	***95***	***169***	***369***	***638***
26	***4***	***10***	***21***	***38***	***10***	***24***	***50***	***89***	***42***	***84***	***150***	***328***	***567***
27	***4***	***9***	***19***	***34***	***9***	***21***	***44***	***80***	***37***	***75***	***134***	***293***	***506***
28	***4***	***8***	***17***	***31***	***8***	***19***	***40***	***71***	***33***	***68***	***120***	***263***	***454***
29	***3***	***7***	***15***	***28***	***8***	***17***	***36***	***64***	***30***	***61***	***108***	***236***	***409***
30	***3***	***7***	***14***	***25***	***7***	***16***	***32***	***58***	***27***	***55***	***98***	***214***	***369***

Note:
1. The ***bold values*** are deflection controlled.
2. Glulam 24F-1.8E used.
3. The beam continuously lateral supported by top diaphragm. (C_L = 1.0)
4. The table values are from Wood Beam software at ***www.Engineering-International.com*** .

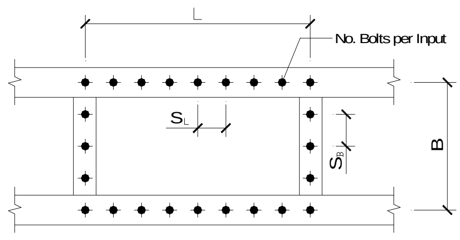
Equipment Anchorage to Wood Roof Based on NDS 05 / IBC 09 / CBC 10

INPUT DATA & DESIGN SUMMARY

LAG SCREW DIAMETER	ϕ	=	1/2	in, NDS APP. L
LAG SCREW LENGTH	Long	=	4	in, NDS APP. L
EQUIPMENT WEIGHT	W	=	1.4	kips
CM HEIGHT	H	=	8.25	ft, 2/3 total height
ANCHORAGE LENGTH	L	=	8	ft
LAG SCREWS ALONG ***L*** EDGE	N_L	=	3	per line
LAG SCREW SPACING	$S_L = L / (N_L - 1) =$		48	in
ANCHORAGE WIDTH	B	=	8	ft
LAG SCREWS ALONG ***B*** EDGE	N_B	=	2	per line
LAG SCREW SPACING	$S_B = B / (N_B - 1) =$		96	in



ELEVATION



PLAN

[THE ANCHORAGE, USING 1/2" x 4" LAG SCREWS, IS ADEQUATE.]

ANALYSIS

ALLOWABLE TENSION & SHEAR VALUES

$Z_{II}' = C_D\, C_d\, C_g\, Z_{II} =$ 252 lbs

where C_D = 1.6 (NDS Tab 2.3.2)

$C_d = p / (8D) =$ 0.814

$p = S + (T\text{-}E) - (\text{Diaphragm Thk}) - t_s =$ 3.256 in

$S + (T\text{-}E) =$ 3.688 in (NDS Appendix L2)

Diaphragm Thk = 0.298 in (0.298 for 15/32, 0.319 for 19/32, UBC Table 23-2-H, page 3-420)

$t_s =$ 0.134 in (10 gage, NDS Tab 11K)

$C_g =$ 0.92 (NDS 10.3.6)

$Z_{II} =$ 210 lbs (NDS Tab 11K)

$Z_\perp' = C_D\, C_d\, C_g\, Z_\perp =$ 168 lbs

$Z_\perp =$ 140 lbs (NDS Tab 11K)

$(W'p) = C_D\, W\, (T\text{-}E) =$ 1323 lbs

where W = 378 lbs / in (NDS Tab 11.2A)

(T-E) = 2.188 in (NDS Appendix L2)

DESIGN LOADS

$F_H = F_p =$ (K_H) MAX{ $0.3S_{DS}I_pW$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p\, W$, $1.6S_{DS}I_pW$] } , (ASCE 7-05, Sec. 13.3.1)

= 1.3 MAX{ 0.24W , MIN[0.65W , 1.30W]}

= 0.84 W , (SD)

= 0.60 W , (ASD) = 0.84 kips

$F_V = K_V\, W =$ 0.10 W , (ASD) = 0.14 kips, up & down

where

$S_{DS} =$	0.54	(ASCE 7-05 Sec 11.4.4)
$I_p =$	1.5	(ASCE Sec. 13.1.3)
$a_p =$	1	(ASCE Tab. 13.6-1)
$R_p =$	1.5	(ASCE Tab. 13.6-1)
z =	h	ft
h =	36	ft
$K_H =$	1.3	(ASCE Sec. 13.4.2a)
$K_V = K_H\, 0.2\, S_{DS} / 1.4 =$	0.10	(vertical seismic factor)

MAXIMUM OVERTURNING MOMENT AT ANCHOR EDGE

$M_{OT} = Fp\, H =$ 6.95 ft-kips

$M_{RES,L} = (0.9W - Fv)(0.5L) =$ 4.48 ft-kips < M_{OT} ,therefore design tension anchors.

$M_{RES,B} = (0.9W - Fv)(0.5B) =$ 4.48 ft-kips < M_{OT} ,therefore design tension anchors.

TENSION LOAD AT CORNER LAG SCREW

$T_L = (F_V - 0.9\, W) / A + M_{OT}\, y / I =$ 248 lbs / bolt

where $A = 2(N_L + N_B) - 4 =$ 6 (total bolts)

$I = \Sigma X_i^2 =$ 9216 in^2-bolts

$y = 0.5\, L =$ 48 in

$T_B = (F_V - 0.9\, W) / A + M_{OT}\, y / I =$ 103 lbs / bolt

where $I = \Sigma Y_i^2 =$ 13824 in^2-bolts

$y = 0.5\, B =$ 48 in

SHEAR LOAD AT EACH LAG SCREW

$V = F_H / A =$ 140 lbs / bolt

CHECK CORNER SCREW CAPACITY AT COMBINED LATERAL AND WITHDRAWAL LOADS

$Z_\alpha' = Z_{II}'(W'p) / (Z_{II}'\sin^2\alpha + W'p\cos^2\alpha)$ (NDS 11.4-1)

= 650 lbs / bolt > $[T_L^2 + V^2]^{0.5} =$ 285 lbs / bolt **[SATISFACTORY]**

where $\alpha = \tan^{-1}(T_L / V) =$ 60.46 ^{o}

$Z_\theta' = Z_\perp'(W'p) / (Z_\perp'\sin^2\alpha + W'p\cos^2\alpha)$ (NDS 11.4-1)

= 241 lbs / bolt > $[T_B^2 + V^2]^{0.5} =$ 174 lbs / bolt **[SATISFACTORY]**

where $\theta = \tan^{-1}(T_B / V) =$ 36.26 ^{o}

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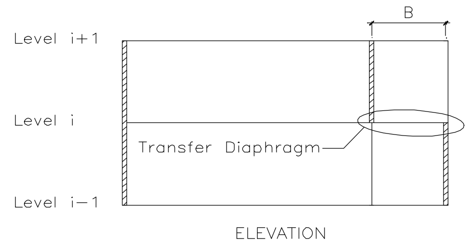
Wood Diaphragm Design for a Discontinuity of Type 4 out-of-plane offset irregularity

INPUT DATA

OFFSET LATERAL SYSTEM DIMENSION **B =** 12 ft
MAX TWO PERPENDICULAR LATERAL SYSTEM SPAN **L =** 32 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor
MINIMUM NOMINAL FRAMING WITH (2 or 3) = 3 in
MINIMUM NOMINAL PANEL THICKNESS = **15/32** in
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 1 8d
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.43
FRAMING OF ***Douglas-Fir-Larch*** OR ***Southern Pine*** ? Yes



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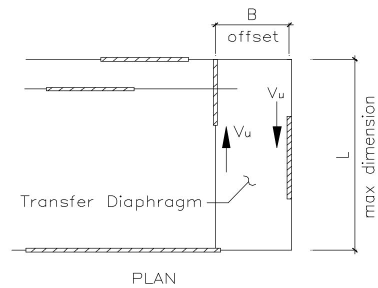
ALLOWABLE SHEAR FORCE ON TRANSFER DIAPHRAGM SIDE V = 10 kips
MAX. ALLOWABLE COLLECTOR (DRAG) FORCE T_{drag} = 11 kips
MAX. ALLOWABLE CHORD FORCE T_{chord} = 3 kips



THE TRANSFER DIAPHRAGM DESIGN IS ADEQUATE.

BLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 2.5 in O.C. BOUNDARY / 4 in O.C. EDGES / 12"O.C. FIELD.
(USE 2-CMST12 COLLECTOR/CHORD SIMPSON STRAP AROUND THE DIAPHRAGM.)

ANALYSIS

CHECK MAX DIAPHRAGM DIMENSION RATIO (SDPWS-08, 4.2.4 & IBC 09, 2308.12.6)

L / B = 2.67 **>** 2.00
< 4.00
[Satisfactory]

CHECK DIAPHRAGM SHEAR CAPACITY (ACI 318-08, 21.11.9)

$v = 1.25\ \rho\ V / L =$ 508 plf, ASD, (ASCE 7-05, 12.3.3.4 & Table 12.3-2.4)
where ρ = 1.3 (ASCE 7-05, 12.10.1.1 & 12.3.4.2)

[Satisfactory]

THE SHEAR CAPACITIES PER IBC Table 2306.2.1 / SDPWS-08 Table 4.2A with ASD reduction factor 2.0 :

Panel Grade	Common Nail	Min. Penetratio (in)	Min. Thickness (in)	Member Width (in)	Blocked Nail Spacing Boundary / Other Edges				Unblocked	
					6 / 6	4 / 6	2.5 / 4	2 / 3	Case 1	Others
Sheathing and Single-Floor	8d	1 1/2	15/32	3	300	400	600	675	265	200

Note: The indicated shear numbers have reduced by specific gravity factor per SDPWS-08 Table 4.2A note 2.

CHECK DIAPHRAGM IN-PLANE FLEXURAL CAPACITY

$M = 1.25\ \rho\ V\ B =$ 195.0 ft-kips, the max value same at both B & L sections, (ASCE 7-05, 12.3.3.4 & Table 12.3-2.4)

$T_{max} = M / MIN(L , B) =$ 16.3 kips (THE 2-CMST12 SIMPSON STRAP REQUIRED.)

[Satisfactory]

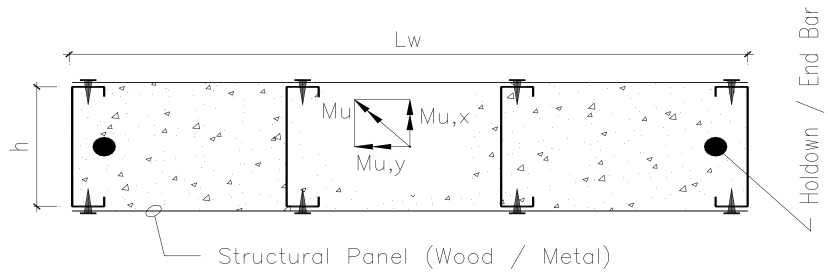
CHECK COLLECTOR / CHORD STRAP CAPACITY

$T = MAX(1.25\ \rho\ T_{chord}\ , 1.25\ \rho\ T_{drag}) =$ 17.9 kips, (ASCE 7-05, 12.3.4, 12.10.1.1 & 12.10.2.1 Exception)
(THE 2-CMST12 SIMPSON STRAP REQUIRED.) **[Satisfactory]**

Composite Strong Wall Design Based on ACI 318-08, AISI S100-2007 & ER-4943P

DESIGN CRITERIA

1. At middle of wall, with out of plan loads (seismic or wind), the inside web of C-Stud within concrete and End Bar are used as tension steel, conservatively.
2. The bottom joint, which shall be clean & rough, is pinned connection out of plan ($M_{u,y}$ zero), so only Holdown as shear wall tension steel used.
3. The Holdown / End Bar shall have full tension capacity with L_d, L_{dh}, & L_s (ACI 318-08 12 & App. D). The two End Bars shall be tied by horizontal strap (not shown), in the wall middle, at 4'-0" o.c. max.
4. The wall, with high stiffness (small drift), can be partially used as shear wall to reach flexible diaphragm condition (ASCE 7-10 Figure 12.3-1).
5. The wall has higher fire-resistance ratings (2010 CBC Table 706.4) and capacity, than pure wood / metal wall, without Type V construction limits.



INPUT DATA & DESIGN SUMMARY

C-STUD SECTION & SPACING **600S162-54** @ **18** in o.c
(50 ksi)

WALL LENGTH L_w = **3** ft

THE DESIGN IS ADEQUATE.

FACTORED SAME HORIZONTAL SECTION LOADS P_u = **120** kips, axial
(SD Level, equal to 1.5 ASD Level Force) $M_{u,x}$ = **150** ft-k, shear wall bending
$M_{u,y}$ = **50** ft-k, out of plan bending
$V_{u,x}$ = **17** kips, max horizontal shear load

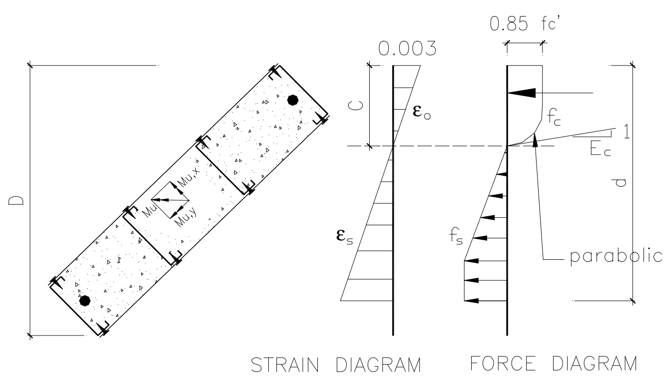
CONCRETE STRENGTH f_c' = **3** ksi
HOLDOWN / END BAR Diameter = **0.875** in, with F_y = **60** ksi
(Total SD level tension capacity at each end, 36.08 kips, # 7 Reinforcement)

ANALYSIS

C-STUD SECTION PROPERTIES OF EACH METAL STUD (SSMA page 7 & 8)

t = 0.0566 in F_y = 50 ksi h = 6 in

CHECK FLEXURAL & AXIAL CAPACITY



$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c} \;,\; E_c = 57\sqrt{f'_c} \;,\; E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] \;,\; for \;\; 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c \;,\; for \;\; \varepsilon_c \ge \varepsilon_o \end{cases}$$

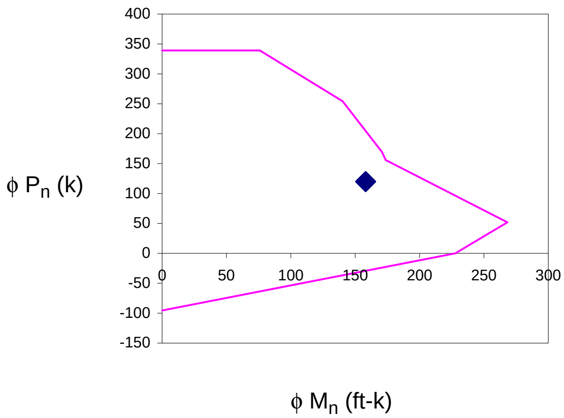
$$f_s = \begin{cases} \varepsilon_s E_s \;,\; for \;\; \varepsilon_s \le \varepsilon_t \\ f_y \;,\; for \;\; \varepsilon_s > \varepsilon_t \end{cases}$$

(cont'd)

$\phi P_{max} = 0.8\ \phi\,[\,0.85 f_c' (A_g - A_{st}) + f_y A_{st}] =$ 339.10 kips

(at max axial load, ACI 318-08 10.3.6.2)

	>	P_u	**[Satisfactory]**
where	$\phi =$	0.65	(ACI 318-08 9.3.2.2)
	$A_g =$	216	in^2
	$A_{st,C} =$	0.68	in^2, C-Stud
	$A_{st,H} =$	1.20	in^2, Holdown / End Bar



$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3) =$ 0.656

(ACI 318-08 Fig. R9.3.2)

where	D =	36.0	in
	Cover =	2.0	in, (ACI 318-08 7.7.1)
	d =	32.8	in
	$\varepsilon_c =$	0.003	
	$\varepsilon_t =$	0.0021	
	C =	19.4	in, at balanced condition

$\phi M_n =$ 206 ft-kips @ $P_u =$ 120 kips > $M_u =$ 158 ft-kips **[Satisfactory]**

ρ_{max}	=	0.025	(ACI 318-08 21.5.2.1)	>	ρ_{provd}	=	0.003	,bending one End Bar
ρ_{min}	=	0.0015	(ACI 318-08 14.2)	<	ρ_{provd}	=	0.009	,total steel

[Satisfactory]

CHECK SHEAR CAPACITY (ACI 318-08 11.1 & 11.2)

$\phi V_{n,x} = \phi\,[2\,(f_c')^{0.5} A_g] =$ 17.75 kips > $V_{u,x}$ **[Satisfactory]**

where ϕ = 0.75 , (ACI 318-08 9.3.2.3)

DETERMINE SHEAR WALL STORY DRIFT

$$\Delta_u = \frac{M_{u,x} H^2}{E_c I_x} + \frac{V_{u,x} H^3}{3 E_c I_x} = \quad 1.07 \text{ in, SD level}$$

where	$E_c =$	3122	ksi
	$I_x =$	23328	in^4
	H =	14	ft, Story Height

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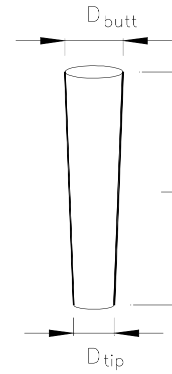
Wood Pole or Pile Design Based on NDS 2012



INPUT DATA & DESIGN SUMMARY

ROUND TIMBER SPECIES **Southern Pine (Grouped)**

F_b	F_v	F_c	E	E_{min}	
1950	160	1250	1500000	600000	(psi, NDS Supplement Table 6)

LENGTH L = **50** ft, (ASTM D25 or D3200)

DIAMETER D_{butt} = **16** in, (ASTM D25 or D3200)

D_{tip} = **8** in, (ASTM D25 or D3200)

AXIAL COMPRESSION FORCE P = **20** kips, ASD

CRITICAL/MAXIMUM BENDING MOMENT M = **17** ft-kips, ASD

The Moment Location from Tip End = **40** ft, (input 0, at tip end, for conservative)

D, (in)	A, (in^2)	I, (in^4)	S, (in^3)
14.4	162.9	2110.7	293.1

CRITICAL/MAXIMUM SECTION SHEAR FORCE V = **8** kips, ASD

The Section Location from Tip End = **0** ft, (input 0, at tip end, for conservative)

D, (in)	A, (in^2)
8.0	50.3

THE POLE/PILE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMPRESSION CAPACITY (NDS Table 6.3.1)

$F_c' = F_c\, C_D\, C_t\, C_{ct}\, C_p\, C_{es}\, C_{ls}$ = 193 psi

> $f_c = P / A$ = 123 psi **[Satisfactory]**

Where

C_D	C_t	C_{ct}	C_p	C_{es}	C_{ls}
0.90	**1.0**	**1.00**	0.14	1.16	**1.06**
(Tab. 2.3.2)	(Tab. 2.3.3)	(Tab. 6.3.5)		(Eq. 6.3-1)	(Tab. 6.3.5)

$C_P = (1+F) / 2c - [((1+F) / 2c)^2 - F / c]^{0.5}$,(NDS 3.7.1)

$F_c^* = F_c' / C_p$ = 1383 psi

$L_e = K_e\, L$ = **1.2** L = 720 in`

$F_{cE} = 0.822\, E'_{min} / (L_e /D)^2$ = 197 psi

E'_{min} = 600000 psi, (NDS Table 6.3.1)

F = F_{cE} / F_c^* = 0.143

c = 0.85

L_e / D = 50.0 < 50 , (NDS 3.7.1.4) **[Satisfactory]**

CHECK BENDING CAPACITY (NDS Table 6.3.1)

$F_b' = F_b\, C_D\, C_t\, C_{ct}\, C_F\, C_{ls}$ = 3253 psi

> $f_b = M / S$ = 696 psi **[Satisfactory]**

Where

C_D	C_t	C_{ct}	C_F	C_{ls}
1.60	**1.0**	**1.00**	0.99	**1.05**
(Tab. 2.3.2)	(Tab. 2.3.3)	(Tab. 6.3.5)	(NDS 6.3.7)	(Tab. 6.3.5)

CHECK BENDING & COMPRESSION CAPACITY (NDS 3.9.2)

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1 - f_c / F_{cE}}\right)\frac{f_b}{F_b'} = \quad 0.97 \quad < \quad 1.0$$ **[Satisfactory]**

$$\frac{f_c}{F'_{cE}} + \left(\frac{f_b}{F'_{bE}}\right)^2 = \quad 0.67 \quad < \quad 1.0$$ **[Satisfactory]**

Where $R_B = (L_e / D)^{0.5}$ = 7.07 , (NDS 3.3.3.6)

$F_{bE} = 1.20\, E'_{min} / R_B^2$ = 14400 psi, (NDS 3.9.2)

CHECK SHEAR CAPACITY (NDS Table 6.3.1)

$F_v' = F_v\, C_D\, C_t\, C_{ct}$ = 256 psi

> $f_v = 4\,V / (3\,A)$ = 212 psi **[Satisfactory]**

Where

C_D	C_t	C_{ct}
1.60	**1.0**	**1.00**
(Tab. 2.3.2)	(Tab. 2.3.3)	(Tab. 6.3.5)

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Wood Member (Beam, Column, Brace, Truss Web & Chord) Design Based on NDS 2012

INPUT DATA & DESIGN SUMMARY

SPECIES & GRADE Glulam 24F-1.8E
LENGTH $L =$ 15 ft
SECTION SIZE GLB 3 1/8 x 6

THE MEMBER DESIGN IS ADEQUATE.

AXIAL COMPRESSION FORCE $P =$ 10 kips, ASD
STRONG AXIS EFFECTIVE LENGTH $k_e L_x =$ 15 ft, (NDS Table G1/Figure 3F)
WEAK AXIS EFFECTIVE LENGTH $k_e L_y =$ 7.5 ft, (NDS Table G1)

STRONG AXIS BENDING MOMENT $M_x =$ 0.4 ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH $L_e =$ 8 ft, (NDS Table 3.3.3)
WEAK AXIS BENDING MOMENT $M_y =$ 0.1 ft-kips, ASD
CRITICAL/MAXIMUM SECTION SHEAR FORCE $V =$ 2.5 kips, ASD

ANALYSIS

DETERMINE DESIGN VALUES (NDS Supplement Tables or ICC ESR)

	F_b	F_v	F_c	$E_{x,min}$	$E_{y,min}$	b, (in)	d, (in)	A, (in^2)	I_x, (in^4)	S_x, (in^3)	I_y, (in^4)	S_y, (in^3)
psi ==>	2,400	265	1600	930000	830000	3.1	6.0	18.8	56.3	18.8	15.3	9.8

CHECK COMPRESSION CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_c' = F_c\, C_D\, C_M\, C_t\, C_F\, C_i\, C_p =$ 763 psi

> $f_c = P / A =$ 533 psi **[Satisfactory]**

Where

C_D	C_M	C_t	C_F	C_i	C_p	C_T
0.90	1.00	1.0	1.00	1.0	0.53	1.0
(Tab. 2.3.2)	(4.4.3/5.5.3)	(Tab. 2.3.3)	(NDS 4.3.6)	(NDS 4.3.8)		(NDS 4.4.2 for Truss only)

$C_P = (1+F) / 2c - [((1+F) / 2c)^2 - F / c]^{0.5}$,(NDS 3.7.1)

$F_c^* = F_c' / C_p =$ 1440 psi

$L_e / d = Max(k_e L_x / d, k_e L_y / b) =$ 30 (NDS Table G1/Figure 3F)

< 50 , (NDS 3.7.1.4) **[Satisfactory]**

$E'_{min} =$ 930000 psi, (Strong Axis Controls.)

$F_{cE} = 0.822\, E'_{min} / (L_e/d)^2 =$ 849 psi

$F = F_{cE} / F_c^* =$ 0.590

$c =$ 0.9

CHECK BENDING CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_{bx}' = F_b\, C_D\, C_M\, C_t\, C_L\, (C_F\, C_i\, C_r)\,(C_V\, C_c\, C_I) =$ 2146 psi

> $f_{bx} = M_x / S_x =$ 256 psi **[Satisfactory]**

Where

C_D	C_L	C_F	C_r	C_V	C_c	C_I
0.90	0.99	1.00	1.00	1.00	1.00	1.00
(Tab. 2.3.2)		(NDS 4.3.6)	(NDS 4.3.9)	(NDS 5.3.6)	(NDS 5.3.8)	(NDS 5.3.9)

$C_L = (1+F) / 1.9 - [((1+F) / 1.9)^2 - F / 0.95]^{0.5}$,(NDS 3.3.3)

$F_b^* = F_{bx}' / (C_V\, C_L) =$ 2160 psi

$R_B = (L_e\, d / b^2)^{0.5} =$ 8 (NDS Eq. 3.3-5)

< 50 , (NDS 3.3.3.7) **[Satisfactory]**

$E'_{min} =$ 930000 psi

$F_{bE} = 1.20\, E'_{min} / R_B^2 =$ 18921 psi

$F = F_{bE} / F_b^* =$ 8.760

$F_{by}' = F_b\, C_D\, C_M\, C_t\, C_{fu}\, (C_F\, C_i\, C_r)\,(C_V\, C_c\, C_I) =$ 2333 psi

> $f_{by} = M_y / S_y =$ 123 psi **[Satisfactory]**

Where C_{fu} = 1.08 (4.3.7/5.3.7)

CHECK BENDING & COMPRESSION CAPACITY (NDS 3.9.2)

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1 - f_c / F_{cEx}}\right)\frac{f_{bx}}{F_{bx}'} + \left(\frac{1}{1 - f_c / F_{cEy} - \left(f_{bx} / F_{bE}\right)^2}\right)\frac{f_{by}}{F_{by}'} = 0.96 < 1.0$$ **[Satisfactory]**

Where $E'_{x,min} =$ 930000 psi

$E'_{y,min} =$ 830000 psi

$F_{cEx} = 0.822\, E'_{x,min} / (k_e L_x / d)^2 =$ 849 psi

$F_{cEy} = 0.822\, E'_{y,min} / (k_e L_y / b)^2 =$ 823 psi

$$\frac{f_c}{F_{cEy}'} + \left(\frac{f_{bx}}{F_{bE}'}\right)^2 = 0.66 < 1.0$$ **[Satisfactory]**

CHECK SHEAR CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_v' = F_v\, C_D\, C_M\, C_t\, (C_i)\,(C_{vr}) =$ 239 psi

> $f_v = 3\,V / (2\,A) =$ 200 psi **[Satisfactory]**

Where

C_D	C_{vr}
0.90	1.0
(Tab. 2.3.2)	(NDS 5.3.10)

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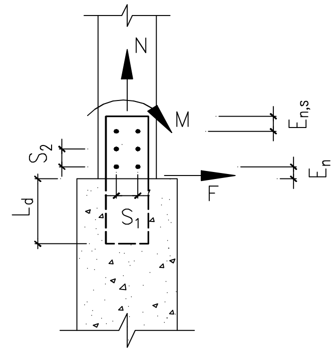
Connection Design for Bending Post at Concrete Column Based on NDS 2012 & ACI 318-11

INPUT DATA & DESIGN SUMMARY



LOADS (ASD) N = 150 lbs
F = 132 lbs (132 lbs, at center of screws.)
M = 396 ft-lbs (341 ft-lbs, at center of screws.)

CONCRETE COLUMN SIZE 16 in thk. x 16 in width

CONCRETE STRENGTH f_c' = 3 ksi

BOTH SIDES STRAP SECTION 5.5 in width x 3/8 in thk.

END DISTANCE OF STEEL $E_{n,s}$ = 1.5 in, vertical dimension

STRAP EMBEDMENT L_d = 10 in

TOP SQUARE POST 10 in thk. x 10 in width

NUMBER OF LAG SCREWS n = 8 , on each side

LAG SCREW DIAMETER ϕ = 1/4 in

LAG SCREW SPACING S_1 = 4 in, (if one row input 0)
S_2 = 2 in

END DISTANCE OF WOOD E_n = 2 in, vertical dimension

LOAD DURATION FACTOR (Tab 2.3.2, NDS 2012) C_D = 1.6

WET SERVICE FACTOR (Tab 10.3.3, NDS 2012) C_M = 1.0

TEMPERATURE FACTOR (Tab 10.3.4, NDS 2012) C_t = 1.0

GROUP ACTION FACTOR APPLY? (0 = No, 1 = Yes) 1 Yes, (C_g = 0.894 , NDS 2012, 10.3.6)

THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

CHECK POST EDGE, END, & SPACING DISTANCE REQUIREMENTS (NDS 2012, Table 11.5.1A, Table 11.5.1B, & Table 11.5.1C)

E_g =	2.75	in	>	1.5 D	[Satisfactory]
E_n =	2	in	>	3.5 D	[Satisfactory]
S =	2	in	>	3 D	[Satisfactory]

CHECK WOOD CAPACITY

$F = [(F_F + F_{MF})^2 + (F_N + F_{MN})^2]^{0.5}$ = 115 lbs / screw, actual load at corner screw

where F_F = 8 lbs / screw F_{MF} = 85 lbs / screw, at corner screw
F_N = 9 lbs / screw F_{MN} = 57 lbs / screw, at corner screw
J = 72 (in^2 Screw Area)

$C_\Delta = \text{Min}(C_{\Delta 1}, C_{\Delta 2}, C_{\Delta 3})$ = 1.000 , (geometry factor, NDS 2012, 11.5.1)

where $C_{\Delta 1}$ = (actual end distance) / (min end distance for full design value) = E_n / 7D = 1.000
$C_{\Delta 2}$ = (actual shear area) / (min shear area for full design value) = 1.000
$C_{\Delta 3}$ = (actual spacing) / (min spacing for full design value) = S / 4D = 2.000

$$C_g = \left[\frac{m\left(1-m^{2n}\right)}{n\left[\left(1+R_{EA}m^n\right)(1+m)-1+m^{2n}\right]}\right]\left[\frac{1+R_{EA}}{1-m}\right] = \quad 0.894$$
, (group action factor, NDS 2012, 10.3.6)

where n = 8 $R_{EA} = \text{Min}[(E_sA_s/E_mA_m), (E_mA_m/E_sA_s)]$ = 0.049
E_sA_s = 6187500 lbs, (one side, NDS Table 10.3.6C) $\gamma = 180000\ D^{1.5}$ = 22500
t_m = 9 1/2 in $u = 1+\gamma\ S/2\ [1 / E_mA_m + 1 / E_sA_s]$ = 1.004
E_mA_m = 1.26E+08 lbs, (E_m fr Tab.10.3.6C) $m = u - (u^2 - 1)^{0.5}$ = 0.916

C_d = P / (8 D) = 1.000 , (penetration depth factor, NDS 2012, tab 11J, footnote 3)

where $P = S + (T - E) - t_s$ = 8 15/32 in, (length of thread penetration in main member)
S = 4 in, (NDS 2012 tab L2)
T - E = 4 27/32 in, (NDS 2012 tab L2)

$$Z_{s\theta} = \frac{Z_{//}Z_{s\perp}}{Z_{//}\sin^2\theta + Z_{s\perp}\cos^2\theta} = \quad 129$$
lbs / screw, side member allowable capacity (NDS 2012, 11.3.3)

where $Z_{//}$ = 150 lbs / screw, (NDS 2012, tab 11J)
$Z_{s\perp}$ = 120 lbs / screw, (NDS 2012, tab 11J)
θ = 54.70 °, angle between F & grain for side member at corner screw

$$Z_{m\theta} = \frac{Z_{//}Z_{m\perp}}{Z_{//}\sin^2\theta + Z_{m\perp}\cos^2\theta} = \quad 138$$
lbs / screw, main member allowable capacity (NDS 2012, 11.3.3)

where $Z_{//}$ = 150 lbs / screw, (NDS 2012, tab 11J)
$Z_{m\perp}$ = 120 lbs / screw, (NDS 2012, tab 11J)
θ = 35.30 °, angle between F & grain for main member at corner screw

$Z_\theta = \text{MIN}(Z_{//}, Z_\perp)\ C_D\ C_g\ C_d$ = 115 lbs / corner screw, connection allowable capacity

(Cont'd)

CHECK STEEL STRAP CAPACITIES (AISC 360-10, ASD)

A_g = 4.13 in^2, yielding criterion F_y = 36.00 ksi

T_{allow} = $0.6\ F_y\ A_g$ = 89.10 k > T = 2 n F = 1.83 k

(0.6 from $1/\Omega_t$, Typ.)   [Satisfactory]   , (conservatively assumed all screws with the max force to vertical.)

A_n = 3.56 in^2, fracture criterion F_u = 58.00 ksi

T_{allow} = $0.5\ F_u\ A_n$ = 103.31 k > T   [Satisfactory]

A_v = 5.63 in^2, block shear

T_{allow} = $0.3\ F_u\ A_v + 0.5\ F_u\ (0.5\ A_n)$ = 149.53 k > T   [Satisfactory]

r_{min} = $t / (12)^{0.5}$ = 0.108 in   L = Max (E_n , S_2) = 2 in

L / r_{min} = 18 < 200   [Satisfactory]   (AISC 360-10 D1)

CHECK STEEL STRAP EMBEDMENT

$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f'_c}}\ ,\ 8d_b\ ,\ 6\ \ in\right) =$$ 9.20 in < L_d [Satisfactory]

, (ACI 318-11 12.5.2)

where η = 0.7

$\rho_{required} / \rho_{provided}$ = 1.0 ψ_e = 1.0

λ = 1.0 d_b = thk. = 0.375 in

T_{allow} = $(1 / 1.5)\ \phi\ MIN(\ 0.2\ f_c'\ ,\ 800\ )\ A_c$ = 24.75 k > T   [Satisfactory]

where ϕ = 0.75   , (ACI 318-11 11.6.5)

A_c = 82.50 in^2, all four friction surfaces.

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Curved Wood Member (Wood Torsion) Design Based on NDS 2015

INPUT DATA & DESIGN SUMMARY

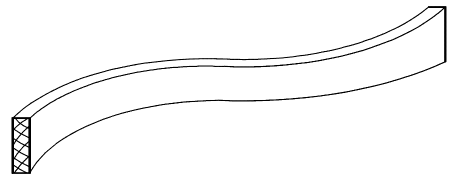
AXIAL COMPRESSION FORCE $P =$ **10** kips, ASD

TOTAL CURVED STRONG AXIS EFFECTIVE LENGTH

$k_e L_x =$ **15** ft, (NDS Table G1/Figure 3F)

TOTAL CURVED WEAK AXIS EFFECTIVE LENGTH

$k_e L_y =$ **7.5** ft, (NDS Table G1)



STRONG AXIS BENDING MOMENT $M_x =$ **0.4** ft-kips, ASD

CURVED BENDING EFFECTIVE LENGTH $L_e =$ **8** ft, (NDS Table 3.3.3)

WEAK AXIS BENDING MOMENT $M_y =$ **0.1** ft-kips, ASD

CRITICAL/MAXIMUM SECTION SHEAR FORCE $V =$ **2.5** kips, ASD

THE MEMBER DESIGN IS ADEQUATE.

TORQUE IN THE RECTANGULAR SECTION $T =$ **0.35** ft-kips, ASD

DESIGN VALUES (NDS Supplement Tables or ICC ESR)

	F_b	F_v	F_c	$E_{x,min}$	$E_{y,min}$	b, (in)	d, (in)	A, (in^2)	I_x, (in^4)	S_x, (in^3)	I_y, (in^4)	S_y, (in^3)
psi ==>	2325	525	2170	788000	788000	3.5	8.625	30.19	187.14	43.39	30.82	17.61

ANALYSIS

CHECK COMPRESSION CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_c' = F_c\, C_D\, C_M\, C_t\, C_F\, C_i\, C_p =$ 902 psi

$>$ $f_c = P / A =$ 331 psi **[Satisfactory]**

Where	C_D	C_M	C_t	C_F	C_i	C_p	C_T
	0.90	1.00	1.0	1.00	1.0	0.46	1.0
	(Tab. 2.3.2)	(4.4.3/5.5.3)	(Tab. 2.3.3)	(NDS 4.3.6)	(NDS 4.3.8)		(NDS 4.4.2 for Truss only)

$C_P = (1+F) / 2c - [((1+F) / 2c)^2 - F / c]^{0.5}$,(NDS 3.7.1)

$F_c^* = F_c' / C_p =$ 1953 psi

$L_e / d = Max(k_e L_x / d , k_e L_y / b) =$ 26 (NDS Table G1/Figure 3F)

$<$ 50 , (NDS 3.7.1.4) **[Satisfactory]**

$E'_{min} =$ 788000 psi, (Weak Axis Controls.)

$F_{cE} = 0.822\, E'_{min} / (L_e/d)^2 =$ 980 psi

$F = F_{cE} / F_c^* =$ 0.502

$c =$ 0.9

CHECK BENDING CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_{bx}' = F_b\, C_D\, C_M\, C_t\, C_L\, (C_F\, C_i\, C_r)\, (C_V\, C_c\, C_I) =$ 2074 psi

$>$ $f_{bx} = M_x / S_x =$ 111 psi **[Satisfactory]**

Where	C_D	C_L	C_F	C_r	C_V	C_c	C_I
	0.90	0.99	1.00	1.00	1.00	1.00	1.00
	(Tab. 2.3.2)		(NDS 4.3.6)	(NDS 4.3.9)	(NDS 5.3.6)	(NDS 5.3.8)	(NDS 5.3.9)

$C_L = (1+F) / 1.9 - [((1+F) / 1.9)^2 - F / 0.95]^{0.5}$,(NDS 3.3.3)

$F_b^* = F_{bx}' / (C_V\, C_L) =$ 2093 psi

$R_B = (L_e\, d / b^2)^{0.5} =$ 8 (NDS Eq. 3.3-5)

$<$ 50 , (NDS 3.3.3.7) **[Satisfactory]**

$E'_{min} =$ 788000 psi

$F_{bE} = 1.20\, E'_{min} / R_B^2 =$ 13990 psi

$F = F_{bE} / F_b^* =$ 7

$F_{by}' = F_b\, C_D\, C_M\, C_t\, C_{fu}\, (C_F\, C_i\, C_r)\, (C_V\, C_c\, C_I) =$ 2171 psi

$>$ $f_{by} = M_y / S_y =$ 68 psi **[Satisfactory]**

Where	C_{fu}
	1.04
	(4.3.7/5.3.7)

CHECK BENDING & COMPRESSION CAPACITY (NDS 3.9.2)

$$\left(\frac{f_c}{F'_c}\right)^2 + \left(\frac{1}{1 - f_c / F_{cEx}}\right)\frac{f_{bx}}{F'_{bx}} + \left(\frac{1}{1 - f_c / F_{cEy} - \left(f_{bx} / F_{bE}\right)^2}\right)\frac{f_{by}}{F'_{by}} = \quad 0.25 \quad < \quad 1.0$$

[Satisfactory]

Where $E'_{x,min} =$ 788000 psi

$E'_{y,min} =$ 788000 psi

$F_{cEx} = 0.822\, E'_{x,min} / (k_e L_x/d)^2 =$ 1487 psi

$F_{cEy} = 0.822\, E'_{y,min} / (k_e L_y/b)^2 =$ 980 psi

$$\frac{f_c}{F'_{cEy}} + \left(\frac{f_{bx}}{F'_{bE}}\right)^2 = \quad 0.36 \quad < \quad 1.0$$

[Satisfactory]

(Cont'd)

CHECK SHEAR CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_v' = F_v\ C_D\ C_M\ C_t\ (C_i)\ (C_{vr}) =$ 473 psi

$>$ $f_v + f_s =$ 273 psi **[Satisfactory]**

Where	C_D	C_{vr}
	0.90	**1.0**
	(Tab. 2.3.2)	(NDS 5.3.10)

$f_v = 3\ V\ /\ (2\ A) =$ 124 psi

$f_s = T\ (\ 1.5\ d + 0.9\ b)\ /\ 0.5\ d^2\ b^2 =$ 148 psi, (torsional stress)

CHECK TORSION CAPACITY (Wood Engineering and Construction Handbook, Page 4.17)

$F_s' = F_v'\ /\ 3 =$ 158 psi $>$ $f_s =$ 148 psi **[Satisfactory]**

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4E-SMF with Wood Nailer Design Based on AISC 358-10 & NDS 2012

INPUT DATA & SUMMARY

COLUMN SIZE **W12X96** BEAM SIZE **W18X35**

DIMENSIONS

$H =$ **15** ft

$L =$ **30** ft

$L_1 =$ **14** ft

LOADS (ASD)

$P =$ **12** kips, (downward)

$F =$ **5.8** kips, (horiz. to right)

$w =$ **1475** lbs / ft, (downward)

BASE PINNED ? **Yes** , (pinned)

BEAM & COLUMN RIGID-ZONE **75** % of (d / 2)

BEAM TO COLUMN BOLTS

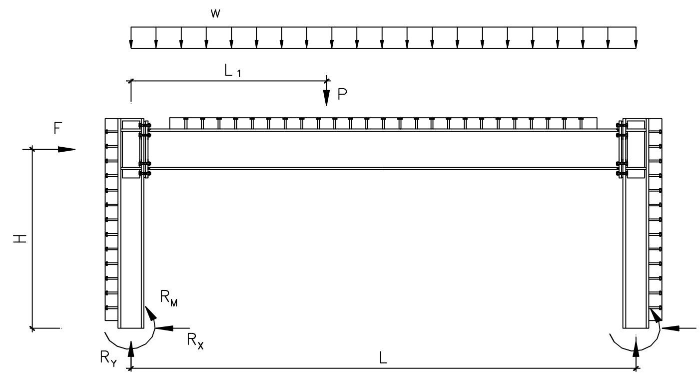
$\phi =$ **1 1/16** in, (Total 2 x 2 x 4 for two moment connections)

GRADES (A325 or A490) **A325**

PLATE & SHIM $t_p =$ **3/4** in

NAILER TYPE 0 Douglas Fir-Larch, G=0.5

(0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir)



THE DESIGN IS ADEQUATE.

BEAM NAILER SIZE 4 thk. x 6 width

THREAD STUD DIAMETER $\phi =$ 3/4

THREAD STUD SPACING $S =$ 12 in

END DISTANCE OF WOOD $E_n =$ 4 in

LOAD DURATION FACTOR (Tab 2.3.2, NDS 2012) $C_D =$ 1.6

	Left	&	Right	
$R_X =$	-6.13	&	11.93	kips
$R_Y =$	25.29	&	30.96	kips
$R_M =$	0.00	&	0.00	ft-kips

ANALYSIS

CHECK STORY DRIFT AND BEAM DEFLECTION

$\Delta_{L,drift} =$ 0.6238 in, (horiz. to right) < $\delta_{xe,allowable,\ ASD} = \Delta_a\ I\ /\ (1.4\ C_d)$, (ASCE 7 12.8.6) = 0.6429 in

$\Delta_{R,drift} =$ 0.6095 in, (horiz. to right)

$C_d =$ 4 ,(ASCE 7 Tab 12.2-1)

[Satisfactory] $\Delta_a =$ 0.02 h_{sx}, (ASCE 7 Tab 12.12-1)

$I =$ 1 , (2012 IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)

$\Delta_{max,\ Beam}\ /\ L =$ 1 / 381 < 1/360 , (2012 IBC Tab 1603) **[Satisfactory]**

CHECK BEAM NAILER CONNECTION

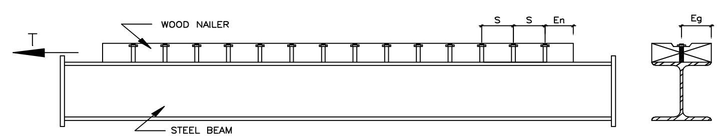
$T = ABS\ (-F) =$ 5.8 kips

$n =$ 27 , number of thread studs

$E_g =$ 2.75 in > 1.5 D

$E_n =$ 4 in > 3.5 D

$S =$ 12 in > 3 D



$C_\Delta = \text{Min}\ (C_{\Delta1}, C_{\Delta2}, C_{\Delta3}) =$ 0.762 , (geometry factor, NDS 2012, 11.5.1)

$$C_g = \left[\frac{m\left(1-m^{2n}\right)}{n\left[\left(1+R_{EA}m^n\right)(1+m)-1+m^{2n}\right]}\right]\left[\frac{1+R_{EA}}{1-m}\right] = \quad 0.185$$

, (group action factor, NDS 2012, 10.3.6)

$Z_{//} =$ 1720 lbs / stud, (interpolated from Tab 11E, NDS 2012, Page 85 or Tab 11B, NDS 2012)

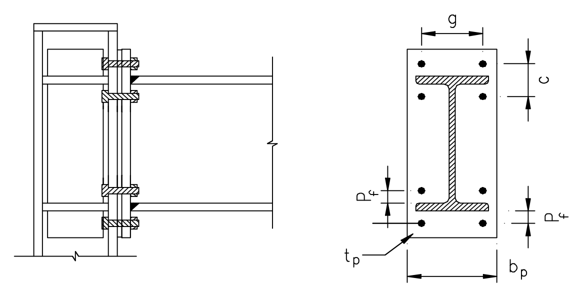
$Z'_{//} = n\ Z_{//}\ C_D\ C_M\ C_t\ C_g\ C_\Delta =$ 10.484 kips > T **[Satisfactory]**

CHECK 4-BOLTED UNSTIFFENED END PLATE MOMENT CONNECTION

$g =$ 5.00 in $P_f =$ 1.75 in

$c =$ 3.93 in $b_p =$ 9.00 in

[Satisfactory]



1. LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)
   [Satisfactory]
2. BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10 Sec. E3.4a)
   [Satisfactory]
3. BENDING MOMENT AT THE COLUMN FACE (FEMA 350 3.6.1.1.2)
   [Satisfactory]
4. SHEAR CAPACITY AT THE COLUMN FACE (FEMA 350 3.6.1.1.3)
   [Satisfactory]
5. END PLATE THICKNESS (AISC 358-10 Eq 6.10-13)
   [Satisfactory]
6. CONTINUITY PLATE REQUIREMENT (AISC 358-10 Eq 6.10-13, FEMA 3.3.3.1)
   [Satisfactory]

(Continuity column stiffeners 7/16 x 6 with 1/4" fillet weld to web & CP to flanges. A doubler plate is not required.)

Purchaser's LOGO

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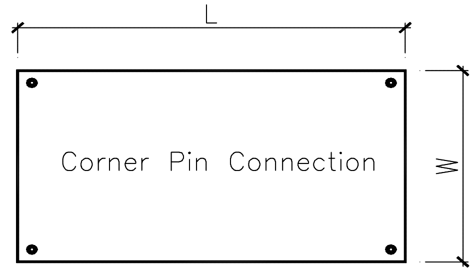
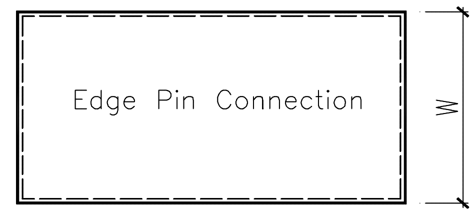
REVIEW BY :

Two-Way Floor Design Based on NDS 2015, using Cross-Laminated Timber (CLT), by Finite Element Method





INPUT DATA & DESIGN SUMMARY

CLT PANEL SIZE W = **8** *ft, (2438 mm)*
L = **12** *ft, (3658 mm)*

CLT PANEL THICKNESS t = **4.5** *in, (114 mm)*

CONNECTION TYPE (0 or 1) **0** *, only corner pinned.*

DESIGN VALUES (NDS 10.2) F_b' = **2.1** *ksi, ASD* *(14 N / mm^2)*
F_v' = **0.5** *ksi, ASD* *(3 N / mm^2)*
E' = **900** *ksi, ASD* *(6207 N / mm^2)*

ALLOWABLE DEFLECTION ***L*** / **240**

UNIFORM LOAD (Including Wt., perpendicular to Plane) w = **100** *psf, ASD level*
(4788 N / m^2)

POINT LOAD (Including Impact Factor) P = **0.5** *kips, ASD level*
(2.22 kN)

THE DESIGN IS ADEQUATE.

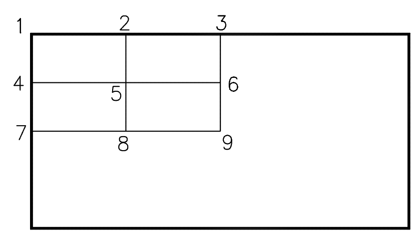
ANALYSIS

CLT PROPERTIES

G = 0.55 *, Specific gravity*
γ = 34 *lbs / ft^3*
Weight = 1236 *lbs, (561 kg)*
υ = 0.45 *, Poisson's ratio*

JOINT DEFLECTIONS, REACTIONS, & PLATE SECTION FORCES

P = 0.5 kips,(*Point load at Joint 9.)*



Joint Number	Δ in	R kips
1	0	-2.53
2	0.44	
3	0.62	
4	0.10	
5	0.44	
6	0.58	
7	0.14	
8	0.44	
9	0.57	

Bending Section	M ft-k/ft
7 - 8	1.37
8 - 9	1.22
3 - 6	3.02
6 - 9	2.86

CHECK BENDING CAPACITY

$M_n / \Omega_b = F_b' \, d \, t^2 / 6 =$ 85.05 *ft-k/ft* **>** M = 3.11 *ft-k/ft*
[Satisfactory]

Where d = 12 *in, (1 ft)*
$M = (M_{6\text{-}9}{}^2 + M_{8\text{-}9}{}^2)^{0.5}$ = 3.11 *ft-k/ft*

CHECK SHEAR CAPACITY

$V_n / \Omega_v = (2/3) \, F_v' \, d \, t =$ 18.00 *kips/ft* **>** V = 1.79 *kips/ft*
[Satisfactory]

Where $V = Max(R\,,P) / (2\ ft^2)^{0.5}$ = 1.7854 *kips/ft*

CHECK DEFLECTION

Δ_{max} = 0.57 *in* **<** *L* / 240 = 0.60 in
[Satisfactory]

Where $L = Max\,(L\,,W)$ = 144.0 *in*

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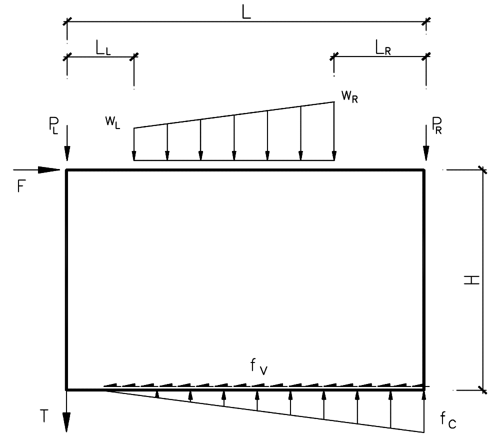
Shear Wall Design, using Cross-Laminated Timber (CLT), Based on NDS 2015



DESIGN CRITERIA

1. PLANE SECTIONS BEFORE BENDING REMAIN PLANAR AFTER BENDING.

$$f_c = \varepsilon_c E' \leq F_b'$$

2. THE TENSION CAPACITY IS ONLY FROM THE LAST END VERTICAL ELEMENTS AND CONNECTED HOLD-DOWN.

$$T = F_t' (A_{parallel}) \leq \text{Hold-Down}$$

THE DESIGN IS ADEQUATE.

INPUT DATA & DESIGN SUMMARY

CLT PANEL SIZE $H =$ **10** *ft, (3048 mm)*
$L =$ **16** *ft, (4877 mm)*

CLT PANEL THICKNESS $t =$ **4.5** *in, (114 mm)*

DESIGN VALUES (NDS 10.2) $F_b' =$ **2.1** *ksi, ASD* $(14\ N/mm^2)$
$F_v' =$ **0.5** *ksi, ASD* $(3\ N/mm^2)$
$E' =$ **900** *ksi, ASD* $(6207\ N/mm^2)$

Hold-Down Capacity = **13** *kips, ASD level (57.82 kN)*
$F_t' =$ **1.8** *ksi, ASD* $(12\ N/mm^2)$, $A_{parallel} =$ **12** in^2 (7742 mm^2)

LOADS $F =$ **60** *kips, ASD level (266.88 kN)*
$P_L =$ **80** *kips, ASD level (355.84 kN)* $P_R =$ **150** *kips, ASD level (667.20 kN)*

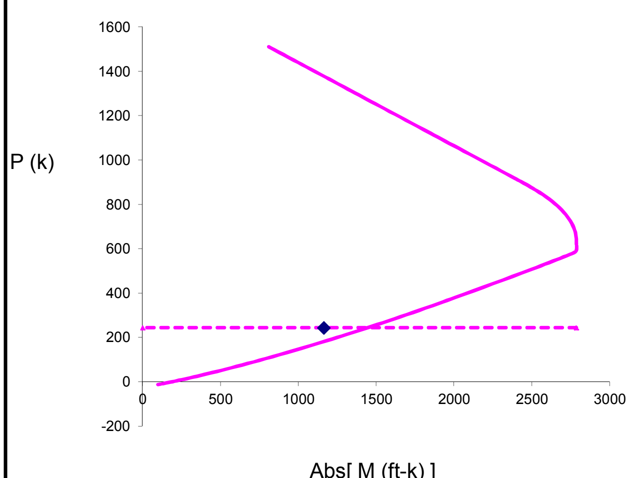
$w_L =$ **600** *plf, ASD (8756 N / m)* $L_L =$ **2** *ft, (610 mm)*
$w_R =$ **1500** *plf, ASD (21891 N / m)* $L_R =$ **3** *ft, (914 mm)*

ANALYSIS

CLT PROPERTIES AND BOTTOM SECTION FORCES

$G =$	0.55	, *Specific gravity*	$P =$	243.61	*kips, ASD level*
$\gamma =$	34	lbs / ft^3	$M =$	1163.30	*ft-kips, ASD level*
Weight =	2059	*lbs, (935 kg)*	$V =$	60.00	*kips, ASD level*

CHECK FLEXURAL & AXIAL CAPACITY BY ALLOWABLE STRESS DESIGN (ASD)



$P =$ 243.61 *kips, ASD level*

$M =$ 1163.30 *ft-kips* < *M (allowable)* = 1438.3 *ft-kips* **[Satisfactory]**

CHECK SHEAR CAPACITY (ASD)

$V =$ 60.00 *kips* < *V (allowable) =* 216 *kips* **[Satisfactory]**

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Hybrid Member (Wood & Metal) Design Based on NDS 2015, AISI S100 & ICBO ER-4943P

DESIGN CRITERIA

1. The only axial and strong axis bending have been calculated for the metal track capacity. The metal on wood has no buckling capacity reduction.
2. The shear force/load is fully supported by wood, so the metal can be nailed to existing structure on job site, without connection changes.

INPUT DATA & DESIGN SUMMARY

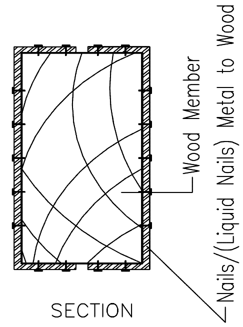
AXIAL COMPRESSION FORCE P = **50** kips, ASD

TOTAL HYBRID STRONG AXIS EFFECTIVE LENGTH

$k_e L_x$ = **15** ft, (NDS Table G1/Figure 3F)

TOTAL HYBRID WEAK AXIS EFFECTIVE LENGTH

$k_e L_y$ = **7.5** ft, (NDS Table G1)



STRONG AXIS BENDING MOMENT M_x = **6** ft-kips, ASD

HYBRID BENDING EFFECTIVE LENGTH L_e = **8** ft, (NDS Table 3.3.3)

WEAK AXIS BENDING MOMENT M_y = **0.1** ft-kips, ASD

CRITICAL/MAXIMUM SECTION SHEAR FORCE V = **2.5** kips, ASD

THE MEMBER DESIGN IS ADEQUATE.

TORQUE IN THE RECTANGULAR SECTION T = **0.35** ft-kips, ASD

WOOD DESIGN VALUES (NDS Supplement Tables or ICC ESR)

	F_b	F_v	F_c	$E_{x,min}$	$E_{y,min}$	b, (in)	d, (in)	A, (in^2)	I_x, (in^4)	S_x, (in^3)	I_y, (in^4)	S_y, (in^3)
psi ==>	**2325**	**525**	**2170**	**788000**	**788000**	**3.5**	**8.625**	**30.19**	**187.14**	**43.39**	**30.82**	**17.61**

METAL DESIGN VALUES (AISI S100 & ICBO ER-4943P)

	F_y	A, (in^2)	S_x, (in^3)
ksi ==>	**50**	**1.17**	**1.80**

ANALYSIS

CHECK COMPRESSION CAPACITY (AISI D6, NDS Table 4.3.1 & Table 5.3.1)

$F_c' = F_c\, C_D\, C_M\, C_t\, C_F\, C_i\, C_p$ = 902 psi

> $f_c = (P - F_y\, A / \Omega_c) / A$ = 576 psi **[Satisfactory]**

Where	C_D	C_M	C_t	C_F	C_i	C_p	C_T
	0.90	**1.00**	**1.0**	1.00	**1.0**	0.46	**1.0**
	(Tab. 2.3.2)	(4.4.3/5.5.3)	(Tab. 2.3.3)	(NDS 4.3.6)	(NDS 4.3.8)		(NDS 4.4.2 for Truss only)

Ω_c = 1.8

$C_P = (1+F) / 2c - [((1+F) / 2c)^2 - F / c]^{0.5}$,(NDS 3.7.1)

$F_c^* = F_c' / C_p$ = 1953 psi

$L_e / d = Max(k_e L_x / d , k_e L_y / b)$ = 26 (NDS Table G1/Figure 3F)

< 50 , (NDS 3.7.1.4) **[Satisfactory]**

E'_{min} = 788000 psi, (Weak Axis Controls.)

$F_{cE} = 0.822\, E'_{min} / (L_e/d)^2$ = 980 psi

F = F_{cE} / F_c^* = 0.502

c = 0.9

CHECK BENDING CAPACITY (AISI C3, NDS Table 4.3.1 & Table 5.3.1)

$F_{bx}' = F_b\, C_D\, C_M\, C_t\, C_L\, (C_F\, C_i\, C_r)(C_V\, C_c\, C_I)$ = 2074 psi

> $f_{bx} = (M_x - F_y\, S_x / \Omega_b) / S_x$ = 417 psi **[Satisfactory]**

Where	C_D	C_L	C_F	C_r	C_V	C_c	C_I
	0.90	0.99	1.00	**1.00**	**1.00**	**1.00**	**1.00**
	(Tab. 2.3.2)		(NDS 4.3.6)	(NDS 4.3.9)	(NDS 5.3.6)	(NDS 5.3.8)	(NDS 5.3.9)

Ω_b = 1.67

$C_L = (1+F) / 1.9 - [((1+F) / 1.9)^2 - F / 0.95]^{0.5}$,(NDS 3.3.3)

$F_b^* = F_{bx}' / (C_V\, C_L)$ = 2093 psi

$R_B = (L_e\, d / b^2)^{0.5}$ = 8 (NDS Eq. 3.3-5)

< 50 , (NDS 3.3.3.7) **[Satisfactory]**

E'_{min} = 788000 psi

$F_{bE} = 1.20\, E'_{min} / R_B^2$ = 13990 psi

F = F_{bE} / F_b^* = 7

$F_{by}' = F_b\, C_D\, C_M\, C_t\, C_{fu}\, (C_F\, C_i\, C_r)(C_V\, C_c\, C_I)$ = 2171 psi

> $f_{by} = M_y / S_y$ = 68 psi **[Satisfactory]**

Where	C_{fu}
	1.04
	(4.3.7/5.3.7)

(Cont'd)

CHECK BENDING & COMPRESSION CAPACITY (AISI C5 & NDS 3.9.2)

$$\left(\frac{f_c}{F'_c}\right)^2+\left(\frac{1}{1-f_c/F_{cEx}}\right)\frac{f_{bx}}{F'_{bx}}+\left(\frac{1}{1-f_c/F_{cEy}-\left(f_{bx}/F_{bE}\right)^2}\right)\frac{f_{by}}{F'_{by}}=$$ 0.81 < 1.0 **[Satisfactory]**

Where $E'_{x,min}$ = 788000 psi

$E'_{y,min}$ = 788000 psi

$F_{c\,Ex} = 0.822\,E'_{x,min} / (k_e L_x/d)^2$ = 1487 psi

$F_{c\,Ey} = 0.822\,E'_{y,min} / (k_e L_y/b)^2$ = 980 psi

$$\frac{f_c}{F'_{cEy}}+\left(\frac{f_{bx}}{F'_{bE}}\right)^2=$$ 0.61 < 1.0 **[Satisfactory]**

$$\frac{f_c+F_y/\Omega_c}{F'_{cEy}}+\frac{f_{bx}+F_y/\Omega b}{F'_{bE}}=$$ 0.77 < 1.0 **[Satisfactory]**

CHECK SHEAR CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_v' = F_v\,C_D\,C_M\,C_t\,(C_i)\,(C_{vr})$ = 473 psi

> $f_v + f_s$ = 273 psi **[Satisfactory]**

Where

C_D	C_{vr}
0.90	**1.0**
(Tab. 2.3.2)	(NDS 5.3.10)

$f_v = 3\,V / (2\,A)$ = 124 psi

$f_s = T\,(1.5\,d + 0.9\,b) / 0.5\,d^2\,b^2$ = 148 psi, (torsional stress)

CHECK TORSION CAPACITY (Wood Engineering and Construction Handbook, Page 4.17)

$F_s' = F_v' / 3$ = 158 psi > f_s = 148 psi **[Satisfactory]**

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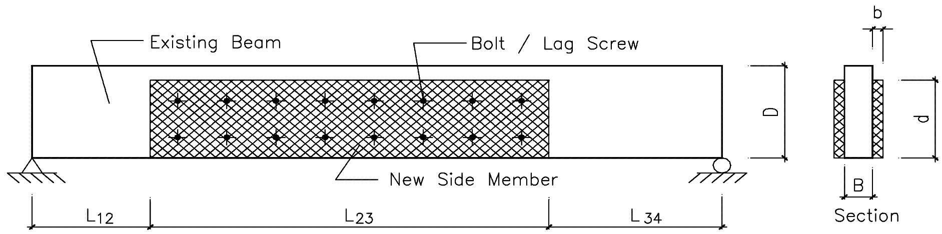
Beam Reinforcement Design by Finite Element Method

INPUT DATA & DESIGN SUMMARY

DIMENSION

L_{12} = 2 *ft*
L_{23} = 14 *ft*
L_{34} = 4 *ft*



EXISTING BEAM D = 13.5 *in*
B = 3.5 *in*

E' = 620 *ksi, Modulus of Elasticity*
F_b' = 0.9 *ksi, ASD, Adjusted Design Value*
F_v' = 0.12 *ksi, ASD, Adjusted Design Value*

NEW SIDE MEMBER d = 12 *in*
b = 0.75 *in*
No. = 2 *, Both Sides*
E' = 900 *ksi, Modulus of Elasticity*
F_b' = 2.1 *ksi, ASD, Adjusted Design Value*

CONNECTOR SHEAR CAPACITY (One Side Face, Wood Control)
Z' = 1470 *lbs per Bolt / Lag Screw*

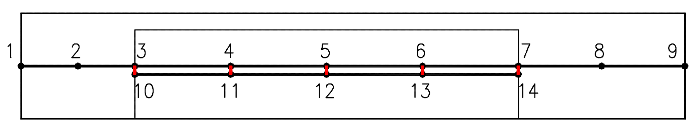
THE DESIGN IS ADEQUATE.

Δ_{sup} = 1.11 in @ 9.00 ft, from left.
(L / 216)
Total 24 Bolts / Lag Screws at One Side.

EXISTING MAX MOMENT BEFORE REINFORCEMENT
M_{int} = 0.7 *ft-kips, ASD, without Shoring Reduced*
V_{int} = 0.3 *kips, ASD*

MAX SECTION FORCES AFTER REINFORCEMENT
M_{sup} = 10 *ft-kips, ASD, from Superimposed Loads*
V_{sup} = 1.2 *kips, ASD*

ANALYSIS



The max Δ on M_{sup} from uniform load, (0.2 kips/ft)

Joint Number	X in	Y in	load kips	Δ in
1	0	7	0.1	0
2	12	7	0.2	0.18
3	24	7	0.45	0.36
4	66	7	0.7	0.86
5	108	7	0.7	1.11
6	150	7	0.7	1.05
7	192	7	0.55	0.69
8	216	7	0.4	0.37
9	240	7	0.2	0
10	24	6	0	0.36
11	66	6	0	0.86
12	108	6	0	1.11
13	150	6	0	1.05
14	192	6	0	0.69

Finite Element Method

Element	Joint		L (in)	A (in^2)	I (in^4)	E (ksi)	M (ft-k)	V (kips)	N (kips)
1	1	2	12.0	47.25	717.61	620	1.90	-1.90	0.00
2	2	3	12.0	47.25	717.61	620	3.60	-1.70	0.00
3	3	4	42.0	47.25	717.61	620	5.49	-0.86	-0.84
4	4	5	42.0	47.25	717.61	620	6.82	-0.38	-1.30
5	5	6	42.0	47.25	717.61	620	6.83	0.10	-1.40
6	6	7	42.0	47.25	717.61	620	6.46	0.58	-1.15
7	7	8	24.0	47.25	717.61	620	6.40	1.40	0.00
8	8	9	24.0	47.25	717.61	620	3.60	1.80	0.00
9	10	11	42.0	18.00	216.00	900	2.43	-0.39	0.84
10	11	12	42.0	18.00	216.00	900	3.00	-0.17	1.30
11	12	13	42.0	18.00	216.00	900	2.99	0.05	1.40
12	13	14	42.0	18.00	216.00	900	2.85	0.27	1.15
13	3	10	0.8	178.50	52479.00	29000	1.11	0.84	0.39
14	4	11	0.8	178.50	52479.00	29000	-0.02	0.46	-0.22
15	5	12	0.8	178.50	52479.00	29000	0.00	0.10	-0.22
16	6	13	0.8	178.50	52479.00	29000	0.02	-0.25	-0.22
17	7	14	0.8	178.50	52479.00	29000	-1.91	-1.15	0.27

The Max Shear Connection Stress
$f = (V^2 + N^2)^{0.5} / d^2 + M / (0.208\ d^3)$
= 72 *psf*

Total Connectors
n = A f / Z' = 24 , one side only

Existing Beam :	F_b' =	900	psi	>	$f_b = (M_{int} + M_{max}) / S$ =	849	psi	**[Satisfactory]**
	F_v' =	120	psi	>	$f_v = 1.2\ (V_{int} + V_{sup}) / A$ =	100	psi	**[Satisfactory]**
New Side Member :	F_b' =	2100	psi	>	$f_b = M_{max} / S$ =	999	psi	**[Satisfactory]**

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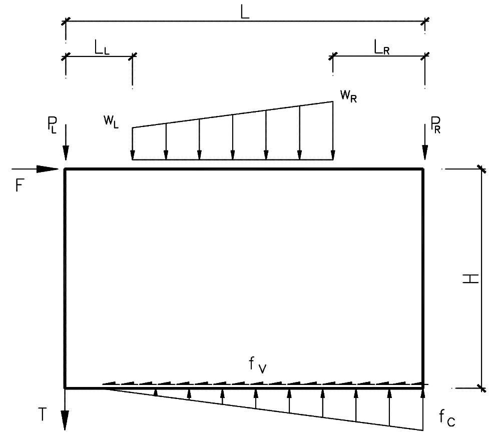
Shear Wall Design, using Laminated Bamboo, Based on NDS 2015



DESIGN CRITERIA

1. PLANE SECTIONS BEFORE BENDING REMAIN PLANAR AFTER BENDING.

   $f_c \;=\; \varepsilon_c\, E' \;\le\; F_b'$

2. THE TENSION CAPACITY IS ONLY FROM THE LAMINATED VERTICAL BAMBOO AREA CONNECTED HOLD-DOWN.

   $T \;=\; F_t'\,(A_{parallel}) \;\le$ *Hold-Down*

THE DESIGN IS ADEQUATE.

INPUT DATA & DESIGN SUMMARY

BAMBOO PANEL SIZE — H = **10** ft, (3048 mm); L = **16** ft, (4877 mm)

BAMBOO PANEL THICKNESS — t = **4** in, (102 mm)

DESIGN VALUES (*from ICC reports or local jurisdiction amendments to the code*)

F_b' = **10.15** ksi, ASD (70 N / mm^2)

F_v' = **3.19** ksi, ASD (22 N / mm^2)

E' = **1.74** ksi, ASD (12 N / mm^2)

Hold-Down Capacity = **13** kips, ASD level (57.82 kN)

F_t' = **13.05** ksi, ASD (90 N / mm^2), $A_{parallel}$ = **8.375** in^2 (5403 mm^2)

LOADS

F = **120** kips, ASD level (533.76 kN)

P_L = **250** kips, ASD level (1112.00 kN) P_R = **550** kips, ASD level (2446.40 kN)

w_L = **600** plf, ASD (8756 N / m) L_L = **2** ft, (610 mm)

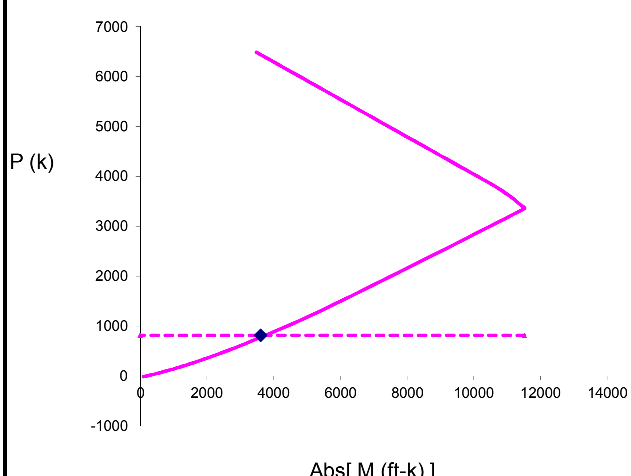
w_R = **1500** plf, ASD (21891 N / m) L_R = **3** ft, (914 mm)

ANALYSIS

BAMBOO PROPERTIES AND BOTTOM SECTION FORCES

G =	0.686	, *Specific gravity*	P =	813.83	*kips, ASD level*
γ =	43	lbs / ft^3	M =	3603.30	*ft-kips, ASD level*
Weight =	2283	*lbs, (1036 kg)*	V =	120.00	*kips, ASD level*

CHECK FLEXURAL & AXIAL CAPACITY BY ALLOWABLE STRESS DESIGN (ASD)



P = 813.83 *kips, ASD level*

M = 3603.30 *ft-kips* < *M (allowable)* = 3726.1 *ft-kips* **[Satisfactory]**

CHECK SHEAR CAPACITY (ASD)

V = 120.00 *kips* < *V (allowable)* = 1224.96 *kips* **[Satisfactory]**

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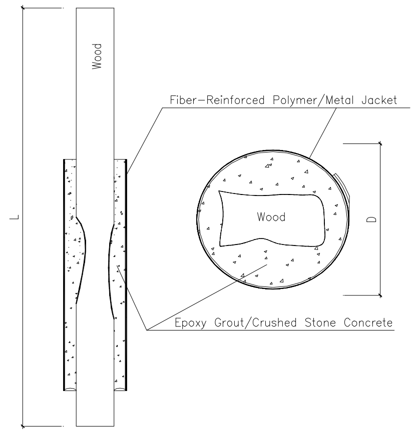
Wood Repair & Protection Design Based on 2016 CEBC, ASCE 41-13, ACI 318-14 & NDS 2015



INPUT DATA & DESIGN SUMMARY

WOOD SPECIES & GRADE — **No. 1, Douglas Fir-Larch**

WOOD LENGTH L = **15** ft

WOOD SECTION SIZE **8 x 10**

FIBER-REINFORCED POLYMER (FRP)/METAL F_y = **55** ksi, tension strength

EPOXY GROUT/CRUSHED STONE CONCRETE f_c' = **3** ksi, compression

MINUMINE FRP/METAL JACKET DIAMETER D = 15 in

EQUIVALENT FRP/METAL JACKET THICKNESS t = **0.0566** in

AXIAL COMPRESSION FORCE P = **10** kips, ASD

STRONG AXIS EFFECTIVE LENGTH $k_e L_x$ = **15** ft, (NDS Table G1/Figure 3F)

WEAK AXIS EFFECTIVE LENGTH $k_e L_y$ = **7.5** ft, (NDS Table G1)

STRONG AXIS BENDING MOMENT M_x = **5** ft-kips, ASD

BENDING EFFECTIVE LENGTH L_e = **8** ft, (NDS Table 3.3.3)

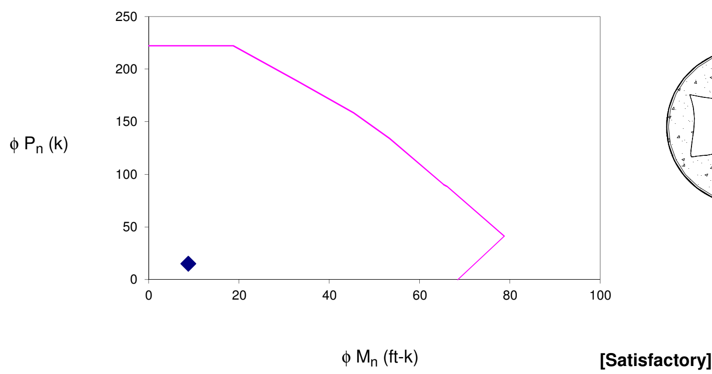
WEAK AXIS BENDING MOMENT M_y = **3** ft-kips, ASD

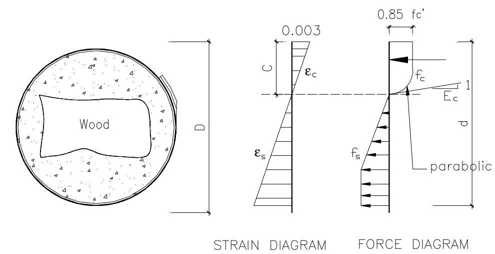
CRITICAL/MAXIMUM SECTION SHEAR FORCE V = **3** kips, ASD

THE REPAIR DESIGN IS ADEQUATE.

ANALYSIS

CHECK ENHANCED SECTION FLEXURAL & AXIAL CAPACITY (ACI 318-14 21 & 22)





$P_u = 1.5\,P =$ 15.0 kips, SD

$M_u = 1.5\,(M_x^{\,2} + M_y^{\,2})^{0.5} =$ 8.7 ft-k, SD

[Satisfactory]

DETERMINE WOOD DESIGN VALUES (NDS Supplement Tables or ICC ESR)

	F_b	F_v	F_c	$E_{x,min}$	$E_{y,min}$	b, (in)	d, (in)	A, (in^2)	I_x, (in^4)	S_x, (in^3)	I_y, (in^4)	S_y, (in^3)
psi ==>	1,350	170	925	580000	580000	7.5	9.5	71.3	535.9	112.8	334.0	89.1

CHECK WOOD COMPRESSION CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_c' = F_c\,C_D\,C_M\,C_t\,C_F\,C_i\,C_p =$ 686 psi

\> $f_c = P / A =$ 140 psi **[Satisfactory]**

Where	C_D	C_M	C_t	C_F	C_i	C_p	C_T
	0.90	**1.00**	**1.0**	1.00	**1.0**	0.82	**1.0**
	(Tab. 2.3.2)	(4.4.3/5.5.3)	(Tab. 2.3.3)	(NDS 4.3.6)	(NDS 4.3.8)		(NDS 4.4.2 for Truss only)

$C_P = (1{+}F) / 2c - [((1{+}F) / 2c)^2 - F / c]^{0.5}$,(NDS 3.7.1)

$F_c^* = F_c' / C_p =$ 833 psi

$L_e / d = Max(\,k_e L_x / d\,,\,k_e L_y / b) =$ 19 (NDS Table G1/Figure 3F)

< 50 , (NDS 3.7.1.4) **[Satisfactory]**

E'_{min} = 580000 psi, (Strong Axis Controls.)

$F_{cE} = 0.822\,E'_{min} / (L_e/d)^2 =$ 1328 psi

$F = F_{cE} / F_c^* =$ 1.595

c = 0.8

CHECK WOOD BENDING CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_{bx}' = F_b\,C_D\,C_M\,C_t\,C_L\,(C_F\,C_i\,C_r)\,(C_V\,C_c\,C_I) =$ 1213 psi

\> $f_{bx} = M_x / S_x =$ 532 psi **[Satisfactory]**

Where	C_D	C_L	C_F	C_r	C_V	C_c	C_I
	0.90	1.00	1.00	**1.00**	1.00	**1.00**	**1.00**
	(Tab. 2.3.2)		(NDS 4.3.6)	(NDS 4.3.9)	(NDS 5.3.6)	(NDS 5.3.8)	(NDS 5.3.9)

$C_L = (1{+}F) / 1.9 - [((1{+}F) / 1.9)^2 - F / 0.95]^{0.5}$,(NDS 3.3.3)

$F_b^* = F_{bx}' / (C_V\,C_L) =$ 1215 psi

$R_B = (\,L_e\,d / b^2\,)^{0.5} =$ 4 (NDS Eq. 3.3-5)

< 50 , (NDS 3.3.3.7) **[Satisfactory]**

(Cont'd)

E'_{min} = 580000 psi

$F_{bE} = 1.20\, E'_{min} / R_B^{\,2}$ = 42928 psi

$F = F_{bE} / F_b^*$ = 35

$F_{by}' = F_b\ C_D\ C_M\ C_t\ C_{fu}\ (C_F\ C_i\ C_r)\ (C_V\ C_c\ C_I)$ = 1247 psi

> $f_{by} = M_y / S_y$ = 404 psi **[Satisfactory]**

Where C_{fu}

1.03

(4.3.7/5.3.7)

CHECK WOOD BENDING & COMPRESSION CAPACITY (NDS 3.9.2)

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1 - f_c / F_{cEx}}\right)\frac{f_{bx}}{F_{bx}'} + \left(\frac{1}{1 - f_c / F_{cEy} - \left(f_{bx} / F_{bE}\right)^2}\right)\frac{f_{by}}{F_{by}'} = \quad 0.87 \quad < \quad 1.0$$ **[Satisfactory]**

Where $E'_{x,min}$ = 580000 psi

$E'_{y,min}$ = 580000 psi

$F_{cEx} = 0.822\, E'_{x,min} / (k_e L_x/d)^2$ = 1328 psi

$F_{cEy} = 0.822\, E'_{y,min} / (k_e L_y/b)^2$ = 3311 psi

$$\frac{f_c}{F_{cEy}'} + \left(\frac{f_{bx}}{F_{bE}'}\right)^2 = \quad 0.04 \quad < \quad 1.0$$ **[Satisfactory]**

CHECK WOOD SHEAR CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_v' = F_v\ C_D\ C_M\ C_t\ (C_i)\ (C_{vr})$ = 153 psi

> $f_v = 3\,V / (2\,A)$ = 63 psi **[Satisfactory]**

Where C_D C_{vr}

0.90 **1.0**

(Tab. 2.3.2) (NDS 5.3.10)

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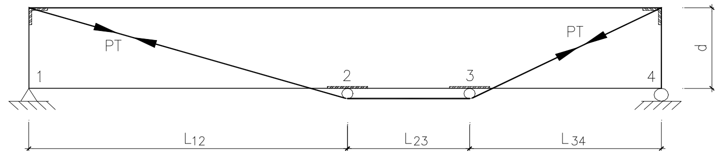
To Fix Sagging Beam, Using External Post-Tensioning Systems, Based on NDS 2015

INPUT DATA & DESIGN SUMMARY



SPECIES & GRADE **Timberstrand LSL 1.55E**

SECTION SIZE **LSL 3 1/2 x 8 5/8**

LENGTH L = **15** ft

L_{12} = **7** ft

L_{23} = **3** ft

L_{34} = 5 ft

THE MEMBER DESIGN IS ADEQUATE.

TENDON FORCE AFTER ALLOWANCE LOSSES

PT = 5 *kips*

PT up camber and section forces ==>

Δ_{max} =	**0.34**	*in @*	8.20	*ft, from left*
M_{max} =	1.78	*ft-kips @*	10.00	*ft, from left*
V_{max} =	0.71	*kips, section shear force*		
P_{max} =	5.00	*kips, section axial compression*		

EXISTING AXIAL LOAD (without PT) P_{Load} = **6** *kips, ASD (Total P = 11.00 kips, ASD)*

STRONG AXIS EFFECTIVE LENGTH $k_e L_x$ = **15** ft, (NDS Table G1/Figure 3F)

WEAK AXIS EFFECTIVE LENGTH $k_e L_y$ = **7.5** ft, (NDS Table G1)

EXISTING STRONG AXIS BENDING LOAD (without PT) M_{Load} = **2** *ft-kips, ASD (Design M_x = 2.00 ft-kips, ASD)*

BENDING EFFECTIVE LENGTH L_e = **8** ft, (NDS Table 3.3.3)

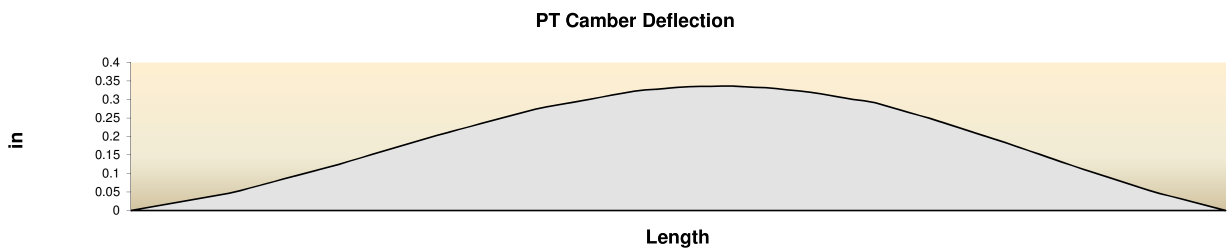
WEAK AXIS BENDING MOMENT M_y = *0.00* ft-kips, ASD, (tendon each side)

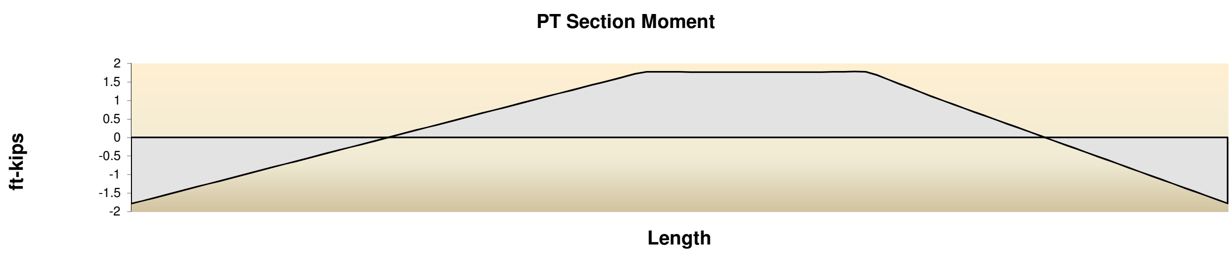
CRITICAL/MAXIMUM SECTION SHEAR FORCE V_{Load} = **3** *kips, ASD (Design V = 3.71 kips, ASD)*

ANALYSIS

DETERMINE DESIGN VALUES (NDS Supplement Tables or ICC ESR)

	F_b	F_v	F_c	$E_{x,min}$	$E_{y,min}$	b, (in)	d, (in)	A, (in^2)	I_x, (in^4)	S_x, (in^3)	I_y, (in^4)	S_y, (in^3)
psi ==>	2,325	525	2170	788000	788000	3.5	8.6	30.2	187.1	43.4	30.8	17.6





CHECK COMPRESSION CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_c' = F_c\, C_D\, C_M\, C_t\, C_F\, C_i\, C_p$ = 902 psi

> $f_c = P / A$ = 364 psi **[Satisfactory]**

Where	C_D	C_M	C_t	C_F	C_i	C_p	C_T
	0.90	**1.00**	**1.0**	1.00	**1.0**	0.46	**1.0**
	(Tab. 2.3.2)	(4.4.3/5.5.3)	(Tab. 2.3.3)	(NDS 4.3.6)	(NDS 4.3.8)		(NDS 4.4.2 for Truss only)

$C_P = (1+F) / 2c - [((1+F) / 2c)^2 - F / c]^{0.5}$,(NDS 3.7.1)

$F_c^* = F_c' / C_p$ = 1953 psi

$L_e / d = Max(k_e L_x / d, k_e L_y / b)$ = 26 (NDS Table G1/Figure 3F)

< 50 , (NDS 3.7.1.4) **[Satisfactory]**

(Cont'd)

E'_{min} = 788000 psi, (Weak Axis Controls.)

$F_{cE} = 0.822\, E'_{min} / (L_e/d)^2 =$ 980 psi

F = F_{cE} / F_c^* = 0.502

c = 0.9

CHECK BENDING CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_{bx}' = F_b\, C_D\, C_M\, C_t\, C_L\, (C_F\, C_i\, C_r)\, (C_V\, C_c\, C_I) =$ 2074 psi

> $f_{bx} = M_x / S_x =$ 553 psi **[Satisfactory]**

Where	C_D	C_L	C_F	C_r	C_V	C_c	C_I
	0.90	0.99	1.00	**1.00**	1.00	**1.00**	**1.00**
	(Tab. 2.3.2)		(NDS 4.3.6)	(NDS 4.3.9)	(NDS 5.3.6)	(NDS 5.3.8)	(NDS 5.3.9)

$C_L = (1+F) / 1.9 - [((1+F) / 1.9)^2 - F / 0.95]^{0.5}$,(NDS 3.3.3)

$F_b^* = F_{bx}' / (C_V\, C_L) =$ 2093 psi

$R_B = (L_e\, d / b^2)^{0.5} =$ 8 (NDS Eq. 3.3-5)

< 50 , (NDS 3.3.3.7) **[Satisfactory]**

E'_{min} = 788000 psi

$F_{bE} = 1.20\, E'_{min} / R_B^2 =$ 13990 psi

F = F_{bE} / F_b^* = 7

$F_{by}' = F_b\, C_D\, C_M\, C_t\, C_{fu}\, (C_F\, C_i\, C_r)\, (C_V\, C_c\, C_I) =$ 2171 psi

> $f_{by} = M_y / S_y =$ 1 psi **[Satisfactory]**

Where	C_{fu}
	1.04
	(4.3.7/5.3.7)

CHECK BENDING & COMPRESSION CAPACITY (NDS 3.9.2)

$$\left(\frac{f_c}{F'_c}\right)^2 + \left(\frac{1}{1 - f_c / F_{cEx}}\right)\frac{f_{bx}}{F'_{bx}} + \left(\frac{1}{1 - f_c / F_{cEy} - \left(f_{bx} / F_{bE}\right)^2}\right)\frac{f_{by}}{F'_{by}} = \quad 0.52 \quad < \quad 1.0$$ **[Satisfactory]**

Where $E'_{x,min}$ = 788000 psi

$E'_{y,min}$ = 788000 psi

$F_{cEx} = 0.822\, E'_{x,min} / (k_e L_x / d)^2 =$ 1487 psi

$F_{cEy} = 0.822\, E'_{y,min} / (k_e L_y / b)^2 =$ 980 psi

$$\frac{f_c}{F'_{cEy}} + \left(\frac{f_{bx}}{F'_{bE}}\right)^2 = \quad 0.39 \quad < \quad 1.0$$ **[Satisfactory]**

CHECK SHEAR CAPACITY (NDS Table 4.3.1 & Table 5.3.1)

$F_v' = F_v\, C_D\, C_M\, C_t\, (C_i)\, (C_{vr}) =$ 473 psi

> $f_v = 3\, V / (2\, A) =$ 184 psi **[Satisfactory]**

Where	C_D	C_{vr}
	0.90	**1.0**
	(Tab. 2.3.2)	(NDS 5.3.10)

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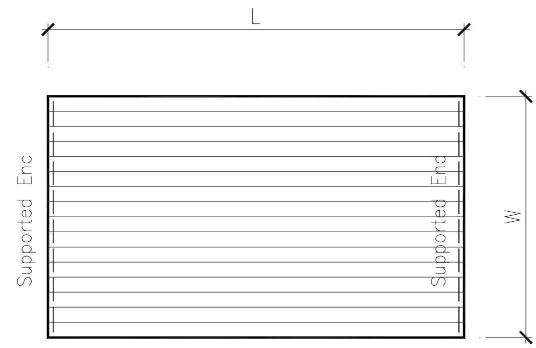
Mechanically Laminated Decking Design Based on 2016 CBC/2015 IBC 2304.9.3

INPUT DATA & DESIGN SUMMARY

NAIL-LAMINATED TIMBER SIZE 2 x 10 No. 1, Douglas Fir-Larch

ONE WAY SPAN L = **12** *ft, (3658 mm)*

DECKING WIDTH W = **8** *ft, (2438 mm)*



Code	Designation
1	Select Structural, Douglas Fir-Larch
2	No. 1, Douglas Fir-Larch
3	No. 2, Douglas Fir-Larch
4	Select Structural, Southern Pine
5	No. 1, Southern Pine
6	No. 2, Southern Pine

Choice => **2**

ALLOWABLE DEFLECTION $\Delta_{Kcr\ D+L}$ = L / **240**

UNIFORM LOAD (Including Wt., perpendicular to Plane)

w = **80** *psf, ASD level* *(3831 N / m^2)*

MOVEABLE POINT LOAD (Including Impact Factor)

P = **0.6** *kips, ASD level* *(2.67 kN)*

THE DECKING DESIGN IS ADEQUATE.

(Total: 65 - 2 x 10 - Long 12 ft)

ANALYSIS

DETERMINE REACTIONS, MOMENT, SHEAR

$R_{Support}$ = 1.08 kips / ft, (CBC/IBC Table 1607.1 Note a.)

V_{Max} = 1.08 kips / ft

M_{Max} = 10.44 ft-kips / ft

DETERMINE SECTION PROPERTIES & ALLOWABLE STRESSES (Single Timber)

b =	1.50 in	E'_{min} =	N/A	$E = E_x$ =	1700 ksi	F_b^* =	N/A
d =	9.25 in	F_{bE} =	N/A	F_b =	1,000 psi	$F = F_{bE} / F_b^*$ =	N/A
A =	13.9 in^2	I =	99 in^4	F_v =	180 psi	F_b' =	1,100 psi
S_x =	21.4 in^3	R_B =	N/A	E' =	1,700 ksi	F_v' =	180 psi
I_E =	N/A						

C_D	C_M	C_t	C_i	C_L	C_F	C_V	C_c	C_r
1.00	1.00	1.00	1.00	1.00	1.10	1.00	1.00	1.00

CHECK BENDING AND SHEAR CAPACITIES

$f_b = M_{Max} / S_x$ = 732 psi < F_b = 1100 psi **[Satisfactory]**

$f_v' = 1.5\ V_{Max} / A$ = 15 psi < F_v' **[Satisfactory]**

CHECK DEFLECTIONS

$\Delta_{(Kcr\ D+L,\ Max)}$ = 0.58 in, at middle span. < $\Delta_{Kcr\ D+L}$ = L / 240 **[Satisfactory]**

Where K_{cr} = 1.50 , (NDS 3.5.2)

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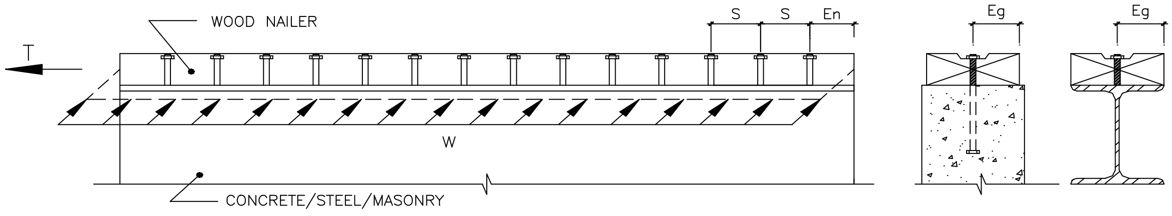
Sill Plate/Nailer Connection Design Based on NDS 2018

DESIGN CRITERIA

1. There are three connection forces from wood to concrete/steel/masonry:
   - (1). Vertical/perpendicular force, which can be uplift or downward. The all uplift/tear force has to fully be supported by end holdown or steel ties. Wood sill plate/nailer cannot transfer uplift/tear force. The downward force is compression on connection surface, which no capacity issues.
   - (2). In plan shear force, which is from wood-frame diaphragm, supported by sill plate/nailer.
   - (3). Out of plan uniformly distributed load, which may be from wind/lateral loads, supported by sill plate/nailer.
2. Since concrete/steel/masonry is stronger than wood, wood capacities of sill plate/nailer always control the connection design, if anchor bolts reached the minimum requirements based on ACI 318-14 or TMS 402-16.



INPUT DATA & DESIGN SUMMARY

OUT OF PLAN UNIFORMLY DISTRIBUTED LOAD (ASD)	w =	180	plf
IN PLAN SHEAR FORCE (ASD)	T =	8	k
NUMBER OF BOLTS	n =	6	
THREAD BOLTS DIAMETER	ϕ =	3/4	
THREAD BOLTS SPACING	S =	12	in
END DISTANCE OF WOOD	E_n =	4	in
NAILER TYPE (0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir)		0	Douglas Fir-Larch, G=0.5
LEDGER GRADE (0=Select Structural, 1=No.1 & Btr, 2=No.1, 3=No.2, 4=No.3, 5=Stud, 6=Construction, 7=Standard, 8=Utility)		2	No.1
NAILER SIZE	4 thk. x	6	width
SECTION AREA OF SIDE MEMBER (Concrete/Steel/Masonry)	A_s =	11.8	in^2
MODULUS OF SIDE MEMBER (Concrete/Steel/Masonry)	E_s =	29000	ksi
LOAD DURATION FACTOR (Tab 2.3.2, NDS 2018)	C_D =	1.6	
WET SERVICE FACTOR (Tab 10.3.3, NDS 2018)	C_M =	1.0	
TEMPERATURE FACTOR (Tab 10.3.4, NDS 2018)	C_t =	1.0	
GROUP ACTION FACTOR APPLY? (0 = No, 1 = Yes)		1	Yes, (C_g = 0.719 , NDS 2018, 11.3.6)

(If T is drag/collector force or bolt spacing less than 12" o.c., the group action factor C_g must apply.)

THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

CHECK EDGE, END, & SPACING DISTANCE REQUIREMENTS (NDS 2018, Table 12.5.1A, Table 12.5.1B, & Table 12.5.1C)

E_g =	2.75	in	>	1.5 D	[Satisfactory]
E_n =	4	in	>	3.5 D	[Satisfactory]
S =	12	in	>	3 D	[Satisfactory]

CHECK WOOD CAPACITY

$C_\Delta = \mathrm{Min}\ (C_{\Delta 1}, C_{\Delta 2}, C_{\Delta 3})$ = 0.762 , (geometry factor, NDS 2018, 12.5.1)

where $C_{\Delta 1}$ = (actual end distance) / (min end distance for full design value) = E_n / 7D = 0.762

$C_{\Delta 2}$ = (actual shear area) / (min shear area for full design value) = 1.000

$C_{\Delta 3}$ = (actual spacing) / (min spacing for full design value) = S / 4D = 4.000

$$C_g = \left[\frac{m\left(1-m^{2n}\right)}{n\left[\left(1+R_{EA}m^{n}\right)(1+m)-1+m^{2n}\right]}\right]\left[\frac{1+R_{EA}}{1-m}\right] =$$ 0.719 , (group action factor, NDS 2018, 11.3.6)

where n = 6

E_sA_s = 3.42E+08 lbs

t_m = 3.5 in

E_mA_m = 26950000 lbs, (E_m fr NDS, Tab.10.3.6C)

$R_{EA} = \mathrm{Min}\ [(E_sA_s/E_mA_m), (E_mA_m/E_sA_s)]$ = 0.079

$\gamma = 180000\ D^{1.5}$ = 116913.4

$u = 1+\gamma\ S/2\ [1 / E_mA_m + 1 / E_sA_s]$ = 1.028

$m = u - (u^2 - 1)^{0.5}$ = 0.789

(cont'd)

$Z'_{//}$ =	$n Z_{//} C_D C_M C_t C_g C_\Delta$ =	8.629	kips	**>**	T		[Satisfactory]	
where $Z_{//}$ =	1640	lbs / bolt, (interpolated from Tab 12E, NDS 2018)						
$Z_\perp'$ =	$n Z_\perp C_D C_M C_t C_g C_\Delta$ =	4.630	kips	**>**	Σ w =	1.080	kips	[Satisfactory]
where $Z_\perp$ =	880	lbs / bolt, (interpolated from Tab 12E, NDS 2018)						
Z_θ' =	$Z_\perp' Z_{\parallel}' / (Z_\perp' \cos^2\theta + Z_{\parallel}' \sin^2\theta)$ =	8.498	kips	**>**	$(T^2 + \Sigma w^2)^{0.5}$ =	8.073	kips	[Satisfactory]
where θ =	7.69	deg, (NDS 2018 12.4.1)						

Technical References:

1. "National Design Specification, NDS", 2018 Edition, AF&AP, AWC, 2018.

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Flitch Plate Beam Design Based on AISC 360-16 & NDS 2018

DESIGN CRITERIA

1. Assume that the steel plate and wood members are full composite, by Wood Bolt Connection/Nailers, to support bending loads.
2. Since there are no building code sections to check if Wood Bolt Connection/Nailers adequate or not, the bolts number spacing have to be determined by test or ICC report, and not less than 3/4" ϕ @ 12" O.C., Each Way.

INPUT DATA & DESIGN SUMMARY

WOOD SPECIES & GRADE: **Timberstrand LSL 1.55E**

BEAM LENGTH L = **30** ft

THE BEAM DESIGN IS ADEQUATE.

WOOD SECTION SIZE **2** x **LSL 3 1/2 x 8 5/8**

STEEL SECTION SIZE **1** x **1** in, (*Thickness*) x 8.6 in, (*Depth*)

STEEL YIELD STRESS F_y = **50** *ksi, (345 MPa)*

STRONG AXIS BENDING MOMENT M = **60** ft-kips, ASD

BENDING EFFECTIVE LENGTH L_e = **1** ft, *(If lateral supported by top diaphragm input bolt spacing. NDS Tab 3.3.3 & AISC 360 F11)*

CRITICAL/MAXIMUM SECTION SHEAR FORCE V = **100** kips, ASD

ANALYSIS

DETERMINE DESIGN VALUES OF SINGLE WOOD MEMBER (NDS Supplement Tables or ICC ESR)

	F_b	F_v	F_c	$E_{x,min}$	$E_{y,min}$	b, (in)	d, (in)	A, (in^2)	I_x, (in^4)	S_x, (in^3)	I_y, (in^4)	S_y, (in^3)
psi ==>	2,325	525	2170	788000	788000	3.5	8.6	30.2	187.1	43.4	30.8	17.6

DETERMINE BENDING CAPACITY OF TOTAL WOOD MEMBER (NDS Table 4.3.1 & Table 5.3.1)

$F_{bx}' = F_b\, C_D\, C_M\, C_t\, C_L\, (C_F\, C_i\, C_r)\,(C_V\, C_c\, C_I) =$ 2091 psi

Where	C_D	C_L	C_F	C_r	C_V	C_c	C_I
	0.90	1.00	1.00	**1.00**	1.00	**1.00**	**1.00**
	(Tab. 2.3.2)		(NDS 4.3.6)	(NDS 4.3.9)	(NDS 5.3.6)	(NDS 5.3.8)	(NDS 5.3.9)

$C_L = (1+F) / 1.9 - [((1+F) / 1.9)^2 - F / 0.95]^{0.5}$,(NDS 3.3.3)

$F_b^* = F_{bx}' / (C_V\, C_L) =$ 2093 psi

$R_B = (L_e\, d / b^2)^{0.5} =$ 3 (NDS Eq. 3.3-5)

$<$ 50 , (NDS 3.3.3.7) **[Satisfactory]**

E'_{min} = 788000 psi

$F_{bE} = 1.20\, E'_{min} / R_B^{\,2} =$ 111919 psi

$F = F_{bE} / F_b^* =$ 53

$M_{x,wood} = \Sigma F_{bx}' S_x =$ 15.1 ft-kips

DETERMINE BENDING CAPACITY OF TOTAL STEEL MEMBER (AISC 360 F11)

$$\frac{M_n}{\Omega_b} = \Sigma \frac{1}{\Omega_b} \begin{cases} M_p \;,\; for\; \frac{L_b d}{t^2} \le \frac{0.08E}{F_y} \\ Min\left(C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \;,\; M_p\right) \;,\; for\; \frac{0.08E}{F_y} < \frac{L_b d}{t^2} \le \frac{1.9E}{F_y} \\ Min\left(F_{cr} S_x \;,\; M_p\right) \;,\; for\; \frac{L_b d}{t^2} > \frac{1.9E}{F_y} \end{cases} = 45.5 \text{ ft-kips}$$

Where $M_p = Min(F_y\, Z\, ,\, 1.6\, M_y) =$ 77.5 ft-kips; C_b = 1.0 , (AISC 360 F1)

$Z = t\, d^2 / 4 =$ 18.6 in^3; $S_x = t\, d^2 / 6 =$ 12.4

$M_y = F_y\, t\, d^2 / 6 =$ 51.7 ft-kips; Ω_b = 1.67 , (AISC 360 F1)

$F_{cr} = Min[1.9\, E\, C_b\, t^2 / (d\, L_b)\, ,\, F_y] =$ 50.0 ksi; E = 29000 ksi

CHECK BENDING CAPACITY

$M_{x,wood} + M_n / \Omega_b =$ 60.6 ft-kips $>$ M = 60.0 ft-kips **[Satisfactory]**

CHECK SHEAR CAPACITY (by Steel only, AISC 360 G2)

$\Sigma V_{ny} / \Omega_v =$ 288.1 / 1.67 = 172.5 kips $>$ V = 100.0 kips **[Satisfactory]**

Purchaser's LOGO
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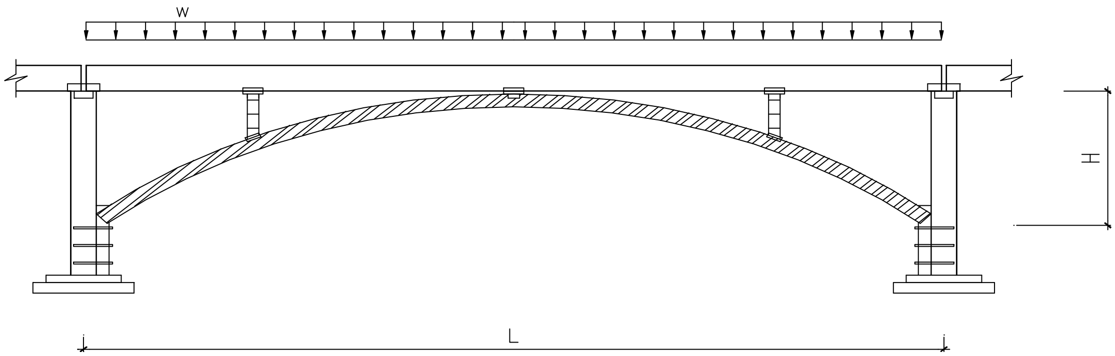
REVIEW BY :

To Fix Sagging Girder, by Bent HSS Tube Arch, Based on NDS 2018 & AISC 360-16

DESIGN CRITERIA

1. The most wood girder/beam/floor sagging is from wood stiffness less, not bending capacity issues. If the passing space under the girder limits, added new arch structure is good enhancing method to increase existing girder stiffness, because arch is axial force member.
2. The new added arch can be flat bent HSS tube, and it not have to be in the same plane with girder. If the girder still in service during repairing, two load cases should be checked: one is the service live load added in w_E before arch, another only in w_N.

INPUT DATA



DIMENSION

L = 24 ft, (7.32 m)

H = 10 ft, (3.05 m)

Section	E (ksi)	A (in^2)	I_x (in^4, in plane)	*E (MPa)*	*A (cm^2)*	*I_x (cm^4, in plane)*
Existing Wood Girder	930	121.5	3280.5	*6412*	*784*	*136545*
New Bent Steel Arch	29000	4.78	6.25	*199948*	*31*	*260*
New Vertical Cripple	1700	19.25	0	*11721*	*124*	*0*

(Note: Input cripple A zero, if arch out of plane and only connected, clip & shim, at middle top.)

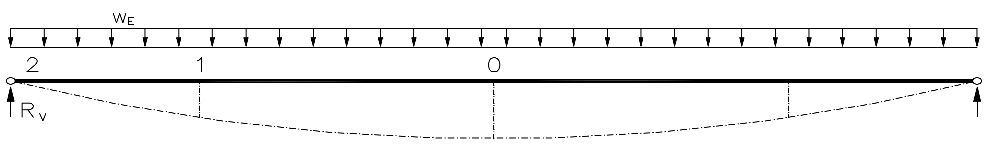
EXISTING GIRDER LOAD BEFORE ARCH

w_E = 8.125 *kips/ft, (118.5 kN/m), (including existing girder weight and impact factor.)*

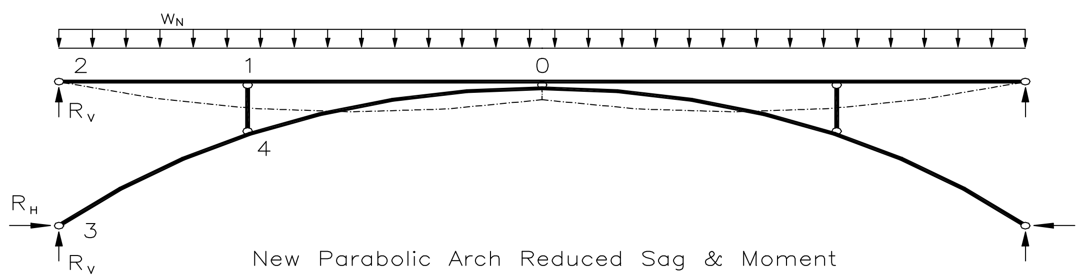
TOTAL GIRDER LOAD AFTER ARCH

w_N = 12.8 *kips/ft, (186.7 kN/m), (including all girder weight and impact factor.)*

ANALYSIS & DESIGN SUMMARY



Existing Girder Section Sag & Moment Before Arch



Existing Girder, before Arch Structure, for load w_E

Element	Design Section Forces		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)
0 - 1	0	585.00	48.75
1 - 2	0	438.75	97.5

Existing Girder, before Arch Structure, for load w_E

Joint	Reaction & Displacements			
	R_H (kips)	R_V (kips)	Δ_x (in)	Δ_y (in)
0			0	19.881
1			0.000	14.165
2	0	97.5	0.000	0.000

Enhanced Girder, after Arch Structure, for load w_N

Element	Design Section Forces by FEM		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)
0 - 1	0.000	0.858	0.228
1 - 2	0.000	0.507	0.085
0 - 4	99.740	1.014	0.288
1 - 4	76.488	0.000	0.000
3 - 4	147.335	1.014	0.106

Enhanced Girder, after Arch Structure, for load w_N

Joint	Reactions & Displacements by FEM			
	R_H (kips)	R_V (kips)	Δ_x (in)	Δ_y (in)
0			0	0.376
1			0.000	0.190
2	0	38.4845	0.000	0
3	91.95719	115.1155	0	0
4			0.046	0.120

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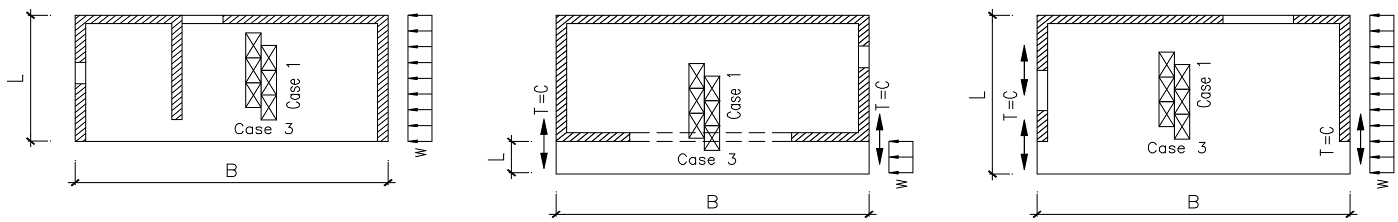
Cantilever Wood Diaphragm Design Based on NDS 2018

INPUT DATA & DESIGN SUMMARY



FRAMING OF *Douglas-Fir-Larch* OR *Southern Pine* ? Yes

THE DIAPHRAGM DESIGN IS ADEQUATE.

LATERAL FORCE ON CANTILEVER DIAPHRAGM: w_{WIND} = 256 plf, ASD

$w_{SEISMIC}$ = 343 plf, ASD

DIMENSIONS: B = 240 ft , L = 36 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL FRAMING WITH (2 or 3) = 3 in

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) = 1 8d

SPECIFIC GRAVITY OF FRAMING MEMBERS 0.43

BLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 6 in O.C. BOUNDARY / 6 in O.C. EDGES / 12"O.C. FIELD.

THE CHORD FORCES: ***T = C =*** 2.78 ***kips, ASD***

THE MAXIMUM DIAPHRAGM DEFLECTION:

Δ = 0.50 ***in***

ANALYSIS

CHECK DIAPHRAGM RATIO (SDPWS-15 4.2.5 & 4.2.7)

L / B = 0.15 < 1.0 / 1.0 **[Satisfactory]**

L = 36.00 < 37.5 ft **[Satisfactory]**

DETERMINE DIAPHRAGM SHEAR STRESS

v_{Max} = 51 plf, (Boundary Spacing = 6 in, Edges ReqD = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.2(1) / SDPWS-15 Table 4.2A with ASD reduction factor 2.0 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Member Width (in)	Blocked Nail Spacing Boundary / Other Edges				Unblocked	
					6 / 6	4 / 6	2.5 / 4	2 / 3	Case 1	Others
Sheathing and Single-Floor	8d	1 1/2	15/32	3	300	400	600	675	265	200

Note: The indicated shear numbers have reduced by specific gravity factor per SDPWS-15 Table 4.2A note 2.

DETERMINE CHORD/DRAG STRUT FORCE (SDPWS-15 4.2.5, ASCE 7-16 12.3, 12.8 & Table 12.2-1):

$T = C = \mathrm{Max}\,(w_{WIND},\ \Omega_0\, v_{SEISMIC})\, L^2 / (\,2B\,)$ = 2.78 kips, ASD

where Ω_0 = 3.0

DETERMINE DIAPHRAGM DEFLECTION: (IBC 2305.2 , / SDPWS-15 4.2.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ \ slip} + \Delta_{Chord\ \ splice\ \ slip} = \frac{5v(2L)^3}{8EAB} + \frac{v(2L)}{4Gt} + 0.188(2L)e_n + \frac{\Sigma(D_C x)}{2B} = 0.497 \text{ in, ASD}$$

Where: v = 51 plf; 2 L = 72 ft; E = 1.7E+06 psi

A = 21.75 in^2; B = 240 ft; G = 9.0E+04 psi,(UBC97 Page3-421)

t = 0.298 in,(UBC97 Page3-420); e_n = 0.041 in, SD; $\Sigma(D_c x)$ = 45.00 in, SD; C_M = 1.0

Note: The deflection, Δ, above is based on completely blocked. For unblocked diaphragm, 2.4Δ should be used.

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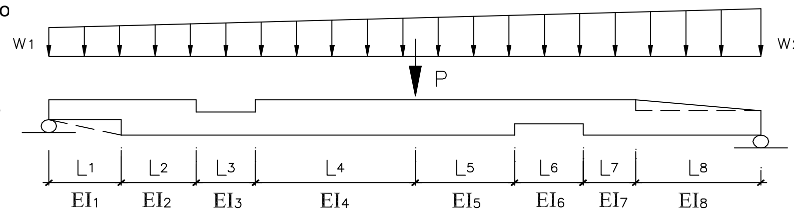
Notching Design for Wood and Steel Beam Based on 2018 IBC, NDS 2018, & AISC 360-16

DESIGN CRITERIA

1. The most notching beams are wood or steel. For notching beam o wood or steel, the deflections should govern the design.
2. The current notching limitations (2018 IBC 2308.7.4, NDS 2018 3.4.3 & 4.4.3, AISC 360) are based on full section design. But this software are based on the reduced notching sections to check the beam deflections.



INPUT DATA & DESIGN SUMMARY

DIMENSIONS, SECTIONS (THE MOMENT OF INERTIA)

Section	1	2	3	4	5	6	7	8
L (in)	**24**	**26**	**13**	**20**	**28**	**5**	**10**	**18**
I (in^4)	**1050**	**2100**	**1260**	**2100**	**2100**	**945**	**2100**	**2100**

L = 12.00 ft
E = **580** ksi

LOADS (ASD)
w_1 = **550** lbs / ft
w_2 = **900** lbs / ft
P = **3500** lbs

Design Section Forces (ASD level):

	Full Section	***Notching Section***
P *(kips)*	0	0
M *(k-ft)*	23.16	20.50
V *(kips)*	6.72	5.48
I *(in^4)*	2100.00	945.00
L *(ft)*	12.00	12.00

ANALYSIS

M_{max} = ***23.16*** *ft-kips, (bending)* *,from left* ***6.92*** *ft*

V_{max} = ***6.72*** *kips, (shear)* *,from left* ***12.00*** *ft*

Δ_{max} = ***0.52*** *in* *,from left* ***5.92*** *ft*

< *L /* ***240*** **=** 0.60 in

[Satisfactory]

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	JOB NO. :	DATE :	REVIEW BY :	

Tudor Arches Design Using Finite Element Method in Structural Mechanics

DESIGN CRITERIA

1. The arch can be not only tudor arch, but also the arches of radial, reverse curve, gothic, parabolic, or A-frame (just change yellow cell locations). For wood or steel arch, the STRONG AXIS EFFECTIVE LENGTH, to check all sections capacity, should be the half curved length.
2. The loads w_1, w_2, w_3, and w_4 may be negative (outward) based on IBC/CBC 1605 wind/seismic load combination results.

INPUT DATA & DESIGN SUMMARY

LOADS

w_1 = **0.3** *kips / ft, (4.4 kN / m)*

w_2 = **0.5** *kips / ft, (7.3 kN / m)*

w_3 = **0.2** *kips / ft, (2.9 kN / m)*

w_4 = **-0.5** *kips / ft, (-7.3 kN / m)*

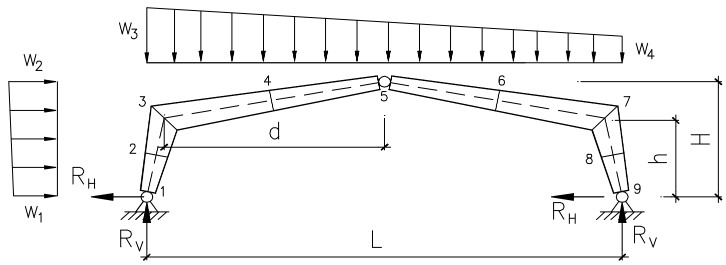
DIMENSIONS

H = **24** *ft, (7.32 m)*

L = **50** *ft, (15.24 m)*

h = **12** *ft, (3.66 m)*

d = **23.5** *ft, (7.16 m)*



E = **1800** ksi

Joint / Section	Location X (ft)	Location Y (ft)	Drift (in) Δx	Drift (in) Δy	Section Depth (in)
1	**0**	**0**	**0**	**0**	**14.5**
2	**0.75**	**6**	-0.06	0.01	**38.425**
3	1.50	12.00	-0.22	0.03	**58**
4	**13.25**	**18**	-0.42	0.43	**36.57**
5	25.00	24.00	-0.59	0.76	**11**
6	36.75	18.00	-0.72	0.52	36.57
7	48.50	12.00	-0.96	0.05	58.00
8	49.25	6.00	-1.19	0.02	38.43
9	50.00	0.00	-0.03	0.00	14.50

Governing Design Section and Forces:

1. Length =	38.48	ft	*N =*	698.87	*kips*
Depth =	14.50	in	*V =*	523.23	*kips*
Width =	**6.75**	in	*M =*	0.00	*ft-kips*
2. Length =	38.48	ft	*N =*	698.87	*kips*
Width =	6.75	in	*V =*	523.23	*kips*
			M =	1647.07	*ft-kips*

No. Element	Joint Start, ***I***	Joint End, ***J***	***I*** Sec. Forces (kips, ft-kips) Axial	Shear	Moment	***J*** Sec. Forces (kips, ft-kips) Axial	Shear	Moment
1	1	2	-3.28	-3.48	0.00	3.28	3.48	-53.09
2	2	3	-3.16	-1.38	53.21	3.16	1.38	-61.56
3	3	4	-1.18	3.71	59.30	1.18	-3.71	-10.31
4	4	5	1.15	5.09	11.84	-1.15	-5.09	0.00
5	5	6	5.30	-0.18	0.00	-5.30	0.18	22.56
6	6	7	3.62	-3.47	-21.14	-3.62	3.47	-24.62
7	7	8	-3.57	-5.28	17.10	3.57	5.28	-49.05
8	8	9	-698.87	523.23	1647.07	698.87	-523.23	0.00

$R_{H,1}$ = 4.80 *kips*

$R_{V,1}$ = -2.75 *kips*

$R_{H,9}$ = 4.80 *kips*

$R_{V,9}$ = -4.75 *kips*

RELEASE HORIZONTAL ARCH ACTION FROM VERTICAL LOADS? (1 = Yes, 2 = No) **1** *Yes, released.*

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Wall of Cross-Laminated Timber (CLT) Design, for Perpendicular to Plane Loads, Based on NDS 2018

INPUT DATA & DESIGN SUMMARY

SPAN DIMENSION H = **18** ft, (5486 mm)

CLT PANEL THICKNESS t = **8** in, (203 mm)

UNIFORM LOAD (Perpendicular to Plane) w = **53.2** psf, ASD level (2547 N / m^2)

POINT LOAD (Including Impact Factor) F = **2** kips, ASD level (8.90 kN)

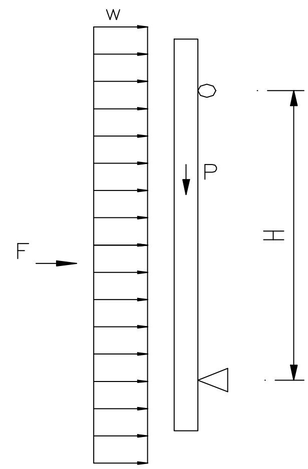
AXIAL LOAD (Including Plane Wt.) P = **8** kips, ASD level (35.58 kN)

DESIGN VALUES (NDS 10.2) F_b' = **2.1** ksi, ASD (14 N / mm^2)

F_v' = **0.5** ksi, ASD (3 N / mm^2)

E' = **900** ksi, ASD (6207 N / mm^2)

ALLOWABLE DEFLECTION H / **240**



THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK AXIAL & BENDING CAPACITY

$f_b = (H^2 w\, d / 8 + F\, H / 4) / (d\, t^2 / 6) + P / (d\, t)$ = 0.67 ksi, (possible max axial stress)

< F_b' **[Satisfactory]**

Where d = 24 in, (effective horizontal width for F & P)

CHECK SHEAR CAPACITY

$f_v = (0.5\, H\, w) / t + F / (d\, t)$ = 0.02 ksi, (possible max shear stress)

< F_v' **[Satisfactory]**

CHECK DEFLECTION

$\Delta_{max}\, K_{cr} = [5\ (H^4 w\, d) / (384\, E'\, I) + (H^3 F) / (48\, E'\, I)]\, K_{cr}$ = 0.70 in, (possible max deflection)

< H / 240 = 0.90 in

[Satisfactory]

Where $I = d\, t^3 / 12$ = 1024.0 in^4 K_{cr} = 1.50 , (NDS 3.5.2)

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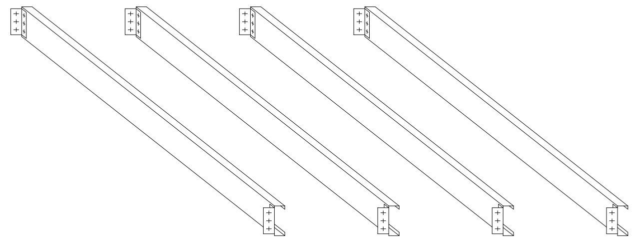
Joist Design Based on AISI S100-2007 & ICBO ER-4943P



INPUT DATA & DESIGN SUMMARY

SECTION & SPACING 600S162-54 @ 24 in o.c
(50 ksi)

JOIST SPAN L = 11.5 ft

DEAD LOAD DL = 20 psf
LIVE LOAD LL = 20 psf

LATERAL SUPPORTED ? 1 compression flange
(0=No, 1=compression flange, 2=tension flange)
DEFLECTION LIMITATION FOR LIVE LOAD ? 1 L /240
(0=No., 1= L /240, 2= L /360, 3= L /180, 4= L /120)

THE DESIGN IS ADEQUATE.

ANALYSIS

SECTION PROPERTIES OF EACH METAL STUD (SSMA page 7 & 8)

t =	0.0566	in	F_y =	50	ksi	I_{xx} =	2.86	in^4	M_n/Ω_b =	27.76	in-kips
h =	6	in	Wt =	1.89	lb/ft	S_{xx} =	0.927	in^3	V_n/Ω_v =	2708	lbs
A =	0.556	in^2	r_x =	2.267	in	r_y =	0.57	in	x_o =	-1.072	in
J =	0.000594	in^4	C_w =	1.318	in^6						

CHECK MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)

h / t = 106.01 **<** 200 **[Satisfactory]**

CHECK FLEXURAL CAPACITY (AISI C3.1)

M_n/Ω_b = 27.76 in-kips **>** **M** **[Satisfactory]**

Where M = [(DL+LL) S + Wt] L^2 / 8 = 16.24 in-kips
S = 24 in o.c., spacing as given.

CHECK SHEAR CAPACITY (AISI C3.2)

V_n/Ω_V = 2708 lbs **>** **V** **[Satisfactory]**

Where V = [(DL+LL) S + Wt] L / 2 = 471 lbs

CHECK LATERAL-TORSIONAL BUCKLING (AISI C3.1.2)

CASE 1: BOTH TOP & BOTTOM FLANGES UNSUPPORTED **<== Does not apply.**

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} = 9.0 \text{ ksi} \quad < \quad 2.78 F_y = 139.0 \text{ ksi}$$
$$< \quad 0.56 F_y = 28.0 \text{ ksi}$$

Where C_b = 1.0
$r_o = (r_x^2 + r_y^2 + x_o^2)^{0.5}$ = 2.572 in
S_f = 0.95 in^3
E = 29500 ksi (AISI pg xiv)
G = 11300 ksi (AISI pg xvi)
K_y = 1.0
K_t = 1.0
L_y = 138 in
L_t = 138 in

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} = 4.967 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{\left(K_t L_t\right)^2}\right] = 7.305 \text{ ksi}$$

$$F_c = \begin{cases} F_y \text{ , } for \ F_e \ge 2.78 F_e \\ \frac{10}{9} F_y\left(1 - \frac{10 F_y}{36 F_e}\right) \text{ , } for \ 2.78 > F_e \ge 0.56 F_e \\ F_e \text{ , } for \ F_e \le 0.56 F_e \end{cases} = 9.0 \text{ ksi}$$

(cont'd)

M_n/Ω_b = 5.16 in-kips **<** **M** **[Satisfactory]**

Where S_c = 0.95 in^3 from SSMA page 7 & 8

Ω_b = 1.67

M_n = S_cF_c = 8.61 in-kips

M = [(DL+LL) S/12 + Wt] L^2 / 8 = 16.24 in-kips

CASE 2: BOTTOM FLANGE SUPPORTED ONLY **<== Does not apply.**

M_n/Ω_b = 19.43 in-kips **>** **M** **[Satisfactory]**

Where S_e = 0.93 in^3, from S_{xx}

Ω_b = 1.67

R = 0.70 (AISI Table D6.1.1-1)

M_n = RS_eF_y = 32.45 in-kips

CHECK CAPACITY COMBINED BENDING & SHEAR AT ANY SAME SECTION (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2} = 0.6105 < 1.0$$ **[Satisfactory]**

Where M = 16.24 in-kips
V = 471 lbs

V_n/Ω_v = 2708 lbs

Ω_b = 1.67

M_n = MIN(Bending , Buckling) = 46.359 in-kips

M_n/Ω_b = 27.76 in-kips, from SSMA page 7 & 8 for bending only.

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.59 > 0.5 \qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.17 < 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right)+\left(\frac{\Omega_v V}{V_n}\right) = 0.525 < 1.3$$ **[Satisfactory]**

CHECK DEFLECTION

$$\Delta_{LL}=\frac{5(LL\ \ S)L^4}{384EI_{xx}}= 0.19 \text{ in} < L/240 = 0.58 \text{ in}$$ **[Satisfactory]**

DETERMINE SCREWS AT EACH LEG OF CONNECTION (SSMA page 48)

V_{max} = 471 lbs

v_{allow} =	344	lbs / screw, for # 8 screws.	= = >	(2)- # 8 screws required.
	370	lbs / screw, for # 10 screws.	= = >	(2)- # 10 screws required.
	384	lbs / screw, for # 12 screws.	= = >	(2)- # 12 screws required.

Technical References:

1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. SSMA, Product Technical Information, ICBO ER-4943P, Steel Stud Manufactures Association, 2001.

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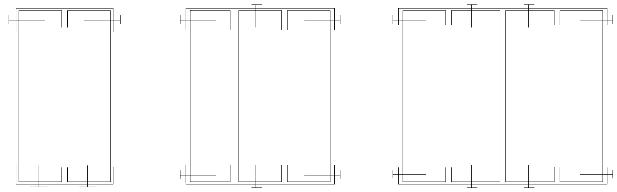
Box Beam Design Based on AISI S100-2007 & ICBO ER-4943P

INPUT DATA & DESIGN SUMMARY



BEAM / HEADER SECTION

VERT. MEMBERS 3 **x** **800S250-68**
(TOTAL SECTION: 8 x 8 , 50 ksi)

SPAN L = **10** ft

DEAD LOAD DL = **0.2** kips / ft
LIVE LOAD LL = **0.36** kips / ft

COMPRESSION FLANGE SUPPORTED ? (0=No, 1=Yes) **0** No.

DEFLECTION LIMITATION FOR LIVE LOAD ? **1** L /240
(0=No., 1= L /240, 2= L /360, 3= L /180, 4= L /120)

THE DESIGN IS ADEQUATE.

ANALYSIS
SECTION PROPERTIES OF EACH STUD (SSMA page 7 & 8)

t =	0.0713	in	F_y =	50	ksi	I_{xx} =	9.261	in^4	M_n/Ω_b =	59.96	in-kips
h =	8	in	Wt =	3.33	lb/ft	S_{xx} =	2.003	in^3	V_n/Ω_v =	4048	lbs
A =	0.978	in^2	r_x =	3.077	in	r_y =	0.877	in	x_o =	-1.674	in
J =	0.001658	in^4	C_w =	9.526	in^6						

CHECK MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)
h / t = 112.20 **<** 200 **[Satisfactory]**

CHECK FLEXURAL CAPACITY (AISI C3.1)
M_n/Ω_b = 14.99 ft-kips **>** **M** **[Satisfactory]**
Where $M = (DL + LL + Wt)\, L^2 / 8 =$ 7.12 ft-kips

CHECK SHEAR CAPACITY (AISI C3.2)
V_n/Ω_V = 12.14 kips **>** **V** **[Satisfactory]**
Where $V = (DL + LL + Wt)\, L / 2 =$ 2.85 kips

CHECK LATERAL-TORSIONAL BUCKLING (AISI C3.1.2.2)

$$L_u = \frac{0.36 C_b \pi}{F_y S_f}\sqrt{E G J I_y} = \quad 1.58 \text{ ft} \quad < \quad L$$

Where C_b = 1.0
S_f = 6.95 in^3 (total vertical studs, SSMA page 7 & 8.)
E = 29500 ksi (AISI pg xiv)
G = 11300 ksi (AISI pg xvi)
I_y = 20.509 in^4 (neglecting top & bottom tracks conservatively.)
J = 0.005 in^4

$$F_e = \frac{C_b \pi}{K_y L_y S_f}\sqrt{E G J I_y} = \quad 22.0 \text{ ksi}$$

< $2.78\,F_y$ = 139.0 ksi
< $0.56\,F_y$ = 28.0 ksi

Where K_y = 1.0
L_y = 120 in

$$F_c = \begin{cases} F_y\ , & for\ F_e \geq 2.78 F_y \\ \frac{10}{9} F_y\left(1 - \frac{10 F_y}{36 F_e}\right)\ , & for\ 2.78 > F_e \geq 0.56 F_y \\ F_e\ , & for\ F_e \leq 0.56 F_y \end{cases} \quad = \quad 22.0 \text{ ksi}$$

(cont'd)

M_n/Ω_b = 7.62 ft-kips **>** **M** **[Satisfactory]**

Where S_c = 6.95 in^3 (total vertical studs, SSMA page 7 & 8.)

Ω_b = 1.67

M_n = S_cF_c = 152.67 in-kips

M = (DL + LL + Wt) L^2 / 8 = 7.12 ft-kips

CHECK CAPACITY COMBINED BENDING & SHEAR AT ANY SAME SECTION (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2} = 0.9643 < 1.0 \quad \textbf{[Satisfactory]}$$

Where M = 7.12 ft-kips

V = 2.85 kips

V_n/Ω_v = 12.14 kips

Ω_b = 1.67

M_n = MIN(Bending , Buckling) = 12.72 ft-kips

M_n/Ω_b = 14.99 ft-kips, for bending only.

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.94 > 0.5 \qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.23 < 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right)+\left(\frac{\Omega_v V}{V_n}\right) = 0.7958 < 1.3 \quad \textbf{[Satisfactory]}$$

CHECK DEFLECTION

$$\Delta LL = \frac{5(LL)L^4}{384EI} = 0.10 \text{ in} < L/240 = 0.50 \text{ in} \quad \textbf{[Satisfactory]}$$

Technical References:

1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. SSMA, Product Technical Information, ICBO ER-4943P, Steel Stud Manufactures Association, 2001.

Daniel T. Li
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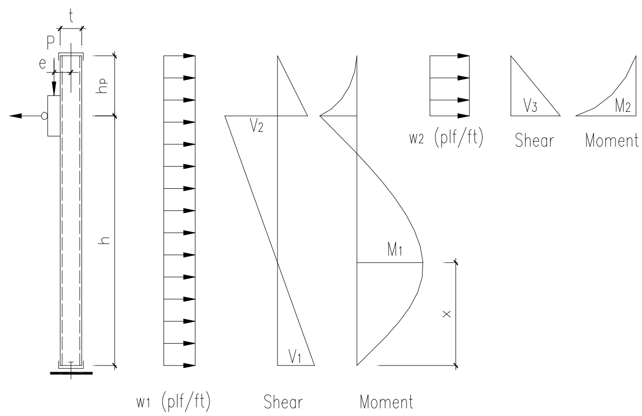
REVIEW BY :

Wall Stud Design Based on AISI S100-2007 & ICBO ER-4943P

INPUT DATA & DESIGN SUMMARY



SECTION & SPACING	600S162-54 @	16	in o.c
		(50 ksi)	
WALL HEIGHT	h =	16	ft
	h_p =	4	ft
GRAVITY LOAD (ASD level)	P =	1500	lbs / ft
LATERAL LOAD (SD level)	w_1 =	25	psf
PARAPET LOAD (SD level)	w_2 =	45	psf
ECCENTRICITY	e =	0	in
DEFLECTION LIMITATION ?		1	h /240

(0=No., 1= h /240, 2= h /360, 3= h /180, 4= h /120)

THE DESIGN IS ADEQUATE.

ANALYSIS

SECTION PROPERTIES OF EACH METAL STUD (SSMA page 7 & 8)

thk =	0.0566	in	F_y =	50	ksi	I_{xx} =	2.86	in^4	M_n/Ω_b =	27.76	in-kips
t =	6	in	Wt =	1.89	lb/ft	S_{xx} =	0.927	in^3	V_n/Ω_v =	2708	lbs
A =	0.556	in^2	r_x =	2.267	in	r_y =	0.57	in	x_o =	-1.072	in
J =	0.000594	in^4	C_w =	1.318	in^6						

CHECK MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)

t / (thk) = 106.01 < 200 **[Satisfactory]**

CHECK FLEXURAL CAPACITY (AISI C3.1)

M_n/Ω_b = 27.76 in-kips / stud **>** **M** **[Satisfactory]**

Where $x = h + h_p - \left[\frac{(h+h_p)^2}{2h} - \frac{Pe}{hw_1}\right] =$ 7.50 ft

$$M_1 = \frac{1}{2w_1h^2}\left[Pe + \frac{w_1}{2}\left(h^2 - h_p^2\right)\right]^2 S = \quad 11.25 \text{ in-kips / stud}$$

S = 1.333 ft o.c., spacing as gaven.

$$M_2 = \frac{w_2h_p^2}{2} S = \quad 5.76 \text{ in-kips / stud}$$

M = (1/1.4) MAX(M_1 , M_2) = 8.04 in-kips / stud, (1/1.4 for wind/seismic, from AISI App. A4.1.2, typical)

CHECK SHEAR CAPACITY (AISI C3.2)

V_n/Ω_V = 2708 lbs / stud **>** **V** **[Satisfactory]**

Where $V_1 = \left[(h+h_p)w_1 - \frac{(h+h_p)^2 w_1}{2h} + \frac{Pe}{h}\right] S =$ 250 lbs / stud

V_2 = h w_1 S - V_1 = 283 lbs / stud

V_3 = h_p w_2 S = 240 lbs / stud

V = (1/1.4) MAX(V_1 , V_2 , V_3) = 202 lbs / stud

CHECK CAPACITY COMBINED BENDING & SHEAR AT ROOF/FLOOR SECTION (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} = \quad 0.166 \quad < \quad 1.0$$ **[Satisfactory]**

Where M = 4.11 in-kips / stud, (1/1.4 included)

V = 202 lbs / stud, (1/1.4 included)

V_n/Ω_v = 2708 lbs / stud, from SSMA page 7 & 8.

M_n/Ω_b = 27.76 in-kips / stud, from SSMA page 7 & 8.

(cont'd)

$\left(\frac{\Omega_b M}{M_n}\right)$ **=** 0.15 **<** 0.5

$\left(\frac{\Omega_v V}{V_n}\right)$ **=** 0.07 **<** 0.7

$0.6\left(\frac{\Omega_b M}{M_n}\right)+\left(\frac{\Omega_v V}{V_n}\right)$ = 0.1637 **<** 1.3 **[Satisfactory]**

CHECK COMPRESSION CAPACITY WITH, AT LEAST, ONE FLANGE THROUGH-FASTENED TO SHEATHING (AISI D6.1.3)

P_n/Ω_c = 4.99 kips / stud **>** **P** **[Satisfactory]**

Where Ω_c = 1.8

$P_n = C_1C_2C_3$ AE / 29500 = 8.97 kips / stud

C_1 = (0.79 x + 0.54) = 0.949

C_2 = (1.17 α t + 0.93) = 0.996

C_3 = α (2.5b - 1.63d) + 22.8 = 17.070

E = 29500 ksi (AISI pg xiv)

P = 2.30 kips / stud (included wall weight, 18psf.)

CHECK CAPACITY COMBINED AXIAL LOAD & BENDING (AISI C5.2.1)

$\frac{\Omega_c P}{P_n}+\frac{\Omega_b C_m M}{M_n \alpha}=$ 0.82 **<** 1.0 **[Satisfactory]**

Where M = 8.04 in-kips / stud, (1/1.4 included)

P = 2.30 kips / stud

P_n/Ω_c = 4.99 kips / stud

M_n/Ω_b = 27.76 in-kips / stud

C_m = 1.0

$P_{Ex}=\frac{\pi^2 E I_x}{(K_x L_x)^2}=$ 22.59 kips / stud

$\alpha = 1-\frac{\Omega_c P}{P_{Ex}}=$ 0.817

CHECK DEFLECTION

$\Delta=\frac{5(w_1 S)L^4}{384 E I_{xx}}=$ 0.58 in **<** h /240 = 0.80 in **[Satisfactory]**

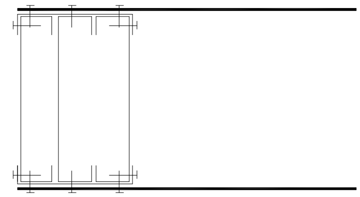
NOTE : 1. STUD FLANGES SHALL BE FASTENED TO SHEATHING AT EACH SIDE OF WALL BEFORE VERTICAL LOAD ADDED.

Technical References:

1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. SSMA, Product Technical Information, ICBO ER-4943P, Steel Stud Manufactures Association, 2001.

Jamb/Column Design Based on AISI S100-2007 & ICBO ER-4943P

INPUT DATA & DESIGN SUMMARY


JAMB / WALL COLUMN SECTION

VERT. MEMBERS 3 **x** **400S200-54** (50 ksi)
(TOTAL SECTION: 4 x 6 , INSIDE 4 in THK. WALL)

HEIGHT h = **9** ft

SERVICE GRAVITY LOAD P = **6.8** kips
SERVICE LATERAL LOAD w = **0.33** kips / ft

DEFLECTION LIMITATION ? **1** h /240
(0=No., 1= h /240, 2= h /360, 3= h /180, 4= h /120)

THE DESIGN IS ADEQUATE.

ANALYSIS
SECTION PROPERTIES OF EACH VERTICAL STUD (SSMA page 7 & 8)

thk =	0.0566	in	F_y =	50	ksi	I_{xx} =	1.292	in^4	M_n/Ω_b =	17.36	in-kips
t =	4	in	Wt =	1.7	lb/ft	S_{xx} =	0.58	in^3	V_n/Ω_v =	3446	lbs
A =	0.5	in^2	r_x =	1.608	in	r_y =	0.758	in	x_o =	-1.695	in
J =	0.000534	in^4	C_w =	1.068	in^6						

CHECK MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)
t / (thk) = 70.67 **<** 200 [Satisfactory]

CHECK FLEXURAL CAPACITY (AISI C3.1)
M_n/Ω_b = 4.34 ft-kips **>** **M** **[Satisfactory]**
Where $M = (1/1.4)\, w\, h^2 / 8$ = 2.39 ft-kips, (1/1.4 for wind/seismic, from AISI App. A4.1.2, typical)

CHECK SHEAR CAPACITY (AISI C3.2)
V_n/Ω_V = 10.34 kips **>** **V** **[Satisfactory]**
Where V = (1/1.4) w L / 2 = 1.06 kips

CHECK COMPRESSION CAPACITY WITH, AT LEAST, ONE FLANGE THROUGH-FASTENED TO SHEATHING (AISI D6.1.3)
P_n/Ω_c = 15.41 kips **>** **P** **[Satisfactory]**
Where Ω_c = 1.8
$P_n = C_1C_2C_3\, AE / 29500$ = 27.73 kips
$C_1 = (0.79\, x + 0.54)$ = 0.956
$C_2 = (1.17\, \alpha\, t + 0.93)$ = 0.996
$C_3 = \alpha\, (2.5b - 1.63d) + 22.8$ = 19.405
E = 29500 ksi (AISI pg xiv)
P = 6.85 kips (included studs weight.)

CHECK CAPACITY COMBINED BENDING & SHEAR AT ANY SAME SECTION (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2} = 0.5594 < 1.0$$ **[Satisfactory]**

Where M = 2.39 ft-kips, (1/1.4 included)
V = 1.06 kips, (1/1.4 included)
V_n/Ω_v = 10.34 kips
M_n/Ω_b = 4.34 ft-kips

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.55 > 0.5 \qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.10 < 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right)+\left(\frac{\Omega_v V}{V_n}\right) = 0.4325 < 1.3$$ **[Satisfactory]**

(cont'd)

CHECK CAPACITY COMBINED AXIAL LOAD & BENDING (AISI C5.2.1)

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_m M}{M_n \alpha} = \quad 0.89 \quad < \quad 1.0$$ **[Satisfactory]**

Where M = 1.70 ft-kips, (1/1.4 included)

P = 6.85 kips

P_n/Ω_c = 15.41 kips

M_n/Ω_b = 4.34 in-kips

C_m = 1.0

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} = \quad 96.75 \text{ kips}$$

$$\alpha = 1 - \frac{\Omega_c P}{P_{Ex}} = \quad 0.873$$

CHECK DEFLECTION

$$\Delta = \frac{5 w L^4}{384 EI} = \quad 0.43 \text{ in} \quad < \quad h/240 = 0.45 \text{ in}$$ **[Satisfactory]**

NOTE : 1. STUD FLANGES SHALL BE FASTENED TO SHEATHING AT EACH SIDE OF WALL BEFORE VERTICAL LOAD ADDED.

Technical References:

1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. SSMA, Product Technical Information, ICBO ER-4943P, Steel Stud Manufactures Association, 2001.

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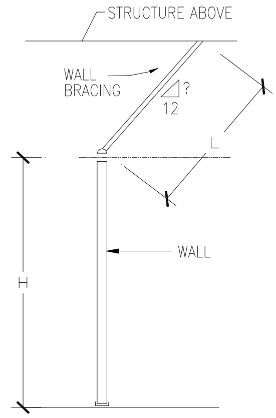
Brace Design Based on AISI S100-2007 & ICBO ER-4943P



WALL BRACING

INPUT DATA & DESIGN SUMMARY

SECTION & SPACING	350S162-33 @	60	in o.c
BRACE LENGTH	L =	12	ft
BRACE SLOPE	12 /	12	
WALL LATERAL LOAD, ASD	F_p =	5	psf
WALL HEIGHT	H =	16	ft

THE DESIGN IS ADEQUATE.

ANALYSIS

Check Brace Compression Capacity (AISI C4.1)

P = F_p (0.5 H) S / Cos α = 0.283 kips / brace

P_n/Ω_c = 0.67 kips / brace **>** **P** **[Satisfactory]**

Where Ω_c = 1.8

Ae = 0.258 in^2 (SSMA page 6-7, ICBO ER-4943P)

r_y = 0.617 in (SSMA page 6-7, ICBO ER-4943P)

E = 29500 ksi (AISI pg xiv)

KL / r_y = 233 **>** **200** [Caution! but Not Mandatory, AISI Commentary page 86]

$F_e = \pi^2 E / (KL / r_y)^2$ = 5 ksi

F_y = 33 ksi

$\lambda_C = (F_y / F_e)^{0.5}$ = 2.48

$$F_n = \begin{cases} 0.658^{\lambda_c^2} F_y \ , & for \ \ \lambda_c \le 1.5 \\ \dfrac{0.877}{\lambda_c^2} F_y \ , & for \ \ \lambda_c > 1.5 \end{cases} = 4.7 \text{ ksi}$$

$P_n = A_e F_n$ = 1.21 kips / brace

NOTE : THE LATERAL LOADS MAY BE REDUCED BY (1/1.4) PER AISI APPENDIX A.4.1.2 .

Technical References:
1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. SSMA, Product Technical Information, ICBO ER-4943P, Steel Stud Manufactures Association, 2001.

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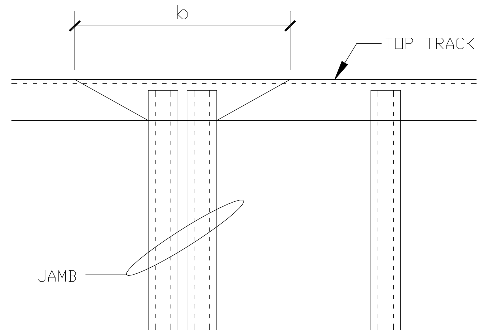
Connection Design of Jamb to Track Based on AISI S100-2007 & ICBO ER-4943P

INPUT DATA & DESIGN SUMMARY



JAMB MEMBERS **3** **x** **600S137-68**

(TOTAL SECTION: 6 x 5 , INSIDE 6 in THK. WALL)

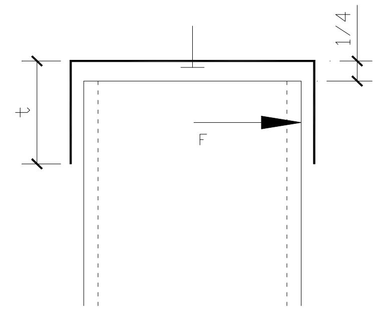
TOP TRACK **600T200-54** (16 GA , 50 ksi)

JAMB LATERAL LOAD F = **0.2** kips

THE DESIGN IS ADEQUATE.

ANALYSIS

SECTION PROPERTIES OF TOP TRACK (SSMA page 10 & 11)



t = 2 in, leg length

F_y = 50 ksi

thk = 0.0566 in, metal thickness

wall = 6 in, wall thickness **<** track width

[Satisfactory]

CHECK BENDING CAPACITY OF TRACK LEG

d = 5 in, jamb width

$b = d + 2\,t\,(\tan 60^o) =$ 11.9 in, effective width

$M = F\,(t + 1/4") / 2 =$ 0.2 in-kips

$S = b\,(thk)^2 / 6 =$ 0.0064 in^3

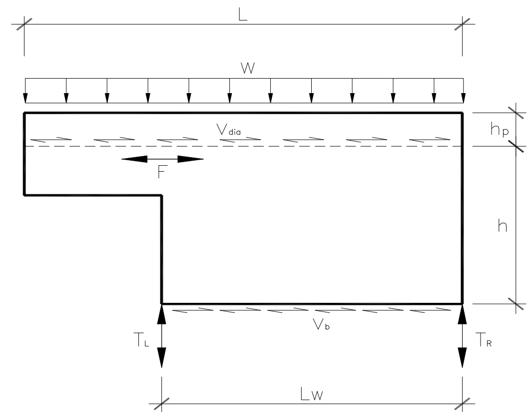
$f_b = M / S =$ 35 ksi **<** (4/3) F_y **[Satisfactory]**

(If jamb lateral load have reduced (1/1.4), the factor 4/3 does not apply. AISI App. A4.1.2)

Technical References:

1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. SSMA, Product Technical Information, ICBO ER-4943P, Steel Stud Manufactures Association, 2001.

Metal Shear Wall Design Based on AISI S100-2007, ER-5762 & ER-4943P



INPUT DATA

LATERAL FORCE ON DIAPHRAGM:	$v_{dia,\ WIND}$ =	350	plf,for wind
	$v_{dia,\ SEISMIC}$ =	500	plf,for seismic
GRAVITY LOADS	w_{DL} =	262	plf,for dead load
	w_{LL} =	0	plf,for live load
DIMENSIONS: L_w = 8 ft ,	h =	16	ft
L = 10 ft ,	h_p =	2	ft
WALL STUD GAGE (20, 18 or 16)		18	Gage
FASTENERS ATTACHING PANELS	#	8	Screws
EDGE STUD SECTION	2 x	**600S200-68**	

DESIGN SUMMARY

ONE SIDE SURE-BOARD SERIES 200 STRUCTURAL PANEL
WITH # 8 SCREWS @ 4" O.C.AT EDGES AND 12" O.C. FIELD.
WITH 18 GAGE METAL STUDS @ 24" O.C. MAX.
WITH #10 SCREWS OF SOLE NAILING (OR 0.145" DIA PINS) @ 3" O.C.
OR WITH 5/8" DIA ANCHOR BOLTS @ 24" O.C. AT FOUNDATION.

THE SHEAR WALL IS ADEQUATE.

HOLD-DOWN FORCES: T_L = 8.27 k , T_R = 8.86 k (USE S/HD10 - 7/8 DIA BOLTS - 30 # 10)
DRAG STRUT FORCES: F = 1.00 k
EDGE STUD: 2 x 600S200-68
SHEAR WALL DEFLECTION: Δ_S = 0.23 in

ANALYSIS

CHECK MAX. SHEAR WALL DIMENSION RATIO

L / B = 2.0 < 2 1/4 (from DSA PA-132) **[Satisfactory]**

CHECK SHEAR STRESS CAPACITY

v_b = 625 plf < $v_{allowable}$ = 770 plf (ASD, from ER-5762, Table 1)

[Satisfactory]

DETERMINE DRAG STRUT FORCE

$F = (L-L_w)\ MAX(\ v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC}\ )$ = 1.00 kips (Ω_0 = 1)

DETERMINE HOLD-DOWN FORCES

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	500	230	82074	Left	17708	0.9	T_L =	8267	S/HD10
				Right	12468	0.9	T_R =	8857	
WIND	350		56000	Left	17708	2/3	T_L =	5524	
				Right	12468	2/3	T_R =	5961	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

DETERMINE MAXIMUM SHEAR WALL DEFLECTION (ER-5762 Table 1 & DSA PA-132)

Δ_S = 0.13 x MAX (h / 9'-0" , 1) = 0.231 in

CHECK EDGE STUD CAPACITY (AISI S100-2007 & ER-4943P)

P_n/Ω_c = 13.18 kips > P_{max} = 8.32 kips, (this value should include upper level DOWNWARD loads if applicable)

[Satisfactory]

Where Ω_c = 1.8

$P_n = C_1C_2C_3\ AE / 29500$ = 23.73 kips

$C_1 = (0.79\ x + 0.54)$ = 0.949

$C_2 = (1.17\ \alpha\ t + 0.93)$ = 1.013

$C_3 = \alpha\ (2.5b - 1.63d) + 22.8$ = 16.145

E = 29500 ksi (AISI pg xiv)

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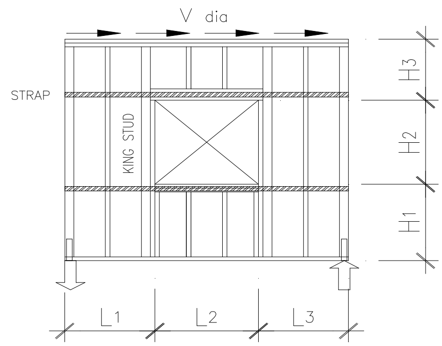
Metal Shear Wall with an Opening Based on AISI S100-2007, ER-5762 & ER-4943P

INPUT DATA

LATERAL FORCE ON DIAPHRAGM:			$V_{dia,\ WIND}$ =	268	plf,for wind			
(SERVICE LOADS)			$V_{dia,\ SEISMIC}$ =	350	plf,for seismic			
DIMENSIONS:	L_1 =	4	ft , L_2 =	16	ft , L_3 =	4	ft	
	H_1 =	4	ft , H_2 =	6	ft , H_3 =	6.5	ft	
WALL STUD GAGE (20, 18 or 16)				18	Gage			
FASTENERS ATTACHING PANELS			#	8	Screws			
EDGE STUD SECTION		2	x	600S162-54				
KING STUD SECTION		2	x	600S162-54				

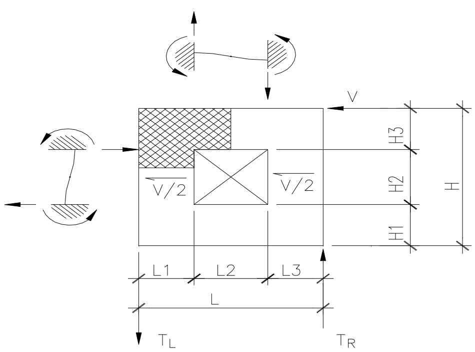


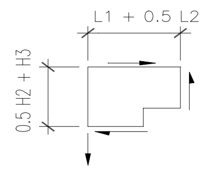
THE SHEAR WALL IS ADEQUATE.

DESIGN SUMMARY

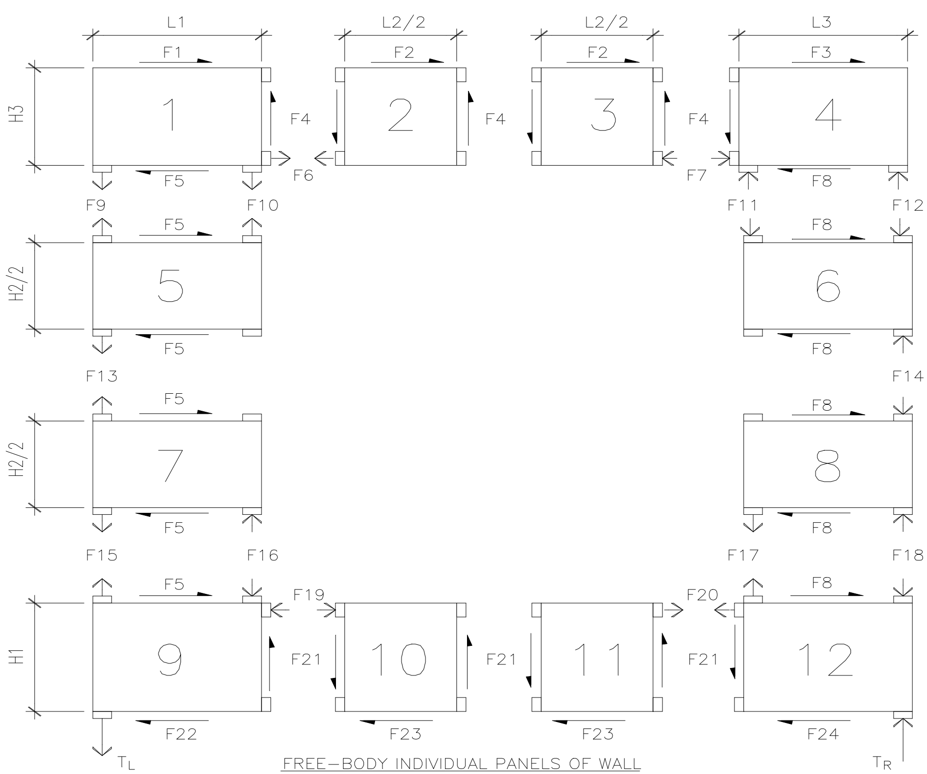
BOTH SIDES SURE-BOARD SERIES 200 STRUCTURAL PANEL
WITH # 8 SCREWS @ 6" O.C.AT EDGES AND 12" O.C. FIELD.
WITH 18 GAGE METAL STUDS @ 24" O.C. MAX.
WITH #10 SCREWS OF SOLE NAILING (OR 0.145" DIA PINS) @ 2" O.C.
OR WITH 5/8" DIA ANCHOR BOLTS @ 16" O.C. AT FOUNDATION.

HOLD-DOWN FORCES:	T_L =	5.99	k ,	T_R =	5.99	k (USE S/LTT20 - 1/2 DIA BOLTS - 5 # 10)
MAX. STRAP FORCE:	T_S =	4.68	k			
EDGE STUD:	2	x	600S162-54	, CONTINUOUS FULL HEIGHT.		
KING STUD:	2	x	600S162-54	, CONTINUOUS FULL HEIGHT.		
SHEAR WALL DEFLECTION:	Δ_S =	0.40	in			





ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

(cont'd)

ANALYSIS

CHECK MAX. SHEAR WALL DIMENSION RATIO

L / B = 1.5 < 2 1/4 (from DSA PA-132) **[Satisfactory]**

DETERMINE FORCES & SHEAR STRESS OF FREE-BODY INDIVIDUAL PANELS OF WALL

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	6.50	27	F1	108	F13	3325
2	8.00	6.50	512	F2	4092	F14	3325
3	8.00	6.50	512	F3	108	F15	6475
4	4.00	6.50	27	F4	3325	F16	3150
5	4.00	3.00	1050	F5	4200	F17	3150
6	4.00	3.00	1050	F6	4092	F18	6475
7	4.00	3.00	1050	F7	4092	F19	4682
8	4.00	3.00	1050	F8	4200	F20	4682
9	4.00	4.00	-121	F9	175	F21	2668
10	8.00	4.00	667	F10	3150	F22	-482
11	8.00	4.00	667	F11	3150	F23	4682
12	4.00	4.00	-121	F12	175	F24	-482

CHECK SHEAR STRESS CAPACITY

v_b = 1050 plf < $v_{allowable}$ = 1124 plf (ASD, from ER-5762, Table 1)

[Satisfactory]

DETERMINE HOLD-DOWN FORCES

THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	350	634	143827	Left	0	0.9	T_L =	5993	S/LTT20
				Right	0	0.9	T_R =	5993	
WIND	268		106128	Left	0	2/3	T_L =	4422	
				Right	0	2/3	T_R =	4422	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

DETERMINE MAXIMUM SHEAR WALL DEFLECTION (ER-5762 Table 1 & DSA PA-132)

Δ_S = 0.22 x MAX (h / 9'-0" , 1) = 0.403 in

CHECK EDGE STUD CAPACITY (AISI S100-2007 & ER-4943P)

P_n/Ω_c = 9.43 kips > P_{max} = 5.99 kips, (this value should include upper level DOWNWARD loads if applicable)

[Satisfactory]

Where Ω_c = 1.8

$P_n = C_1C_2C_3$ AE / 29500 = 16.98 kips

$C_1 = (0.79\,x + 0.54)$ = 0.949

$C_2 = (1.17\,\alpha\,t + 0.93)$ = 0.996

$C_3 = \alpha\,(2.5b - 1.63d) + 22.8$ = 16.145

E = 29500 ksi (AISI pg xiv)

CHECK KING STUD CAPACITY (AISI S100-2007 & ER-4943P)

P_n/Ω_c = 9.43 kips > P_{max} = 3.15 kips

[Satisfactory]

Where $P_n = C_1C_2C_3$ AE / 29500 = 16.98 kips

$C_1 = (0.79\,x + 0.54)$ = 0.949

$C_2 = (1.17\,\alpha\,t + 0.93)$ = 0.996

$C_3 = \alpha\,(2.5b - 1.63d) + 22.8$ = 16.145

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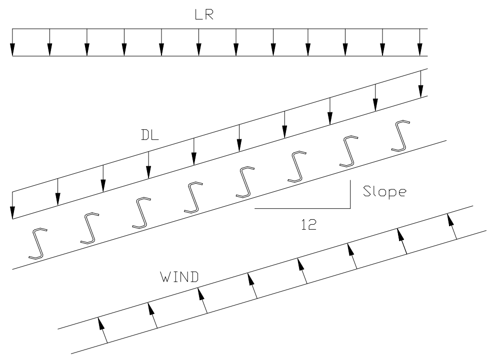
Metal Z-Purlins Design Based on AISI S100-2007



INPUT DATA & DESIGN SUMMARY

SECTION & SPACING **10ZS2.25x85** @ **24** in o.c

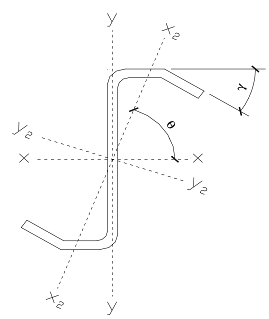
ROOF SLOPE **4** : 12 (18.43 ^{0})
MAXIMUM PURLIN SPAN L = **25** ft
YIELD STRESS (33 or 55) F_y = **55** ksi
SLOPED DEAD LOAD DL = **8** psf
PROJECTED ROOF LIVE LOAD LR = **20** psf
NET UPWARD WIND PRESSURE WIND = **25** psf

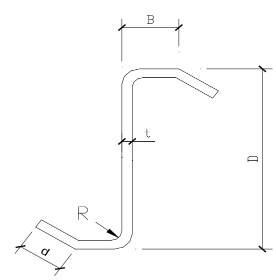
LATERAL SUPPORTED BY DIAPHRAGM? **1** top flange
(0=No, 1=top flange, 2=bottom flange)
DEFLECTION LIMITATION FOR LIVE LOAD ? **1** L /240
(0=No., 1= L /240, 2= L /360, 3= L /180, 4= L /120)

THE DESIGN IS ADEQUATE.

ANALYSIS
SECTION PROPERTIES OF EACH Z PURLIN (AISI Manual, 2002, Table I-4 & Table II-4)

t	=	0.085	in	r_y	=	1.08	in
D	=	10	in	Wt	=	4.6278	lb/ft
A	=	1.36	in^2	r_x	=	3.74	in
J	=	0.00326	in^4	C_w	=	30	in^6
I_e	=	18.8	in^4	M_n	=	194	in-kips
S_e	=	3.53	in^3	V_n	=	9.25	lbs
B	=	2.25	in	d	=	0.96	in
R	=	0.1875	in	S_x	=	3.79	in^3





CHECK MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)
D / t = 117.65 **<** 200 **[Satisfactory]**

DETERMINE ASD LEVEL LOADS
w_{DL+LR} = [(DL Cosθ + LR Cos^{2}θ) S + Wt Cosθ] = 55.569 lb/ft, down inward to roof
$M_{mid} = w_{DL+LR} L^2 / 8$ = 52.096 in-kips, conservative value at middle of span
$M_{support} = - w_{DL+LR} L^2 / 12$ = -34.731 in-kips, conservative value at supports
$V_{DL+LR} = 2\ w_{DL+LR}\ L / 3$ = 0.9262 kips, conservative value at supports

$w_{0.9DL+WIND}$ = [(0.9 DL Cosθ - WIND) S + Wt Cosθ] = -31.9487 lb/ft, uplift inward to roof
$M_{mid} = w_{0.9DL+WIND} L^2 / 8$ = -29.952 in-kips, conservative value at middle of span
$M_{support} = - w_{0.9DL+WIND} L^2 / 12$ = 19.968 in-kips, conservative value at supports
$V_{0.9DL+WIND} = 2w_{0.9DL+WIND}\ L / 3$ = -0.5325 kips, conservative value at supports

CHECK FLEXURAL CAPACITY (AISI C3.1)
M_n/Ω_b = 116.17 in-kips **>** **M** **[Satisfactory]**
Where M = Max(M_{mid}, $M_{support}$) = 52.10 in-kips
Ω_b = 1.67

CHECK SHEAR CAPACITY (AISI C3.2)
V_n/Ω_V = 5.78 lbs **>** **V** **[Satisfactory]**
Where V = Max(V_{DL+LR}, $V_{0.9DL+WIND}$) = 0.9262 lbs
Ω_v = 1.60

CHECK LATERAL-TORSIONAL BUCKLING (AISI C3.1.2)
CASE 1: BOTH TOP & BOTTOM FLANGES UNSUPPORTED **<== Does not apply.**

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} =$$ 6.9 ksi **<** 2.78 F_y = 152.9 ksi
< 0.56 F_y = 30.8 ksi

(cont'd)

Where C_b = 1.0 x_o = 0 in

$r_o = (r_x^2 + r_y^2 + x_o^2)^{0.5}$ = 3.893 in

S_f = 3.79 in^3, (S_x)

E = 29500 ksi (AISI pg xiv)

G = 11300 ksi (AISI pg xvi)

K_y = 1.0

K_t = 1.0

L_y = 300 in

L_t = 300 in

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} = 3.773 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right] = 6.496 \text{ ksi}$$

$$F_c = \begin{cases} F_y \,, & for\ F_e \geq 2.78 F_e \\ \frac{10}{9} F_y \left(1 - \frac{10 F_y}{36 F_e}\right), & for\ 2.78 > F_e \geq 0.56 F_e \\ F_e \,, & for\ F_e \leq 0.56 F_e \end{cases} = 6.9 \text{ ksi}$$

M_n/Ω_b = 15.70 in-kips **<** **M** **[Satisfactory]**

Where S_c = 3.79 in^3

Ω_b = 1.67

M_n = $S_c F_c$ = 26.21 in-kips

M = 52.10 in-kips

CASE 2: TOP OR BOTTOM FLANGE SUPPORTED ONLY

M_n/Ω_b = 66.85 in-kips **>** **M** **[Satisfactory]**

Where S_e = 3.53 in^3

Ω_b = 1.67

R = 0.575 (AISI Table D6.1.1-1)

M_n = $R S_e F_y$ = 111.64 in-kips

M = 34.73 in-kips

CHECK CAPACITY COMBINED BENDING & SHEAR AT ANY SAME SECTION (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} = 0.7956 < 1.0 \quad \textbf{[Satisfactory]}$$

Where M = 52.10 in-kips

V = 0.93 lbs

V_n/Ω_v = 5.78125 lbs

M_n = MIN(Bending , Buckling) = 111.64 in-kips

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.78 > 0.5 \qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.16 < 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right) + \left(\frac{\Omega_v V}{V_n}\right) = 0.6278 < 1.3 \quad \textbf{[Satisfactory]}$$

CHECK LIVE LOAD DEFLECTION

$$\Delta_{LR} = \frac{5(w_{LR})L^4}{384 E I_e} = 0.57 \text{ in} < L/240 = 1.25 \text{ in} \quad \textbf{[Satisfactory]}$$

Where $w_{LR} = LR \cos^2\theta\, S$ = 36 lb/ft, down inward to roof

Technical References:

1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. AISI MANUAL, 2001 Edition. American Iron and Steel Institute.

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Seismic Design for Ordinary Concentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

BRACE SECTION (Tube or Pipe)	=>	**HSS6X6X5/8**	**Tube**	**A**	$\mathbf{r_{min}}$	**t**	**h**
BRACE AXIAL LOAD AT SERVICE LEVEL	D =	**20** kips		11.70	2.17	0.58	6.00
	L =	**10** kips					
BRACE AXIAL LOAD AT HORIZ. SEISMIC	Q_E =	**50** kips (ASCE 7-05 12.4.2.1)					
SEISMIC PARAMETER	S_{DS} =	**0.533** (ASCE 7-05 11.4.4)	**THE DESIGN IS ADEQUATE.**				
UNBRACED LENGTH OF THE BRACE	l =	**18** ft					
BUILDING LIMITATION FOR SDC D or E	H =	**28** ft (< 35 ft, ASCE 7-05 Tab. 12.2-1)					

REQUIRED CONNECTION = > (5/8 in Gusset Plate with 14 in Length, 4 leg, 5/16 in Fillet Weld. Cover Plate 3/4 x 5 at Each Sides.)

CHECK LIMITING WIDTH THICKNESS RATIO λ_{ps} FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

$D / t = 0.044\ E_s / F_y$ = 27.74 , for Pipe

($D / t = 1300 / F_y$ for AISC-Seismic 97, Tab. 1-9-1) **>** Actual **[Satisfactory]**

$h / t = 0.64\ (E_s / F_y)^{0.5}$ = 16.07 , for Tube

[$h / t = 110 / (F_y)^{0.5}$ for AISC-Seismic 97, Tab. 1-9-1]

Where F_y = 46 ksi

E_s = 29000 ksi

CHECK LIMITING SLENDERNESS RATIO FOR V OR INVERTED-V CONFIGURATIONS (AISC 341-05 Sec. 14.2)

$4\ (E_s / F_y)^{0.5}$ = 100.4 **>** $K\,l / r$ = 99.5 **[Satisfactory]**

[$720 / (F_y)^{0.5}$ for AISC-Seismic 97, Sec. 14.2]

Where K = 1.0

DETERMINE FACTORED DESIGN LOADS TO WITHSTAND LIMITED INELASTIC DEFORMATIONS (AISC 341-05 Sec.14.1)

$P_{ut} = 0.9D - (C_d / I)\ Q_E - 0.2S_{DS}D$ = -146.6 kips (Tension, ASCE 7-05 Sec. 12.8.6)

$P_{uc} = 1.2D + f_1L + (C_d / I)\ Q_E + 0.2S_{DS}D$ = 193.6 kips (Compression Governs, ASCE 7-05 Sec. 12.8.6)

Where C_d = 3 1/4 (ASCE 7-05 Tab. 12.2-1)

I = 1 (IBC Tab.1604.5)

f_1 = 0.5 (IBC 1605.2)

CHECK DESIGN STRENGTH IN COMPRESSION (AISC 360-05 E3)

$\phi_c P_n = \phi_c A_g F_{cr}$ = 248.75 kips **>** P_{uc} **[Satisfactory]**

Where ϕ_c = 0.9

F_e = $\pi^2 E / (K\,l / r)^2$ = 28.891 ksi

λ_c = $(K\,l / r)\ (F_y / E)^{0.5}$ = 3.96

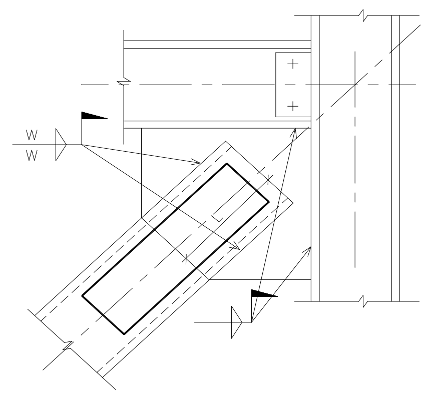
$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = 23.62 \text{ kis, for } \lambda_c \le 4.71 \\ 0.877\ F_e = \text{N/A kis, for } \lambda_c > 4.71 \end{cases}$

DETERMINE CONNECTION DESIGN FORCE (AISC 341-05 Sec. 14.4)

$P_{ut} = \text{Min}(R_y F_y A_g\ ,\ \Omega_0 P_u)$ = 387.26 kips (Tension)

Where R_y = 1.4 (AISC 341-05 Tab. I-6-1) (1.6 for Pipe)

Ω_0 = 2



DETERMINE BEST FILLET WELD SIZE (AISC 360-05 Sec.J2.2b)

w = 5/16 in

> w_{MIN} = 0.1875 in

< w_{MAX} = 0.4375 in

[Satisfactory]

DETERMINE REQUIRED WELD LENGTH (AISC 360-05 Sec.J2.4)

$L = P_{ut} / [(4)\ \phi\ F_w\ (0.707\ w)]$

= 387.26 / [(4) 0.75 (0.6x70)(0.707x5/16)] = 13.91 in

(USE 14 **in)**

CHECK DESIGN SHEAR RUPTURE OF SLOTTED BRACE (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nv}$ = 849.19 kips **>** P_{ut}

Where ϕ = 0.75 **[Satisfactory]**

F_u = 58 ksi (LRFD Tab.1-4, Pg. 1-21)

A_{nv} = 4 t L = 4 x 0.581 x 14 = 32.54 in^2

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE (AISC 360-05 Tab. J2.4)

t_g = 5/8 in

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nu}$ = 456.75 kips **>** P_{ut} **[Satisfactory]**

Where ϕ = 0.75

F_u = 58 ksi (A36 Steel)

A_{nu} = 2 tg L = 2 x 5/8 x 14 = 17.50 in^2

(cont'd)

CHECK TENSION CAPACITY AT SLOTTED BRACE (AISC 360-05 D.2 b)

$\phi P_n = \phi R_t F_u A_e =$ 513.94 kips **>** P_{ut} **[Satisfactory]**

Where $\phi =$ 0.75

$F_u =$ 58 ksi (AISC 13th Tab.2-3)

$x =$ 3 h / 8 = 2.25 , for Tube (HSS Specification 2.1-4)

$D / \pi =$ 1.91 , for Pipe (HSS Specification 2.1-3)

$U =$ MIN(1 - x / L , 0.9) = 0.84 ,(AISC 360-05 Tab. D3.1.)

$A_n = A_g - 2 (t_g + 1/8)\ t =$ 10.83 in^2

$A_e = U A_n =$ 9.09 in^2

$R_t =$ 1.3 (AISC 341-05 Tab. I-6-1)

Try Cover Plate 3/4 x 5 , at Each Sides.

Region	x	0.5 A_n	x A
HSS	2.25	5.41	12.18
Cover Plate	3.38	3.75	12.66
Σ		9.16	24.84

x = 24.84 / 9.16 = 2.71

U = MIN(1 - x / L , 0.9) = 0.81

A_n = 10.83 + 7.50 = 18.33 in^2

A_e = $U A_n =$ 14.78 in^2

Thus, $\phi P_n = \phi R_t F_u A_e =$ 771.52 kips **>** P_{ut} **[Satisfactory]**

Where $F_u =$ 58 ksi, use plate value

$R_t =$ 1.2 (AISC 341-05 Tab. I-6-1)

Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.

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Seismic Design for Ordinary Concentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

WF BRACE SECTION => W14X370 ==>

A	r_y	k	d	b_f
109.00	4.27	3.26	17.90	16.50
			t_w	t_f
			1.66	2.66

BRACE AXIAL LOAD AT SERVICE LEVEL D = **100** kips
L = **100** kips
BRACE AXIAL LOAD AT HORIZ. SEISMIC Q_E = **100** kips (ASCE 7-05 12.4.2.1)
SEISMIC PARAMETER S_{DS} = **0.533** (ASCE 7-05 11.4.4)
BRACE YIELD STRESS F_y = **50** ksi

THE DESIGN IS ADEQUATE.

UNBRACED LENGTH OF THE BRACE l = **31.4** ft
BUILDING LIMITATION FOR SDC D or E H = **35** ft (< 35 ft, ASCE 7-05 Tab. 12.2-1)
REQUIRED CONNECTION = > (2 in Gusset Plate with 8 in Length, 8 leg, 3/4 in Fillet Weld. .)

CHECK LIMITING WIDTH THICKNESS RATIO λ_{ps} FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

$b_f / t_f = 0.30 (E_s / F_y)^{0.5}$ = 7.22 > Actual **[Satisfactory]**

($b_f / t_f = 52 / (F_y)^{0.5}$ for AISC-Seismic 97, Tab. 1-9-1)

Where E_s = 29000 ksi

CHECK LIMITING SLENDERNESS RATIO FOR V OR INVERTED-V CONFIGURATIONS (AISC 341-05 Sec. 14.2)

$4 (E_s / F_y)^{0.5}$ = 96.3 > $K l / r$ = 88.2 **[Satisfactory]**

[$720 / (F_y)^{0.5}$ for AISC-Seismic 97, Sec. 14.2]

Where K = 1.0

DETERMINE FACTORED DESIGN LOADS TO WITHSTAND LIMITED INELASTIC DEFORMATIONS (AISC 341-05 Sec.14.1)

$P_{ut} = 0.9D - (C_d / I) Q_E - 0.2S_{DS}D$ = -245.7 kips (Tension, ASCE 7-05 Sec. 12.8.6)

$P_{uc} = 1.2D + f_1L + (C_d / I) Q_E + 0.2S_{DS}D$ = 505.7 kips (Compression Governs, ASCE 7-05 Sec. 12.8.6)

Where C_d = 3 1/4 (ASCE 7-05 Tab. 12.2-1)
I = 1 (IBC Tab.1604.5)
f_1 = 0.5 (IBC 1605.2)

CHECK DESIGN STRENGTH IN COMPRESSION (AISC 360-05 E3)

$\phi_c P_n = \phi_c A_g F_{cr}$ = 2777.76 kips > P_{uc} **[Satisfactory]**

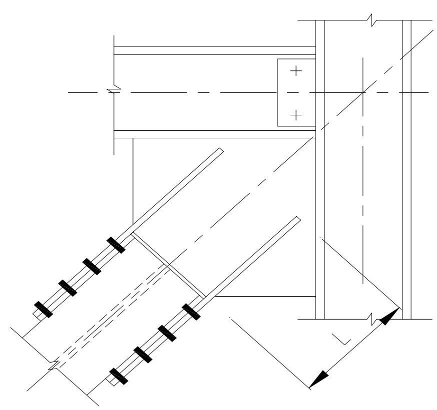
Where ϕ_c = 0.9

F_e = $\pi^2 E / (K l / r)^2$ = 36.805 ksi

λ_c = $(K l / r) (F_y / E)^{0.5}$ = 3.66

$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = 28.32 & \text{kis, for } \lambda_c \le 4.71 \\ 0.877 F_e = \text{N/A} & \text{kis, for } \lambda_c > 4.71 \end{cases}$

DETERMINE CONNECTION DESIGN FORCE (AISC 341-05 Sec. 14.4)



$P_{ut} = \mathrm{Min}(R_y F_y A_g , \Omega_0 P_u)$ = 1011.32 kips (Tension)

Where R_y = 1.1 (AISC Seismic 02 & 97 Tab. I-6-1)
Ω_0 = 2

DETERMINE BEST FILLET WELD SIZE (AISC 360-05 Sec.J2.2b)

w = 3/4 in > w_{MIN} = 0.3125 in
< w_{MAX} = 0.9375 in

[Satisfactory]

DETERMINE REQUIRED WELD LENGTH (AISC 360-05 Sec.J2.4)

$L = P_{ut} / [(8) \phi F_w (0.707 w)]$
$= 1011.32 / [(8) 0.75 (0.6x70)(0.707x3/4)]$ = 7.57 in

(USE 8 **in)**

CHECK SHEAR RUPTURE OF SLOTTED CON. PLATES (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nv}$ = 1386.43 kips > P_{ut}

Where ϕ = 0.75 **[Satisfactory]**
F_u = 58 ksi (A36 Steel)
A_{nv} = 4 t L = 4 x 1.66 x 8 = 53.12 in^2

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE (AISC 360-05 Tab. J2.4)

t_g = 2 in, Center with Column Web.

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nu}$ = 1670.40 kips > P_{ut} **[Satisfactory]**

Where ϕ = 0.75
F_u = 58 ksi (A36 Steel)
A_{nu} = 4 tg L = 4 x 2 x 8 = 64.00 in^2

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Seismic Design for Ordinary Concentrically Braced Frames Based on CBC 10 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

				Tube	A	r_{min}	t	h
BRACE SECTION (Tube or Pipe)	= >	HSS6X6X5/8						
BRACE AXIAL LOAD AT SERVICE LEVEL	D =	20	kips		11.70	2.17	0.58	6.00
	L =	10	kips					
BRACE AXIAL LOAD AT HORIZ. SEISMIC	Q_E =	50	kips (ASCE 7-05 12.4.2.1)					
SEISMIC PARAMETER	S_{DS} =	0.533	(ASCE 7-05 11.4.4)	**THE DESIGN IS ADEQUATE.**				
UNBRACED LENGTH OF THE BRACE	l =	18	ft					
BUILDING LIMITATION FOR SDC D or E	H =	28	ft (< 35 ft, ASCE 7-05 Tab. 12.2-1)					

REQUIRED CONNECTION = > (5/8 in Gusset Plate with 14 in Length, 4 leg, 5/16 in Fillet Weld. Cover Plate 3/4 x 5 at Each Sides.)

CHECK LIMITING WIDTH THICKNESS RATIO λ_{ps} FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

$D / t = 0.044\ E_s / F_y$ = 27.74 , for Pipe

($D / t = 1300 / F_y$ for AISC-Seismic 97, Tab. 1-9-1) **>** Actual **[Satisfactory]**

$h / t = 0.64\ (E_s / F_y)^{0.5}$ = 16.07 , for Tube

[$h / t = 110 / (F_y)^{0.5}$ for AISC-Seismic 97, Tab. 1-9-1]

Where F_y = 46 ksi

E_s = 29000 ksi

CHECK LIMITING SLENDERNESS RATIO FOR V OR INVERTED-V CONFIGURATIONS (AISC 341-05 Sec. 14.2)

$4\ (E_s / F_y)^{0.5}$ = 100.4 **>** K l / r = 99.5 **[Satisfactory]**

[$720 / (F_y)^{0.5}$ for AISC-Seismic 97, Sec. 14.2]

Where K = 1.0

DETERMINE FACTORED DESIGN LOADS TO WITHSTAND LIMITED INELASTIC DEFORMATIONS (AISC 341-05 Sec.14.1)

$P_{ut} = 0.9D - (C_d / I)\ Q_E - 0.2S_{DS}D$ = -146.6 kips (Tension, ASCE 7-05 Sec. 12.8.6)

$P_{uc} = 1.2D + f_1L + (C_d / I)\ Q_E + 0.2S_{DS}D$ = 193.6 kips (Compression Governs, ASCE 7-05 Sec. 12.8.6)

Where C_d = 3 1/4 (ASCE 7-05 Tab. 12.2-1)

I = 1 (CBC Tab.1604.5)

f_1 = 0.5 (CBC 1605.4)

CHECK DESIGN STRENGTH IN COMPRESSION (AISC 360-05 E3)

$\phi_c P_n = \phi_c A_g F_{cr}$ = 248.75 kips **>** P_{uc} **[Satisfactory]**

Where ϕ_c = 0.9

F_e = $\pi^2 E / (KL / r)^2$ = 28.891 ksi

λ_c = $(KL / r)\ (F_y / E)^{0.5}$ = 3.96

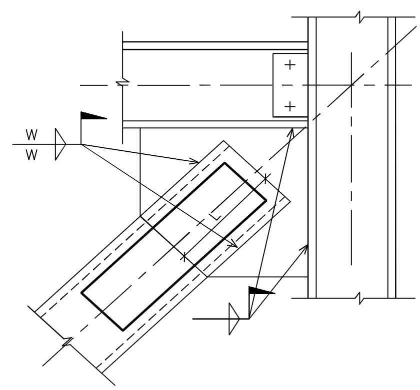
$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = & 23.62 \text{ kis, for } \lambda_c \le 4.71 \\ 0.877\ F_e = & \text{N/A kis, for } \lambda_c > 4.71 \end{cases}$

DETERMINE CONNECTION DESIGN FORCE (AISC 341-05 Sec. 14.4)

$P_{ut} = \mathrm{Min}(R_y F_y A_g\ ,\ \Omega_0 P_u)$ = 387.26 kips

Where R_y = 1.4 (AISC 341-05 Tab. I-6-1) (1.6 for Pipe)

Ω_0 = 2



DETERMINE BEST FILLET WELD SIZE (AISC 360-05 Sec.J2.2b)

w = 5/16 in

> w_{MIN} = 0.1875 in

< w_{MAX} = 0.4375 in

[Satisfactory]

DETERMINE REQUIRED WELD LENGTH (AISC 360-05 Sec.J2.4)

$L = P_{ut} / [(4)\ \phi\ F_w\ (0.707\ w)]$

= 387.26 / [(4) 0.75 (0.6x70)(0.707x5/16)] = 13.91 in

(USE 14 **in)**

CHECK DESIGN SHEAR RUPTURE OF SLOTTED BRACE (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nv}$ = 849.19 kips **>** P_{ut}

Where ϕ = 0.75 **[Satisfactory]**

F_u = 58 ksi (LRFD Tab.1-4, Pg. 1-21)

A_{nv} = 4 t L = 4 x 0.581 x 14 = 32.54 in^2

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE (AISC 360-05 Tab. J2.4)

t_g = 5/8 in

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nu}$ = 456.75 kips **>** P_{ut} **[Satisfactory]**

Where ϕ = 0.75

F_u = 58 ksi (A36 Steel)

A_{nu} = 2 tg L = 2 x 5/8 x 14 = 17.50 in^2

(cont'd)

CHECK TENSION CAPACITY AT SLOTTED BRACE (AISC 360-05 D.2 b)

$\phi P_n = \phi R_t F_u A_e =$ 513.94 kips **>** P_{ut} **[Satisfactory]**

Where $\phi =$ 0.75

$F_u =$ 58 ksi (AISC 13th Tab.2-3)

$x =$ 3 h / 8 = 2.25 , for Tube (HSS Specification 2.1-4)

$D / \pi =$ 1.91 , for Pipe (HSS Specification 2.1-3)

$U =$ MIN(1 - x / L , 0.9) = 0.84 ,(AISC 360-05 Tab. D3.1.)

$A_n = A_g - 2 (t_g + 1/8) t =$ 10.83 in^2

$A_e = U A_n =$ 9.09 in^2

$R_t =$ 1.3 (AISC 341-05 Tab. I-6-1)

Try Cover Plate 3/4 x 5 , at Each Sides.

Region	**x**	**0.5 A_n**	**x A**
HSS	2.25	5.41	12.18
Cover Plate	3.38	3.75	12.66
Σ		9.16	24.84

x = 24.84 / 9.16 = 2.71

U = MIN(1 - x / L , 0.9) = 0.81

A_n = 10.83 + 7.50 = 18.33 in^2

A_e = $U A_n =$ 14.78 in^2

Thus, $\phi P_n = \phi R_t F_u A_e =$ 771.52 kips **>** P_{ut} **[Satisfactory]**

Where $F_u =$ 58 ksi, use plate value

$R_t =$ 1.2 (AISC 341-05 Tab. I-6-1)

Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.

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Seismic Design for Ordinary Concentrically Braced Frames Based on CBC 10 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

WF BRACE SECTION	=>	W14X370		==>	**A**	**r_y**	**k**	**d**	**b_f**
BRACE AXIAL LOAD AT SERVICE LEVEL	D =	100	kips		109.00	4.27	3.26	17.90	16.50
	L =	100	kips					**t_w**	**t_f**
BRACE AXIAL LOAD AT HORIZ. SEISMIC	Q_E =	100	kips (ASCE 7-05 12.4.2.1)					1.66	2.66
SEISMIC PARAMETER	S_{DS} =	0.533	(ASCE 7-05 11.4.4)						
BRACE YIELD STRESS	F_y =	50	ksi		**THE DESIGN IS ADEQUATE.**				
UNBRACED LENGTH OF THE BRACE	l =	31.4	ft						
BUILDING LIMITATION FOR SDC D or E	H =	35	ft (< 35 ft, ASCE 7-05 Tab. 12.2-1)						

REQUIRED CONNECTION => (2 in Gusset Plate with 8 in Length, 8 leg, 3/4 in Fillet Weld. .)

CHECK LIMITING WIDTH THICKNESS RATIO λ_{ps} FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

$b_f / t_f = 0.30 (E_s / F_y)^{0.5}$ = 7.22 > Actual **[Satisfactory]**

($b_f / t_f = 52 / (F_y)^{0.5}$ for AISC-Seismic 97, Tab. 1-9-1)

Where E_s = 29000 ksi

CHECK LIMITING SLENDERNESS RATIO FOR V OR INVERTED-V CONFIGURATIONS (AISC 341-05 Sec. 14.2)

$4 (E_s / F_y)^{0.5}$ = 96.3 > K l / r = 88.2 **[Satisfactory]**

[$720 / (F_y)^{0.5}$ for AISC-Seismic 97, Sec. 14.2]

Where K = 1.0

DETERMINE FACTORED DESIGN LOADS TO WITHSTAND LIMITED INELASTIC DEFORMATIONS (AISC 341-05 Sec.14.1)

$P_{ut} = 0.9D - (C_d / I)\, Q_E - 0.2S_{DS}D$ = -245.7 kips (Tension, ASCE 7-05 Sec. 12.8.6)

$P_{uc} = 1.2D + f_1L + (C_d / I)\, Q_E + 0.2S_{DS}D$ = 505.7 kips (Compression Governs, ASCE 7-05 Sec. 12.8.6)

Where C_d = 3 1/4 (ASCE 7-05 Tab. 12.2-1)

I = 1 (CBC Tab.1604.5)

f_1 = 0.5 (CBC 1605.4)

CHECK DESIGN STRENGTH IN COMPRESSION (AISC 360-05 E3)

$\phi_c P_n = \phi_c A_g F_{cr}$ = 2777.76 kips > P_{uc} **[Satisfactory]**

Where ϕ_c = 0.9

F_e = $\pi^2 E / (KL / r)^2$ = 36.805 ksi

λ_c = $(KL / r) (F_y / E)^{0.5}$ = 3.66

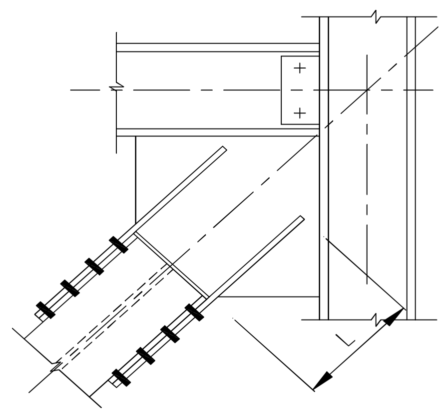
$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = 28.32 & \text{kis, for } \lambda_c \le 4.71 \\ 0.877 F_e = \text{N/A} & \text{kis, for } \lambda_c > 4.71 \end{cases}$

DETERMINE CONNECTION DESIGN FORCE (AISC 341-05 Sec. 14.4)

$P_{ut} = \mathrm{Min}(R_yF_yA_g\ ,\ \Omega_0P_u)$ = 1011.32 kips (Tension)

Where R_y = 1.1 (AISC Seismic 02 & 97 Tab. I-6-1)

Ω_0 = 2



DETERMINE BEST FILLET WELD SIZE (AISC 360-05 Sec.J2.2b)

w = 3/4 in

> w_{MIN} = 0.3125 in

< w_{MAX} = 0.9375 in

[Satisfactory]

DETERMINE REQUIRED WELD LENGTH (AISC 360-05 Sec.J2.4)

$L = P_{ut} / [(8)\, \phi\, F_w\, (0.707\, w)]$

= 1011.32 / [(8) 0.75 (0.6x70)(0.707x3/4)] = 7.57 in

(USE 8 **in)**

CHECK SHEAR RUPTURE OF SLOTTED CON. PLATES (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nv}$ = 1386.43 kips > P_{ut}

Where ϕ = 0.75 **[Satisfactory]**

F_u = 58 ksi (A36 Steel)

A_{nv} = 4 t L = 4 x 1.66 x 8 = 53.12 in^2

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE (AISC 360-05 Tab. J2.4)

t_g = 2 in, Center with Column Web.

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nu}$ = 1670.40 kips > P_{ut} **[Satisfactory]**

Where ϕ = 0.75

F_u = 58 ksi (A36 Steel)

A_{nu} = 4 tg L = 4 x 2 x 8 = 64.00 in^2

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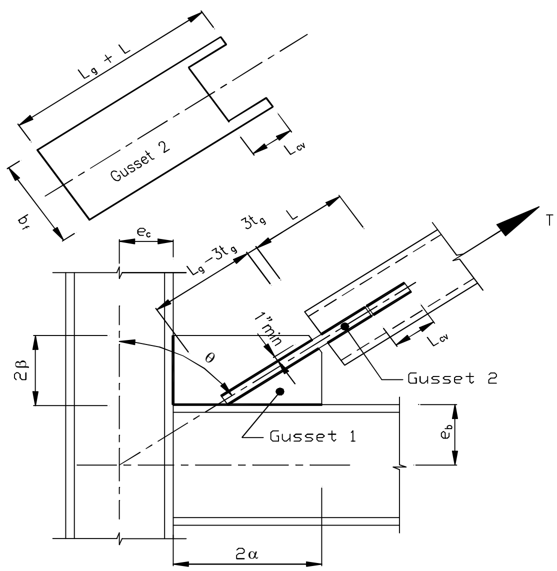
Bracing Connection Design, with Perpendicular Gusset, Based on CBC/IBC & AISC

DESIGN CRITERIA

This bracing connection, with a added perpendicular Gusset 2 plate, may force the brace buckling in-plane frame. There are no cover plate required since the Gusset 2 has concave end. This software can determine two gusset dimensions based on geometry and can check the gusset interface weld capacities on beam and column with moment loads.

INPUT DATA & DESIGN SUMMARY



BRACE AXIAL LOAD AT SERVICE LEVEL $T =$ **396** kips
(AISC 341-05 13.3a)

ANGLE BETWEEN BRACE & COLUMN $\theta =$ **35** ^{0}

THE TWO GUSSET PLATE THICKNESS $t_g =$ **2** in

BRACE SECTION (Tube or Pipe) => **HSS8X8X5/8**

Tube	A	r_{min}	t	b	h
	16.40	2.98	0.63	8.00	8.00

COLUMN SECTION => **W12X65**
ORIENTATION = **x-x** , $e_c =$ **6.05** in

BEAM SECTION => **W16X40**
ORIENTATION = **x-x** , $e_b =$ **8.00** in

4 Legs with 5/8" Fillet Weld $L =$ 11.00 in
$L_g =$ 19.65 in $L_{cv} =$ 3.00 in
2 Legs with 5/8" Fillet Weld $2\beta =$ 16.73 in
$2\alpha =$ 11.27 in

THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

CHECK LIMITING WIDTH THICKNESS RATIO FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

$D / t = 0.044\ E_s / F_y$ = 27.74 , for Pipe > Actual **[Satisfactory]**

$b / t = 0.64\ (E_s / F_y)^{0.5}$ = 16.07 , for Tube (AISC 360-05, B4.2.d)

Where $E_s =$ 29000 ksi
$F_y =$ 46 ksi

DETERMINE BEST FILLET WELD SIZE PER BRACE THICKNESS (AISC 360-05 J2.2b)

$w =$ 0.625 in > $w_{MIN} =$ 0.25 in

(USE w = 0.625 in) < $w_{MAX} = (\phi\ 0.6\ F_u\ t) / (\phi\ 0.707\ F_{EXX}) = (0.75 \times 0.6 \times 58\ \text{ksi})\ t / (0.75 \times 0.707 \times 70\ \text{ksi})$

[Satisfactory] $= 1.1795\ t =$ 0.74 in

DETERMINE REQUIRED WELD LENGTH AT BRACE (AISC 360-05 J2.4)

$L = \Omega\ T / [(4)\ (0.6)\ F_{EXX}\ (0.707\ w)]$

$= (2.0)\ (396.00) / [(4)\ (0.6)\ (70)\ (0.707 \times 5/8)] =$ 10.67 in **(USE L = 11.00 in)**

DETERMINE REQUIRED CONCAVE END AT GUSSET 2 PLATE (AISC 360-05 J2.4)

$L_{cv} = \Omega\ F_y\ A_e / [(4)\ (0.6)\ F_{EXX}\ (0.707\ w)]$

$= (2.0)\ (46)(2.66) / [(4)\ (0.6)\ (70)\ (0.707 \times 5/8)] =$ 3.29 in **(USE $L_{cv} =$ 3.00 in)**

< 0.5 L **[Satisfactory]**

DETERMINE GUSSET DIMENSIONS BASED ON GEOMETRY

$b_f =$ 11.4 in , min wide of Gusset 2 plate < Max($b_{f,col}$, $b_{f,bm}$) = 12.0 in

$L_g =$ **19.65** in > L **[Satisfactory]**

$2\beta =$ 16.7 in , $2\alpha =$ 11.3 in

DETERMINE CONNECTION INTERFACE FORCES (AISC Manual 13th Edition, Page 13-10)

$\beta =$ 8.37 in

$\alpha =$ 5.63 in > $(e_b + \beta)\tan\theta - e_c =$ 5.41 in

[The original Uniform Force Method may not apply]

$K = e_b \tan\theta - e_c =$ -0.45 in

$D = \tan^2\theta + (\alpha / \beta)^2 =$ 0.9438

$K' = \alpha\ (\tan\theta + \alpha / \beta) =$ 7.7394

$\alpha_{Ideal} = (K' \tan\theta + K\ (\alpha / \beta)^2\ ) / D =$ 5.53 in

(cont'd)

$\beta_{Ideal} = (\alpha_{Ideal} - K) / \tan\theta =$ 8.53 in

$r = [(e_b + \beta_{Ideal})^2 + (e_c + \alpha_{Ideal})^2]^{0.5} =$ 20.18 in

$V_c = (\beta_{Ideal} / r)\, T =$ 167.4 kips

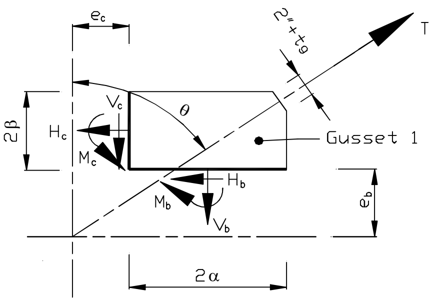
$H_c = (e_c / r)\, T =$ 118.7 kips

$M_c = H_c [\, \beta_{Ideal} - \beta \,] =$ 1.6 ft-kips

$V_b = (e_b / r)\, T =$ 157.0 kips

$H_b = (\alpha_{Ideal} / r)\, T =$ 108.4 kips

$M_b = V_b [\, \alpha_{Ideal} - \alpha \,] =$ -1.4 ft-kips



CHECK WELD CAPACITY AT INTERFACES (AISC 360-05 J2.4)

$f_{Vc} = V_c / (4\, \beta\, 0.707\, w) =$ 11.32 ksi

$f_{Hc} = H_c / (4\, \beta\, 0.707\, w) =$ 8.03 ksi

$f_{Mc} = 3\, M_c / (4\, \beta^2\, 0.707\, w) =$ 0.48 ksi

$f_{Vb} = V_b / (4\, \alpha\, 0.707\, w) =$ 15.76 ksi

$f_{Hb} = H_b / (4\, \alpha\, 0.707\, w) =$ 10.89 ksi

$f_{Mb} = 3\, M_b / (4\, \alpha^2\, 0.707\, w) =$ 0.90 ksi

$\Omega =$ 2.0

$f_{v,c} = [(f_{Vc})^2 + (f_{Hc} + f_{Mc})^2]^{0.5} =$ 14.16 ksi $<$ $0.6\, F_{EXX} / \Omega =$ 21.00 ksi **[Satisfactory]**

$f_{v,b} = [(f_{Vb})^2 + (f_{Hb} + f_{Mb})^2]^{0.5} =$ 19.68 ksi $<$ $0.6\, F_{EXX} / \Omega =$ 21.00 ksi **[Satisfactory]**

CHECK SHEAR RUPTURE CAPACITY OF SLOTTED BRACE (AISC 360-05 J4.2)

$R_{n,rup,brace} / \Omega = (0.6 F_u) A_{nu} / \Omega =$ 478.5 kips $>$ T **[Satisfactory]**

Where $F_u =$ 58 ksi (AISC Manual 13th Edition, Pg. 2-39)

$A_{nu} = 4\, t\, L = 4 \times 0.625 \times 11 =$ 27.50 in^2

$\Omega =$ 2.0

CHECK SHEAR RUPTURE CAPACITY OF GUSSET 2 PLATE (AISC 360-05 J4.2)

$R_{n,rup,gusset2} / \Omega = (0.6 F_u) A_{nv2} / \Omega =$ 765.6 kips $>$ T **[Satisfactory]**

Where $F_u =$ 58 ksi (A36 Steel)

$A_{nv2} = 2\, t_g\, L = 2 \times 2 \times 11 =$ 44.00 in^2

$\Omega =$ 2.0

CHECK SHEAR RUPTURE CAPACITY OF GUSSET 1 PLATE (AISC 360-05 J4.2)

$R_{n,rup,gusset1} / \Omega = (0.6 F_u) A_{nv1} / \Omega =$ 949.77 kips $>$ T **[Satisfactory]**

Where $F_u =$ 58 ksi (A36 Steel)

$A_{nv1} = 2\, t_g\, (L_g - 3 t_g) = 2 \times 2 \times 13.6 =$ 54.58 in^2

$\Omega =$ 2.0

CHECK GUSSET 2 TENSION YIELDING CAPACITY (AISC 360-05 D2 a)

$P_n / \Omega = F_y b_f t_g / \Omega =$ 410.19 kips $>$ T **[Satisfactory]**

Where $F_y =$ 36 ksi (plate value)

CHECK GUSSET 1 & 2 COMPRESSION CAPACITY (AISC 341-05 13.3c)

$P_n / \Omega = F_{cr} b_f t_g / \Omega =$ 403.78 kips $>$ T **[Satisfactory]**

Where $K =$ 1.2 (SEAOC Vol.3 page 40)

$b =$ 6.88 in, (Gusset 1 effective wide)

$A =$ 32.54 in^2

$I =$ 60.50 in^4

$r_g = (I / A)^{0.5} =$ 1.36 in

$K L_g / r_g < 200$ **[Satisfactory]**

$\lambda_c = (K L_g / r_g) (F_y / E)^{0.5} =$ 0.609

$F_e =$ 957.27 ksi (AISC 360-05 E3)

$F_{cr} =$ 35.44 ksi (AISC 360-05 E3)

CHECK GUSSET 1 & 2 BLOCK SHEAR CAPACITY (AISC 360-05 J4.3)

$R_{n,guss} / \Omega = \text{Min}\, [0.6 F_u A_{nv}, 0.6 F_y A_{gv}] / \Omega + U_{bs} F_u A_{nt} / \Omega =$ 765.6 $+ U_{bs} F_u A_{nt} / \Omega$

$>$ T = 396.0 **[Satisfactory]**

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Seismic Design for Special Concentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

BEAM SECTION => **W14X605** =>

A	d	t_w	b_f	t_f	S_x
178	20.9	2.60	17.40	4.16	1040
	I_x	r_x	r_y	Z_x	k
	10800	7.79	4.55	1320	4.76

BEAM DISTRIBUTED SERVICE LOADS D = **0.24** kips / ft

L = **0.5** kips / ft

BEAM LENGTH l = **26** ft

TOP FLANGE CONTINUOUSLY BRACED **?** **0** No

BEAM YIELD STRESS F_y = **50** ksi

THE BEAM DESIGN IS ADEQUATE.

DETERMINE FACTORED AXIAL LOAD ON THE BEAM (AISC 341-05 13.4a)

$P_u = 0.5\,(R_y F_y A_g + 0.3\phi_c P_n)\cos\alpha =$ 386.09 kips

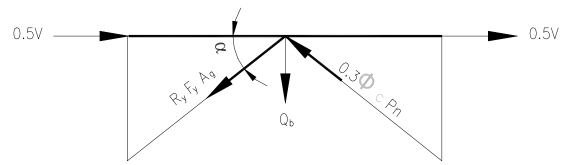
Where R_y = 1.4 (see brace sheet)

α = 49.46 ^{o}

F_y = 46 ksi (see brace sheet)

A_g = 16.40 in^2 (see brace sheet)

$\phi_c P_n$ = 439.36 kips (see brace sheet)



CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 2.09 **<** $0.3\,(E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

Where E_s = 29000 ksi

$h / t_w =$ 4.38 **<**
$3.14(E_s/F_y)^{0.5}(1-1.54P_u/\phi_b P_y) =$ 70.01 , for $P_u/\phi_b P_y \le 0.125$
$(E_s/F_y)^{0.5}\mathrm{Max}[1.49\ ,\ 1.12\,(2.33-P_u/\phi_b P_y)] =$ N/A , for $P_u/\phi_b P_y > 0.125$

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_y A$ = 8900 kips

DETERMINE UNBALANCED VERTICAL FORCE ON BEAM (AISC 341-05 13.4a)

$Q_b = (R_y F_y A_g - 0.3\phi_c P_n)\sin\alpha =$ 702.45 kips (Vertical)

DETERMINE FACTORED MOMENT ON THE BEAM (IBC 09 1605.2 & ASCE 7-05 12.4.2.3)

$M_{nt} = (1.2D + L)\, l^2 / 8 + Q_b\, l / 4 =$ 4632.48 ft-kips

DETERMINE UNBALANCED SEGMENT LENGTH ABOUT X - AND Y - AXES

l_x = 26 ft

l_y = 13 ft (AISC Seismic Sec.13.4a ,lateral support at the intersection of chevron braces required.)

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$\phi_c P_n = \phi_c F_{cr} A =$ 7123.39 kips **>** P_u **[Satisfactory]**

Where ϕ_c = 0.9

K = 1.0

$\mathrm{MAX}(K l_x/r_x,\ K l_y/r_y)$ = 40.05 **<** 200 **[Satisfactory]**

λ_c = $(K l\ /\ r)\ (F_y\ /\ E)^{0.5}$ = 1.663

F_e = $\pi^2\, E\ /\ (K l\ /\ r)^2$ = 178.4 ksi (AISC 360-05 E3)

F_{cr} = 44.47 ksi (AISC 360-05 E3)

DETERMINE FLEXURAL DESIGN STRENGTH (AISC-AISC 360-05 F1)

L_b = 13.00 ft

$L_p = 1.76\, r_y\, (E / F_{yf})^{0.5}$ = 16.06 ft

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_0}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x h_0}{Jc}\right)^2}} = 231.67 \text{ ft}$$

$M_p = F_y Z_x$ = 5500.0 ft-kips

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\frac{Jc}{S_x h_0}\left(\frac{L_b}{r_{ts}}\right)^2} = 928.14 \text{ ksi}$$

Where $r_{ts} = [(\,I_y\, C_w)^{0.5} / S_x]^{0.5}$ = 5.44

c = 1.00

$h_0 = d - t_f$ = 16.74 in

C_b = 1.30 , (AISC 360-05 F1)

I_y	G	J	C_w
3680	11200	869	258000

(cont'd)

M_n	=	M_p	=	5500	ft-kips, for L_b @ [0 , L_p]
		MIN{C_b [M_p - (M_p - 0.75 F_y S_x) (L_b - L_p) / (L_r - L_p)] , M_p}	=	N/A	ft-kips, for L_b @ (L_p , L_r]
		MIN(F_{cr} S_x , M_p)	=	N/A	ft-kips, for L_b @ (L_r , Larger)

$\phi_b\ M_n$ = 0.9 M_n = 4950 ft-kips

CHECK FLEXURAL CAPACITY (AISC 360-05 C2.1b)

$M_u = B_1 M_{nt}$ = 4689.50 ft-kips **<** $\phi_b M_{nx}$ = 4950 ft-kips

Where $P_{e1} = \pi^2 E_s I_x / (K\ell_x)^2$ = 31755 kips **[Satisfactory]**

C_m = 1.0 (AISC 360-05 C2.1b)

α = 1.0

B_1 = MAX[$C_m/(1 - \alpha\ P_u/P_{e1})$, 1.0] = 1.012

CHECK INTERACTION CAPACITY (AISC 360-05 H1.1)

For $P_u/\phi_c P_n \geq 0.2$, $P_u / \phi_c P_n + 8 / 9\ (M_{ux} / \phi_b M_{nx})$ = N/A

For $P_u/\phi_c P_n < 0.2$, $P_u / (2\phi_c P_n) + M_{ux} / \phi_b M_{nx}$ = 0.97 **<** **1** **[Satisfactory]**

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Seismic Design for Special Concentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION	=>	**W8X40**		=>	**A**	**d**	$\mathbf{t_w}$	$\mathbf{b_f}$	$\mathbf{t_f}$	$\mathbf{S_x}$
COLUMN AXIAL SERVICE LOADS	D =	**22**	kips		11.7	8.3	0.36	8.07	0.56	35.5
	L =	**29**	kips			$\mathbf{I_x}$	$\mathbf{r_x}$	$\mathbf{r_y}$	$\mathbf{Z_x}$	**k**
COLUMN AXIAL LOAD AT HORIZ. SEISMIC	Q_E =	**18**	kips, (ASCE 7-05 12.4.2.1)			146	3.53	2.05	40	0.95
UNBRACED COLUMN LENGTH	ℓ =	**14**	ft							
COLUMN YIELD STRESS (36 or 50)	F_y =	**50**	ksi							

THE COLUMN DESIGN IS ADEQUATE.

DETERMINE FACTORED DESIGN LOADS (IBC 09 1605.2 & ASCE 7-05 12.4.2.3)

$P_{ut} = 0.9D - \rho Q_E - 0.2S_{DS}D =$ -2 kips (Tension)

$P_{uc} = 1.2D + L + \rho Q_E + 0.2S_{DS}D =$ 77 kips (Compression, Governs)

Where $\rho =$ 1

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 7.21 **<** $0.3\,(E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

Where $E_s =$ 29000 ksi

$h / t_w =$ 17.62 **<**
- $3.14(E_s/F_y)^{0.5}(1-1.54P_u/\phi_bP_y) =$ N/A , for $P_u/\phi_bP_y < 0.125$
- $(E_s/F_y)^{0.5}\mathrm{Max}[1.49\ ,\ 1.12\ (2.33-P_u/\phi_bP_y)] =$ 58.89 , for $P_u/\phi_bP_y > 0.125$

[Satisfactory]

Where $\phi_c =$ 0.9

$P_y = F_yA =$ 585 kips

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$\phi_cP_n = \phi_cF_{cr}A =$ 321.98 kips **>** P_u **[Satisfactory]**

Where $\phi_c =$ 0.9

K = 1.0

$\mathrm{MAX}(K\ell_x/r_x,\ K\ell_y/r_y) =$ 82.01 **<** 200 **[Satisfactory]**

$\lambda_c =$ $(K\ell\ /\ r)\ (F_y\ /\ E)^{0.5} =$ 3.405 (AISC 360-05 E2-4, Pg 6-47)

$F_e =$ $\pi^2\ E\ /\ (K\ell\ /\ r)^2 =$ 42.557 ksi (AISC 360-05 E3)

$F_{cr} =$ 30.58 ksi (AISC 360-05 E3)

CHECK AMPLIFIED SEISMIC LOAD EFFECTS FOR P_u / f P_n > 0.4 (AISC 341-05 8.3)

$P_{uc} / \phi\ P_n =$ 0.24 **<** 0.4 **[Amplified Seismic Load Do Not Need to Checke]**

$P_{uc} = 1.2D + L + \Omega_0Q_E + 0.2S_{DS}D =$ 95 kips **<** ϕ_cP_n **[Satisfactory]**

Where $\Omega_0 =$ 2

Seismic Design for Special Concentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION	= >	**HSS8X8X5/8**		**= > Tube**	**A**	**r_{min}**	**t**	**h**
COLUMN AXIAL SERVICE LOADS	D =	**22**	kips		16.40	2.98	0.63	8.00
	L =	**29**	kips					
COLUMN AXIAL LOAD AT HORIZ. SEISMIC	Q_E =	**18**	kips, (ASCE 7-05 12.4.2.1)					
UNBRACED COLUMN LENGTH	ℓ =	**14**	ft					
COLUMN YIELD STRESS (42 or 46)	F_y =	**46**	ksi					

THE COLUMN DESIGN IS ADEQUATE.

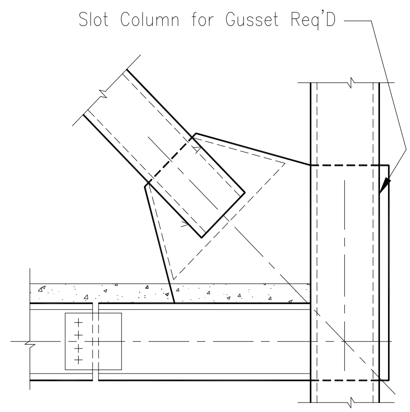
ANALYSIS

CHECK LIMITING WIDTH THICKNESS RATIO FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 02 Tab. I-8-1)

$D / t = 0.044\ E_s / F_y$ = 27.74 , for Pipe

$h / t = 0.64\ (E_s / F_y)^{0.5}$ = 16.07 , for Tube **>** Actual **[Satisfactory]** (AISC 360-05, B4.2.d)

Where E_s = 29000 ksi



DETERMINE FACTORED DESIGN LOADS (IBC 09 1605.2 & ASCE 7-05 12.4.2.3)

$P_{ut} = 0.9D - \rho Q_E - 0.2S_{DS}D$ = -2 kips (Tension)

$P_{uc} = 1.2D + L + \rho Q_E + 0.2S_{DS}D$ = 77 kips (Compression, Governs)

Where ρ = 1

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$\phi_c P_n = \phi_c F_{cr} A$ = 548.56 kips **>** P_u **[Satisfactory]**

Where ϕ_c = 0.9

K = 1.0

$MAX(K\ell_x/r_x,\ K\ell_y/r_y)$ = 56.31 **<** 200 **[Satisfactory]**

λ_c = $(K\ell / r)\ (F_y / E)^{0.5}$ = 2.243 (AISC 360-05 E2-4, Pg 6-47)

F_e = $\pi^2 E / (K\ell / r)^2$ = 90.279 ksi (AISC 360-05 E3)

F_{cr} = 37.17 ksi (AISC 360-05 E3)

CHECK AMPLIFIED SEISMIC LOAD EFFECTS FOR P_u / f P_n > 0.4 (AISC 341-05 8.3)

$P_{uc} / \phi P_n$ = 0.14 **<** 0.4 **[Amplified Seismic Load Do Not Need to Checke]**

$P_{uc} = 1.2D + L + \Omega_0 Q_E + 0.2S_{DS}D$ = 95 kips **<** $\phi_c P_n$ **[Satisfactory]**

Where Ω_0 = 2

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Seismic Design for Special Concentrically Braced Frames Based on CBC/IBC & AISC

INPUT DATA & DESIGN SUMMARY

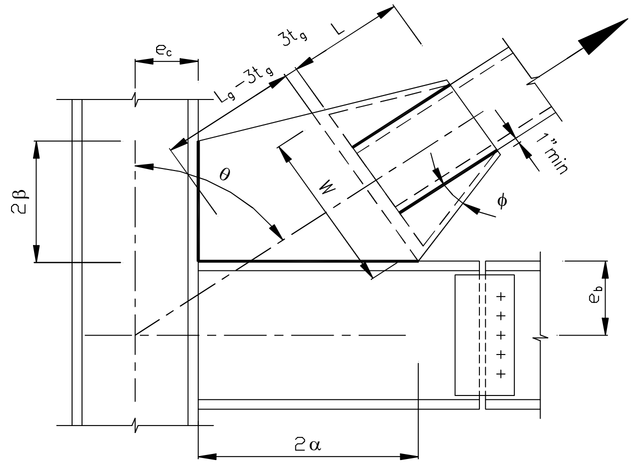
BRACE SECTION (Tube or Pipe) => **HSS8X8X5/8**

Tube	A	r_{min}	t	b	h
	16.40	2.98	0.63	8.00	8.00

BRACE AXIAL LOAD AT SERVICE LEVEL D = **10** kips
L = **10** kips
SEISMIC AXIAL LOAD (ASCE 7-05 12.4.2.1) Q_E = **190** kips
SEISMIC PARAMETER (ASCE 7-05 11.4.4) S_{DS} = **0.877**
REDUNDANCY FACTOR (ASCE 7-05 12.3.4) ρ = **1**

UNBRACED LENGTH OF THE BRACE L = **20** ft
ANGLE BETWEEN BRACE & COLUMN θ = **36** ^{0}
ANGLE BTW BRACE & GUSSET EDGE ϕ = **34** ^{0}
COLUMN SECTION e_c = **4** in
BEAM SECTION e_b = **8.1** in

LENGTH OF END BRACE TO JUNCTION L_g = 12.023 in
LENGTH OF GUSSET TO COLUMN 2β = 15.6 in
LENGTH OF GUSSET TO BEAM 2α = 17.2 in
THE WHITMORE WIDTH W = 17.2 in



CBC 10 Chapter A (DSA or OSHPD) APPLY? ==> **Yes**

(1 in Gusset Plate with 8 in Length, 4 leg, 5/8" Fillet Weld at Brace.)
(Fill 3000 psi Solid Cement Grout in All Tube Bracings.)

THE BRACE DESIGN IS ADEQUATE.

ANALYSIS

CHECK LIMITING WIDTH THICKNESS RATIO FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

$D / t = 0.044\, E_s / F_y$ = 27.74 , for Pipe
$b / t = 0.64\, (E_s / F_y)^{0.5}$ = 16.07 , for Tube
> Actual **[Satisfactory]** (AISC 360-05, B4.2.d)

Where E_s = 29000 ksi
F_y = 46 ksi

CHECK LIMITING SLENDERNESS RATIO FOR V OR INVERTED-V CONFIGURATIONS (AISC 341-05 Sec. 13.2a)

$4.0\,(E_s / F_y)^{0.5}$ = 100.4 > $K\,L / r$ = 80.4 **[Satisfactory]**
Where K = 1.0

DETERMINE FACTORED DESIGN LOADS (CBC 10 1605.2 & ASCE 7-05 12.4.2.3)

$P_{ut} = 0.9D - \rho Q_E - 0.2S_{DS}D$ = -182.75 kips (Tension)
$P_{uc} = 1.2D + L + \rho Q_E + 0.2S_{DS}D$ = 213.75 kips (Compression, Governs)

CHECK DESIGN STRENGTH IN COMPRESSION (AISC 360-05 E3)

$\phi_c P_n = \phi_c A_g F_{cr}$ = 439.36 kips > P_{uc} **[Satisfactory]**

Where ϕ_c = 0.9
F_e = $\pi^2 E / (KL / r)^2$ = 44.237 ksi
λ_c = $(KL / r)\,(F_y / E)^{0.5}$ = 3.20

$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = 29.77 & \text{kis, for } \lambda_c \le 4.71 \\ 0.877\,F_e = \text{N/A} & \text{kis, for } \lambda_c > 4.71 \end{cases}$

DETERMINE CONNECTION DESIGN FORCE (AISC 341-05 Sec. 13.3a)

$P_{ut} = \text{MIN}(R_y F_y A_g\,,\, P_{max})$ = 403.75 kips (Tension)
Where R_y = 1.4 (AISC 341-05 Tab. I-6-1)
P_{max} = 403.75 kips, (the estimated maximum earthquake force, that can be transferred to the brace by the system.)

DETERMINE BEST FILLET WELD SIZE (AISC 360-05 Sec.J2.2b)

w = 5/8 in > w_{MIN} = 0.25 in
(USE w = 0.625 in) < w_{MAX} = $(\phi\, 0.6\, F_u\, t) / (\phi\, 0.707\, F_{EXX})$ = (0.75 x 0.6 x 58 ksi) t / (0.75 x 0.707 x 70 ksi)
[Satisfactory] = 1.1795 t = 0.74 in

(cont'd)

DETERMINE REQUIRED WELD LENGTH (AISC 360-05 Sec.J2.4)

$L = P_{ut} / [(4)\ \phi\ F_w\ (0.707\ w)]$

$= 403.75 / [(4)\ 0.75\ (0.6x70)(0.707x5/8)] =$ 7.25 in

(USE 8 in)

CHECK SHEAR RUPTURE CAPACITY OF SLOTED BRACE

(AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nu} =$ 522.00 kips **>** P_{ut}

Where ϕ = 0.75 **[Satisfactory]**

F_u = 58 ksi (AISC 13th Tab.2-3)

A_{nu} = 4 t L = 4 x 0.625 x 8 = 20.00 in^2

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE (AISC 360-05 Tab. J2.4)

t_g = 1 in **(USE 1 in)**

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 Sec.J4.2)

$\phi P_n = \phi(0.6F_u)A_{nv} =$ 417.60 kips **>** P_{ut} **[Satisfactory]**

Where ϕ = 0.75

F_u = 58 ksi (A36 Steel)

A_{nv} = 2 tg L = 2 x 1 x 8 = 16.00 in^2

CHECK GUSSET BLOCK SHEAR CAPACITY (AISC 360-05 J4.3)

$\phi R_n = \phi(0.6F_u)A_{nv} + \phi F_y A_{gt} =$ 417.60 + $\phi F_y A_{gt}$

> P_{ut} = 403.75

[Satisfactory]

GUSSET COMPRESSION CAPACITY (AISC 341-05 13.3c)

$\phi_c P_n = \phi_c F_{cr} L_w t_g =$ 489.68 kips **<** 1.1 R_y P_n

Where ϕ_c = 0.9 **[Unsatisfactory]**

K = 1.2 (SEAOC Vol.3 page 40)

$r_g = t_g / (12)^{0.5} =$ 0.29 in

$K\ L_g / r_g$ < 200 **[Satisfactory]**

$\lambda_c = (KL_g / r_g)\ (F_y / E)^{0.5} =$ 1.761

F_e = 114.58 ksi (AISC 360-05 Sec.E3)

F_{cr} = 31.564 ksi (AISC 360-05 Sec.E3)

(Gusset Stiffer Reqd, or Increase tg.)

CHECK GUSSET TENSION YIELDING CAPACITY (AISC 360-05 D2 a)

$\phi_t P_n = \phi_t F_y L_w t_g =$ 558.50 kips **>** P_{ut}

Where ϕ_t = 0.9 **[Satisfactory]**

F_y = 36 ksi (plate value)

L_w = W = 17.2 in

CHECK SHEAR LAG FRACTURE OF BRACE (AISC 360-05 D.2 b)

$\phi P_n = \phi\ R_t\ F_u\ A_e =$ 529.94 kips **>** P_{ut} **[Satisfactory]**

Where ϕ = 0.75

F_u = 58 ksi (AISC 13th Tab.2-3)

x = B^2+2BH / 4(B+H) = 3.00 , for Tube (AISC Tab. D3.1)

D / π = 2.55 , for Pipe (AISC 360 Tab. D3.1)

U = MIN(1 - x / L , 0.9) = 0.63 ,(AISC 360-05 Tab. D3.1.)

$A_n = A_g - 2\ (t_g + 1/8)\ t =$ 14.99 in^2

$A_e = U\ A_n =$ 9.37 in^2

R_t = 1.3 (AISC 341-05 6.2)

Try Cover Plate 0 x 7 , at Each Sides.

(0 for no cover required)

Region	**x**	**0.5 A_n**	**x A**
HSS	3.00	7.50	22.49
Cover Plate	4.00	0.00	0.00
Σ		7.50	22.49

x = 22.49 / 7.50 = 3.00

U = MIN(1 - x / L , 0.9) = 0.63

$\mathbf{A_n}$ = 14.99 + 0.00 = 14.99

$\mathbf{A_e}$ = U A_n = 9.37 in^2

Thus, $\phi P_n = \phi\ R_t\ F_u\ A_e =$ 529.94 kips **>** P_{ut} **[Satisfactory]**

Where F_u = 58 ksi (plate value)

Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.

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Seismic Design for Buckling-Restrained Braced Frames Based on AISC 360-05 & AISC 341-05

DESIGN CRITERIA

1. Bracing member self shall be composed of a structural steel core & a system that restrains the steel core from buckling. The each end of brace to gusset plate may be standard bolted connections, modified bolted connections, or true pin connections. Selecting bracing members are based on the tested & approved manufacture's lists.
2. The lateral capacity of BRBF, selected bracing members with steel beams and columns, must also be checked by a special loads combination of DL + LL + 2 $\delta_x/1.5$ (AISC 341-05, 16.2b, 16.5b & ASCE 7-05, 12.8.6).
3. For the connection of gusset plate to beam and column, the interface dimensions of α and β may not satisfy the basic relationship of the Uniform Force Method: $\alpha - \beta \tan\theta = e_b \tan\theta - e_c$. This software can determine gusset dimensions based on geometry and can check the gusset interface weld capacities with moment loads.

INPUT DATA & DESIGN SUMMARY

BRACE CORE AXIAL YIELD STRENGTH P_{ysc} = **150** kips, SD
(from tested & approved manufacture's lists, AISC 341-05, 16.2a)

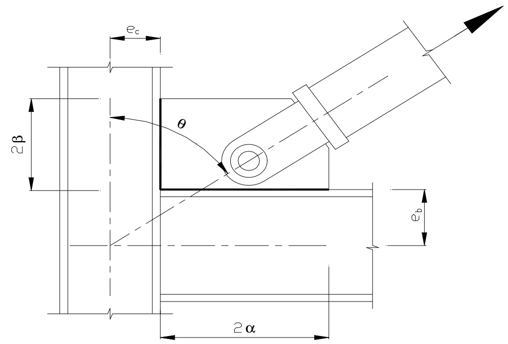
ANGLE BETWEEN BRACE & COLUMN θ = **25** ^{0}

GUSSET DIMENSIONS 2 β = **24.0** in
2 α = 10.1 in

COLUMN SECTION => **W12X96**
ORIENTATION = **x-x** , e_c = 6.35 in

BEAM SECTION => **W16X67**
ORIENTATION = **x-x** , e_b = 8.15 in

(1" Gusset Plate with 7/16" Fillet Weld, 2 leg x 24" at Column Interface, 2 leg x 11" at Beam Interface.)



THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE REQUIRED STRENGTH OF BRACING CONNECTION (AISC 341-05, 16.3a & 16.2d)

$T = (1.1 / 1.5)\ \beta\ \omega\ R_y\ P_{ysc}$ = 247.1 kips, ASD

Where β = 1.20 , (AISC 341-05, 16.2d. May be verified by nonlinear time-history or analyses.)
ω = 1.44 , (AISC 341-05, 16.2d. May be verified by nonlinear time-history or analyses.)
R_y = 1.30 , (AISC 341-05, Table I-6-1)

DETERMINE BEST FILLET WELD SIZE PER THICKNESS OF GUSSET & FLANGES (AISC 360-05 J2.2b)

w = 7/16 in > w_{MIN} = 0.25 in
< w_{MAX} = 0.5625 in **[Satisfactory]**

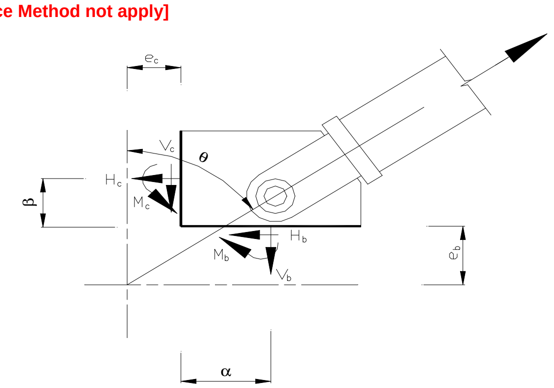
DETERMINE REQUIRED THICKNESS OF GUSSET PLATE

t_g = 1 in **(USE t_g = 1 in)**

DETERMINE CONNECTION INTERFACE FORCES (AISC Manual 13th Edition, Page 13-10)

β = 12.00 in
α = 5.07 in > $(e_b + \beta)\tan\theta - e_c$ = 3.05 in
[the original Uniform Force Method not apply]

$K = e_b \tan\theta - e_c$ = -2.55 in
$D = \tan^2\theta + (\alpha / \beta)^2$ = 0.396
$K' = \alpha (\tan\theta + \alpha / \beta)$ = 4.5081
$\alpha_{Ideal} = (K' \tan\theta + K (\alpha / \beta)^2) / D$ = 4.16 in
$\beta_{Ideal} = (\alpha_{Ideal} - K) / \tan\theta$ = 14.38 in
$r = [(e_b + \beta_{Ideal})^2 + (e_c + \alpha_{Ideal})^2]^{0.5}$ = 24.86 in
$V_c = (\beta_{Ideal} / r)\ T$ = 143.0 kips
$H_c = (e_c / r)\ T$ = 63.1 kips
$M_c = H_c [\beta_{Ideal} - \beta]$ = 12.5 ft-kips
$V_b = (e_b / r)\ T$ = 81.0 kips
$H_b = (\alpha_{Ideal} / r)\ T$ = 41.3 kips
$M_b = V_b [\alpha_{Ideal} - \alpha]$ = -6.2 ft-kips



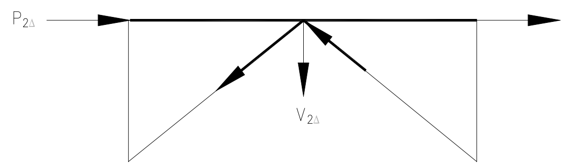
CHECK WELD CAPACITY AT INTERFACES (AISC 360-05 J2.4)

$f_{Vc} = V_c / (4\ \beta\ 0.707\ w)$ = 10.37 ksi
$f_{Hc} = H_c / (4\ \beta\ 0.707\ w)$ = 4.58 ksi
$f_{Mc} = 3\ M_c / (4\ \beta^2\ 0.707\ w)$ = 2.73 ksi
$f_{Vb} = V_B / (4\ \alpha\ 0.707\ w)$ = 13.90 ksi
$f_{Hb} = H_B / (4\ \alpha\ 0.707\ w)$ = 7.09 ksi
$f_{Mb} = 3\ M_b / (4\ \alpha^2\ 0.707\ w)$ = 7.51 ksi
Ω = 2.0
$f_{v,c} = [(f_{Vc})^2 + (f_{Hc} + f_{Mc})^2]^{0.5}$ = 12.68 ksi < $0.6\ F_{EXX} / \Omega$ = 21.00 ksi **[Satisfactory]**
$f_{v,b} = [(f_{Vb})^2 + (f_{Hb} + f_{Mb})^2]^{0.5}$ = 20.16 ksi < $0.6\ F_{EXX} / \Omega$ = 21.00 ksi **[Satisfactory]**

Seismic Design for Buckling-Restrained Braced Frames Based on AISC 360-05 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

BEAM SECTION	=>	W16X67		=>	**A**	**d**	**t_w**	**b_f**	**t_f**	**S_x**
BEAM DISTRIBUTED SERVICE LOADS	D =	0.24	kips / ft		20	16.3	0.40	10.20	0.67	119
	L =	0.5	kips / ft			**I_x**	**r_x**	**r_y**	**Z_x**	**k**
BEAM LENGTH	$\mathcal{L}$ =	26	ft			970	6.96	2.44	132	1.37
TOP FLANGE CONTINUOUSLY BRACED **?**		1	Yes							
BEAM AXIAL LOAD AT $2\delta_x$/1.5 STORY DRIFT	$P_{2\delta x}$ =	150	kips,ASD (AISC 341, 16.5b)							
BEAM VERT LOAD AT $2\delta_x$/1.5 STORY DRIFT	$V_{2\delta x}$ =	25	kips							
BEAM YIELD STRESS	F_y =	50	ksi							



THE BEAM DESIGN IS ADEQUATE.

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1) <= does not apply for top flange continuously braced.

$b_f / (2t_f)$ = 7.67 < $0.3 (E_s / F_y)^{0.5}$ = N/A **[Satisfactory]**

Where E_s = 29000 ksi

h / t_w = 34.33 <
- $3.14(E_s/F_y)^{0.5}(1-1.54P_{2\Delta}\Omega_o/P_y)$ = N/A , for $P_{2\Delta}\Omega_o/P_y < 0.125$
- $1.12(E_s/F_y)^{0.5}(2.33-P_{2\Delta}\Omega_o/P_y)$ = N/A , for $P_{2\Delta}\Omega_o/P_y > 0.125$

[Satisfactory] Where Ω_o = 2.0 , (AISC 341-05, Table R3-1)

$P_y = F_yA$ = 1000 kips

DETERMINE MOMENT ON THE BEAM

$M = (D + L) \mathcal{L}^2/ 8 + V_{2\delta x} \mathcal{L} / 4$ = 225.03 ft-kips, ASD

DETERMINE UNBALANCED SEGMENT LENGTH ABOUT X - AND Y - AXES

$\mathcal{L}_x$ = 26 ft

$\mathcal{L}_y$ = 13 ft

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$P_n/\Omega_o = F_{cr}A/\Omega_o$ = 370.76 kips > $P_{2\delta x}$ **[Satisfactory]**

Where Ω_o = 2.0 , (AISC 341-05, Table R3-1)

K = 1.0

MAX($K\mathcal{L}_x/r_x$, $K\mathcal{L}_y/r_y$) = 63.95 < 200 **[Satisfactory]**

λ_c = $(K\mathcal{L} / r) (F_y / E)^{0.5}$ = 2.656

F_e = $\pi^2 E / (K\mathcal{L} / r)^2$ = 69.979 ksi (AISC 360-05 E3)

F_{cr} = 37.08 ksi (AISC 360-05 E3)

DETERMINE FLEXURAL DESIGN STRENGTH (AISC-AISC 360-05 F1)

L_b = 13.00 ft

$L_p = 1.76\, r_y (E / F_{yf})^{0.5}$ = 8.62 ft

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_0}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x h_0}{Jc}\right)^2}} = 26.24 \text{ ft}$$

$M_p = F_y Z_x$ = 550.0 ft-kips

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\frac{Jc}{S_x h_0}\left(\frac{L_b}{r_{ts}}\right)^2} = 138.69 \text{ ksi}$$

Where $r_{ts} = [(I_y C_w)^{0.5}/ S_x]^{0.5}$ = 2.80

c = 1.00

$h_0 = d - t_f$ = 15.64 in

C_b = 1.30 , (AISC 360-05 F1)

I_y	**G**	**J**	**C_w**
119	11200	2.62	7300

(cont'd)

M_n	=	M_p	=	N/A	ft-kips, for L_b @ $[0, L_p]$
		$MIN\{C_b [M_p - (M_p - 0.75 F_y S_x) (L_b - L_p) / (L_r - L_p)], M_p\}$	=	550	ft-kips, for L_b @ $(L_p, L_r]$
		$MIN(F_{cr} S_x, M_p)$	=	N/A	ft-kips, for L_b @ $(L_r$, Larger)

M_n/Ω_o = 275 ft-kips

CHECK FLEXURAL CAPACITY (AISC 360-05 C2.1b)

$M = B_1 M$ = 245.71 ft-kips **<** M_n/Ω_o = 275 ft-kips

Where $P_{e1} = \pi^2 E_s I_x / (K\ell_x)^2$ = 2852 kips **[Satisfactory]**

C_m = 1.0 (AISC 360-05 C2.1b)

α = 1.6

$B_1 = MAX[C_m/(1 - \alpha P_{2\Delta}/P_{e1}), 1.0]$ = 1.092

CHECK INTERACTION CAPACITY (AISC 360-05 H1.1)

For $P_{2\Delta}\Omega_o/P_n \geq 0.2$,	$P_{2\Delta}\Omega_o/P_n + 8/9 (M\Omega_o/M_n)$ =	1.20	**<**	**4/3**	**[Satisfactory]**
For $P_{2\Delta}\Omega_o/P_n < 0.2$,	$P_{2\Delta}\Omega_o/(2P_n) + M\Omega_o/M_n$ =	N/A			

Seismic Design for Buckling-Restrained Braced Frames Based on AISC 360-05 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION	= >	**W12X96**		= >	**A**	**d**	**t_w**	**b_f**	**t_f**	**S_x**
COLUMN AXIAL SERVICE LOADS	D =	**22**	kips		28.2	12.7	0.55	12.20	0.90	131
	L =	**29**	kips			**I_x**	**r_x**	**r_y**	**Z_x**	**k**
COL AXIAL LOAD AT $2\delta_x/1.5$ STORY DRIFT	$P_{2\delta x}$ =	**100**	kips, ASD (AISC 341, 16.5b)			833	5.43	3.09	147	1.50
UNBRACED COLUMN LENGTH	ℓ =	**14**	ft							
COLUMN YIELD STRESS (36 or 50)	F_y =	**50**	ksi							

THE COLUMN DESIGN IS ADEQUATE.

DETERMINE DESIGN LOAD (AISC 341-05, 16.2b & 16.5b)

$P = D + L + P_{2\delta x} =$ 151 kips, ASD (Compression, Governs)

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 6.78 **<** $0.3 (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

Where $E_s =$ 29000 ksi

$h / t_w =$ 17.64 **<**
- $3.14(E_s/F_y)^{0.5}(1-1.54P\Omega_o/P_y) =$ N/A , for $P\Omega_o/P_y < 0.125$
- $1.12(E_s/F_y)^{0.5}(2.33-P\Omega_o/P_y) =$ 57.07 , for $P\Omega_o/P_y > 0.125$

[Satisfactory] Where $\Omega_o =$ 2.0 , (AISC 341-05, Table R3-1)

$P_y = F_yA =$ 1410 kips

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$P_n/\Omega_o = F_{cr}A/\Omega_o =$ 568.31 kips **>** P **[Satisfactory]**

Where $\Omega_o =$ 2.0 , (AISC 341-05, Table R3-1)

K = 1.0

$MAX(K\ell_x/r_x, K\ell_y/r_y) =$ 54.29 **<** 200 **[Satisfactory]**

$\lambda_c =$ $(K\ell / r) (F_y / E)^{0.5} =$ 2.254 (AISC 360-05 E2-4, Pg 6-47)

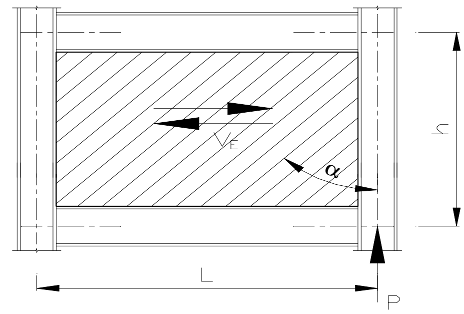
$F_e =$ $\pi^2 E / (K\ell / r)^2 =$ 97.094 ksi (AISC 360-05 E3)

$F_{cr} =$ 40.31 ksi (AISC 360-05 E3)

Seismic Design for Special Plate Shear Wall Based on AISC 341-05 & AISC 360-05

DESIGN CRITERIA

1. Design the boundary elements, columns & beams, under gravity loads, DL & LL, ***without*** web plate exists. (AISC 341-05, 17.4a)
2. Determine seismic forces of boundary elements ***with*** web plate support. Use two layers steel plate strips, one layer angled α & another **-** α, in 3D modeling software. Do not use Orthotropic Finite Element modeling method.
3. Use this software to check the seismic capacities of SPSW per AISC 341-05 Chapter 17.



INPUT DATA & DESIGN SUMMARY

VERTICAL BOUNDARY ELEMENT, VBE => **W14X193**

A	d	t_w	b_f	t_f	S_x
56.8	15.5	0.89	15.70	1.44	310
I_x	r_x	r_y	Z_x	k	
2400	6.50	4.05	355	2.04	

HORIZONTAL BOUNDARY ELEMENT, HBE => **W21X50**

A	d	t_w	b_f	t_f	S_x
14.7	20.8	0.38	6.53	0.54	94.5
I_x	r_x	r_y	Z_x	k	I_y
984	8.18	1.30	110	1.04	25

THE SPSW DESIGN IS ADEQUATE.
(Continuity VBE/column stiffeners 5/16 x 4.8 with 1/4" fillet weld to web & CP to flanges. A doubler plate is not required.)

BOUNDARY STEEL YIELD STRESS	F_y =	**50**	ksi	WEB PLATE YIELD STRESS	$F_{y,WEB}$ =	**36**	ksi
GRAVITY AXIAL LOAD ON VBE	P_{DL} =	**50**	kips	SEISMIC AXIAL LOAD ON VBE	P_E =	**50**	kips
	P_{LL} =	**85**	kips	SEISMIC SHEAR FORCE ON WEB	V_E =	**897**	kips
BEAM LENGTH BETWEEN COL. CENTERS	L =	**30**	ft	PARAMETER (ASCE 7-05 11.4.4)	S_{DS} =	**0.877**	
THICKNESS OF THE WEB PLATE	t_w =	**0.25**	in	STORY HEIGHT	h =	**15**	ft

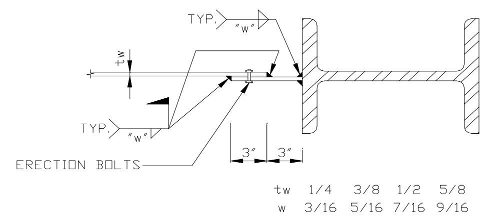
ANALYSIS

CHECK PANEL ASPECT RATIO (AISC 341-05, 17.2b)

L / h = 2.0 **>** 0.8
< 2.5 **[Satisfactory]**

CHECK WEB PLATES SHEAR STRENGTH (AISC 341-05, 17.2a)

$$\alpha = ATAN\left(\frac{1+\frac{t_w L}{2A_c}}{1+t_w h\left(\frac{1}{A_b}+\frac{h^3}{360 I_c L}\right)}\right)^{\left(\frac{1}{4}\right)} = 36.3\ ^{O}$$



$\phi V_n = \phi\ (0.42\ F_y\ t_w\ L_{cf}\ SIN\ 2\alpha)$ = 1118.9 kips **>** V_E
[Satisfactory]

Where ϕ = 0.9
L_{cf} = 345 in

CHECK VBE/COLUMN CAPACITY UNDER SEISMIC LOAD COMBINATION (AISC 341-05, 17.4a & 8.3)

$P_u = (1.2 + S_{DS})\ P_{DL} + f_1\ P_{LL} + P_E$ = 238.9 kips, (IBC 09, Eq. 16-5)
Where f_1 = 1 (IBC 09, 1605.2.1)
I_c = 2400 in^4 **>** $0.00307\ t_w\ h^4 / L$ = 2238 in^4 (AISC 341-05, 17.4g)
[Satisfactory]

$\phi_c P_n = \phi_c F_{cr} A$ = 2212.03 kips **>** P_u **[Satisfactory]**
Where ϕ_c = 0.9
K = 1
MAX($K\angle_x/r_x$, $K\angle_y/r_y$) = 44.46 **<** 200 **[Satisfactory]** (AISC 360-05, E3)
E_s = 29000 ksi
λ_c = $(K\angle\ /\ r)\ (F_y\ /\ E)^{0.5}$ = 1.846 (AISC 360-05, E2-4, Pg 6-47)
F_e = $\pi^2\ E\ /\ (K\angle\ /\ r)^2$ = 144.8 ksi (AISC 360-05, E3)
F_{cr} = 43.27 ksi (AISC 360-05, E3)

$P_u\ /\ \phi\ P_n$ = 0.11 **<** 0.4 **[Amplified Seismic Load Do Not Need to Checke]**

$P_{uc} = (1.2 + S_{DS})\ P_{DL} + P_{LL} + \Omega_0 P_E$ = 314 kips **<** $\phi_c P_n$ **[Satisfactory]**
Where Ω_0 = 2.5 (AISC 341-05, Table R3-1)

(Cont'd)

CHECK HBE/BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-05, Tab. I-8-1)

$b_f / (2t_f) =$ 6.10 < $0.3 (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

$[52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

$h / t_w =$ 49.26 < $2.45 (E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**

$[418 / (F_y)^{0.5}$ for FEMA Sec. 3.3.1.2]

CHECK VBE/COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-05, Tab. I-8-1)

$b_f / (2t_f) =$ 5.45 < $0.3 (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

$[52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

$h / t_w =$ 12.83 <

- $3.14(E_s/F_y)^{0.5}(1\text{-}1.54C_a) =$ 64.74 , for $C_a = P_u/\phi_b P_y \le 0.125$
- $[520 / (F_y)^{0.5}(1\text{-}1.54P_u/\phi_b P_y)$ for AISC Seismic 97, Tab. I-9-1]
- $1.12(E_s/F_y)^{0.5}MAX(1.49, 2.33 - C_a) =$ N/A , for $C_a = P_u/\phi_b P_y > 0.125$
- $\{MAX[191 / (F_y)^{0.5}(2.33\text{-}P_u/\phi_b P_y) , 253 / (F_y)^{0.5}]$ for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where $\phi_b =$ 0.9 , $P_y = F_y A =$ 2840 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-05, 17.4a & 9.6)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*) =$ 4.89 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c Z_c (F_{yc} - P_u / A_g) + V_{col} (d_b / 2) =$ 2710 + 0 = 2710 ft-kips

$N_c =$ 2 , (if only one column below, input 1)

$\Sigma M_{pb}^* = N_b M_B + V_{bm} (d_c / 2) =$ 555 + 0 = 555 ft-kips

$N_b =$ 1 , (if double side connected to SPSW, input 2)

$M_B = C_{pr} R_y F_{yb} Z_b =$ 555 ft-kips

$R_y =$ 1.1 (AISC 341-05 Tab. I-6-1)

$C_{pr} =$ 1.1 (AISC 341-05 Sec. 9.6 & AISC 358-05 Sec. 2.4.3)

CHECK CONTINUITY PLATE REQUIREMENT (AISC 341-05, 17.4b & AISC 358-05, 2.4.4)

$t_{cf} = MIN\{ b_{bf} / 6 , 0.4[1.8 b_{bf} t_{bf} (F_{yb} R_{yb}) / (F_{yb} R_{yb})]^{0.5} \} =$ 1.00 in < actual t_{cf}

(The continuity plates may not be required.)

$t_{st} = t_{bf}$ for interior connection, or $(t_{bf} /2)$ for exterior connection = 0.27 in, USE 0.31 in, (5/16 in)

$b_{st} =$ 4.8 in < $0.56 (E / F_{yst})^{0.5} t_{st} =$ 4.97 in, (AISC 360-05 Sec. G3.3)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr} A =$ 214.0 kips

Where $\phi_c =$ 0.9 (AISC 360-05, E1) $h_{st} = d_c - 2k =$ 11.42 in

$K =$ 0.75 $K h_{st} / r_{st} < 200$ (AISC 360, E2) **[Satisfactory]**

$I = t_{st} (2b_{st} + t_{wc})^3 / 12 =$ 26 in^4 $F_e = \pi^2 E / (K h_{st} / r_{st})^2 =$ 15279 ksi, (AISC 360, E3)

$A = 2b_{st} t_{st} + 25(t_{wc})^2 =$ 7 in^2 $\lambda_c = (K h_{st} / r_{st}) (F_y / E)^{0.5} =$ 0.15

$r_{st} = (I / A)^{0.5} =$ 1.98 in $F_{cr} = \{$ $(0.658^{(Fy/Fe)})F_y =$ 35.96 , $\lambda_c \le 4.71$

$F_{yst} =$ 36 kips, stiff. yield stress $0.877 F_e =$ N/A kis, for $\lambda_c > 4.71$

$P_{u,st} = R_{yb} F_{yb} b_{fb} t_{fb} =$ 192.1 kips < $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360-05, J2.2b)

$w =$ 1/4 in > $w_{MIN} =$ 0.1875 in

< $w_{MAX} =$ 0.25 in

[Satisfactory]

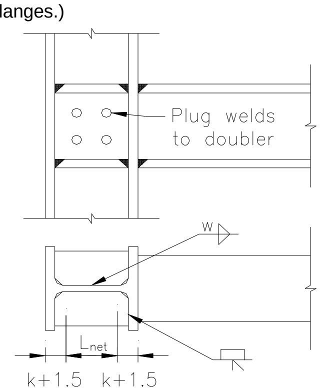
The required weld length between continuity plates and column web (AISC 360-05, J2.2b)

$L_w = 0.6 t_{st} L_{nst} F_y / [(2) \phi F_w (0.707 w)]$ = 0.6 x (0.3125 x 8.4) x 36 / [(2) 0.75 (0.6x70)(0.707x1/4)] = 3.61 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 8.4 < $2(L_{net} - 0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341-05, 17.4f & 9.3)



$t_{ReqD} = MAX (t_1, t_2) =$ 0.57 in

$t_1 = C_y M_c (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h] =$ 0.57 in

Where $C_y = S_b / (C_{pr} Z_b) =$ 0.91

$S_b = 2I_b / d_b =$ 95 in^2

$I_b = I_x =$ 984 in^4

$M_c = \Sigma M_{pb}^* =$ 555 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b - 2t_{st} + d_c - 2k) / 90 =$ 0.35 in

Since $t_{wc} =$ 0.89 in > t_{ReqD} , a doubler plate is not required.

Technical References:

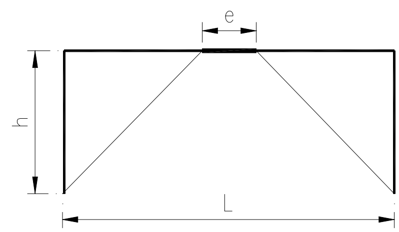
1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.
3. AISC 358-05: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, Dec 13, 2005.

Seismic Design for Eccentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

LINK SECTION	= >	**W18X143**		= >	**A**	**d**	**t_w**	**b_f**	**t_f**	**S_x**
MAX SERVICE LOADS AT LINK END	V_{DL} =	**1.8**	kips		42.1	19.5	0.73	11.20	1.32	282
	P_{DL} =	**7.4**	kips			**I_x**	**r_x**	**r_y**	**Z_x**	**k**
	M_{DL} =	**14.4**	ft-kips			2750	8.08	2.72	322	1.72
	V_{LL} =	**1.3**	kips							
	P_{LL} =	**5.3**	kips							
	M_{LL} =	**9.6**	ft-kips							
MAX HORIZ. SEISMIC LOADS AT LINK END	V_E =	**84**	kips (ASCE 7-05, 12.4.2.1)							
	P_E =	**5.5**	kips							
	M_E =	**168**	ft-kips							
LINK LENGTH	e =	**4**	ft							
LINK YIELD STRESS	F_y =	**50**	ksi							
REDUNDANCY FACTOR	ρ =	**1.19**								
SEISMIC PARAMETER	S_{DS} =	**1**	(ASCE 7-05, 11.4.4)							
BEAM LENGTH BETWEEN COL. CENTERS	L =	**30**	ft (including link)							
STORY HEIGHT	h =	**12.5**	ft							
MAXIMUM INELASTIC STORY DRIFT	δ =	**0.7**	in (ASCE 7-05, 12.8.6)							

THE LINK DESIGN IS ADEQUATE.
(USE 3/4 x 4-7/8 @ 24 in o.c. INTERMEDIATE & END STIFFENERS WITH 5/16" FILLET WELD.)



DETERMINE FACTORED DESIGN LOADS AT LINK END (IBC 09 1605.2 & ASCE 7-05 12.4.2.3)

$V_u = (1.2 + 0.2S_{DS})V_{DL} + f_1V_{LL} + \rho V_E =$ 103.13 kips

$P_u = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + \rho P_E =$ 19.56 kips

$M_u = (1.2 + 0.2S_{DS})M_{DL} + f_1M_{LL} + \rho M_E =$ 224.88 ft-kips

Where f_1 = 0.5

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 4.24 **<** $0.3 (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

[$52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

$h / t_w =$ 22.00 **<**

- $3.14(E_s/F_y)^{0.5}(1-1.54P_u/\phi_bP_y) =$ 74.42 , for $P_u/\phi_bP_y < 0.125$
- [$520 / (F_y)^{0.5}(1-1.54P_u/\phi_bP_y)$ for AISC Seismic 97, Tab. I-9-1]
- $(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = N/A , for $P_u/\phi_bP_y > 0.125$
- { MAX[$191 / (F_y)^{0.5}(2.33-P_u/\phi_bP_y)$, $253 / (F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 2105 kips

CHECK SHEAR CAPACITY (AISC 341-05, Sec. 15.2b)

$\phi V_n = \phi$ MIN(V_p, $2M_p/e$) = 332.3 kips **>** V_u **[Satisfactory]**

Where ϕ = 0.9 (Ignored axial force effect since Pu < 0.15 Py = 0.15 Fy Ag, AISC 341-05, 15.2)

$A_w = (d - 2t_f)t_w =$ 12.31 in^2

$V_p = 0.6F_yA_w =$ 369.2 kips

$M_p = F_yZ =$ 1341.7 ft-kips

CHECK FLEXURAL CAPACITY (AISC 360-05 F1)

$\phi_bM_p =$ 1207.5 **>** M_u **[Satisfactory]**

Where ϕ_b = 0.9

CHECK ADDITIONAL SHEAR CAPACITY REQUIREMENT FOR Pu>0.15Py ONLY (AISC 341-05, 15.2b) **< = DOES NOT APPLY.**

$\phi V_{na} = \phi$ MIN(V_{pa}, $2M_{pa}/e$) = 332.3 kips **>** V_u **[Satisfactory]**

Where ϕ = 0.9

$V_{pa} = V_p[1 - (P_u/P_y)^2]^{0.5} =$ 369.2 kips

$M_{pa} = 1.18\, M_p(1 - P_u/P_y) =$ 1568.5 ft-kips

CHECK ADDITIONAL LINK LENGTH REQUIREMENT FOR Pu>0.15Py ONLY (AISC 341-05, 15.2b) **< = DOES NOT APPLY.**

e **<** {
- $[1.15 - 0.5\rho' (A_w/A_g)](1.6M_p/V_p)$ = N/A ft, for $\rho' (A_w/A_g) > 0.3$
- $(1.6M_p/V_p)$ = 5.81 ft, for $\rho' (A_w/A_g) < 0.3$

[Satisfactory] Where $\rho' = P_u / V_u =$ 0.19

$A_w / A_g =$ 0.29

(cont'd)

CHECK LINK ROTATION ANGLE LIMITATION (AISC 341-05, 15.2c)

$\gamma_p = L\ \delta / (h\ e)$ = 0.04 rad **<** $\gamma_{p,allowable}$ = 0.080 rad **[Satisfactory]**

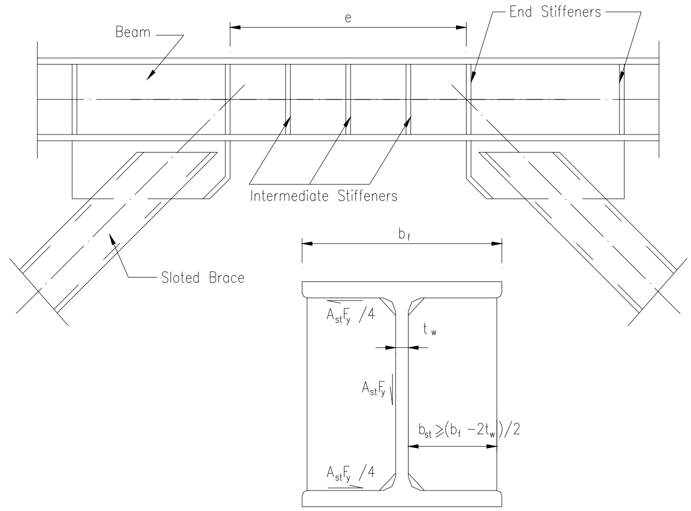
Where $\gamma_{p,allowable}$ = 0.08 rad for e < $1.6M_p/V_p$;

= 0.02 rad for e > $2.6M_p/V_p$;

= linear interpolation [0.02, 0.08] by e value.

$1.6M_p/V_p$ = 5.81 ft, $2.6M_p/V_p$ = 9.45 ft

CHECK LINK STIFFENER REQUIREMENT (AISC 341-05, 15.3)



$b_{st} = (b_f - 2t_w) /2$ = 4.87 in

t_{st} = MAX (0.75 t_w , 3/8) = 0.548 in

USE 9/16 x 4-7/8 END STIFFENERS AT EACH SIDE.

s = see table following = 30.0 in

Provide 1 stiffeners to give s = 24.0 in

Where $1.6\ M_p / V_p$ = 5.81 ft

$2.6\ M_p / V_p$ = 9.45 ft

$5.0\ M_p / V_p$ = 18.17 ft

γ_p = 0.04 rad

e = 4 ft

d = 19.5 in

t_{st} = MAX (t_w , 3/8) = 0.730 in

USE 3/4 x 4-7/8 @ 24 in o.c. INTERMEDIATE STIFFENERS AT EACH SIDE.

	γ_p		
e	[0 ~ 0.02]	(0.02 ~ 0.08)	0.08
[0~$1.6M_p/V_p$]	$52t_w$-d/5	$178t_w$/3-d/5-1100$\gamma_p t_w$/3	$30t_w$-d/5
($1.6M_p/V_p$~$2.6M_p/V_p$]	MIN($52t_w$-d/5 , b_f)	Min($178t_w$/3-d/5-1100$\gamma_p t_w$/3 , $1.5b_f$)	MIN($30t_w$-d/5 , $1.5b_f$)
($2.6M_p/V_p$~$5M_p/V_p$)	$1.5b_f$	$1.5b_f$	$1.5b_f$
[$5.0M_p/V_p$~Greater]	Not ReqD	Not ReqD	Not ReqD

The best fillet weld size (AISC 360-05 Sec.J2.2b)

w = 5/16 in > w_{MIN} = 0.1875 in

< w_{MAX} = 0.4375 in

[Satisfactory]

The required weld length between A36 stiffener and web (AISC 360-05 Sec.J2.4)

$L_w = A_{st}F_y / [(2)\ \phi\ F_w\ (0.707\ w)]$ = (9/16 x 4-7/8) x 36 / [(2) 0.75 (0.6x70)(0.707x5/16)] = 5.01 in

< (d - 2k), **[Satisfactory]**

The required weld length between A36 stiffener and flange (AISC 360-05 Sec.J2.4)

$L_f = 0.25A_{st}F_y / [(2)\ \phi\ F_w\ (0.707\ w)]$ = 0.25(9/16 x 4-7/8) x 36 / [(2) 0.75 (0.6x70)(0.707x5/16)] = 1.25 in

< (b_{st} - k), **[Satisfactory]**

CHECK COMBINED LINK CAPACITY (AISC 360-05 Sec.H.1)

$f = P_{u,link} / (2A_f) + M_{u,link} / Z_f$ = 35.3 **<** F_y **[Satisfactory]**

Where $P_{u,link} = \Omega\ P_u$ = 70.0 kips

$M_{u,link} = V_p\ (e/2)$ = 738.5 ft-kips

$Z_f = (d - t_f)\ b_f\ t_f$ = 268.8 in^3

$\Omega = V_n / V_u$ = 3.58

$A_f = b_f\ t_f$ = 14.78 in^2

Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.

Seismic Design for Eccentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

BRACE SECTION (Tube or Pipe)	= >	**HSS10X10X5/8**	**Tube**	**A**	**r_{min}**	**t**	**h**
MAX SERVICE LOADS	P_{DL} =	**11.8** kips		21.00	3.80	0.58	10.00
	P_{LL} =	**8.3** kips					
UNBRACED LENGTH OF THE BRACE	L_b =	**11** ft					

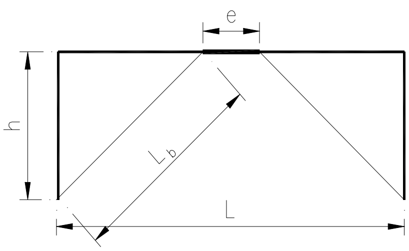
(SEE LINK DESIGN SPREADSHEET FOR BALANCE OF INPUT DATA)

THE BRACE DESIGN IS ADEQUATE.
REQUIRED CONNECTION = > (5/8 in Gusset Plate with 23 in Length, 4 leg, 5/16 in Fillet Weld.)

DETERMINE LIMITING WIDTH THICKNESS RATIO FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

$D / t = 0.044\ E_s / F_y$ = 27.74 , for Pipe
$h / t = 1.12\ (E_s / F_y)^{0.5}$ = 28.12 , for Tube
> Actual **[Satisfactory]**

Where F_y = 46 ksi
E_s = 29000 ksi



DETERMINE FACTORED DESIGN LOADS (AISC 341-05 Sec.15.6)

$P_u = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + P_E$ = 629.9 kips
Where $P_E = 1.25\ R_y\ [V_n\ L\ L_b / (L\text{-}e)\ h]$ = 609.2 kips
R_y = 1.3 (1.4 for Pipe.)

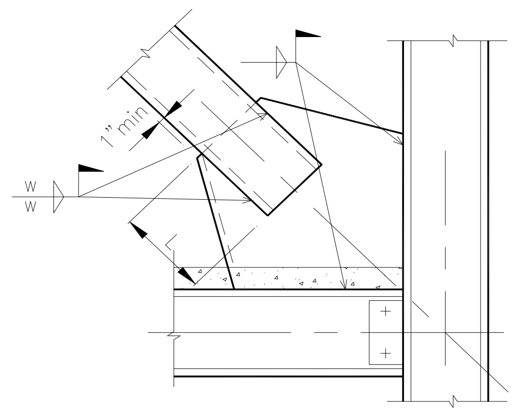
DETERMINE DESIGN STRENGTH IN COMPRESSION (AISC 360-05 E3)

$\phi_cP_n = \phi_cA_gF_{cr}$ = 757.24 kips **>** P_u **[Satisfactory]**
ϕ_c = 0.85
F_e = $\pi^2 E / (K\ell / r)^2$ = 237.8 ksi
λ_c = $(K\ell / r)\ (F_y / E)^{0.5}$ = 1.38
K = 1.0

$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = 42.42 & \text{kis, for } \lambda_c \le 4.71 \\ 0.877\ F_e = \text{N/A} & \text{kis, for } \lambda_c > 4.71 \end{cases}$

DETERMINE CONNECTION DESIGN FORCE

$P_{ut} = P_u$ = 629.91 kips (Tension)



DETERMINE BEST FILLET WELD SIZE (AISC 360-05 Sec.J2.2b)

w = 5/16 in
> w_{MIN} = 0.1875 in
< w_{MAX} = 0.4375 in
[Satisfactory]

DETERMINE REQUIRED WELD LENGTH (AISC 360-05 Sec.J2.4)

$L = P_{ut} / [(4)\ \phi\ F_w\ (0.707\ w)]$
= 629.9 / [(4) 0.75 (0.6x70)(0.707x5/16)] = 22.63 in
(USE 23 in)

CHECK DESIGN SHEAR RUPTURE CAPACITY OF SLOTTED BRACE
(AISC 360-05 Sec.J4.2)

$\phi R_n = \phi(0.6F_u)A_{nu}$ = 1395.10 kips **>** P_{ut}
Where ϕ = 0.75 **[Satisfactory]**
F_u = 58 ksi (AISC 13th Tab.2-3)
A_{nu} = 4 t L = 4 x 0.581 x 23 = 53.45 in^2

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE (AISC 360-05 Tab. J2.4)

t_g = 5/8 in

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 J4.2)

$\phi R_n = \phi(0.6F_u)A_{nv}$ = 750.38 kips **>** P_{ut} **[Satisfactory]**
Where ϕ = 0.75
F_u = 58 ksi (plate value)
A_{nv} = 2 tg L = 2 x 5/8 x 23 = 28.75 in^2

CHECK TENSION AT SLOTTED BRACE (AISC 360-05 D.2 b)

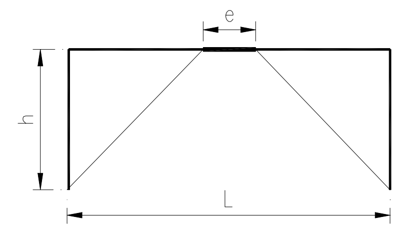
$\phi_tP_n = \phi_tR_yF_uUA_n$ = 1146.48 kips **>** P_{ut}
Where ϕ_t = 0.75 **[Satisfactory]**
U = 1 (LRFD Sec.B3.2d)
A_n = A - 2 t t_g = 20.274 in^2

THE GUSSET BLOCK SHEAR CAPACITY (AISC 360-05 J4.3)

$\phi R_n = \phi(0.6F_u)A_{nv} + \phi F_yA_{gt}$ = 750.38 + ϕF_yA_{gt}
> P_{ut} = 629.91
[Satisfactory]

Seismic Design for Eccentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY



MAX SERVICE LOADS AT OUTSIDE OF LINK	V_{DL} =	**6.8**	kips
	P_{DL} =	**1**	kips
	M_{DL} =	**17**	ft-kips
	V_{LL} =	**4.8**	kips
	P_{LL} =	**0.7**	kips
	M_{LL} =	**11.3**	ft-kips
SEISMIC LOADS AT OUTSIDE OF LINK	V_E =	**8.7**	kips (ASCE 7-05 12.4.2.1)
	P_E =	**100**	kips
	M_E =	**100**	ft-kips

(SEE LINK DESIGN SPREADSHEET FOR BALANCE OF INPUT DATA)

THE BEAM DESIGN IS ADEQUATE.

DETERMINE FACTORED DESIGN LOADS AT SECTION OF LINK AND BEAM (AISC 360-05 Sec. 15.6b & ASCE 7-05 12.4.2.3)

$V_u = (1.2 + 0.2S_{DS})V_{DL} + f_1V_{LL} + \rho V_E =$ 67.0 kips

$P_u = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + P_E =$ 537.9 kips

$M_u = (1.2 + 0.2S_{DS})M_{DL} + f_1M_{LL} + M_E =$ 923.0 ft-kips

Where f_1 = 0.5

R_y = 1.1 (AISC 341-05 Tab. I-6-1)

V_n = 369.2 kips (from link design)

$M_n = V_n e / 2 =$ 738.47 ft-kips

$V_E = (1.1R_y V_n / V_{E,\ link}) V_E =$ 46.3 kips

$P_E = 1.1R_y V_n L / 2h =$ 536.1 kips

$M_E = 1.1R_y M_n =$ 893.5 ft-kips

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 4.24 **<** $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

[$52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

$h / t_w =$ 22.00 **<**

$3.14(E_s/F_y)^{0.5}(1-1.54P_u/\phi_bP_y) =$ N/A , for $P_u/\phi_bP_y < 0.125$

[$520 / (F_y)^{0.5}(1-1.54P_u/\phi_bP_y)$ for AISC Seismic 97, Tab. I-9-1]

$(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = 55.19 , for $P_u/\phi_bP_y > 0.125$

{ MAX[$191 / (F_y)^{0.5}(2.33-P_u/\phi_bP_y)$, $253 / (F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 2105 kips

CHECK UNBALANCED SEGMENT LENGTH

$\angle_1 = (L - e - d_c) / 2 =$ 12.38 ft, (top & bottom flange bracing with a design strength greater than below will be provided

Brace Load : $P_{b,link} = 0.06R_y F_y b_f t_f =$ 48.8 kips, [AISC Seismic Sec.15.5] at each end of the link segment.)

$\angle_2 = \angle_1 / 2 =$ 6.19 ft, (lateral supported at middle of beam outside of link with following design strength.)

Brace Load : $P_{b,mid} = 0.02F_y b_f t_f =$ 14.8 kips, [AISC 341-05 Sec.15.6.(2)]

$M_{b,\ mid} = 0.02F_y b_f t_f d =$ 24.0 ft-kips, [AISC 341-05 Sec.15.6.(2)]

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$\phi_cP_n = \phi_cF_{cr}A =$ 1694.11 kips **>** P_u **[Satisfactory]**

Where ϕ_c = 0.85

K = 1.0

MAX($K\angle_1/r_x$, $K\angle_2/r_y$) = 27.34 **<** 200 **[Satisfactory]**

λ_c = 0.361 π

F_{cr} = 47.34 ksi

(cont'd)

DETERMINE FLEXURAL DESIGN STRENGTH (AISC-AISC 360-05 F1)

L_b = 6.19 ft

$L_p = 1.76\, r_y\, (E / F_{yf})^{0.5}$ = 9.60 ft

$L_r = r_y X_1 [1 + (1 + X_2 F_L^{\,2})^{0.5}]^{0.5} / F_L$ = 35.27 ft

$M_p = \mathrm{MIN}(F_y Z_x , 1.5 F_y S_x)$ = 1341.7 ft-kips

$M_r = F_L S_x$ = 940.0 ft-kips

$M_{cr} = C_b S_x r_y X_1 (2 + X_1^{\,2} X_2 r_y^{\,2} / L_b^{\,2})^{0.5} / L_b$ = 17113 ft-kips

Where $X_1 = \pi (0.5\, E\, G\, J\, A)^{0.5} / S_x$ = 4036.3

$X_2 = 4\, C_w [Sx / (G\, J)]^2 / I_y$ = 0.0006

F_r = 10.00 ksi

$F_L = \mathrm{MIN}(F_{yf} - F_r , F_{yw})$ = 40.00 ksi

C_b = 1.30 , (AISC 360-05 F1)

A	**I_y**	**t_f**	**r_y**	**S_x**
42.1	311	1.32	2.72	282
E	**G**	**J**	**C_w**	**Z_x**
29000	11200	19.2	25700	322.0

M_n =

M_p	=	1341.7	ft-kips, for L_b @ [0 , L_p]
$\mathrm{MIN}\{C_b [M_p - (M_p - M_r) (L_b - L_p) / (L_r - L_p)] , M_p\}$	=	N/A	ft-kips, for L_b @ (L_p , L_r]
$\mathrm{MIN}(M_{cr} , M_p)$	=	N/A	ft-kips, for L_b @ (L_r , Larger)

$\phi_b M_n = 0.9 M_n$ = 1208 ft-kips

CHECK FLEXURAL CAPACITY (AISC 360-05 C2.1b)

$M_{ux} = B_1 M_u$ = 923.00 ft-kips **<** $\phi_b M_{nx} = \mathrm{Min}(R_y \phi_b F_y Z, \phi_b M_n)$ = 1208 ft-kips

Where $P_{e1} = \pi^2 E_s I_x / (K L_x)^2$ = 35645 kips **[Satisfactory]**

C_m = 0.6 (AISC 360-05 C2.1b) Where ϕ_b = 0.9

$B_1 = C_m/(1 - P_u/P_{e1})$ = 1.000

CHECK INTERACTION CAPACITY (AISC 360-05 H1.1)

For $P_u/\phi_c P_n > 0.2$, $P_u / \phi_c P_n + 8 / 9\, (M_{ux} / \phi_b M_{nx})$ = 1.00

For $P_u/\phi_c P_n < 0.2$, $P_u / (2\phi_c P_n) + M_{ux} / \phi_b M_{nx}$ = N/A

< **1** **[Satisfactory]**

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PROJECT :
CLIENT :
JOB NO. :
DATE :

PAGE :
DESIGN BY :
REVIEW BY :

Seismic Design for Eccentrically Braced Frames Based on IBC 09 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W14X145** =>

A	d	t_w	b_f	t_f	S_x
42.7	14.8	0.68	15.50	1.09	232
	I_x	r_x	r_y	Z_x	k
	1710	6.33	3.98	260	1.69

COLUMN AXIAL SERVICE LOADS P_{DL} = **151** kips

P_{LL} = **46** kips

NUMBER OF STORIES n = **4**

COLUMN YIELD STRESS (36 or 50) F_y = **50** ksi

THE COLUMN DESIGN IS ADEQUATE.

UNBRACED COLUMN LENGTH ℓ = h = 14 ft

DETERMINE COLUMN AXIAL SEISMIC LOAD (AISC 341-05 Sec. 15.8)

P_E = (n - 1) 1.1 R_y V_n = 1340.3 kips

DETERMINE FACTORED DESIGN LOADS (AISC 360-05 Sec. 15.6b & ASCE 7-05 12.4.2.3)

$P_{u,t}$ = (0.9 - 0.2S_SD) P_{DL} - P_E = -1235 kips (Tension)

$P_{u,c}$ = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + P_E = 1575 kips (Compression)

Where f_1 = 0.5 (IBC 1605.2)

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

b_f / (2t_f) = 7.11 **<** 0.3 (E_s / F_y)$^{0.5}$ = 7.22 **[Satisfactory]**

[52 / (F_y)$^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

h / t_w = 16.79 **<**

3.14(E_s/F_y)$^{0.5}$(1-1.54P_u/$\phi_b P_y$) = N/A , for $P_u/\phi_b P_y$ < 0.125

[520 / (F_y)$^{0.5}$(1-1.54P_u/$\phi_b P_y$) for AISC Seismic 97, Tab. I-9-1]

(E_s/F_y)$^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = 40.74 , for $P_u/\phi_b P_y$ > 0.125

{ MAX[191 / (F_y)$^{0.5}$(2.33-P_u/$\phi_b P_y$) , 253 / (F_y)$^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_c = 0.9

P_y = F_yA = 2135 kips

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$\phi_c P_n$ = $\phi_c F_{cr}$A = 1593.27 kips **>** P_u **[Satisfactory]**

Where ϕ_c = 0.85

K = 1.0

MAX(Kℓ_x/r_x, Kℓ_y/r_y) = 42.19 **<** 200 **[Satisfactory]**

λ_c = 0.558 π

F_{cr} = 43.90 ksi

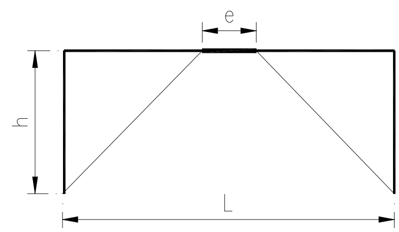
Seismic Design for Eccentrically Braced Frames Based on CBC 07 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

LINK SECTION	= >	**W18X143**		= >	A	d	t_w	b_f	t_f	S_x
MAX SERVICE LOADS AT LINK END	V_{DL} =	**1.8**	kips		42.1	19.5	0.73	11.20	1.32	282
	P_{DL} =	**7.4**	kips			I_x	r_x	r_y	Z_x	k
	M_{DL} =	**14.4**	ft-kips			2750	8.08	2.72	322	1.72
	V_{LL} =	**1.3**	kips							
	P_{LL} =	**5.3**	kips							
	M_{LL} =	**9.6**	ft-kips							
MAX HORIZ. SEISMIC LOADS AT LINK END	V_E =	**84**	kips (ASCE 7-05, 12.4.2.1)							
	P_E =	**5.5**	kips							
	M_E =	**168**	ft-kips							
LINK LENGTH	e =	**4**	ft							
LINK YIELD STRESS	F_y =	**50**	ksi							
REDUNDANCY FACTOR	ρ =	**1.19**								
SEISMIC PARAMETER	S_{DS} =	**1**	(ASCE 7-05, 11.4.4)							
BEAM LENGTH BETWEEN COL. CENTERS	L =	**30**	ft (including link)							
STORY HEIGHT	h =	**12.5**	ft							
MAXIMUM INELASTIC STORY DRIFT	δ =	**0.7**	in (ASCE 7-05, 12.8.6)							

THE LINK DESIGN IS ADEQUATE.

(USE 3/4 x 4-7/8 @ 24 in o.c. INTERMEDIATE & END STIFFENERS WITH 5/16" FILLET WELD.)



DETERMINE FACTORED DESIGN LOADS AT LINK END (CBC 07 1605.2 & ASCE 7-05 12.4.2.3)

$V_u = (1.2 + 0.2S_{DS})V_{DL} + f_1V_{LL} + \rho V_E =$ 103.13 kips

$P_u = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + \rho P_E =$ 19.56 kips

$M_u = (1.2 + 0.2S_{DS})M_{DL} + f_1M_{LL} + \rho M_E =$ 224.88 ft-kips

Where f_1 = 0.5

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 4.24 **<** $0.3\,(E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

[$52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

$h / t_w =$ 22.00 **<**

- $3.14(E_s/F_y)^{0.5}(1\text{-}1.54P_u/\phi_bP_y) =$ 74.42 , for $P_u/\phi_bP_y < 0.125$
  [$520 / (F_y)^{0.5}(1\text{-}1.54P_u/\phi_bP_y)$ for AISC Seismic 97, Tab. I-9-1]
- $(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = N/A , for $P_u/\phi_bP_y > 0.125$
  { MAX[$191 / (F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y)$, $253 / (F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 2105 kips

CHECK SHEAR CAPACITY (AISC 341-05, Sec. 15.2b)

$\phi V_n = \phi\,\text{MIN}(V_p, 2M_p/e) =$ 332.3 kips **>** V_u **[Satisfactory]**

Where ϕ = 0.9 (Ignored axial force effect since Pu < 0.15 Py = 0.15 Fy Ag, AISC 341-05, 15.2)

$A_w = (d - 2t_f)t_w =$ 12.31 in^2

$V_p = 0.6F_yA_w =$ 369.2 kips

$M_p = F_yZ =$ 1341.7 ft-kips

CHECK FLEXURAL CAPACITY (AISC 360-05 F1)

$\phi_bM_p =$ 1207.5 **>** M_u **[Satisfactory]**

Where ϕ_b = 0.9

CHECK ADDITIONAL SHEAR CAPACITY REQUIREMENT FOR Pu>0.15Py ONLY (AISC 341-05, 15.2b) **< = DOES NOT APPLY.**

$\phi V_{na} = \phi\,\text{MIN}(V_{pa}, 2M_{pa}/e) =$ 332.3 kips **>** V_u **[Satisfactory]**

Where ϕ = 0.9

$V_{pa} = V_p[1 - (P_u/P_y)^2]^{0.5} =$ 369.2 kips

$M_{pa} = 1.18\,M_p(1 - P_u/P_y) =$ 1568.5 ft-kips

CHECK ADDITIONAL LINK LENGTH REQUIREMENT FOR Pu>0.15Py ONLY (AISC 341-05, 15.2b) **< = DOES NOT APPLY.**

e **<**
- $[1.15 - 0.5\rho' (A_w/A_g)](1.6M_p/V_p) =$ N/A ft, for $\rho' (A_w/A_g) > 0.3$
- $(1.6M_p/V_p) =$ 5.81 ft, for $\rho' (A_w/A_g) < 0.3$

[Satisfactory] Where $\rho' = P_u / V_u =$ 0.19

$A_w / A_g =$ 0.29

(cont'd)

CHECK LINK ROTATION ANGLE LIMITATION (AISC 341-05, 15.2c)

$\gamma_p = L\,\delta / (h\,e)$ = 0.04 rad **<** $\gamma_{p,allowable}$ = 0.080 rad **[Satisfactory]**

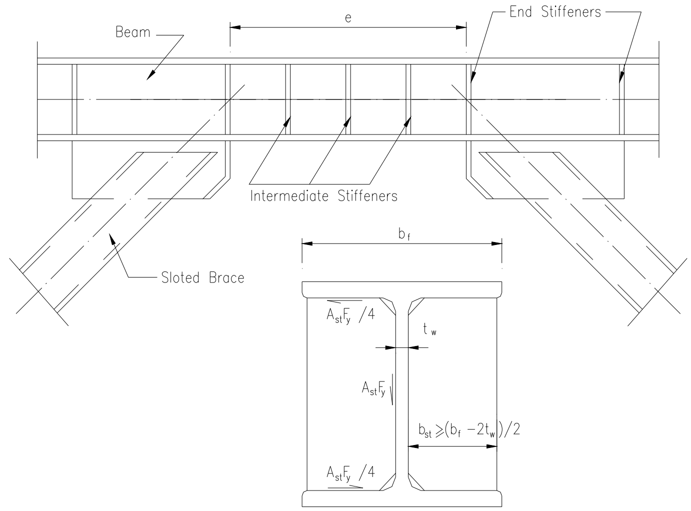
Where $\gamma_{p,allowable}$ = 0.08 rad for $e < 1.6M_p/V_p$;

= 0.02 rad for $e > 2.6M_p/V_p$;

= linear interpolation [0.02, 0.08] by e value.

$1.6M_p/V_p$ = 5.81 ft, $2.6M_p/V_p$ = 9.45 ft

CHECK LINK STIFFENER REQUIREMENT (AISC 341-05, 15.3)



$b_{st} = (b_f - 2t_w)/2$ = 4.87 in

$t_{st} = MAX(0.75\,t_w, 3/8)$ = 0.548 in

USE 9/16 x 4-7/8 END STIFFENERS AT EACH SIDE.

s = see table following = 30.0 in

Provide 1 stiffeners to give s = 24.0 in

Where $1.6\,M_p / V_p$ = 5.81 ft

$2.6\,M_p / V_p$ = 9.45 ft

$5.0\,M_p / V_p$ = 18.17 ft

γ_p = 0.04 rad

e = 4 ft

d = 19.5 in

$t_{st} = MAX(t_w, 3/8)$ = 0.730 in

USE 3/4 x 4-7/8 @ 24 in o.c. INTERMEDIATE STIFFENERS AT EACH SIDE.

e	γ_p [0 ~ 0.02]	(0.02 ~ 0.08)	0.08
$[0 \sim 1.6M_p/V_p]$	$52t_w$-d/5	$178t_w/3$-d/5-$1100\gamma_p t_w/3$	$30t_w$-d/5
$(1.6M_p/V_p \sim 2.6M_p/V_p]$	MIN($52t_w$-d/5 , b_f)	Min($178t_w/3$-d/5-$1100\gamma_p t_w/3$, $1.5b_f$)	MIN($30t_w$-d/5 , $1.5b_f$)
$(2.6M_p/V_p \sim 5M_p/V_p)$	$1.5b_f$	$1.5b_f$	$1.5b_f$
$[5.0M_p/V_p \sim$Greater]	Not ReqD	Not ReqD	Not ReqD

The best fillet weld size (AISC 360-05 Sec.J2.2b)

w = 5/16 in

> w_{MIN} = 0.1875 in

< w_{MAX} = 0.4375 in

[Satisfactory]

The required weld length between A36 stiffener and web (AISC 360-05 Sec.J2.4)

$L_w = A_{st}F_y / [(2)\,\phi\,F_w\,(0.707\,w)]$ = (9/16 x 4-7/8) x 36 / [(2) 0.75 (0.6x70)(0.707x5/16)] = 5.01 in

< (d - 2k), **[Satisfactory]**

The required weld length between A36 stiffener and flange (AISC 360-05 Sec.J2.4)

$L_f = 0.25A_{st}F_y / [(2)\,\phi\,F_w\,(0.707\,w)]$ = 0.25(9/16 x 4-7/8) x 36 / [(2) 0.75 (0.6x70)(0.707x5/16)] = 1.25 in

< (b_{st} - k), **[Satisfactory]**

CHECK COMBINED LINK CAPACITY (AISC 360-05 Sec.H.1)

$f = P_{u,link} / (2A_f) + M_{u,link} / Z_f$ = 35.3 **<** F_y **[Satisfactory]**

Where $P_{u,link} = \Omega\,P_u$ = 70.0 kips

$M_{u,link} = V_p\,(e/2)$ = 738.5 ft-kips

$Z_f = (d - t_f)\,b_f\,t_f$ = 268.8 in^3

$\Omega = V_n / V_u$ = 3.58

$A_f = b_f\,t_f$ = 14.78 in^2

Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.

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Seismic Design for Eccentrically Braced Frames Based on CBC 07 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

BRACE SECTION (Tube or Pipe) => **HSS10X10X5/8**

Tube	A	r_{min}	t	h
	21.00	3.80	0.58	10.00

MAX SERVICE LOADS P_{DL} = **11.8** kips

P_{LL} = **8.3** kips

UNBRACED LENGTH OF THE BRACE L_b = **11** ft

(SEE LINK DESIGN SPREADSHEET FOR BALANCE OF INPUT DATA)

THE BRACE DESIGN IS ADEQUATE.

REQUIRED CONNECTION = > (5/8 in Gusset Plate with 23 in Length, 4 leg, 5/16 in Fillet Weld.)

DETERMINE LIMITING WIDTH THICKNESS RATIO FOR COMPRESSION ELEMENT, LOCAL BUCKLING (AISC 341-05 Tab. I-8-1)

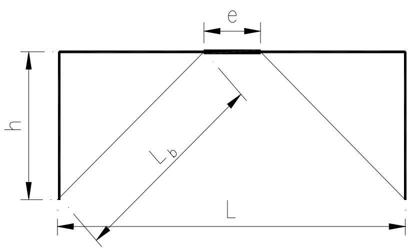
$D / t = 0.044\, E_s / F_y$ = 27.74 , for Pipe

$h / t = 1.12\, (E_s / F_y)^{0.5}$ = 28.12 , for Tube

> Actual **[Satisfactory]**

Where F_y = 46 ksi

E_s = 29000 ksi



DETERMINE FACTORED DESIGN LOADS (AISC 341-05 Sec.15.6)

$P_u = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + P_E$ = 629.9 kips

Where $P_E = 1.25\, R_y\, [V_n\, L\, L_b / (L\text{-}e)\, h]$ = 609.2 kips

R_y = 1.3 (1.4 for Pipe.)

DETERMINE DESIGN STRENGTH IN COMPRESSION (AISC 360-05 E3)

$\phi_cP_n = \phi_cA_gF_{cr}$ = 757.24 kips **>** P_u **[Satisfactory]**

ϕ_c = 0.85

F_e = $\pi^2 E / (K\ell / r)^2$ = 237.8 ksi

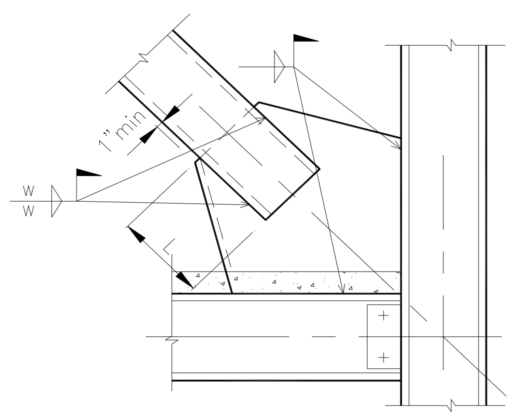
λ_c = $(K\ell / r)\,(F_y / E)^{0.5}$ = 1.38

K = 1.0

$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = 42.42 & \text{kis, for } \lambda_c \le 4.71 \\ 0.877\, F_e = \text{N/A} & \text{kis, for } \lambda_c > 4.71 \end{cases}$

DETERMINE CONNECTION DESIGN FORCE

$P_{ut} = P_u$ = 629.91 kips (Tension)



DETERMINE BEST FILLET WELD SIZE (AISC 360-05 Sec.J2.2b)

w = 5/16 in

> w_{MIN} = 0.1875 in

< w_{MAX} = 0.4375 in

[Satisfactory]

DETERMINE REQUIRED WELD LENGTH (AISC 360-05 Sec.J2.4)

$L = P_{ut} / [(4)\, \phi\, F_w\, (0.707\, w)]$

= 629.9 / [(4) 0.75 (0.6x70)(0.707x5/16)] = 22.63 in

(USE 23 in)

CHECK DESIGN SHEAR RUPTURE CAPACITY OF SLOTTED BRACE

(AISC 360-05 Sec.J4.2)

$\phi R_n = \phi(0.6F_u)A_{nu}$ = 1395.10 kips **>** P_{ut}

Where ϕ = 0.75 **[Satisfactory]**

F_u = 58 ksi (AISC 13th Tab.2-3)

A_{nu} = 4 t L = 4 x 0.581 x 23 = 53.45 in^2

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE (AISC 360-05 Tab. J2.4)

t_g = 5/8 in

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 J4.2)

$\phi R_n = \phi(0.6F_u)A_{nv}$ = 750.38 kips **>** P_{ut} **[Satisfactory]**

Where ϕ = 0.75

F_u = 58 ksi (plate value)

A_{nv} = 2 tg L = 2 x 5/8 x 23 = 28.75 in^2

CHECK TENSION AT SLOTTED BRACE (AISC 360-05 D.2 b)

$\phi_tP_n = \phi_tR_yF_uUA_n$ = 1146.48 kips **>** P_{ut}

Where ϕ_t = 0.75 **[Satisfactory]**

U = 1 (LRFD Sec.B3.2d)

$A_n = A - 2\, t\, t_g$ = 20.274 in^2

THE GUSSET BLOCK SHEAR CAPACITY (AISC 360-05 J4.3)

$\phi R_n = \phi(0.6F_u)A_{nv} + \phi F_yA_{gt}$ = 750.38 + ϕF_yA_{gt}

> P_{ut} = 629.91

[Satisfactory]

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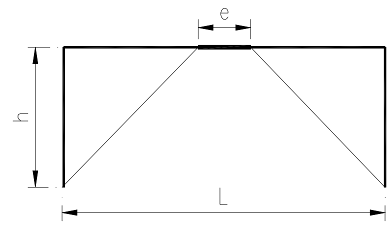
Seismic Design for Eccentrically Braced Frames Based on CBC 07 & AISC 341-05

INPUT DATA & DESIGN SUMMARY



MAX SERVICE LOADS AT OUTSIDE OF LINK	V_{DL} =	6.8	kips
	P_{DL} =	1	kips
	M_{DL} =	17	ft-kips
	V_{LL} =	4.8	kips
	P_{LL} =	0.7	kips
	M_{LL} =	11.3	ft-kips
SEISMIC LOADS AT OUTSIDE OF LINK	V_E =	8.7	kips (ASCE 7-05 12.4.2.1)
	P_E =	100	kips
	M_E =	100	ft-kips

(SEE LINK DESIGN SPREADSHEET FOR BALANCE OF INPUT DATA)

THE BEAM DESIGN IS ADEQUATE.

DETERMINE FACTORED DESIGN LOADS AT SECTION OF LINK AND BEAM (AISC 360-05 Sec. 15.6b & ASCE 7-05 12.4.2.3)

$V_u = (1.2 + 0.2S_{DS})V_{DL} + f_1V_{LL} + \rho V_E =$ 67.0 kips

$P_u = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + P_E =$ 537.9 kips

$M_u = (1.2 + 0.2S_{DS})M_{DL} + f_1M_{LL} + M_E =$ 923.0 ft-kips

Where f_1 = 0.5

R_y = 1.1 (AISC 341-05 Tab. I-6-1)

V_n = 369.2 kips (from link design)

$M_n = V_n\, e / 2 =$ 738.47 ft-kips

$V_E = (1.1R_y\, V_n / V_{E\,,\,link})\, V_E =$ 46.3 kips

$P_E = 1.1R_y\, V_n\, L / 2h =$ 536.1 kips

$M_E = 1.1R_y\, M_n =$ 893.5 ft-kips

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 4.24 **<** $0.3\,(E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

[$52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

$h / t_w =$ 22.00 **<**

$3.14(E_s/F_y)^{0.5}(1\text{-}1.54P_u/\phi_bP_y) =$ N/A , for $P_u/\phi_bP_y < 0.125$

[$520 / (F_y)^{0.5}(1\text{-}1.54P_u/\phi_bP_y)$ for AISC Seismic 97, Tab. I-9-1]

$(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = 55.19 , for $P_u/\phi_bP_y > 0.125$

{ MAX[$191 / (F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y)$, $253 / (F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 2105 kips

CHECK UNBALANCED SEGMENT LENGTH

$\angle_1 = (L - e - d_c) / 2 =$ 12.38 ft, (top & bottom flange bracing with a design strength greater than below will be provided at each end of the link segment.)

Brace Load : $P_{b,link} = 0.06R_y\, F_y\, b_f\, t_f =$ 48.8 kips, [AISC Seismic Sec.15.5]

$\angle_2 = \angle_1 / 2 =$ 6.19 ft, (lateral supported at middle of beam outside of link with following design strength.)

Brace Load : $P_{b,mid} = 0.02F_y\, b_f\, t_f =$ 14.8 kips, [AISC 341-05 Sec.15.6.(2)]

$M_{b,\,mid} = 0.02F_y\, b_f\, t_f\, d =$ 24.0 ft-kips, [AISC 341-05 Sec.15.6.(2)]

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$\phi_cP_n = \phi_cF_{cr}A =$ 1694.11 kips **>** P_u **[Satisfactory]**

Where ϕ_c = 0.85

K = 1.0

MAX($K\angle_1/r_x$, $K\angle_2/r_y$) = 27.34 **<** 200 **[Satisfactory]**

λ_c = 0.361 π

F_{cr} = 47.34 ksi

(cont'd)

DETERMINE FLEXURAL DESIGN STRENGTH (AISC-AISC 360-05 F1)

L_b = 6.19 ft

$L_p = 1.76\, r_y\, (E / F_{yf})^{0.5}$ = 9.60 ft

$L_r = r_y X_1 [1 + (1 + X_2 F_L^{\,2})^{0.5}]^{0.5} / F_L$ = 35.27 ft

$M_p = MIN(F_y Z_x , 1.5 F_y S_x)$ = 1341.7 ft-kips

$M_r = F_L S_x$ = 940.0 ft-kips

$M_{cr} = C_b S_x r_y X_1 (2 + X_1^2 X_2 r_y^2 / L_b^2)^{0.5} / L_b$ = 17113 ft-kips

Where $X_1 = \pi\, (0.5\, E\, G\, J\, A)^{0.5} / S_x$ = 4036.3

$X_2 = 4\, C_w\, [Sx / (G\, J)]^2 / I_y$ = 0.0006

F_r = 10.00 ksi

$F_L = MIN(F_{yf} - F_r , F_{yw})$ = 40.00 ksi

C_b = 1.30 , (AISC 360-05 F1)

A	I_y	t_f	r_y	S_x
42.1	311	1.32	2.72	282
E	**G**	**J**	**C_w**	**Z_x**
29000	11200	19.2	25700	322.0

M_n =	M_p	=	1341.7	ft-kips, for L_b @ [0 , L_p]
	$MIN\{C_b [M_p - (M_p - M_r) (L_b - L_p) / (L_r - L_p)] , M_p\}$	=	N/A	ft-kips, for L_b @ (L_p , L_r]
	$MIN(M_{cr} , M_p)$	=	N/A	ft-kips, for L_b @ (L_r , Larger)

$\phi_b\, M_n$ = 0.9 M_n = 1208 ft-kips

CHECK FLEXURAL CAPACITY (AISC 360-05 C2.1b)

$M_{ux} = B_1 M_u$ = 923.00 ft-kips **<** $\phi_b M_{nx} = Min(R_y\, \phi_b F_y Z, \phi_b M_n)$ = 1208 ft-kips

Where $P_{e1} = \pi^2 E_s I_x / (K L_x)^2$ = 35645 kips **[Satisfactory]**

C_m = 0.6 (AISC 360-05 C2.1b) Where ϕ_b = 0.9

$B_1 = C_m/(1 - P_u/P_{e1})$ = 1.000

CHECK INTERACTION CAPACITY (AISC 360-05 H1.1)

For $P_u/\phi_c P_n > 0.2$, $P_u / \phi_c P_n + 8 / 9\, (M_{ux} / \phi_b M_{nx})$ = 1.00 **<** **1** **[Satisfactory]**

For $P_u/\phi_c P_n < 0.2$, $P_u / (2\phi_c P_n) + M_{ux} / \phi_b M_{nx}$ = N/A

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Seismic Design for Eccentrically Braced Frames Based on CBC 07 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION	= >	**W14X145**		= >	**A**	**d**	**t_w**	**b_f**	**t_f**	**S_x**
COLUMN AXIAL SERVICE LOADS	P_{DL} =	**151**	kips		42.7	14.8	0.68	15.50	1.09	232
	P_{LL} =	**46**	kips			**I_x**	**r_x**	**r_y**	**Z_x**	**k**
NUMBER OF STORIES	n =	**4**				1710	6.33	3.98	260	1.69
COLUMN YIELD STRESS (36 or 50)	F_y =	**50**	ksi							

THE COLUMN DESIGN IS ADEQUATE.

UNBRACED COLUMN LENGTH ℓ = h = 14 ft

DETERMINE COLUMN AXIAL SEISMIC LOAD (AISC 341-05 Sec. 15.8)

P_E = (n - 1) 1.1 R_y V_n = 1340.3 kips

DETERMINE FACTORED DESIGN LOADS (AISC 360-05 Sec. 15.6b & ASCE 7-05 12.4.2.3)

$P_{u,t} = (0.9 - 0.2S_SD)\ P_{DL} - P_E$ = -1235 kips (Tension)

$P_{u,c} = (1.2 + 0.2S_{DS})P_{DL} + f_1P_{LL} + P_E$ = 1575 kips (Compression)

Where f_1 = 0.5 (CBC 1605.4)

CHECK LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f)$ = 7.11 < $0.3 (E_s / F_y)^{0.5}$ = 7.22 **[Satisfactory]**

[$52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

h / t_w = 16.79 <

$3.14(E_s/F_y)^{0.5}(1-1.54P_u/\phi_bP_y)$ = N/A , for $P_u/\phi_bP_y < 0.125$

[$520 / (F_y)^{0.5}(1-1.54P_u/\phi_bP_y)$ for AISC Seismic 97, Tab. I-9-1]

$(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = 40.74 , for $P_u/\phi_bP_y > 0.125$

{ MAX[$191 / (F_y)^{0.5}(2.33-P_u/\phi_bP_y)$, $253 / (F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_c = 0.9

$P_y = F_yA$ = 2135 kips

CHECK COMPRESSION CAPACITY (AISC 360-05 E3)

$\phi_cP_n = \phi_cF_{cr}A$ = 1593.27 kips > P_u **[Satisfactory]**

Where ϕ_c = 0.85

K = 1.0

MAX($K\ell_x/r_x$, $K\ell_y/r_y$) = 42.19 < 200 **[Satisfactory]**

λ_c = 0.558 π

F_{cr} = 43.90 ksi

Seismic Design for Intermediate/Ordinary Moment Resisting Frames Based on IBC 09, AISC 341-05 & AISC 358-05

INPUT DATA & DESIGN SUMMARY

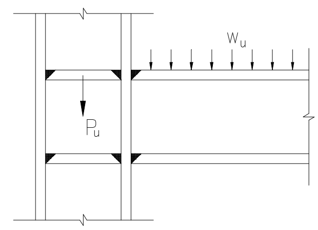
COLUMN SECTION => **W8X48**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.1	8.5	0.40	8.11	0.69	43.2	184	3.61	2.08	49	1.08

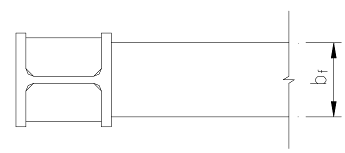
BEAM SECTION => **W14X48**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.1	13.8	0.34	8.03	0.60	70.2	484	5.86	1.91	78	1.19

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi
THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **27** kips
BEAM LENGTH BETWEEN COL. CENTERS L = **33** ft
AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **9** ft



THE DESIGN IS ADEQUATE.
(Continuity column stiffeners 0.75 x 4
with 1/4" fillet weld to web & CP to flanges.
A doubler plate is required with thickness of 1-5/8 in.)



ANALYSIS

THE SEISMIC DESIGN FACTOR COMPARISON (ASCE 7-05, Table 12.2-1)

FRAME TYPE	R	Ω_o	C_d
SMRF	8	3	5 1/2
IMRF	4.5	3	4
OMRF	3.5	3	3

CHECK BEAM LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f)$ = 6.75 **<** $0.3 (E_s / F_y)^{0.5}$ = 7.22 **[Satisfactory]**
[$52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]
Where E_s = 29000 ksi

h / t_w = 33.59 **<** $2.45 (E_s / F_y)^{0.5}$ = 59.00 **[Satisfactory]**
[$418 / (F_y)^{0.5}$ for FEMA Sec. 3.3.1.2]

CHECK COLUMN LOCAL BUCKING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f)$ = 5.92 **<** $0.3 (E_s / F_y)^{0.5}$ = 7.22 **[Satisfactory]**
[$52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

h / t_w = 15.85 **<**
$3.14(E_s/F_y)^{0.5}(1\text{-}1.54P_u/\phi_b P_y)$ = 70.67 , for $P_u/\phi_b P_y < 0.125$
[$520 / (F_y)^{0.5}(1\text{-}1.54P_u/\phi_b P_y)$ for AISC Seismic 97, Tab. I-9-1]
$(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = N/A , for $P_u/\phi_b P_y > 0.125$
{ MAX[$191 / (F_y)^{0.5}(2.33\text{-}P_u/\phi_b P_y)$, $253 / (F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_y A$ = 705 kips

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-05 Sec. 2.4.4)

t_{cf} = MIN{ $b_{bf} / 6$, $0.4[1.8b_{bf}\, t_{bf}\, (F_{yb}R_{yb}) / (F_{yb}R_{yb})]^{0.5}$ }= 1.17 in **>** actual t_{cf}
(The continuity plates required.)

$t_{st} = t_{bf}$ for interior connection, or (t_{bf} /2) for exterior connection = 0.60 in,USE 0.75 in

b_{st} = 4 in **<** $1.79 (E_s / F_{yst})^{0.5}\, t_{st}$ = 38.10 in, (LRFD Sec. K1.9)
[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr}\, A$ = 288.0 kips

Where ϕ_c = 0.9 — $h_{st} = d_c - 2k$ = 6.34
K = 0.75 — $K\, h_{st} / r_{st} < 200$ (AISC 360, E2) **[Satisfactory]**
$I = t_{st}\,(2b_{st} + t_{wc})^3 / 12$ = 36 in^4 — λ_c = 0.026
$A = 2b_{st}t_{st} + 25(t_{wc})^2$ = 9 in^2 — F_{cr} = 35.99 ksi
$r_{st} = (I / A)^{0.5}$ = 2.02 in — F_{yst} = 36 kips, plate yield stress

$P_{u,st} = R_{yb}\, F_{yb}\, b_{fb}\, t_{fb}$ = 262.8 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360-05 Sec.J2.2b)

w = 1/4 in > w_{MIN} = 0.1875 in
< w_{MAX} = 0.3125 in

[Satisfactory]

The required weld length between A36 continuity plates and column web (AISC 360-05 Sec.J2.2b)

$L_w = 0.6t_{st}L_{nst}F_y / [(2)\ \phi\ F_w\ (0.707\ w)]$ = (0.75 x 3.3) x 36 / [(2) 0.75 (0.6x70)(0.707x1/4)] = 3.44 in

Where $L_{net} = d_c - 2(k + 1.5)$ = 3.3 **<** $2(L_{net}$ -0.5) **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

(cont'd)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341-05 Sec. 10.2d)

t_{ReqD} = MAX (t_1, t_2) = 2.02 in

$t_1 = C_y M_{pr} (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h]$ = 2.02 in

Where $C_y = S_b / (C_{pr} Z_b)$ = 0.78

C_{pr} = 1.15 (AISC 341-05 Sec. 9.6 & AISC 358-05 Sec. 2.4.3)

R_y = 1.1 (AISC 341-05 Tab. I-6-1)

$S_b = 2I_b / d_b$ = 70 in^2

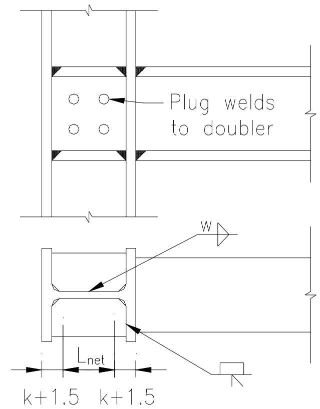
$I_b = I_x$ = 484 in^4

$M_{pr} = N_b C_{pr} R_y F_{yb} Z_b$ = 826 ft-kips

N_b = 2 , (if double side connection of beams, input 2)

$t_2 = (d_z + w_z) / 90 = (d_b - 2t_{st} + d_c - 2k) / 90$ = 0.21 in

Since t_{wc} = 0.40 in < t_{ReqD} , a doubler plate is required with thickness of 1-5/8 in.



Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.
3. AISC 358-05: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, Dec 13, 2005.
4. Thomas A. Sabol, Ph.D., S. E.: "2005 AISC Seismic Provisions and Seismic Design Manual Seminar", AISC, Oct. 12, 2006.

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Seismic Design for Intermediate/Ordinary Moment Resisting Frames Based on CBC 10, AISC 341-05 & AISC 358-05

INPUT DATA & DESIGN SUMMARY

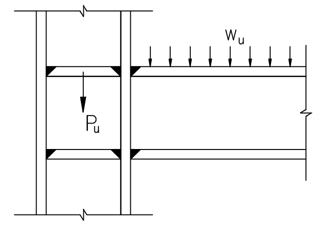
COLUMN SECTION => **W8X48**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.1	8.5	0.40	8.11	0.69	43.2	184	3.61	2.08	49	1.08

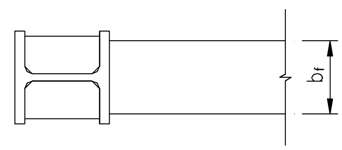
BEAM SECTION => **W14X48**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.1	13.8	0.34	8.03	0.60	70.2	484	5.86	1.91	78	1.19

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi
THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **27** kips
BEAM LENGTH BETWEEN COL. CENTERS L = **33** ft
AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **9** ft



THE DESIGN IS ADEQUATE.
(Continuity column stiffeners 0.75 x 4
with 1/4" fillet weld to web & CP to flanges.
A doubler plate is required with thickness of 1-5/8 in.)



ANALYSIS

THE SEISMIC DESIGN FACTOR COMPARISON (ASCE 7-05, Table 12.2-1)

FRAME TYPE	R	Ω_o	C_d
SMRF	8	3	5 1/2
IMRF	4.5	3	4
OMRF	3.5	3	3

CHECK BEAM LOCAL BUCKLING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 6.75 < $0.3\,(E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**
$[\,52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]
Where E_s = 29000 ksi

$h / t_w =$ 33.59 < $2.45\,(E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**
$[\,418 / (F_y)^{0.5}$ for FEMA Sec. 3.3.1.2]

CHECK COLUMN LOCAL BUCKING LIMITATION (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 5.92 < $0.3\,(E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**
$[\,52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

$h / t_w =$ 15.85 <
$3.14(E_s/F_y)^{0.5}(1\text{-}1.54P_u/\phi_bP_y) =$ 70.67 , for $P_u/\phi_bP_y < 0.125$
$[\,520 / (F_y)^{0.5}(1\text{-}1.54P_u/\phi_bP_y)$ for AISC Seismic 97, Tab. I-9-1]
$(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = N/A , for $P_u/\phi_bP_y > 0.125$
{ MAX[$191 / (F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y)$, $253 / (F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 705 kips

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-05 Sec. 2.4.4)

$t_{cf} = \text{MIN}\{\, b_{bf} / 6\ ,\ 0.4[1.8b_{bf}\,t_{bf}\,(F_{yb}R_{yb}) / (F_{yb}R_{yb})]^{0.5} \}=$ 1.17 in > actual t_{cf}
(The continuity plates required.)

$t_{st} = t_{bf}$ for interior connection, or (t_{bf} /2) for exterior connection = 0.60 in,USE 0.75 in, (5/8 in)

$b_{st} =$ 4 in < $1.79\,(E_s / F_{yst})^{0.5}\,t_{st} =$ 38.10 in, (LRFD Sec. K1.9)
[Satisfactory]

$\phi_cP_{n,st} = \phi_cF_{cr}\,A =$ 288.0 kips

Where $\phi_c =$ 0.9 $h_{st} = d_c - 2k =$ 6.34
K = 0.75 $K\,h_{st} / r_{st}$ < 200 (AISC 360, E2) **[Satisfactory]**
$I = t_{st}\,(2b_{st} + t_{wc})^3 / 12 =$ 36 in^4 $\lambda_c =$ 0.026
$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 9 in^2 F_{cr} = 35.99 ksi
$r_{st} = (\,I / A\,)^{0.5} =$ 2.02 in F_{yst} = 36 kips, plate yield stress

$P_{u,st} = R_{yb}\,F_{yb}\,b_{fb}\,t_{fb} =$ 262.8 kips < $\phi_cP_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360-05 Sec.J2.2b)

w = 1/4 in
> w_{MIN} = 0.1875 in
< w_{MAX} = 0.3125 in

[Satisfactory]

The required weld length between A36 continuity plates and column web (AISC 360-05 Sec.J2.2b)

$L_w = 0.6t_{st}L_{nst}F_y / [(2)\ \phi\ F_w\,(0.707\ w)]$ = (0.75 x 3.3) x 36 / [(2) 0.75 (0.6x70)(0.707x1/4)] = 3.44 in

Where $L_{net} = d_c - 2(k + 1.5) =$ 3.3 < $2(L_{net}\,\text{-}0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

(cont'd)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341-05 Sec. 10.2d)

t_{ReqD} = MAX (t_1, t_2) = 2.02 in

$t_1 = C_y M_{pr} (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h]$ = 2.02 in

Where $C_y = S_b / (C_{pr} Z_b)$ = 0.78

C_{pr} = 1.15 (AISC 341-05 Sec. 9.6 & AISC 358-05 Sec. 2.4.3)

R_y = 1.1 (AISC 341-05 Tab. I-6-1)

$S_b = 2I_b / d_b$ = 70 in^2

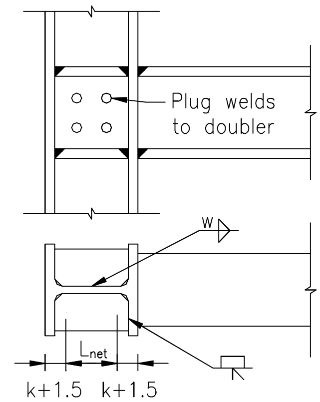
$I_b = I_x$ = 484 in^4

$M_{pr} = N_b C_{pr} R_y F_{yb} Z_b$ = 826 ft-kips

N_b = 2 , (if double side connection of beams, input 2)

$t_2 = (d_z + w_z) / 90 = (d_b - 2t_{st} + d_c - 2k) / 90$ = 0.21 in

Since t_{wc} = 0.40 in < t_{ReqD} , a doubler plate is required with thickness of 1-5/8 in.



Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.
3. AISC 358-05: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, Dec 13, 2005.
4. Thomas A. Sabol, Ph.D., S. E.: "2005 AISC Seismic Provisions and Seismic Design Manual Seminar", AISC, Oct. 12, 2006.

Seismic Design for Special Moment Resisting Frames Based on IBC 09, AISC 341-05 & 358-05

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W14X257**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
75.6	16.4	1.18	16.00	1.89	415	3400	6.71	4.13	487	2.49

BEAM SECTION => **W24X76**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
22.4	23.9	0.44	8.99	0.68	176	2100	9.68	1.92	200	1.18

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

FACTOR GRAVITY LOAD ON THE BEAM w_u = **1.31** klf

FACTOR AXIAL LOAD ON THE COLUMN P_u = **199.6** kips

BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft

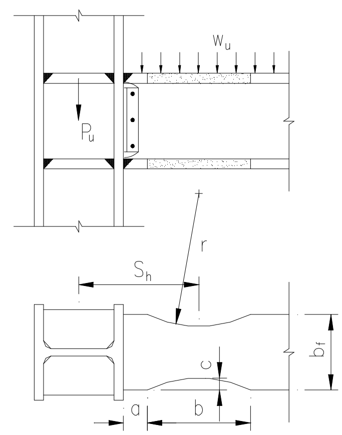
AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **13.25** ft

REDUCED SECTION DIMENSIONS a = **5.5** in, [0.5~0.75b_f]

(AISC 358-05 Sec. 5.8) b = **18** in, [0.65~0.85d_b]

c = **2.125** in, [0.1~ 0.25b_f]

THE SMRF DESIGN IS ADEQUATE.
(Continuity column stiffeners 11/16 x 7
with 7/16" fillet weld to web & CP to flanges.
A doubler plate is not required.)



ANALYSIS

$r = (4c^2 + b^2) / 8c =$ 20.1 in, (AISC 358-05 Fig. 5.1)

$S_h = d_c/2 + a + b/2 =$ 22.7 in

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 6.61 < $0.3 (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

[52 / $(F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

$h / t_w =$ 48.95 < $2.45 (E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**

[418 / $(F_y)^{0.5}$ for FEMA Sec. 3.3.1.2]

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f) =$ 4.23 < $0.3 (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

[52 / $(F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

$h / t_w =$ 9.68 <

$3.14(E_s/F_y)^{0.5}(1\text{-}1.54C_a) =$ 68.79 , for $C_a = P_u/\phi_b P_y \le 0.125$

[520 / $(F_y)^{0.5}(1\text{-}1.54P_u/\phi_b P_y)$ for AISC Seismic 97, Tab. I-9-1]

$(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = N/A , for $C_a = P_u/\phi_b P_y > 0.125$

{ MAX[191 / $(F_y)^{0.5}(2.33\text{-}P_u/\phi_b P_y)$, 253 / $(F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_y A$ = 3780 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-05 Sec. 9.6)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*) =$ 2.40 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c Z_c (F_{yc} - P_u / A_g) + V_{col} (d_b / 2) =$ 3844 + 0 = 3844 ft-kips

N_c = 2 , (if only one column below, input 1)

$\Sigma M_{pb}^* = N_b (M_{RBS} + M_v) =$ 1598 ft-kips, at center of column

N_b = 2 , (if double side connection of beams, input 2)

$M_v = V_{RBS} S_h = [2M_{RBS} / (L\text{-}2S_h) + w_u (L\text{-}2S_h)/2] S_h =$ 129 ft-kips

$M_{RBS} = C_{pr} R_y F_{yb} Z_{RBS} =$ 670 ft-kips

R_y = 1.1 (AISC 341-05 Tab. I-6-1)

$Z_{RBS} = Z_b - 2c\, t_f (d - t_f) =$ 133 in^3

C_{pr} = 1.1 (1.1 from AISC 341-05 Sec. 9.6, or $F_y+F_u / 2F_y$ from AISC 358-05 Sec. 2.4.3)

CHECK BENDING MOMENT AT THE COLUMN FACE (AISC 358-05 Sec. 5.8)

$M_f = M_{RBS} + [2M_{RBS} / (L - 2S_h) + w_u(L - 2S_h)/2] (a + b/2)$

= 753 ft-kips < $\phi_d M_{pe} = \phi_d R_y F_{yb} Z_b =$ 917 ft-kips **[Satisfactory]**

Where ϕ_d = 1.00 (AISC 358-05 Sec. 2.4.1a)

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-05 Sec. 2.4.4)

$t_{cf} = \text{MIN}\{ b_{bf} / 6 , 0.4[1.8b_{bf} t_{bf} (F_{yb}R_{yb}) / (F_{yb}R_{yb})]^{0.5} \} =$ 1.33 in < actual t_{cf}

(The continuity plates may not be required.)

$t_{st} = t_{bf}$ for interior connection, or (t_{bf} /2) for exterior connection = 0.68 in, USE 0.69 in, (11/16 in)

b_{st} = 7 in < $0.56 (E / F_{yst})^{0.5} t_{st} =$ 10.93 in, (AISC 360-05 Sec. G3.3)

[Satisfactory]

(Cont'd)

$\phi_c P_{n,st} = \phi_c F_{cr}\, A =$ 468.5 kips

Where $\phi_c =$ 0.9 (AISC 360-05 E1)

$K =$ 0.75

$I = t_{st}\,(2b_{st} + t_{wc})^3 / 12 =$ 200 in^4

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 14 in^2

$r_{st} = (I / A)^{0.5} =$ 3.72 in

$F_{yst} =$ 36 kips, plate yield stress

$h_{st} = d_c - 2k =$ 11.42 in

$K\, h_{st} / r_{st} < 200$ (AISC 360, E2) **[Satisfactory]**

$F_e = \pi^2 E / (K\, h_{st} / r_{st})^2 =$ 54054 ksi, (AISC 360, E3)

$\lambda_c = (K\, h_{st} / r_{st})\,(F_y / E)^{0.5} =$ 0.08

$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = & 35.99 \quad , \lambda_c \le 4.71 \\ 0.877\, F_e = & \text{N/A} \quad \text{kis, for } \lambda_c > 4.71 \end{cases}$

$P_{u,st} = R_{yb}\, F_{yb}\, b_{fb}\, t_{fb} =$ 336.2 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360-05 Sec.J2.2b)

w = 7/16 in

> w_{MIN} = 0.25 in

< w_{MAX} = 0.5625 in

[Satisfactory]

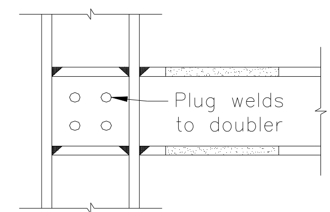
The required weld length between continuity plates and column web (AISC 360-05 Sec.J2.2b)

$L_w = 0.6t_{st}L_{nst}F_y / [(2)\ \phi\ F_w\ (0.707\ w)]$ = 0.6 x (0.6875 x 8.4) x 36 / [(2) 0.75 (0.6x70)(0.707x7/16)] = 4.89 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 8.4 **<** $2(L_{net} - 0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341-05 Sec. 9.3)



$t_{ReqD} = \text{MAX}\ (t_1, t_2) =$ 1.09 in

$t_1 = C_y\, M_c\, (h - d_b) / [0.9\ (0.6)\ F_{yc}\, R_{yc}\, d_c\, (d_b - t_{fb})\, h] =$ 1.09 in

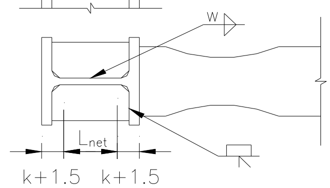
Where $C_y = S_b / (C_{pr}\, Z_{RBS}) =$ 0.76

$S_b = 2I_b / d_b =$ 111 in^2

$I_b = I_x - (4\ c\ t_{fb})(0.5d_b - 0.5t_{fb})^2 =$ 1321 in^4

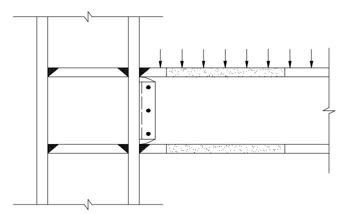
$M_c = \Sigma M_{pb}^* =$ 1598 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b - 2t_{st} + d_c - 2k) / 90 =$ 0.38 in



Since $t_{wc} =$ 1.18 in **>** t_{ReqD}, a doubler plate is not required.

CHECK BEAM TO COLUMN CP FLANGES WELD AT 0.04 RADIANS STORY DRIFT (AISC 341-05 Sec. 9.2a)



$n = \dfrac{I_b}{I_c}\dfrac{h}{L} =$ 0.273 $\qquad a = h\left(\dfrac{3n}{6n+1}\right) =$ 4.11 ft

$b = h\left(\dfrac{3n+1}{6n+1}\right) =$ 9.14 ft $\qquad \Delta = 0.04h =$ 6.36 in

$M_{Beam} = \dfrac{3nh}{2(1+6n)}\dfrac{6E}{\left[\dfrac{b^3}{I_c} + a^2\left(\dfrac{a}{I_c} + \dfrac{L}{2I_b}\right)\right]}\Delta =$ 3601 ft-kips, (conservative value at center line of column)

$M_{u,f} = \text{MAX}(\psi M_{Beam}\ , 0.8M_p) =$ 3241 ft-kips, (AISC 341-05 Sec. 9.2a.2)

< $\phi M_n = \phi\ 0.6F_{EXX}\ 0.5t_f b_f (d - t_f)^2 =$ 4326 ft-kips, (AISC 360-05 Sec. J2.4)

[Satisfactory]

Where $\psi =$ 0.9 , changeable factor for beam moment at column face

$M_p =$ 833 ft-kips

Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.
3. AISC 358-05: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, Dec 13, 2005.
4. Thomas A. Sabol, Ph.D., S. E.: "2005 AISC Seismic Provisions and Seismic Design Manual Seminar", AISC, Oct. 12, 2006.

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CLIENT :
JOB NO. : DATE :

PAGE :
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REVIEW BY :

Seismic Design for Special Moment Resisting Frames Based on CBC 10, AISC 341-05 & 358-05

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W14X257**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
75.6	16.4	1.18	16.00	1.89	415	3400	6.71	4.13	487	2.49

BEAM SECTION => **W8X67**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
19.7	9.0	0.57	8.28	0.94	60.4	272	3.72	2.12	70	1.33

CBC 2010 Sec. 2205A.4.1.1 APPLY? ==> **Yes**

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

FACTOR GRAVITY LOAD ON THE BEAM w_u = **1.31** klf

FACTOR AXIAL LOAD ON THE COLUMN P_u = **199.6** kips

BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft

AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **21** ft

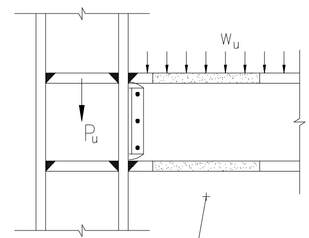
REDUCED SECTION DIMENSIONS a = **5.5** in, $[0.5\sim0.75b_f]$

(AISC 358-05 Sec. 5.8) b = **6.3** in, $[0.65\sim0.85d_b]$

c = **1.242** in, $[0.1\sim0.25b_f]$

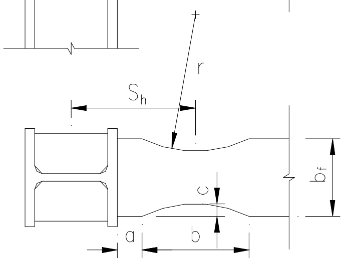
THE SMRF DESIGN IS ADEQUATE.

(Continuity column stiffeners 15/16 x 7.4
with 3/4" fillet weld to web & CP to flanges.
A doubler plate is required with thickness of 1/4 in.)



ANALYSIS $r = (4c^2 + b^2) / 8c$ = 4.6 in, (AISC 358-05 Fig. 5.1)

$S_h = d_c/2 + a + b/2$ = 16.9 in



CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f)$ = 4.43 < $0.3 (E_s / F_y)^{0.5}$ = 7.22 **[Satisfactory]**

$[52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

Where E_s = 29000 ksi

h / t_w = 11.12 < $2.45 (E_s / F_y)^{0.5}$ = 59.00 **[Satisfactory]**

$[418 / (F_y)^{0.5}$ for FEMA Sec. 3.3.1.2]

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-05 Tab. I-8-1)

$b_f / (2t_f)$ = 4.23 < $0.3 (E_s / F_y)^{0.5}$ = 7.22 **[Satisfactory]**

$[52 / (F_y)^{0.5}$ for AISC Seismic 97, Tab. I-9-1]

h / t_w = 9.68 <

$3.14(E_s/F_y)^{0.5}(1\text{-}1.54C_a)$ = 68.79 , for $C_a = P_u/\phi_bP_y \le 0.125$

$[520 / (F_y)^{0.5}(1\text{-}1.54P_u/\phi_bP_y)$ for AISC Seismic 97, Tab. I-9-1]

$(E_s/F_y)^{0.5}$ MAX[1.49, 1.12(2.33 - C_a)] = N/A , for $C_a = P_u/\phi_bP_y > 0.125$

{ MAX[191 / $(F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y)$, 253 / $(F_y)^{0.5}$] for AISC Seismic 97, Tab. I-9-1}

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 3780 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-05 Sec. 9.6)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*)$ = 6.19 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c Z_c (F_{yc} - P_u / A_g) + V_{col} (d_b / 2)$ = 3844 + 0 = 3844 ft-kips

N_c = 2 , (if only one column below, input 1)

$\Sigma M_{pb}^* = N_b (M_{RBS} + M_v)$ = 621 ft-kips, at center of column

N_b = 2 , (if double side connection of beams, input 2)

$M_v = V_{RBS} S_h = [2M_{RBS} / (L\text{-}2S_h) + w_u (L\text{-}2S_h)/2] S_h$ = 52 ft-kips

$M_{RBS} = C_{pr}R_yF_{yb} Z_{RBS}$ = 259 ft-kips

R_y = 1.1 (AISC 341-05 Tab. I-6-1)

$Z_{RBS} = Z_b - 2c\, t_f (d - t_f)$ = 51 in^3

C_{pr} = 1.1 (1.1 from AISC 341-05 Sec. 9.6, or F_y+F_u / $2F_y$ from AISC 358-05 Sec. 2.4.3)

CHECK BENDING MOMENT AT THE COLUMN FACE (AISC 358-05 Sec. 5.8)

$M_f = M_{RBS} + [2M_{RBS} / (L - 2S_h) + w_u(L - 2S_h)/2] (a + b/2)$

= 286 ft-kips < $\phi_d M_{pe} = \phi_d R_y F_{yb} Z_b$ = 321 ft-kips **[Satisfactory]**

Where ϕ_d = 1.00 (AISC 358-05 Sec. 2.4.1a)

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-05 Sec. 2.4.4)

t_{cf} = MIN{ b_{bf} / 6 , $0.4[1.8b_{bf}\, t_{bf} (F_{yb}R_{yb}) / (F_{yb}R_{yb})]^{0.5}$ }= 1.38 in < actual t_{cf}

(The continuity plates may not be required.)

$t_{st} = t_{bf}$ for interior connection, or (t_{bf} /2) for exterior connection = 0.94 in, USE 0.94 in, (15/16 in)

b_{st} = 7.4 in < $0.56 (E / F_{yst})^{0.5} t_{st}$ = 14.90 in, (AISC 360-05 Sec. G3.3)

[Satisfactory]

(Cont'd)

$\phi_c P_{n,st} = \phi_c F_{cr}\, A =$ 712.5 kips

Where $\phi_c =$ 0.9 (AISC 360-05 E1)

$K =$ 0.75

$I = t_{st}\,(2b_{st} + t_{wc})^3 / 12 =$ 319 in^4

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 22 in^2

$r_{st} = (I / A)^{0.5} =$ 3.81 in

$F_{yst} =$ 36 kips, plate yield stress

$h_{st} = d_c - 2k =$ 11.42 in

$K\,h_{st} / r_{st} < 200$ (AISC 360, E2) **[Satisfactory]**

$F_e = \pi^2 E / (K\,h_{st} / r_{st})^2 =$ 56544 ksi, (AISC 360, E3)

$\lambda_c = (K\,h_{st} / r_{st})\,(F_y / E)^{0.5} =$ 0.08

$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = & 35.99 \quad , \lambda_c \le 4.71 \\ 0.877\,F_e = & \text{N/A} \quad \text{kis, for } \lambda_c > 4.71 \end{cases}$

$P_{u,st} = R_{yb}\,F_{yb}\,b_{fb}\,t_{fb} =$ 425.8 kips < $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360-05 Sec.J2.2b)

w = 3/4 in

> w_{MIN} = 0.3125 in

< w_{MAX} = 0.8125 in

[Satisfactory]

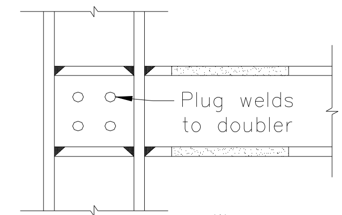
The required weld length between continuity plates and column web (AISC 360-05 Sec.J2.2b)

$L_w = 0.6 t_{st} L_{nst} F_y / [(2)\,\phi\,F_w\,(0.707\,w)]$ = 0.6 x (0.9375 x 8.4) x 36 / [(2) 0.75 (0.6x70)(0.707x3/4)] = 3.61 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 8.4 < $2(L_{net} - 0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341-05 Sec. 9.3)



$t_{ReqD} = \text{MAX}\,(t_1, t_2) =$ 1.41 in

$t_1 = C_y\,M_c\,(h - d_b) / [0.9\,(0.6)\,F_{yc}\,R_{yc}\,d_c\,(d_b - t_{fb})\,h] =$ 1.41 in

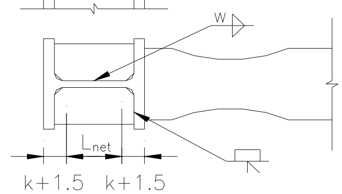
Where $C_y = S_b / (C_{pr}\,Z_{RBS}) =$ 0.77

$S_b = 2I_b / d_b =$ 44 in^2

$I_b = I_x - (4\,c\,t_{fb})(0.5d_b - 0.5t_{fb})^2 =$ 196 in^4

$M_c = \Sigma M_{pb}{}^* =$ 621 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b - 2t_{st} + d_c - 2k) / 90 =$ 0.21 in



Since $t_{wc} =$ 1.18 in < t_{ReqD}, a doubler plate is required with thickness of 1/4 in.

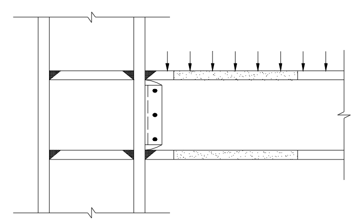
CHECK BEAM TO COLUMN CP FLANGES WELD AT 0.04 RADIANS STORY DRIFT (AISC 341-05 Sec. 9.2a)

$$n = \frac{I_b}{I_c}\frac{h}{L} = \quad 0.056 \qquad a = h\left(\frac{3n}{6n+1}\right) = \quad 2.64 \text{ ft}$$

$$b = h\left(\frac{3n+1}{6n+1}\right) = \quad 18.36 \text{ ft} \qquad \Delta = 0.04h = \quad 10.08 \text{ in}$$

$$M_{Beam} = \frac{3nh}{2(1+6n)}\,\frac{6E}{\left[\frac{b^3}{I_c} + a^2\left(\frac{a}{I_c} + \frac{L}{2I_b}\right)\right]}\Delta = \quad 606$$

ft-kips, (conservative value at center line of column)



$M_{u,f} = \text{MAX}(\psi M_{Beam}, 0.8M_p) =$ 546 ft-kips, (AISC 341-05 Sec. 9.2a.2)

< $\phi M_n = \phi\, 0.6F_{EXX}\, 0.5 t_f b_f (d - t_f)^2 =$ 661 ft-kips, (AISC 360-05 Sec. J2.4)

[Satisfactory]

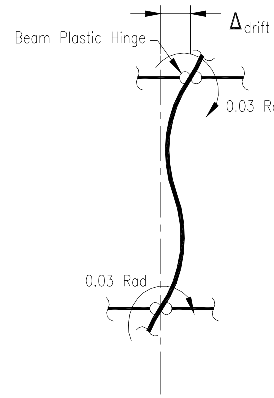
Where $\psi =$ 0.9 , changeable factor for beam moment at column face

$M_p =$ 292 ft-kips

(Cont'd)

CHECK COLUMN UNDER CONNECTION INELASTIC ROTATION OF 0.03 RADIANS (CBC 2010 Sec. 2205A.4.1.1)

$$\{F\} = [K]\{\Delta\} = \begin{vmatrix} 72 & 9075 & -72 & 9075 \\ 9075 & 1534760 & -9075 & 752220 \\ -72 & -9075 & 72 & -9075 \\ 9075 & 752220 & -9075 & 1534760 \end{vmatrix} \begin{vmatrix} 0 \\ 0.03 \\ 10.08 \\ 0.03 \end{vmatrix} = \begin{vmatrix} -182 \\ -22870 \\ 182 \\ -22870 \end{vmatrix}$$



Where Δ_{drift} = [0 ~ 0.04 h] = 10.08 in

$M_{u,\, max}$ = **22870** in-kips, = 1905.8 ft-kips

> $\phi M_n = \phi F_y Z_x$ = 1826.3 ft-kips **[Caution !]**

Technical References:

1. AISC 341-05: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Nov. 16, 2005.
2. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.
3. AISC 358-05: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, Dec 13, 2005.

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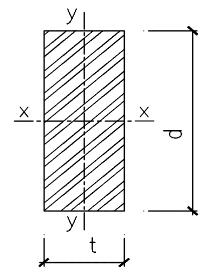
Rectangular Section Member Design Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY



STEEL YIELD STRESS	F_y =	**36**	ksi
SECTION DIMENSIONS	t =	**0.625**	in
	d =	**5**	in
STRONG AXIS, x-x, UNBRACED BENDING LENGTH	L_b =	**3.5**	ft
STRONG AXIS, x-x, UNBRACED AXIAL LENGTH	$K L_x$ =	**3.5**	ft
WEAK AXIS, y-y, UNBRACED AXIAL LENGTH	$K L_y$ =	**3.5**	ft
AXIAL LOAD, ASD	P =	**6**	kips
STRONG AXIS, x-x, BENDING LOAD, ASD	M_x =	**0.9**	ft-kips
WEAK AXIS, y-y, BENDING LOAD, ASD	M_y =	**0.8**	ft-kips
STRONG DIRECTION SHEAR LOAD, ASD	V =	**0.2**	kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK BENDING CAPACITY (AISC 360-05, F11)

$$\frac{M_n}{\Omega_b}=\frac{1}{\Omega_b}\begin{cases} M_p \ , \ for \ \frac{L_b d}{t^2}\le\frac{0.08E}{F_y} \\ Min\left(C_b\left[1.52-0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \ , \ M_p\right) \ , \ for \ \frac{0.08E}{F_y}<\frac{L_b d}{t^2}\le\frac{1.9E}{F_y} \\ Min\left(F_{cr}S_x \ , \ M_p\right) \ , \ for \ \frac{L_b d}{t^2}>\frac{1.9E}{F_y}\end{cases} = 6.3 \text{ ft-kips} > M_x$$

[Satisfactory]

Where M_p = Min(F_y Z , 1.6 M_y) = 11.7 ft-kips
$Z = t\,d^2/4$ = 3.9 in^3
$M_y = F_y\, t\, d^2/6$ = 7.8 ft-kips
F_{cr} = Min[1.9 E C_b t^2 / (d L_b) , F_y] = 36.0 ksi

C_b = 1.0 , (AISC 360-05, F1)
$S_x = t\,d^2/6$ = 2.6 in^3
Ω_b = 1.67 , (AISC 360-05, F1)
E = 29000 ksi

CHECK COMPRESSION CAPACITY (AISC 360-05, E7)

$$\frac{P_n}{\Omega_c}=\frac{A_g}{\Omega_c}\begin{cases} Q\left[0.658^{QF_y/F_e}\right]F_y \ , \ for \ \frac{KL}{r}\le 4.71\sqrt{\frac{E}{QF_y}} \ or \ F_e\ge 0.44QF_y \\ 0.877F_e \ , \ for \ \frac{KL}{r}>4.71\sqrt{\frac{E}{QF_y}} \ and \ F_e<0.44QF_y\end{cases} = 8.7 \text{ kips} > P$$

[Satisfactory]

Where $KL/r = 12^{0.5}$ Max($K L_x$ / d , $K L_y$ / t) = 232.8
$F_e = \pi^2 E / (KL/r)^2$ = 5.28 ksi
Q_a = 1.00 , (AISC 360-05, E7.2.b)
$Q = Q_s Q_a$ = 1.00

Ω_c = 1.67 , (AISC 360-05, E1)
Q_s = 1.00 , (AISC 360-05, E7.1.a)
F_{cr} = 4.63 ksi, for Q = 1

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases}\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) \ , \ for \ \frac{P_r}{P_c}\ge 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) \ , \ for \ \frac{P_r}{P_c}<0.2\end{cases} = 0.97 < 1.0$$

[Satisfactory]

Where	P_r =	6.0	kips		$P_c = P_n / \Omega_c$ =	8.7 kips, (AISC 360-05 Chapter E)
	M_{rx} =	0.9	ft-kips		$M_{cx} = M_n / \Omega_b$ =	6.3 ft-kips, (AISC 360-05 Chapter F)
	M_{ry} =	0.8	ft-kips	<	$M_{cy} = M_n / \Omega_b$ =	4.7 ft-kips, (AISC 360-05 Chapter F)

CHECK SHEAR CAPACITY (AISC 360-05, G2)

V_{ny} / Ω_v = 75.2 / 1.67 = 45.0 kips > V = 0.2 kips **[Satisfactory]**

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Simply Supported Member of Triple W-Shapes Design Based on AISC 13th (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

W-SHAPE YIELD STRESS F_y = **50** ksi

DOUBLE PARALLEL W-SHAPE SECTION => **W18X65**

A	d	r_x	r_y	I_x	S_x	I_y	S_y	λ	t_w	b_f	t_f
19.1	18.4	7.48	1.69	1070	117	55	14.4	0.0158	0.45	7.59	0.75

THE DOUBLE W-SHAPE SPAN S = **32** ft, (simply supported)

ONE PERPENDICULAR W-SHAPE SECTION => **W16X45**

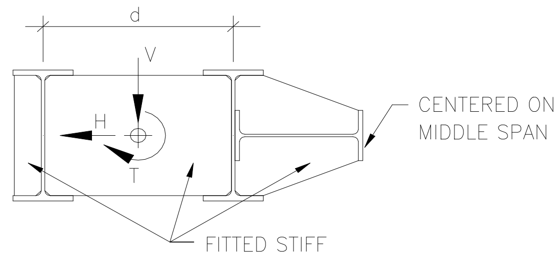
A	d	r_x	r_y	I_x	S_x	I_y	S_y	λ	t_w	b_f	t_f
13.3	16.1	6.64	1.57	586	72.7	33	9.34	0.0147	0.35	7.04	0.57

THE ONE W-SHAPE LENGTH L = **29** ft, (centered on the middle of span)
(may not full span)

DOUBLE W-SHAPE DISTANCE d = **24** in

FITTED STIFF THICKNESS t = **0.5** in

FITTED STIFF SPACING s = **96** in



LATERAL DISTRIBUTED LOAD V = **1.5** kips / ft
H = **1.5** kips / ft

DISTRIBUTED TORSION LOAD T = **0.7333** ft-kips / ft

AXIAL LOAD (BY RIGID END PLATE OR EQUAL) P = **96** kips

THE TRIPLE W-SHAPES DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE GOVERNING DESIGN LOADS ON SIGNAL W-SHAPES

Assume conservatively that the perpendiculat W-Shape only supports the H direction bending load, and the torsion load, T, convert to a coupling force on double W-Shapes.

For one of double W-Shapes, W18X65

$M_x = (0.5\ V + T / d)\ S^2 / 8$ =	142.9	ft-kips at middle	L_b =	8.0	ft
P = 0.5 P =	48.0	kips at middle	L_x =	8.0	ft
$V_x = (0.5\ V + T / d)\ S / 2$ =	17.9	kips at end	L_y =	8.0	ft
$V_y = (0.5\ H)\ S / 2$ =	12.0	kips at end			

For one perpendicular W-Shapes, W16X45

$M_x = (0.5\ H)\ S^2 / 8$ =	96.0	ft-kips at middle	L_b =	8.0	ft

For stiff. plate vert. section, 0.5" x 16.9"

$M_x = V\ d / 4$ =	0.8	ft-kips at center	L_b =	1.4	ft
P = H s =	12.0	kips at center	L_y =	2.0	ft

CHECK COMBINED COMPRESSION AND BENDING CAPACITY OF SINGLE DOUBLE W-SHAPE (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}\ge 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}< 0.2 \end{cases} = 0.67 < 1.0$$ **[Satisfactory]**

Where P_r = 48.0 kips
M_{rx} = 142.9 ft-kips
M_{ry} = 9.6 ft-kips, 10% perpendicular bending load for over design
$P_c = P_n / \Omega_c$ = 944 / 1.67 = 565.0 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**
$M_{cx} = M_n / \Omega_b$ = 520.7 / 1.67 = 311.8 ft-kips, (AISC 360-05 Chapter F)
> M_{rx} **[Satisfactory]**
$M_{cy} = M_n / \Omega_b$ = 93.8 / 1.67 = 56.1 ft-kips, (AISC 360-05 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK WEB SHEAR CAPACITY OF SINGLE DOUBLE W-SHAPE (AISC 360-05, G2)

V_{nx} / Ω_v = 248.4 / 1.67 = 148.7 kips > V_x = 17.9 kips **[Satisfactory]**

(cont'd)

CHECK FLANGES SHEAR CAPACITY OF SINGLE DOUBLE W-SHAPE (AISC 360-05, G2)

V_{ny} / Ω_v = 380.3 / 1.67 = 227.7 kips **>** V_y = 12.0 kips **[Satisfactory]**

CHECK FLEXURAL CAPACITY OF PERPENDICULAR W-SHAPE (AISC 360-05 Chapter F)

M_n / Ω_b = 518.8 / 1.67 = 310.7 kips **>** M_x = 96.0 kips **[Satisfactory]**

CHECK COMBINED COMPRESSION AND BENDING CAPACITY OF FITTED STIFF PLATE (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) \ , \ for \ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}\right) \ , \ for \ \frac{P_r}{P_c}< 0.2 \end{cases} = 0.74 < 1.0$$ **[Satisfactory]**

Where P_r = 12.0 kips

M_{rx} = 0.8 ft-kips

$P_c = P_n / \Omega_c$ = 72.9 / 1.67 = 43.7 kips, (AISC 360-05 Chapter E)

> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 2.4 / 1.67 = 1.4 ft-kips, (AISC 360-05 Chapter F)

> M_{rx} **[Satisfactory]**

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Steel Beam Design with Gravity Loading Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

BEAM SECTION (WF, Tube or WT) => **W14X22**

WF	I_x	S_x	J	b_f	t_f	t_w
	199	29	0.21	5.00	0.34	0.23

SLOPED DEAD LOADS $w_{DL,1}$ = **0.2** kips / ft
$w_{DL,2}$ = kips / ft

PROJECTED LIVE LOADS $w_{LL,1}$ = **0.16** kips / ft
$w_{LL,2}$ = kips / ft

CONCENTRATED LOADS P_{DL} = kips
P_{LL} = kips

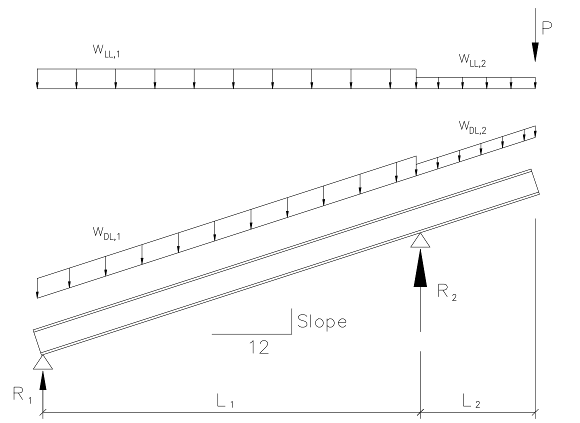
BEAM SPAN LENGTH L_1 = **20** ft

CANTILEVER LENGTH L_2 = ft, (0 for no cantilever)

BEAM SLOPE **0** : **12** (θ = 0.00 °)

DEFLECTION LIMIT OF LIVE LOAD Δ_{LL} = L / **240**

BEAM YIELD STRESS F_y = **50** ksi



THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

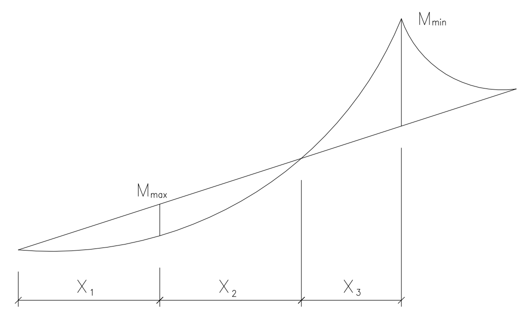
DETERMINE REACTIONS, MOMENTS & SHEARS

$$R_2 = 0.5\left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)L_1+\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)(L_1+0.5L_2)\frac{L_2}{L_1}+P\frac{L_1+L_2}{L_1}$$

= 3.60 kips

$$R_1=\left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)L_1+\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)L_2+P-R_2$$

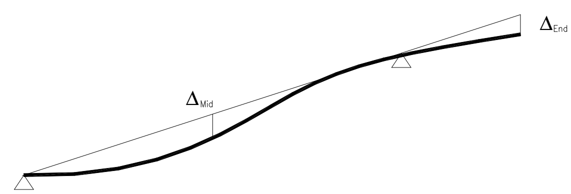
= 3.60 kips



X_1 = 10.00 ft

X_2 = 10.00 ft

X_3 = 0.00 ft



$$M_{Min}=0.5\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)L_2^2+PL_2=$$ 0.0 ft-kips

$$M_{Max}=\left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)\frac{(X_1+X_2)^2}{8}=$$ 18.0 ft-kips

V_{max} = 3.60 kips, at R1 right.

CHECK MMin BENDING CAPACITY ABOUT MAJOR AXIS (AISC 360-05 Chapter F)

$$l = Max(L_2\ ,\ X_3) =$$ 0.00 ft, unbraced length

Required Conditions	Chapter F Sections for WF				Tube	WT
	F2	F3	F4	F5	F7	F9
Double Symmetric	x	x			x	
Compact Web	x	x	x			
Noncompact Web			x			x
Slender Web						x
Compact Flanges	x					
Noncompact Flanges						x
Slender Flanges						x
Applicable Section	**ok**					

$M_{allowable} = M_n\ /\ \Omega_b$ = **82.8** ft-kips

> M_{Min} **[Satisfactory]**

where

Ω_b = 1.67 , (AISC 360-05 F1)

CHECK MMax BENDING CAPACITY ABOUT MAJOR AXIS (AISC 360-05 Chapter F)

$M_{allowable} = M_n\ /\ \Omega_b$ = **82.8** ft-kips, top flange fully supported

> M_{Max} **[Satisfactory]**

(cont'd)

CHECK SHEAR CAPACITY ABOUT MAJOR AXIS (AISC 360-05 Chapter G2 or G5)

$V_{allowable} = V_n \ / \ \Omega_V =$ **56.6** kips

$>$ V_{Max} **[Satisfactory]**

where $\Omega_V =$ 1.67 , (AISC 360-05 G1)

DETERMINE CAMBER AT DEAD LOAD CONDITION

$L = L_1 / \cos\ \theta =$	20.00	ft, beam sloped span
$a = L_2 / \cos\ \theta =$	0.00	ft, beam sloped cantilever length
$P = P_{DL}\ \cos\ \theta =$	0.00	kips, perpendicular to beam
$w_1 = w_{DL,1}\ \cos\ \theta =$	0.20	klf, perpendicular to beam
$w_2 = w_{DL,2}\ \cos\ \theta =$	0.00	klf, perpendicular to beam

$$\Delta_{End} = \frac{Pa^2(L+a)}{3EI} - \frac{w_1 L^3 a}{24EI} + \frac{w_2 a^3(4L+3a)}{24EI} =$$ 0.00 in, downward perpendicular to beam.

USE C = 0/4" AT CANTILEVER.

$$\Delta_{Mid} = -\frac{PaL^2}{16EI} + \frac{5w_1 L^4}{384EI} - \frac{w_2 L^2 a^2}{32EI} =$$ 0.12 in, downward perpendicular at middle of beam.

USE C = 0/4" AT MID BEAM.

CHECK DEFLECTION AT LIVE LOAD CONDITION

$P = P_{LL}\ \cos\ \theta =$	0.00	kips, perpendicular to beam
$w_1 = w_{LL,1}\ \cos^2\ \theta =$	0.16	klf, perpendicular to beam
$w_2 = w_{LL,2}\ \cos^2\ \theta =$	0.00	klf, perpendicular to beam

$$\Delta_{End} = \left[\frac{Pa^2(L+a)}{3EI} - \frac{w_1 L^3 a}{24EI} + \frac{w_2 a^3(4L+3a)}{24EI}\right]\cos\theta =$$ 0.00 in, downward to vertical direction.

$<$ $2L_2 \ / \ 240 =$ 0.00 in **[Satisfactory]**

$$\Delta_{Mid} = \left[-\frac{PaL^2}{16EI} + \frac{5w_1 L^4}{384EI} - \frac{w_2 L^2 a^2}{32EI}\right]\cos\theta =$$ 0.10 in, downward to vertical direction.

$<$ $L_1 \ /$ 240 = 1.00 in **[Satisfactory]**

Steel Beam Design with Gravity Loading Based on AISC-ASD 9th

INPUT DATA & DESIGN SUMMARY

BEAM SECTION => **W18X40** =>

I_x	S_x	r_T	b_f	t_f	t_w
612	68.4	1.52	6.02	0.53	0.32

SLOPED DEAD LOADS $w_{DL,1}$ = **1.15** kips / ft
$w_{DL,2}$ = **0.5** kips / ft

PROJECTED LIVE LOADS $w_{LL,1}$ = **0.8** kips / ft
$w_{LL,2}$ = **0.5** kips / ft

CONCENTRATED LOADS P_{DL} = **3** kips
P_{LL} = **3** kips

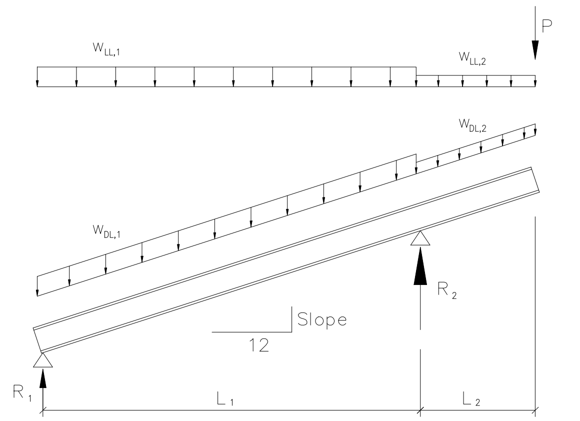
BEAM SPAN LENGTH L_1 = **30** ft

CANTILEVER LENGTH L_2 = **10** ft, (0 for no cantilever)

BEAM SLOPE **4** : 12 (θ = 18.43 ^{0})

DEFLECTION LIMIT OF LIVE LOAD Δ_{LL} = L / **360**

BEAM YIELD STRESS F_y = **50** ksi



THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

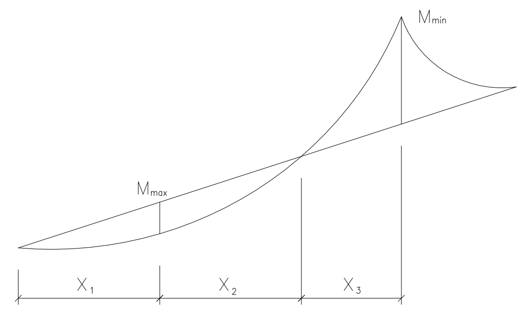
DETERMINE REACTIONS, MOMENTS & SHEARS

$$R_2 = 0.5\left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)L_1+\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)(L_1+0.5L_2)\frac{L_2}{L_1}+P\frac{L_1+L_2}{L_1}$$

= 50.17 kips

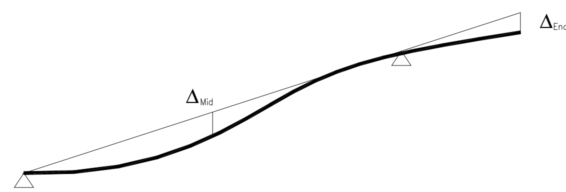
$$R_1 = \left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)L_1+\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)L_2+P-R_2$$

= 26.47 kips



X_1 = 13.16 ft
X_2 = 13.16 ft
X_3 = 3.69 ft

$$M_{Min} = 0.5\left(\frac{w_{DL,2}}{\cos\theta}+w_{LL,2}\right)L_2^2+PL_2 =$$ 111.4 ft-kips



$$M_{Max} = \left(\frac{w_{DL,1}}{\cos\theta}+w_{LL,1}\right)\frac{(X_1+X_2)^2}{8} =$$ 174.1 ft-kips

V_{max} = 39.89 kips, at R2 left.

CHECK M_{Min} BENDING CAPACITY (AISC-ASD, F1.3, page 5-46)

$$l = Max(L_2\ ,\ X_3) =$$ 10.00 ft, unbraced length

$$C_b = 1.75+1.05\left(\frac{M_1}{M_2}\right)+0.3\left(\frac{M_1}{M_2}\right)^2 =$$ 1.75 , since M_1 is 0

r_T = 1.52 in , A_f = 3.16 in^2

$$L_c = MIN\left(\frac{76b_f}{\sqrt{F_y}}\ ,\ \frac{20000}{(d/A_f)F_y}\right) =$$ 5.39 ft

$$L_u = MAX\left(r_T\sqrt{\frac{102000C_b}{F_y}}\ ,\ \frac{12000C_b}{0.6(d/A_f)F_y}\right) =$$ 10.30 ft

$$L_3 = r_T\sqrt{\frac{510000C_b}{F_y}} =$$ 16.92 ft

(cont'd)

$$F_{b1} = MIN\left(\left[\frac{2}{3} - \frac{F_y(l/r_T)^2}{1530000C_b}\right]F_y \quad , \quad 0.6F_y\right) = \quad 27.5 \text{ ksi}$$

$$F_{b2} = MIN\left(\frac{170000C_b}{(l/r_T)^2} \quad , \quad \frac{F_y}{3}\right) = \quad 16.7 \text{ ksi}$$

$$F_{b3} = MIN\left(\frac{12000C_b}{l(d/A_f)} \quad , \quad 0.6F_y\right) = \quad 30.0 \text{ ksi}$$

$$F_b = \begin{cases} 0.66F_y \quad , \quad for \quad l \le L_c \\ 0.6F_y \quad , \quad for \quad L_c < l < L_u \\ MAX\left(F_{b1} \quad , \quad F_{b3}\right) \quad , \quad for \quad L_u \le l < L_3 \\ MAX\left(F_{b2} \quad , \quad F_{b3}\right) \quad , \quad for \quad l \ge L_3 \end{cases} = \quad 30.0 \text{ ksi}$$

$f_b = M_{Min} / S_x$ = 19.5 ksi **<** ***F_b*** **[Satisfactory]**

CHECK LOCAL BUCKLING (AISC-ASD Tab. B5.1)

$b_f / (2t_f)$ = 5.73 **<** $65 / (F_y)^{0.5}$ = 9.19

[Satisfactory]

d / t_w = 56.83 **<** $640 / (F_y)^{0.5}$ = 90.51

[Satisfactory]

CHECK ***M***$_{Max}$ BENDING CAPACITY (AISC-ASD, F1.3, page 5-46)

$f_b = M_{Max} / S_x$ = 30.5 ksi **<** ***$F_b = 0.66 F_y$ =*** 33.0 ksi **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC-ASD, F4, page 5-49)

$f_v = V_{Max} / t_w d$ = 7.1 ksi **<** ***$F_v = 0.4 F_y$ =*** 20.0 ksi **[Satisfactory]**

DETERMINE CAMBER AT DEAD LOAD CONDITION

$L = L_1 / \cos\theta$ = 31.62 ft, beam sloped span

$a = L_2 / \cos\theta$ = 10.54 ft, beam sloped cantilever length

$P = P_{DL} \cos\theta$ = 2.85 kips, perpendicular to beam

$w_1 = w_{DL,1} \cos\theta$ = 1.09 klf, perpendicular to beam

$w_2 = w_{DL,2} \cos\theta$ = 0.47 klf, perpendicular to beam

$$\Delta_{End} = \frac{Pa^2(L+a)}{3EI} - \frac{w_1 L^3 a}{24EI} + \frac{w_2 a^3(4L+3a)}{24EI} = \quad -0.69$$ in, uplift perpendicular to beam.

<u>USE C = 3/4" AT CANTILEVER.</u>

$$\Delta_{Mid} = -\frac{PaL^2}{16EI} + \frac{5w_1L^4}{384EI} - \frac{w_2L^2a^2}{32EI} = \quad 1.36$$ in, downward perpendicular at middle of beam.

<u>USE C = 5/4" AT MID BEAM.</u>

CHECK DEFLECTION AT LIVE LOAD CONDITION

$P = P_{LL} \cos\theta$ = 2.85 kips, perpendicular to beam

$w_1 = w_{LL,1} \cos^2\theta$ = 0.72 klf, perpendicular to beam

$w_2 = w_{LL,2} \cos^2\theta$ = 0.45 klf, perpendicular to beam

$$\Delta_{End} = \left[\frac{Pa^2(L+a)}{3EI} - \frac{w_1 L^3 a}{24EI} + \frac{w_2 a^3(4L+3a)}{24EI}\right]\cos\theta = \quad -0.19$$ in, uplift to vertical direction.

< ***$2L_2$ / 360 =*** 0.67 in **[Satisfactory]**

$$\Delta_{Mid} = \left[-\frac{PaL^2}{16EI} + \frac{5w_1L^4}{384EI} - \frac{w_2L^2a^2}{32EI}\right]\cos\theta = \quad 0.84$$ in, downward to vertical direction.

< ***L_1 /*** 360 = 1.00 in **[Satisfactory]**

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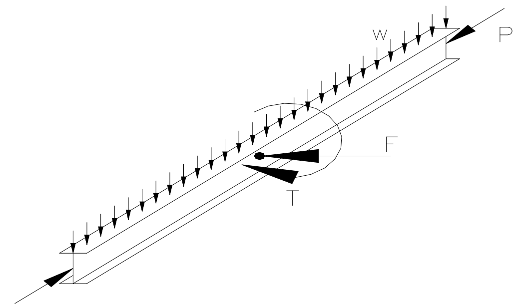
WF Simply Supported Beam Design with Torsional Loading Based on AISC 13th (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

BEAM SECTION	=>	W10X54	
GRAVITY DISTRIBUTED LOAD	w =	1.15	kips / ft
LATERAL POINT LOAD AT MID	F =	5	kips
TORSION AT MID SPAN	T =	5.1	ft-kips
AXIAL LOAD	P =	96	kips
BEAM LENGTH	L =	15	ft
BEAM YIELD STRESS	F_y =	50	ksi
VERTICAL BENDING UNBRACED LENGTH	L_b =	15	ft
AXIAL VERTICAL UNBRACED LENGTH	L_x =	15	ft
AXIAL HORIZONTAL UNBRACED LENGTH	L_y =	7.5	ft

=>

A	d	r_x	r_y	I_x	S_x
15.8	10.1	4.38	2.55	303	60
I_y	S_y	λ	t_w	b_f	t_f
103	20.6	0.0174	0.37	10.00	0.62



ANALYSIS

DETERMINE GOVERNING MOMENTS AT MIDDLE OF SPAN

$M_x = w\,L^2 / 8$ = 32.3 ft-kips

$M_y = F\,L\ / 4$ = 18.8 ft-kips

$M_0 = T\,L\ / (4d)$ = 22.7 ft-kips

$M_T = \beta M_0$ = 13.3 ft-kips

THE BEAM DESIGN IS ADEQUATE.

$$\beta = \frac{4\left(\sinh\frac{\lambda L}{2}\right)^2}{\lambda L \sinh \lambda L} =$$ 0.584 ,(Philip page 101)

CHECK TORSIONAL CAPACITY (AISC 360-05 H3.3 & Philip page 100)

f_{bx} / F_{nx} = 0.73 < 1.00 **[Satisfactory]**

Where ***$f_{bx} = M_x / S_x + 2M_T / S_y$ =*** 21.93 ksi

$F_{nx} = F_y / \Omega_T = F_y / 1.67$ = 29.94 ksi

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) \ , \ for \ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) \ , \ for \ \frac{P_r}{P_c}< 0.2 \end{cases}$$ = 1.08 < 4/3 **[Satisfactory]**

Where ***P_r =*** 96 kips

$M_{rx} = (M_x / S_x + 2M_T / S_y) S_x$ = 109.7 ft-kips, (Sim. from Philip page 100)

M_{ry} = 18.8 ft-kips

$P_c = P_n / \Omega_c$ = 721 / 1.67 = 431.97 kips, (AISC 360-05 Chapter E)

> *3/4 P_r* [Satisfactory]

$M_{cx} = M_n / \Omega_b$ = 252.62 / 1.67 = 151.27 ft-kips, (AISC 360-05 Chapter F)

> *M_{rx}* [Satisfactory]

$M_{cy} = M_n / \Omega_b$ = 130.42 / 1.67 = 78.094 ft-kips, (AISC 360-05 Chapter F)

> *3/4 M_{ry}* [Satisfactory]

DETERMINE DEFLECTIONS

$$\phi = \frac{T}{2GJ\lambda}\left(\frac{\lambda L}{2} - \frac{2\sinh\frac{\lambda L}{2}}{\sinh(\lambda L)}\right)\sinh\frac{\lambda L}{2} =$$ 0.2213 °, max twist angle at middle (Philip page 100)

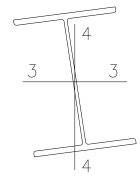
Where G = 11200 ksi E_s = 29000 ksi

J = 1.82 in^4

(cont'd)

$$\Delta vert = \frac{5wL^4}{384EI_3} =$$ 0.15 in = L / 1207 , vertical deflection at middle

Where $I_3 = I_x \sin^2(90-\phi) + I_y \cos^2(90-\phi) =$ 303 in^4 , (AISC 13th Page 17-42)



$$\Delta horiz = \frac{FL^3}{48EI_4} =$$ 0.20 in = L / 885 , horizontal deflection at middle

Where $I_4 = I_x \cos^2(90-\phi) + I_y \sin^2(90-\phi) =$ 103 in^4 , (AISC 13th Page 17-42)

Technical References:

1. AISC: "Steel Construction Manual 13th Edition", American Institute of Steel Construction, 2005.
2. Philip H. Lin: "Simplified Design for Torsional Loading of Rolled Steel Members", Engineering Journal, AISC, 1977.

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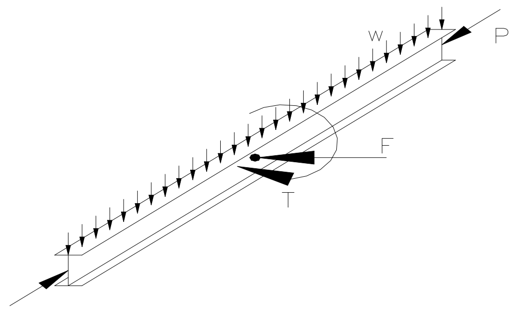
WF Simply Supported Beam Design with Torsional Loading Based on AISC Manual 9th

INPUT DATA & DESIGN SUMMARY

BEAM SECTION	= >	W10X54	
GRAVITY DISTRIBUTED LOAD	w =	1.15	kips / ft
LATERAL POINT LOAD AT MID	F =	5	kips
TORSION AT MID SPAN	T =	5.1	ft-kips
AXIAL LOAD	P =	96	kips
BEAM LENGTH	L =	15	ft
BEAM YIELD STRESS	F_y =	50	ksi
VERTICAL BENDING UNBRACED LENGTH	L_b =	15	ft
AXIAL VERTICAL UNBRACED LENGTH	L_x =	15	ft
AXIAL HORIZONTAL UNBRACED LENGTH	L_y =	7.5	ft

= >

A	d	r_x	r_y	I_x	S_x
15.8	10.1	4.38	2.55	303	60
I_y	S_y	λ	t_w	b_f	t_f
103	20.6	0.0174	0.37	10.00	0.62



ANALYSIS

CHECK LOCAL BUCKLING (AISC-ASD Tab. B5.1)

$b_f / (2t_f) =$ 8.13 **<** $65 / (F_y)^{0.5} =$ 9.19
[Satisfactory]

$d / t_w =$ 27.30 **<** $640 / (F_y)^{0.5} =$ 90.51
[Satisfactory]

THE BEAM DESIGN IS ADEQUATE.

DETERMINE GOVERNING MOMENTS AT MIDDLE OF SPAN

$M_x = w L^2 / 8 =$ 32.3 ft-kips
$M_y = F L / 4 =$ 18.8 ft-kips
$M_0 = T L / (4d) =$ 22.7 ft-kips
$M_T = \beta M_0 =$ 13.3 ft-kips

$$\beta = \frac{4\left(\sinh\frac{\lambda L}{2}\right)^2}{\lambda L \sinh \lambda L} = 0.584$$,(Philip page 101)

DETERMINE GOVERNING UNBALANCED SEGMENT LENGTH (AISC-ASD F1)

$L_c = \mathrm{MIN}[76b_f/(F_y)^{0.5}, 20000/(d/A_f)F_y] =$ 8.96 ft
$L_u = \mathrm{MAX}[r_T(102000C_b/F_y)^{0.5}, 12000C_b/(d/A_f)0.6F_y] =$ 20.30 ft
$L_3 = r_T(510000C_b/F_y)^{0.5} =$ 22.39 ft

Where $(d/A_f) =$ 1.64 in^{-1}
$r_T =$ 2.66
$C_b =$ 1.00

DETERMINE ALLOWABLE BENDING STRESSES (AISC-ASD F1)

$F_{bx} = \{$

=	$0.66F_y$	=	N/A	ksi, for L_b @ [0, L_c]
=	$0.60F_y$	=	30.00	ksi, for L_b @ (L_c, L_u]
=	MAX(F_{b1}, F_{b3})	=	N/A	ksi, for L_b @ (L_u, L_3]
=	MAX(F_{b2}, F_{b3})	=	N/A	ksi, for L_b @ (L_3, Larger)

Where $F_{b1} = \mathrm{MIN}\{[2/3 - F_y(L/r_T)^2/(1530000C_b)]F_y, 0.6F_y\} =$ 25.85 ksi
$F_{b2} = \mathrm{MIN}[170000C_b/(L/r_T)^2, F_y/3] =$ 16.67 ksi
$F_{b3} = \mathrm{MIN}[12000C_b/(L\,d/A_f), 0.6F_y] =$ 30.00 ksi

CHECK VERTICAL FLEXURAL CAPACITY (AISC-ASD F & Philip page 100)

$f_{bx} / F_{bx} =$ 0.73 **<** 1.00 **[Satisfactory]**
Where $f_{bx} = M_x / S_x + 2M_T / S_y =$ 21.93 ksi

(cont'd)

CHECK COMPRESSION CAPACITY (AISC-ASD E2)

$f_a / F_a =$ 0.24 < 1.33 **[Satisfactory]**

Where $f_a = P / A =$ 6.08 ksi

$K =$ 1.0

$E_s =$ 29000 ksi

$C_c = (2\pi^2 E_s/F_y)^{0.5} =$ 107

$KL/r = MAX(KL_x/r_x, KL_y/r_y) =$ 41.10 < 200 **[Satisfactory]**

$F = (KL/r) / C_c =$ 0.38

$F_a = \begin{cases} (1-F^2/2)F_y / (5/3+3F/8-F^3/8) = 25.68 \text{ ksi, for } C_c > (Kl/r) \\ 12\pi^2 E_s/[23(KL/r)^2] = \text{N/A ksi, for } C_c < (Kl/r) \end{cases}$

CHECK COMBINED STRESS (AISC-ASD H1)

$f_a / F_a =$ 0.24 > 0.15

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1-\frac{f_a}{F'_{ex}}\right)F_{bx}} + \frac{C_{my} f_{by}}{\left(1-\frac{f_a}{F'_{ey}}\right)F_{by}} = \quad 1.33 \quad < \quad 1.33$$

Where $C_m =$ 1.00

$f_{by} = M_y / S_y =$ 10.92 ksi

$F_{by} = 0.75 F_y =$ 37.50 ksi

$$F'_{ex} = \frac{12\pi^2 E}{23\left(\frac{K l_x}{r_x}\right)^2} = 88.39 \text{ ksi} \qquad F'_{ey} = \frac{12\pi^2 E}{23\left(\frac{K l_y}{r_y}\right)^2} = 120.18 \text{ ksi}$$

$$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} = \quad 1.22 \quad < \quad 1.33$$

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} = \quad 1.26 \quad < \quad 1.33$$

<== Not applicable.

[Satisfactory]

DETERMINE DEFLECTIONS

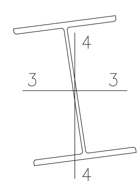
$$\phi = \frac{T}{2GJ\lambda}\left(\frac{\lambda L}{2} - \frac{2\sinh\frac{\lambda L}{2}}{\sinh(\lambda L)}\right)\sinh\frac{\lambda L}{2} =$$ 0.2213 °, max twist angle at middle (Philip page 100)

Where $G =$ 11200 ksi

$J =$ 1.82 in^4

$$\Delta vert = \frac{5wL^4}{384EI_3} =$$ 0.15 in = L / 1207 , vertical deflection at middle

Where $I_3 = I_x \sin^2(90-\phi) + I_y \cos^2(90-\phi) =$ 303 in^4 , (AISC-ASD Page 6-23)



$$\Delta horiz = \frac{FL^3}{48EI_4} =$$ 0.20 in = L / 885 , horizontal deflection at middle

Where $I_4 = I_x \cos^2(90-\phi) + I_y \sin^2(90-\phi) =$ 103 in^4 , (AISC-ASD Page 6-23)

Technical References:
1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. Philip H. Lin: "Simplified Design for Torsional Loading of Rolled Steel Members", Engineering Journal, AISC, 1977.

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HSS (Tube, Pipe) Member Design with Torsional Loading Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY

MEMBER SHAPE (Tube or Pipe) & SIZE		HSS12X12X5/8	< == **Tube**
STEEL YIELD STRESS	F_y =	46	ksi
TORSIONAL FORCE	T_r =	63	ft-kips, ASD
AXIAL COMPRESSION FORCE	P_r =	46	kips, ASD
STRONG AXIS EFFECTIVE LENGTH	kL_x =	20	ft
WEAK AXIS EFFECTIVE LENGTH	kL_y =	20	ft
STRONG AXIS BENDING MOMENT	M_{rx} =	250	ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH	L_b =	20	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	13	kips
WEAK AXIS BENDING MOMENT	M_{ry} =	30	ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	10	kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK TORSIONAL CAPACITY (AISC 360-05, H3.1)

$$T_c=\frac{1}{\Omega_T}T_n=\frac{1}{\Omega_T}\begin{cases}\left[2(B-t)(H-t)t-4.5(4-\pi)t^3\right]\begin{bmatrix}0.6F_y\ ,\ for\ \frac{h}{t}\le 2.45\sqrt{\frac{E}{F_y}}\\ 0.6F_y 2.45\sqrt{\frac{E}{F_y}}\frac{t}{h}\ ,\ for\ \frac{h}{t}\le 3.07\sqrt{\frac{E}{F_y}}\\ 0.458\frac{E\pi^2}{(h/t)^2},\ for\ \frac{h}{t}\le 260\end{bmatrix},\ for\ HSS\ Tube\\ \frac{\pi(D-t)^2 t}{2}Max\left[\frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{(5/4)}}\ ,\ \frac{0.60E}{\left(\frac{D}{t}\right)^{(3/2)}}\right],\ for\ HSS\ Pipe\end{cases} = 221.5 \text{ ft-kips}$$

> T_r **[Satisfactory]**

Where	B	H	h	t	D	E
	12.00	12.00	10.13	0.63		29000

Ω_T = 1.67 ,ASD

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases}\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)\ ,\ for\ \frac{P_r}{P_c}\ge 0.2\\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)\ ,\ for\ \frac{P_r}{P_c}<0.2\end{cases} = 1.16 < 1.3$$

[Satisfactory]
(IBC 06 / CBC 07, 1605.3.2)

Where $P_c = P_n / \Omega_c$ = 986 / 1.67 = 590.28 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 417.83 / 1.67 = 250.20 ft-kips, (AISC 360-05 Chapter F)
> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 417.83 / 1.67 = 250.20 ft-kips, (AISC 360-05 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong} / \Omega_v$ = 276.0 / 1.67 = 165.3 kips > V_{strong} = 13.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 276.0 / 1.67 = 165.3 kips > V_{weak} = 10.0 kips **[Satisfactory]**

CHECK COMBINED TORSION, SHEAR, COMPRESSION, AND BENDING CAPACITY (AISC 360-05, H3.2)

$$\begin{cases}\frac{P_r}{P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)+\left[Max\left(\frac{V_{strong}}{V_{c,strong}}\ ,\ \frac{V_{weak}}{V_{c,weak}}\right)+\frac{T_r}{T_c}\right]^2\ ,\ for\ \frac{T_r}{T_c}>0.2\\ Torsion\ \ Neglected\ ,\ for\ \frac{T_r}{T_c}\le 0.2\end{cases} = 1.3 < 1.3$$

[Satisfactory]
(IBC 06 / CBC 07, 1605.3.2)

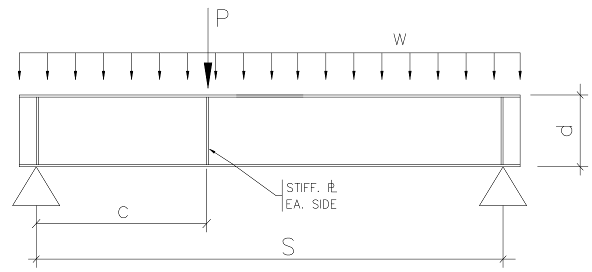
Daniel T. Li
Engineering International

PROJECT :
CLIENT :
JOB NO. : DATE :
PAGE :
DESIGN BY :
REVIEW BY :

Plate Girder Design Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	50	ksi
SIMPLY SUPPORTED SPAN	S =	72	ft
SUPERIMPOSED UNIFORM DEAD LOAD	DL =	1	kips / ft
UNIFORM LIVE LOAD	LL =	1	kips / ft
POINT DEAD LOAD	P_{DL} =	60	kips
POINT LIVE LOAD	P_{LL} =	60	kips
DISTANCE POINT LOAD TO END	c =	24	ft
TOP FLANGE WIDTH	$b_{f,top}$ =	16	in
TOP FLANGE THICKNESS	$t_{f,top}$ =	1.5	in
BOTTOM FLANGE WIDTH	$b_{f,bot}$ =	16	in
BOTTOM FLANGE THICKNESS	$t_{f,bot}$ =	1.5	in
WEB THICKNESS	t_w =	0.5	in
BEAM DEPTH	d =	53	in
UNBRACED LENGTH	L_b =	8	ft



1/4 x 7 STIFFENER, E. SIDES, AT SUPPORTS ONLY.

FLANGE TO WEB WELDING USE 5/16 in - 24 in @ 41 in o.c.

THE GIRDER DESIGN IS ADEQUATE.

ANALYSIS

CHECK LIMITING WIDTH-THICKNESS RATIOS FOR WEB (AISC 360-05 Table B4.1)

h_c / t_w = 100.00 < λ_r = 137.27

\> λ_p = 90.55

Noncompact Web

where E = 29000 ksi

$\lambda_r = 5.7\,(E / F_y)^{0.5}$ = 137.27

$\lambda_p = (h_c / h_p)\,(E / F_y)^{0.5} / (0.54\,M_p / M_y - 0.09)^2$ = 93.26 ,for $A_{f,top} \neq A_{f,bot}$

$\lambda_p = 3.76\,(E / F_y)^{0.5}$ = 90.55 ,for $A_{f,top} = A_{f,bot}$

h_c = 50.00 in h_p = 50.00 in

M_p = 6452.1 ft-kips M_y = 5824.6 ft-kips

CHECK LIMITING WIDTH-THICKNESS RATIOS FOR FLANGES (AISC 360-05 Table B4.1)

$0.5\,b_{f,top} / t_{f,top}$ = 5.33 < λ_r = 17.04

< λ_p = 9.15

Compact Flanges

where $\lambda_r = 0.95\,(k_c\,E / F_L)^{0.5}$ = 17.04

$\lambda_p = 0.38\,(E / F_y)^{0.5}$ = 9.15

$k_c = Min\,[0.76\,,\,Max\,(0.35\,,\,4 / (h / t_w)^{0.5}\,)]$ = 0.39

S_{xt} = 1398 in^3 S_{xc} = 1398 in^3

F_L = 35 ksi, (AISC 360-05 Table note B4.1 & Eq F4-6)

DETERMINE CRITERIA FOR ALLOWABLE FLEXURAL STRENGTH (AISC 360-05 Table F1.1)

Required Conditions	Chapter F Sections F2	F3	F4	F5
Double Symmetric	x	x		
Compact Web	x	x	x	
Noncompact Web				
Slender Web				
Compact Flanges	x			
Noncompact Flanges				
Slender Flanges				
Applicable Section			**ok**	

$M_{allowable} = M_n / \Omega_b$ = **3806.0** ft-kips
(from following analysis)

DETERMINE ALLOWABLE FLEXURAL STRENGTH , Mn / Wb , BASED ON AISC 360-05 Chapter F2

<== Not Applicable.

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = \quad 13.23 \text{ ft}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_0}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{S_x h_0}{Jc}\right)^2}} \approx \pi r_{ts} \sqrt{\frac{E}{0.7F_y}} = \quad 32.74 \text{ ft}$$

where r_y = 3.75 in S_x = 1398 in^3

h_o = 51.50 in I_y = 1025 in^4

(cont'd)

$C_w = I_y\, h_0^2 / 4 =$ 679321 , (AISC 360-05 F2.2)

$J = [2\, b_f\, t_f^3 + (d - t_f)\, t_f^3] / 3 =$ 38.146 in^4, (Galambos 1968)

(Use $J =$ 4.0314 in^4)

$r_{ts} = [(I_y\, C_w)^{0.5} / S_x)]^{0.5} =$ 4.34 in

$c =$ 1.0 $C_b =$ 1.0 , (AISC Manual 13th Table 3-1, page 3-10)

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_0}\left(\frac{L_b}{r_{ts}}\right)^2} =$$ 586.73 ksi

$$M_{n,F2} = \begin{cases} M_p \ , \ for \ \ L_b \le L_p \\ Min\left(C_b\left[M_p - \left(M_p - 0.7 F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \ , \ M_p\right) \ , \ for \ \ L_p < L_b \le L_r \\ Min\left(F_{cr} S_x \ , \ M_p\right) \ , \ for \ \ L_r \le L_b \end{cases} =$$ 6452.1 ft-kips

$M_{allowable,\,F2} = M_n / \Omega_b =$ **3863.5** ft-kips where $\Omega_b =$ 1.67 , (AISC 360-05 F1)

DETERMINE ALLOWABLE FLEXURAL STRENGTH , Mn / Wb , BASED ON AISC 360-05 Chapter F3

<== Not Applicable.

$$M_{n,F3} = \begin{cases} \left[M_p - \left(M_p - 0.7 F_y S_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \ , \ for \ \ Noncompact \ \ Flanges \\ \frac{0.9 E k_c S_x}{\lambda^2} \ , \ for \ \ Slender \ \ Flanges \end{cases} =$$ 7600.9 ft-kips

where $\lambda = b_f / (2\, t_f) =$ 5.33

$\lambda_{pf} = \lambda_p =$ 9.15 $\lambda_{rf} = \lambda_r =$ 17.04

$M_{allowable,\,F3} = Min(M_{n,F2} , M_{n,F3}) / \Omega_b =$ **3863.5** ft-kips

DETERMINE ALLOWABLE FLEXURAL STRENGTH , Mn / Wb , BASED ON AISC 360-05 Chapter F4

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} =$$ 9.59 ft

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_0}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{F_L}{E}\frac{S_{xc} h_0}{J}\right)^2}} =$$ 33.05 ft

where $a_w = h_c\, t_w / (b_{fc}\, t_{fc}) =$ 1.04

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h_0}{d} + \frac{1}{6} a_w \frac{h^2}{h_0 d}\right)}} =$$ 4.34 in

$M_p = Min\,[Z_x\, F_y , 1.6 S_{xc}\, F_y] =$ 6452.1 ft-kips

$M_{yc} = S_{xc}\, F_y =$ 5824.6 ft-kips $M_{yt} = S_{xt}\, F_y =$ 5824.6 ft-kips

$\lambda = h_c / t_w =$ 100.00

$\lambda_{pw} = \lambda_p =$ 93.26 $\lambda_{rw} = \lambda_r =$ 137.27

$$R_{pc} = \begin{cases} \frac{M_p}{M_{yc}} \ , \ for \ \ h_c / t_w \le \lambda_{pw} \\ Min\left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right) \ , \ \frac{M_p}{M_{yc}}\right] \ , \ for \ \ h_c / t_w > \lambda_{pw} \end{cases} =$$ 1.0912

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_0}\left(\frac{L_b}{r_t}\right)^2} =$$ 586.58 ksi, (for $I_{yc} / I_y =$ 0.50 > 0.23, AISC 360-05 F4-5)

$$M_{n,F4.2} = \begin{cases} R_{pc} M_{yc} \ , \ for \ \ L_b \le L_p \\ Min\left(C_b\left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \ , \ R_{pc} M_{yc}\right) \ , \ for \ \ L_p < L_b \le L_r \\ Min\left(F_{cr} S_{xc} \ , \ R_{pc} M_{yc}\right) \ , \ for \ \ L_r \le L_b \end{cases} =$$ 6355.97 ft-kips

(cont'd)

$$M_{n,F5.3}=\begin{cases}R_{pc}M_{yc}\ , & for\ \ Compact\ \ Flanges\\ \left[R_{pc}M_{yc}-\left(R_{pc}M_{yc}-F_{L}S_{xc}\right)\left(\frac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)\right]\ , & for\ \ Noncompact\ \ Flanges\\ \frac{0.9Ek_cS_{xc}}{\lambda^2}\ , & for\ \ Slender\ \ Flanges\end{cases}\quad = \quad 6356 \text{ ft-kips}$$

$$R_{pt}=\begin{cases}\frac{M_p}{M_{yt}}\ , & for\ \ h_c/t_w\le\lambda_{pw}\\ Min\left[\frac{M_p}{M_{yt}}-\left(\frac{M_p}{M_{yt}}-1\right)\left(\frac{\lambda-\lambda_{pw}}{\lambda_{rw}-\lambda_{pw}}\right)\ ,\ \frac{M_p}{M_{yt}}\right]\ , & for\ \ h_c/t_w>\lambda_{pw}\end{cases}\quad = \quad 1.0912$$

$M_{allowable,\ F4}$ = Min($M_{n,F4.2}$, $M_{n,F4.3}$, $R_{pt}M_{yt}$) / Ω_b = **3806.0** ft-kips

DETERMINE ALLOWABLE FLEXURAL STRENGTH , Mn / Wb , BASED ON AISC 360-05 Chapter F5

<== Not Applicable.

$$L_r=\pi r_t\sqrt{\frac{E}{0.7F_y}}= \quad 32.73 \text{ ft}$$

$$F_{cr,F5.2}=\begin{cases}F_y\ , & for\ \ L_b\le L_p\\ Min\left(C_b\left[F_y-(0.3F_y)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]\ ,\ F_y\right)\ , & for\ \ L_p<L_b\le L_r\\ Min\left(\frac{C_b\pi^2E}{\left(\frac{L_b}{r_t}\right)^2}\ ,\ F_y\right)\ , & for\ \ L_r\le L_b\end{cases}\quad = \quad 50 \text{ ksi}$$

$$F_{cr,F5.3}=\begin{cases}F_y\ , & for\ \ Compact\ \ Flanges\\ \left[F_y-(0.3F_y)\left(\frac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)\right]\ , & for\ \ Noncompact\ \ Flanges\\ \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2}\ , & for\ \ Slender\ \ Flanges\end{cases}\quad = \quad 50 \text{ ksi}$$

$$R_{pg}=Min\left(1-\frac{Min(a_w\ ,\ 10)}{1200+300Min(a_w\ ,\ 10)}\left(\frac{h_c}{t_w}-5.7\sqrt{\frac{E}{F_y}}\right)\ ,\ 1.0\right)= \quad 1$$

$M_{allowable,\ F5}$ = Min($R_{pg}F_yS_{xc}$, $R_{pg}F_{cr,F5.2}S_{xc}$, $R_{pg}F_{cr,F5.3}S_{xc}$, F_yS_{xt}) / Ω_b = **3487.8** ft-kips

DETERMINE ALLOWABLE SHEAR STRENGTH, Vn / Wv , BASED ON AISC 360-05 Chapter G2

$h = d - t_{f,top} - t_{f,bot}$ = 50 in , h/t_w = 100 , **A_w =** 26.50 in^2 ,

a = 71.4 ft

$$k_v=\begin{cases}5+\frac{5}{(a/h)^2}\ , & for\ \ a/h\le 3\\ 5\ , & for\ \ a/h>3\end{cases}\quad = \quad 5.00$$

$$C_v=\begin{cases}1.0\ , & for\ \ h/t_w\le 1.10\sqrt{\frac{k_vE}{F_y}}\\ \frac{1.10}{h/t_w}\sqrt{\frac{k_vE}{F_y}}\ , & for\ \ 1.10\sqrt{\frac{k_vE}{F_y}}<h/t_w\le 1.37\sqrt{\frac{k_vE}{F_y}}\\ \frac{1.51E\ \ k_v}{(h/t_w)^2F_y}\ , & for\ \ 1.37\sqrt{\frac{k_vE}{F_y}}<h/t_w\end{cases}\quad = \quad 0.438$$

$V_n = 0.6\,F_yA_wC_v$ = 348.13 kips

$V_{allowable}$ = V_n / Ω_v = **208.46** kips Ω_v = 1.67 , (AISC 360-05 G1)

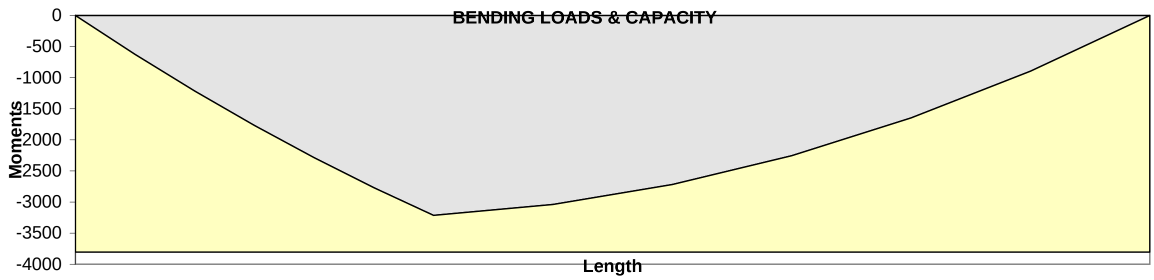
TOTAL SUPERIMPOSED GRAVITY LOAD

w = DL + LL = 2.000 kips / ft , P = P_{DL} + P_{LL} = 120.00 kips

CHECK EACH SECTION CAPACITIES

Section	Left	0.06 S	0.11 S	0.17 S	0.22 S	0.28 S	Point	0.44 S	0.56 S	0.67 S	0.78 S	0.89 S	Right
Distance	0	4.00	8.00	12.00	16.00	20.00	24.00	32.00	40.00	48.00	56.00	64.00	72.00
d (in)	53	53	53	53	53	53	53	53	53	53	53	53	53
y (in)	27	27	27	27	27	27	27	27	27	27	27	27	27
I (in^4)	37044	37044	37044	37044	37044	37044	37044	37044	37044	37044	37044	37044	37044
Wt (plf)	248.4	248.4	248.4	248.4	248.4	248.4	248.4	248.4	248.4	248.4	248.4	248.4	248.4
V (kips)	160.94	151.95	142.96	133.96	124.97	115.97	0.00	31.01	48.99	66.98	84.97	102.96	120.94
M (ft-k)	0	626	1216	1769	2287	2769	3215	3039	2719	2255	1647	896	0

(cont'd)

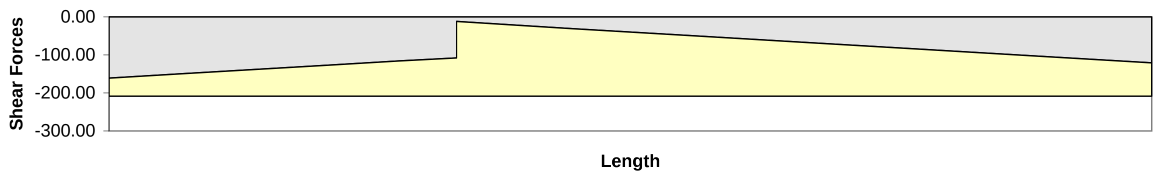


M_{max} = 3215.08 ft-kips @ 24.00 ft, from heel.

< $M_{allowable}$ = 3806 ft-kips

[Satisfactory]

SHEAR LOADS & CAPACITY



V_{max} = 160.94 kips @ 0.00 ft, from heel.

< $V_{allowable}$ **=** 208.46 kips **[Satisfactory]**

DETERMINE DEFLECTION AT MID SPAN

$$\Delta_{DL} = \frac{5wL^4}{384EI} + \frac{0.06415Pb}{EIL}\left(L^2 - b^2\right)^{3/2} =$$ 0.72 in (L / 1195)

(for camber, self Wt included.)

where	E =	29000	ksi	w =	1.248	kips / ft
	I =	37044	in^4	P =	60	kips
	b =	0.6	ft	L =	72.0	ft

$$\Delta_{LL} = \frac{5wL^4}{384EI} + \frac{0.06415Pb}{EIL}\left(L^2 - b^2\right)^{3/2} =$$ 0.58 in (L / 1482)

where P = 60 kips w = 1.000 kips / ft

DETERMINE FLANGE TO WEB WELDING (AISC 360-05 J2.4)

***w* =** 5/16 in

w_{min} = 3/16 in, < ***w***

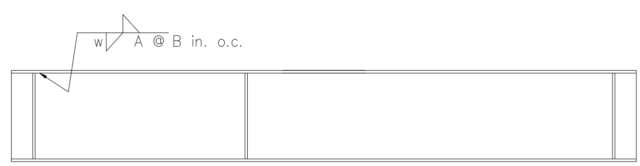
w_{max} = 7/16 in, > ***w***

Ω = 2.0

V_{max} = 160.94 kips

$Q = A_f(d - y - 0.5\ t_{f,top})$ = 618 in^3

$v_{max} = V_{max}\ Q / I$ = 2.68 kips / in



A = 24 in

$$B = \frac{(0.6F_{EXX})(0.707w)A}{v_{max}\ \ \Omega} =$$ 41 in. o.c.

USE 5/16 in - 24 in @ 41 in o.c.

DESIGN STIFFENERS

1. ***BEARING STIFFENERS ARE REQUIRED AT EACH END SUPPORT.*** (AISC 360-05, J10.8)

2. CHECK LOCAL WEB YIELDING FOR THE CONCENTRATED LOAD. (AISC 360-05, J10.2)

$R = P$ = 120.00 kips

N = 0 in, bearing length, point.

$k = t_{f,top} + w$ = 1.81 in

Ω = 1.5

$$\begin{cases} \dfrac{R}{t_w(N+5k)}\ , & for\ \ c > d \\ \dfrac{R}{t_w(N+2.5k)}\ , & for\ \ c \le d \end{cases} =$$ 26.48 **<** F_y / Ω **[Satisfactory]**

3. CHECK WEB CRIPPLING FOR THE CONCENTRATED LOAD. (AISC 360-05, J10.3)

Ω = 2.0

$$R_n / \Omega = 1/\Omega \begin{cases} 0.80 t_w^2 \left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \ , \ for \ c \geq 0.5d \\ 0.40 t_w^2 \left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \ , \ for \ c < 0.5d \end{cases} = 208.57 \quad > \quad P$$

[Satisfactory]

(Note : If item 2, local web yielding is Satisfactory, this item does not need to be checked.)

4. CHECK SIDESWAY WEB BUCKING FOR THE CONCENTRATED LOAD. (AISC 360-05, J10.4)

d_c = d - 2k = 49.38 in

C_r = 960000 ksi

(d_c / t_w) / (l / b_f) = 16.46

Ω = 1.76

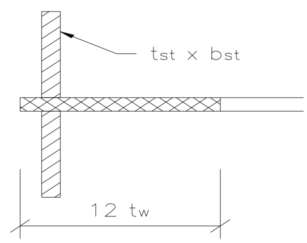
$$R_n / \Omega = 1/\Omega \begin{cases} \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \ , \ for \ \frac{d_c/t_w}{l/b_f} < 1.7 \\ \frac{C_r t_w^3 t_f}{h^2}\left[1+0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \ , \ for \ 1.7 \leq \frac{d_c/t_w}{l/b_f} < 2.3 \\ \Omega P \ , \ for \ \frac{d_c/t_w}{l/b_f} \geq 2.3 \end{cases} = 120.00 \quad > \quad P$$

[Satisfactory]

(Note : If item 2, local web yielding is Satisfactory, this item does not need to be checked.)

5. DETERMINE STIFFENER SIZE.

t_w = 5/8 in **,** ***b_{st} =*** 7 in



b_{st} / t_w = 11.20 < 0.56 (E / F_y)$^{0.5}$, (AISC 360-05 Table B4.1)
[Satisfactory]

A_g = 11.75 in^2 **,** ***I =*** 159 in^4

R = 160.9 kips

Ω_c = 1.67 , (AISC 360-05 E1)

K l / r = ***0.75 h / (I / A_{eff})$^{0.5}$ =*** 10.2

C_c = 4.71 (E / F_y)$^{0.5}$ = 113

$$F_e = \frac{\pi^2 E}{(kl/r)^2} = 2751.51 \text{ ksi}$$

$$R_n / \Omega_c = A_g / \Omega_c \begin{cases} \left[0.658^{\frac{F_y}{F_e}}\right] F_y \ , \ for \ \frac{kl}{r} \leq C_c \\ 0.877 F_e \ , \ for \ \frac{kl}{r} > C_c \end{cases} = 349.1 \text{ kips, (AISC 360-05 E2)} \quad > \quad R$$

[Satisfactory]

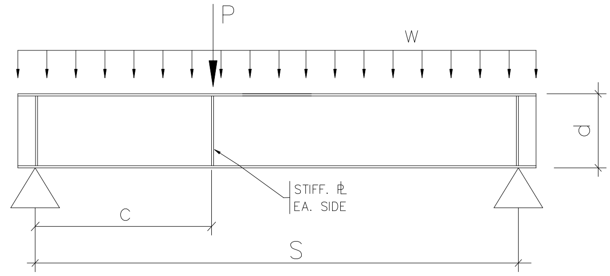
Technical Reference:

1. AISC: "Steel Construction Manual 13th Edition", American Institute of Steel Construction, 2005.

Plate Girder Design Based on AISC-ASD 9th, Chapter G

INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	50	ksi
SIMPLY SUPPORTED SPAN	S =	72	ft
SUPERIMPOSED UNIFORM DEAD LOAD	DL =	1	kips / ft
UNIFORM LIVE LOAD	LL =	1	kips / ft
POINT DEAD LOAD	P_{DL} =	60	kips
POINT LIVE LOAD	P_{LL} =	60	kips
TOP FLANGE WIDTH	$b_{f,top}$ =	16	in
TOP FLANGE THICKNESS	$t_{f,top}$ =	1.5	in
BOTTOM FLANGE WIDTH	$b_{f,bot}$ =	16	in
BOTTOM FLANGE THICKNESS	$t_{f,bot}$ =	1.5	in
WEB THICKNESS	t_w =	0.5	in
BEAM DEPTH	d =	70	in
DISTANCE POINT LOAD TO END	c =	24	ft
UNBRACED LENGTH	l =	8	ft



1/4 x 7 STIFFENER, E. SIDES, AT SUPPORTS ONLY.
FLANGE TO WEB WELDING USE 5/16 in - 24 in @ 57 in o.c.

THE GIRDER DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE F_b (AISC-ASD, F1.3, page 5-46)

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \approx 1.75 + 1.05\left(\frac{4l^2 - S^2}{S^2}\right) + 0.3\left(\frac{4l^2 - S^2}{S^2}\right)^2 = \quad 1.02$$

r_T = 4.16 in , A_f = 24.00 in^2

$$L_c = MIN\left(\frac{76 b_f}{\sqrt{F_y}} \ ,\ \frac{20000}{(d/A_f)F_y}\right) = \quad 11.43 \text{ ft}$$

$$L_u = MAX\left(r_T\sqrt{\frac{102000 C_b}{F_y}} \ ,\ \frac{12000 C_b}{0.6(d/A_f)F_y}\right) = \quad 15.84 \text{ ft}$$

$$L_3 = r_T\sqrt{\frac{510000 C_b}{F_y}} = \quad 35.42 \text{ ft}$$

$$F_{b1} = MIN\left(\left[\frac{2}{3} - \frac{F_y (l/r_T)^2}{1530000 C_b}\right]F_y \ ,\ 0.6F_y\right) = \quad 30.0 \text{ ksi}$$

$$F_{b2} = MIN\left(\frac{170000 C_b}{(l/r_T)^2} \ ,\ \frac{F_y}{3}\right) = \quad 16.7 \text{ ksi}$$

$$F_{b3} = MIN\left(\frac{12000 C_b}{l(d/A_f)} \ ,\ 0.6F_y\right) = \quad 30.0 \text{ ksi}$$

$$F_b = \begin{cases} 0.66F_y \ , & for \ \ l \le L_c \\ 0.6F_y \ , & for \ \ L_c < l < L_u \\ MAX\,(F_{b1}\ ,\ F_{b3}) \ , & for \ \ L_u \le l < L_3 \\ MAX\,(F_{b2}\ ,\ F_{b3}) \ , & for \ \ l \ge L_3 \end{cases} = \quad 33.0 \text{ ksi}$$

CHECK WEB SLENDERNESS (AISC-ASD, G1, page 5-51)

a = 71.4 ft, the max clear distance between stiffeners.

h/t_w = 134.00 **>** $\frac{760}{\sqrt{F_b}} =$ 132.30 **[Satisfactory]**

$$< \begin{cases} \dfrac{14000}{\sqrt{F_{yf}(F_{yf}+16.5)}} \ , & for \ \ a > 1.5h \\ \dfrac{2000}{\sqrt{F_{yf}}} \ , & for \ \ a \le 1.5h \end{cases} = \quad 282.84$$

[Satisfactory]

(cont'd)

DETERMINE ALLOWABLE FLEXURAL STRESS (AISC-ASD G2, pg 5-51)

$A_w =$ 33.50 in^2 , $\alpha = 0.6\,F_{yw} / F_b =$ 0.91

$$R_{PG} = MIN\left(1 - 0.0005\frac{A_w}{A_f}\left(\frac{h}{t} - \frac{760}{\sqrt{F_b}}\right)\ ,\ 1.0\right) = \quad 0.999$$

$$R_e = MIN\left[\frac{12 + \left(\frac{A_w}{A_f}\right)\left(3\alpha - \alpha^3\right)}{12 + 2\left(\frac{A_w}{A_f}\right)}\ ,\ 1.0\right] = \quad 0.998$$

$$F'_b = F_b R_{PG} R_e = \quad 32.89 \text{ ksi}$$

DETERMINE ALLOWABLE SHEAR STRESS (F4-2, pg 5-49)

$h = d - t_{f,top} - t_{f,bot} =$ 67 in , $h / t_w =$ 134

$$k_v = \begin{cases} 4.0 + \frac{5.34}{(a/h)^2}\ ,\ for\ \ a/h \le 1.0 \\ 5.34 + \frac{4.0}{(a/h)^2}\ ,\ for\ \ a/h > 1.0 \end{cases} = \quad 5.36$$

$$C_v = \begin{cases} \frac{45000 k_v}{F_y (h/t_w)^2}\ ,\ for\ \ C_V \le 0.8 \\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}}\ ,\ for\ \ C_V > 0.8 \end{cases} = \quad 0.27$$

$$F_v = MIN\left(\frac{C_v F_y}{2.89}\ ,\ 0.4 F_y\right) = \quad 4.65 \text{ ksi}$$

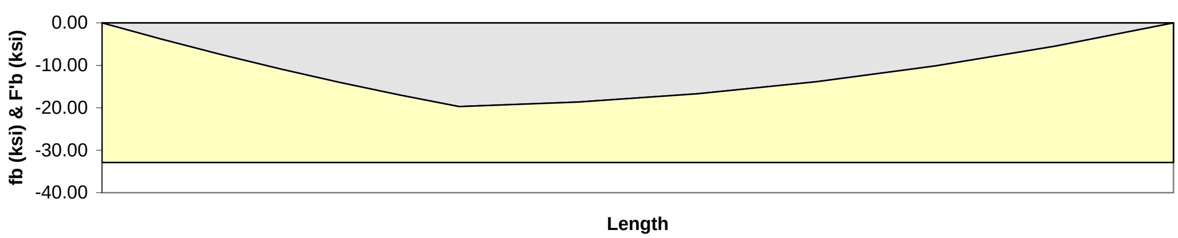
TOTAL SUPERIMPOSED GRAVITY LOAD

w = DL + LL = 2.000 kips / ft , P = $P_{DL} + P_{LL}$ = 120.00 kips

CHECK EACH SECTION CAPACITIES

Section	Left	0.06 S	0.11 S	0.17 S	0.22 S	0.28 S	Point	0.44 S	0.56 S	0.67 S	0.78 S	0.89 S	Right
Distance	0	4.00	8.00	12.00	16.00	20.00	24.00	32.00	40.00	48.00	56.00	64.00	72.00
d (in)	70	70	70	70	70	70	70	70	70	70	70	70	70
y (in)	35	35	35	35	35	35	35	35	35	35	35	35	35
I (in^4)	68848	68848	68848	68848	68848	68848	68848	68848	68848	68848	68848	68848	68848
Wt (plf)	277.3	277.3	277.3	277.3	277.3	277.3	277.3	277.3	277.3	277.3	277.3	277.3	277.3
V (kips)	161.98	152.87	143.77	134.66	125.55	116.44	0.00	30.89	49.11	67.33	85.55	103.77	121.98
M (ft-k)	0	630	1223	1780	2300	2784	3232	3057	2737	2272	1660	903	0
f_v (ksi)	4.63	4.37	4.11	3.85	3.59	3.33	0.00	0.88	1.40	1.92	2.44	2.96	3.49
f_b (ksi)	0.00	3.84	7.46	10.86	14.03	16.98	19.71	18.65	16.70	13.86	10.13	5.51	0.00

BENDING STRESS

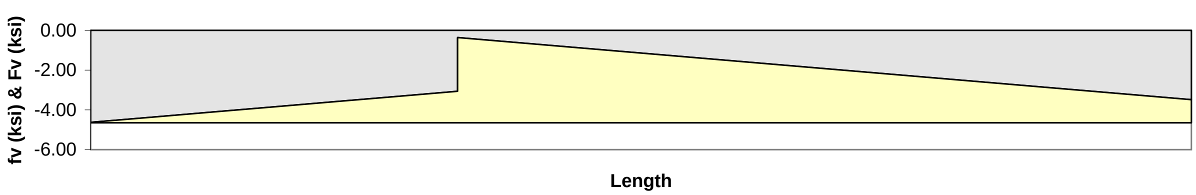


$f_{b,max}$ = 19.71 ksi @ 24.00 ft, from heel.

< F'_b = 32.89 ksi

[Satisfactory]

SHEAR STRESS



(cont'd)

$f_{v,max}$ = 4.63 ksi @ 0.00 ft, from heel.
< F_v = 4.65 ksi **[Satisfactory]**

DETERMINE DEFLECTION AT MID SPAN

$$\Delta_{DL} = \frac{5wL^4}{384EI} + \frac{0.06415Pb}{EIL}\left(L^2 - b^2\right)^{3/2} =$$ 0.40 in (L / 2173)
(for camber, self Wt included.)

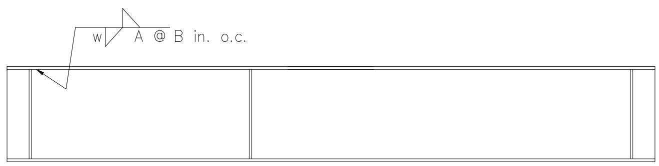
where	E =	29000	ksi	w =	1.277	kips / ft
	I =	68848	in^4	P =	60	kips
	b =	0.6	ft	L =	72.0	ft

$$\Delta_{LL} = \frac{5wL^4}{384EI} + \frac{0.06415Pb}{EIL}\left(L^2 - b^2\right)^{3/2} =$$ 0.31 in (L / 2755)

where P = 60 kips w = 1.000 kips / ft

DETERMINE FLANGE TO WEB WELDING

w **=** 5/16 in
w ***min*** **=** 3/16 in, < ***w***
w ***max*** **=** 7/16 in, > ***w***



V ***max*** **=** 161.98 kips
Q = A**_f_**(d - y - 0.5 t**_f,top_**) = 822 in^3
v$_{max}$ ***= V***$_{max}$ ***Q / I =*** 1.93 kips / in

A = 24 in

$$B = \frac{(0.3F_u)(0.707w)A}{v_{max}} =$$ 57 in. o.c.

USE 5/16 in - 24 in @ 57 in o.c.

DESIGN STIFFENERS

1. ***BEARING STIFFENERS ARE REQUIRED AT EACH END SUPPORT.*** (AISC-ASD, K1.8, page 5-82)

2. CHECK LOCAL WEB YIELDING FOR THE CONCENTRATED LOAD. (AISC-ASD, K3, page 5-81)
R = P = 120.00 kips
N = 0 in, bearing length, point.
k = t$_{f,top}$ ***+ w =*** 1.81 in

$$\begin{cases} \dfrac{R}{t_w(N+5k)} \ , \ for \ \ c>d \\ \dfrac{R}{t_w(N+2.5k)} \ , \ for \ \ c\le d \end{cases} =$$ 26.48 < $0.66F_y$ **[Satisfactory]**

3. CHECK WEB CRIPPLING FOR THE CONCENTRATED LOAD. (AISC-ASD, K4, page 5-81)

$$R = \begin{cases} 67.5t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}} \ , \ for \ \ c\ge 0.5d \\ 34t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}} \ , \ for \ \ c<0.5d \end{cases} =$$ 206.68 > P **[Satisfactory]**

(Note : If item 2, local web yielding is Satisfactory, this item does not need to be checked.)

4. CHECK SIDESWAY WEB BUCKING FOR THE CONCENTRATED LOAD. (AISC-ASD, K5, page 5-81)
d$_c$ ***= d - 2k =*** 66.38 in
(d$_c$ ***/ t***$_w$ ***) / (l / b***$_f$***) =*** 22.13

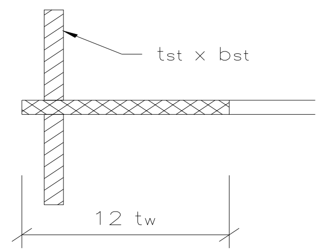
$$R = \begin{cases} \frac{6800t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \ , \ for \ \ \frac{d_c/t_w}{l/b_f}<1.7 \\ \frac{6800t_w^3}{h}\left[1+0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \ , \ for \ \ 1.7\le\frac{d_c/t_w}{l/b_f}<2.3 \\ P \ , \ for \ \ \frac{d_c/t_w}{l/b_f}\ge 2.3 \end{cases} =$$ 120.00 > P **[Satisfactory]**

(Note : If item 2, local web yielding is Satisfactory, this item does not need to be checked.)

(cont'd)

5. DETERMINE STIFFENER SIZE.

t_w = 5/8 in , b_{st} = 7 in



b_{st} / t_w = 11.20 < $95 / F_y^{0.5}$, AISC-ASD, B5.1
[Satisfactory]

A_{eff} = 11.75 in^2 , I = 159 in^4

f_a = 13.8 ksi

E_s = 29000 ksi

$K l / r = 0.75\, h / (I / A_{eff})^{0.5}$ = 13.7

$C_c = (2 \pi^2 E_s / F_y)^{0.5}$ = 107

$$F_a = \begin{cases} \dfrac{\left[1 - \dfrac{(kl/r)^2}{2C_c^2}\right] F_y}{\dfrac{5}{3} + \dfrac{3(kl/r)}{8C_c} - \dfrac{(kl/r)^3}{8C_c^3}} & , \quad for \quad \dfrac{kl}{r} \le C_c \\ \dfrac{12\pi^2 E}{23(kl/r)^2} & , \quad for \quad \dfrac{kl}{r} > C_c \end{cases} = 28.9$$

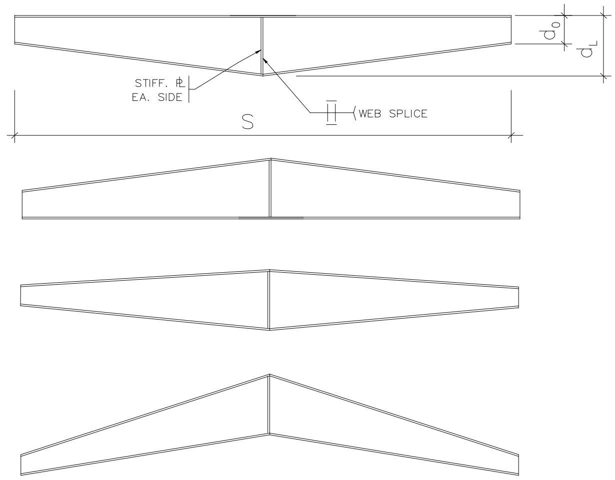
ksi, (AISC-ASD, E2, page 5-42)

> f_a **[Satisfactory]**

Technical Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.

Web Tapered Girder Design Based on AISC-ASD 9th, Appendix F



INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	50	ksi
SIMPLY SUPPORTED SPAN	S =	96	ft
SUPERIMPOSED DEAD LOAD	DL =	1	kips / ft
LIVE LOAD	LL =	0.75	kips / ft
FLANGE WIDTH	b_f =	13	in
FLANGE THICKNESS	t_f =	1	in
WEB THICKNESS	t_w =	0.5	in
HEEL DEPTH	d_0 =	24	in
MID-SPAN DEPTH	d_L =	72	in
DISTANCE BETWEEN STIFFENERS	a =	24	ft
UNBRACED LENGTH / PURLIN SPACING	L =	10	ft

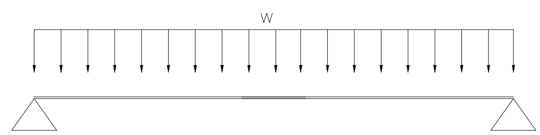
(Diaphragm is not bracing member. L is different with " l " in F1.3, pg 5-47)

THE GIRDER DESIGN IS ADEQUATE.

ANALYSIS

TOTAL SUPERIMPOSED GRAVITY LOAD

w = DL + LL = 1.750 kips / ft



ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \ \text{for} \ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \ \text{for} \ F_{b\gamma} \le F_y/3 \end{cases} = 30.00 \text{ ksi}$$

where A_f = 13.00 in^2

γ = MIN[(d_L - d_0) / d_0 , 0.268 L/d_0, 6.0] = 1.34

A_{To} = t_f b_f + d_0 t_w / 6 = 15.00 in^2

I_{To} = (t_f b_f^3 + d_0 t_w^3 / 6) / 12 = 183 in^4

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}} = 3.49 \text{ in}$$

$$F_{s\gamma} = \frac{12000}{h_s L d_o / A_f} = 37.13 \text{ ksi}$$

$$h_s = 1.0 + 0.0230\gamma\sqrt{\frac{Ld_0}{A_f}} = 1.46 \text{ in}$$

$$F_{w\gamma} = \frac{170000}{(h_w L / r_{To})^2} = 103.99 \text{ ksi}$$

$$h_w = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}} = 1.18 \text{ in}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} = 1.21$$

ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ \text{for} \ h/t_w \le 380\sqrt{F_y} \\ \frac{C_v F_y}{2.89} \le 0.4F_y \ , \ \text{for} \ h/t_w > 380\sqrt{F_y} \end{cases} = 10.01 \text{ ksi}$$

where $h = d_{under\ investigation} - 2\,t_f$ = 46 in, at first stiffer from end.

h / t_w = 92 > 380 / $F_y^{0.5}$ = 54

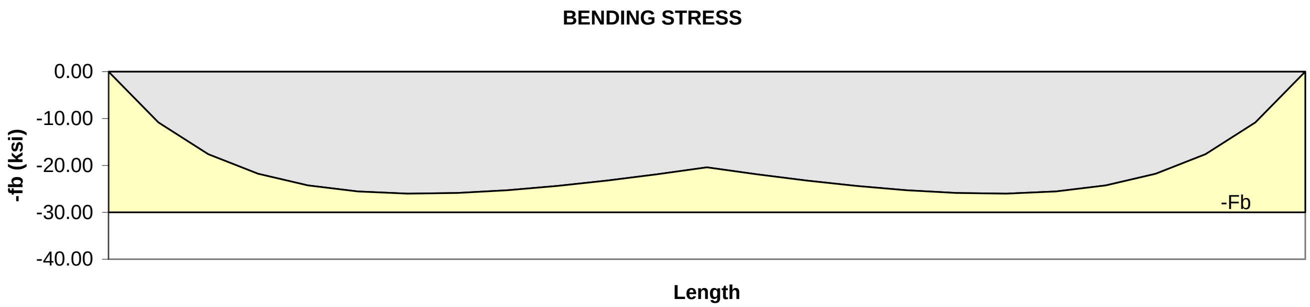
$$C_v = \begin{cases} \frac{45000 k_v}{F_y (h/t_w)^2} \ , \ \text{for} \ C_v \le 0.8 \\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \ , \ \text{for} \ C_v > 0.8 \end{cases} = 0.58$$

$$k_v = \begin{cases} 4.0 + \frac{5.34}{(a/h)^2} \ , \ \text{for} \ a/h \le 1.0 \\ 5.34 + \frac{4.0}{(a/h)^2} \ , \ \text{for} \ a/h > 1.0 \end{cases} = 5.44$$

(cont'd)

CHECK EACH SECTION CAPACITIES

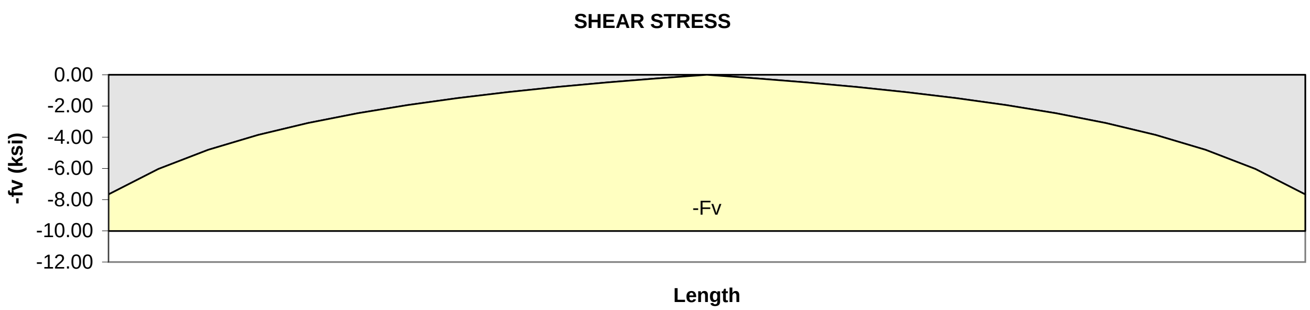
Section	HEEL	1/24 **S**	1/12 **S**	1/8 **S**	1/6 **S**	5/24 **S**	1/4 **S**	7/24 **S**	1/3 **S**	3/8 **S**	5/12 **S**	11/24 **S**	MID
Distance	0	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	44.00	48.00
d (in)	24	28	32	36	40	44	48	52	56	60	64	68	72
I (in^4)	3884	5473	7374	9602	12175	15108	18416	22117	26226	30758	35731	41160	47060
Wt (plf)	125.9	132.7	139.5	146.3	153.1	159.9	166.7	173.5	180.3	187.2	194.0	200.8	207.6
V (kips)	92.00	84.49	76.94	69.37	61.77	54.15	46.49	38.81	31.10	23.37	15.61	7.82	0.00
M (ft-k)	0	353	676	968	1231	1463	1664	1835	1974	2083	2161	2208	2224
f_v (ksi)	7.67	6.03	4.81	3.85	3.09	2.46	1.94	1.49	1.11	0.78	0.49	0.23	0.00
f_b (ksi)	0.00	10.84	17.60	21.79	24.26	25.56	26.02	25.88	25.30	24.38	23.23	21.89	20.41



$f_{b,max}$ = 26.02 ksi @ 24.00 ft, from heel.

< F_b = 30.00 ksi

[Satisfactory]



$f_{v,max}$ = 7.67 ksi @ 0.00 ft, from heel.

< F_v = 10.01 ksi **[Satisfactory]**

DETERMINE DEFLECTION AT MID SPAN

$$\Delta_{DL} = \int_0^S \frac{m_{DL}\, m_{unit}}{E I_S} ds =$$ 3.06 in (Span / 377)
(for camber, self Wt included.)

$$\Delta_{LL} = \int_0^S \frac{m_{LL}\, m_{unit}}{E I_S} ds =$$ 1.95 in (Span / 591)

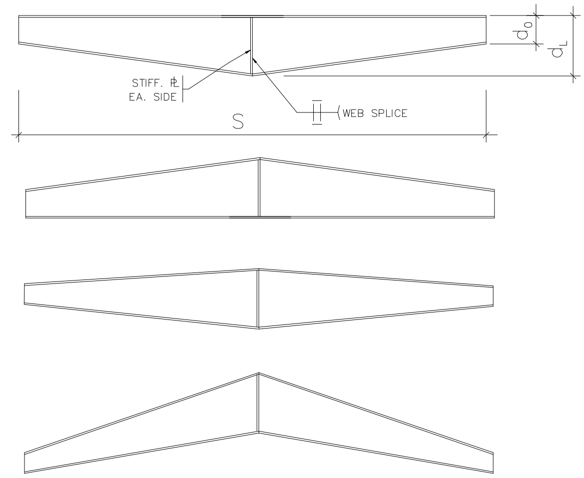
where E = 29000 ksi

Section	HEEL	1/24 **S**	1/12 **S**	1/8 **S**	1/6 **S**	5/24 **S**	1/4 **S**	7/24 **S**	1/3 **S**	3/8 **S**	5/12 **S**	11/24 **S**	MID
Distance	0.00	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	44.00	48.00
I_S (in^4)	3884	5473	7374	9602	12175	15108	18416	22117	26226	30758	35731	41160	47060
m_{DL} (ft-k)	0	215	412	590	751	893	1016	1121	1206	1273	1321	1350	1360
m_{LL} (ft-k)	0	138	264	378	480	570	648	714	768	810	840	858	864
m_{unit}	0.00	2.00	4.00	6.00	8.00	10.00	12.00	14.00	16.00	18.00	20.00	22.00	24.00
$\Delta_{DL,S}$	0.00	0.01	0.04	0.07	0.10	0.13	0.15	0.16	0.17	0.18	0.18	0.17	0.17
$\Delta_{LL,S}$	0.00	0.00	0.02	0.05	0.07	0.08	0.10	0.10	0.11	0.11	0.11	0.11	0.11

Technical Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.

Web Tapered Girder Design Based on AISC-ASD 9th, Appendix F



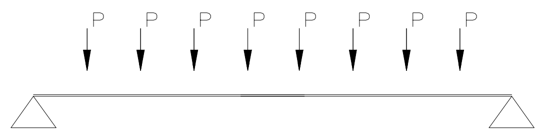
INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	50	ksi
SIMPLY SUPPORTED SPAN	S =	96	ft
NUMBER OF CONCENTRATED LOADS	n =	10	(@ 8.7 ft o.c.)
SUPERIMPOSED DEAD LOAD	P_D =	8.7273	kips
LIVE LOAD	P_L =	6.5455	kips
FLANGE WIDTH	b_f =	13	in
FLANGE THICKNESS	t_f =	1	in
WEB THICKNESS	t_w =	0.5	in
HEEL DEPTH	d_0 =	24	in
MID-SPAN DEPTH	d_L =	72	in
DISTANCE BETWEEN STIFFENERS	a =	24	ft
UNBRACED LENGTH / PURLIN SPACING	L =	10	ft

(Diaphragm is not bracing member. L is different with " / " in F1.3, pg 5-47)

THE GIRDER DESIGN IS ADEQUATE.

ANALYSIS



TOTAL SUPERIMPOSED GRAVITY LOAD

$P = P_D + P_L =$ 15.273 kips @ 8.7 ft o.c.

ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \ for\ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \ for\ F_{b\gamma} \le F_y/3 \end{cases} \quad = \quad 30.00 \text{ ksi}$$

where A_f = 13.00 in^2

$\gamma = MIN[(d_L - d_0) / d_0 , 0.268\ L/d_0, 6.0] =$ 1.34

$A_{To} = t_f b_f + d_0 t_w / 6 =$ 15.00 in^2

$I_{To} = (t_f b_f^3 + d_0 t_w^3 / 6) / 12 =$ 183 in^4

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}} = 3.49 \text{ in}$$

$$F_{s\gamma} = \frac{12000}{h_s L d_o / A_f} = 37.13 \text{ ksi}$$

$$h_s = 1.0 + 0.0230\gamma\sqrt{\frac{Ld_o}{A_f}} = 1.46 \text{ in}$$

$$F_{w\gamma} = \frac{170000}{(h_w L / r_{To})^2} = 103.99 \text{ ksi}$$

$$h_w = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}} = 1.18 \text{ in}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} = 1.21$$

ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ for\ h/t_w \le 380\sqrt{F_y} \\ \frac{C_v F_y}{2.89} \le 0.4F_y \ , \ for\ h/t_w > 380\sqrt{F_y} \end{cases} \quad = \quad 10.01 \text{ ksi}$$

where $h = d_{under\ investigation} - 2\,t_f =$ 46 in, at first stiffer from end.

$h / t_w =$ 92 > $380 / F_y^{0.5}$ = 54

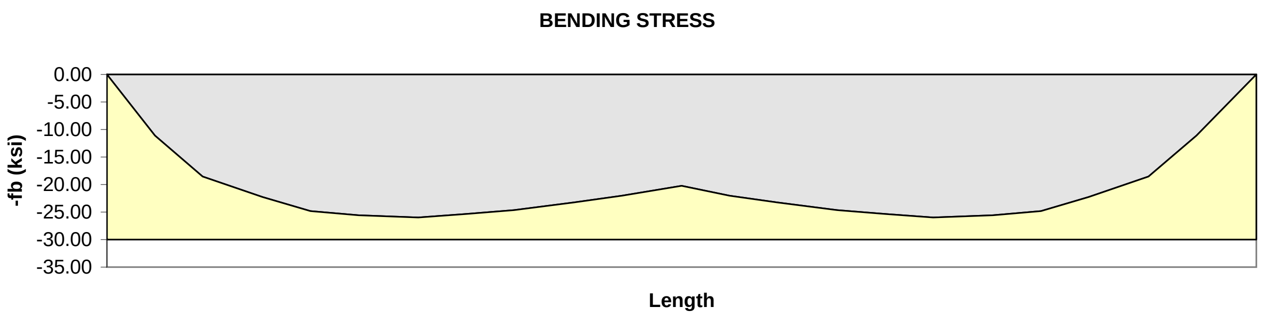
$$C_v = \begin{cases} \frac{45000 k_v}{F_y (h/t_w)^2} \ , \ for\ C_v \le 0.8 \\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \ , \ for\ C_v > 0.8 \end{cases} \quad = \quad 0.58$$

$$k_v = \begin{cases} 4.0 + \frac{5.34}{(a/h)^2} \ , \ for\ a/h \le 1.0 \\ 5.34 + \frac{4.0}{(a/h)^2} \ , \ for\ a/h > 1.0 \end{cases} \quad = \quad 5.44$$

(cont'd)

CHECK EACH SECTION CAPACITIES

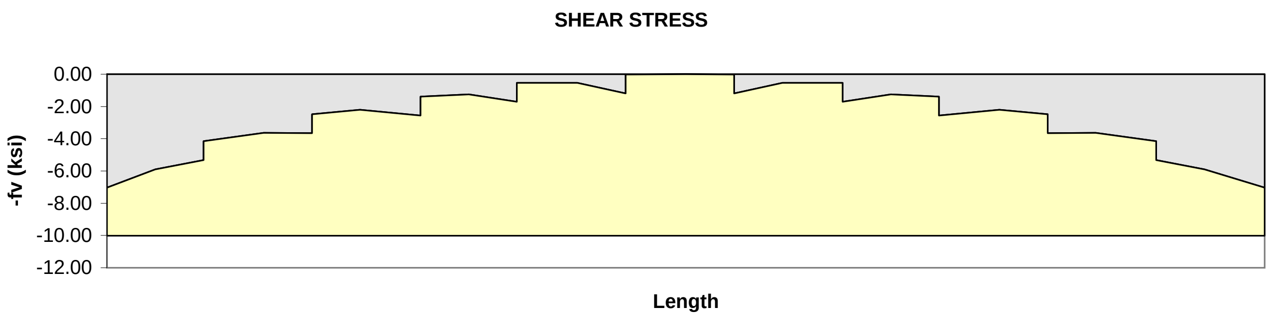
Section	HEEL	1/22 S	2/22 S	3/22 S	4/22 S	5/22 S	6/22 S	7/22 S	8/22 S	9/22 S	10/22 S		MID
Distance	0	4.36	8.73	13.09	17.45	21.82	26.18	30.55	34.91	39.27	43.64		48.00
d (in)	24	28	33	37	41	46	50	55	59	63	68		72
I (in^4)	3884	5633	7754	10269	13199	16564	20385	24683	29479	34793	40647		47060
Wt (plf)	125.9	133.3	140.8	148.2	155.6	163.0	170.4	177.9	185.3	192.7	200.1		207.6
V (kips)	84.37	83.80	67.93	67.30	51.36	50.67	34.67	33.91	17.84	17.02	0.89		0.00
M (ft-k)	0	367	731	1026	1319	1541	1761	1910	2057	2133	2205		2207
f_v (ksi)	7.03	5.91	4.15	3.63	2.48	2.21	1.38	1.24	0.61	0.54	0.03		0.00
f_b (ksi)	0.00	11.09	18.52	22.24	24.85	25.58	26.01	25.33	24.66	23.27	22.02		20.26



$f_{b,max}$ = 26.01 ksi @ 17.45 ft, from heel.

< F_b = 30.00 ksi

[Satisfactory]



$f_{v,max}$ = 7.03 ksi @ 0.00 ft, from heel.

< F_v = 10.01 ksi **[Satisfactory]**

DETERMINE DEFLECTION AT MID SPAN

$$\Delta_{DL} = \int_0^S \frac{m_{DL}\, m_{unit}}{E I_S} ds =$$ 3.04 in (Span / 379) (for camber, self Wt included.)

$$\Delta_{LL} = \int_0^S \frac{m_{LL}\, m_{unit}}{E I_S} ds =$$ 1.94 in (Span / 595)

where E = 29000 ksi

Section	HEEL	1/22 S	2/22 S	3/22 S	4/22 S	5/22 S	6/22 S	7/22 S	8/22 S	9/22 S	10/22 S		MID
Distance	0.00	4.36	8.73	13.09	17.45	21.82	26.18	30.55	34.91	39.27	43.64		48.00
I_S (in^4)	3884	5633	7754	10269	13199	16564	20385	24683	29479	34793	40647		47060
m_{DL} (ft-k)	0	224	446	627	804	941	1075	1168	1257	1304	1348		1350
m_{LL} (ft-k)	0	143	286	400	514	600	685	743	800	828	857		857
m_{unit}	0.00	2.18	4.36	6.55	8.73	10.91	13.09	15.27	17.45	19.64	21.82		24.00
$\Delta_{DL,S}$	0.00	0.01	0.04	0.09	0.12	0.15	0.17	0.18	0.19	0.19	0.19		0.18
$\Delta_{LL,S}$	0.00	0.01	0.03	0.05	0.08	0.10	0.11	0.12	0.12	0.12	0.12		0.12

Technical Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.

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Composite Beam Design with Verco Floor Deck Based on AISC Manual 9th

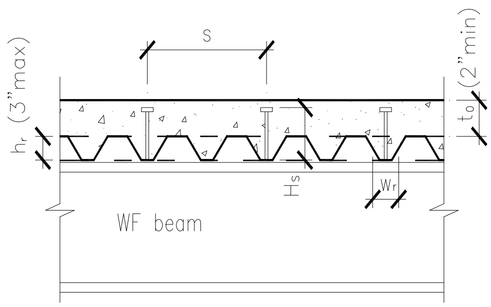
INPUT DATA & DESIGN SUMMARY

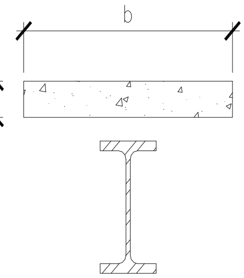
BEAM SECTION => **W30X132** =>

A	d	I_x	S_x
38.9	30.3	5770	380

FLOOR DECK TYPE => **W3-6 1/4" LW**

RIBS PERPENDICULAR TO BEAM ?		**Yes**	(perpendicular)
BEAM SPAN	L =	**52**	ft
BEAM SPACING (TRIB. WIDTH)	B =	**16**	ft, o.c.
SUPERIMPOSED LOAD	w_s =	**246**	lbs / ft^2
BEAM YIELD STRESS	F_y =	**50**	ksi
CONCRETE STRENGTH	f_c'=	**3**	ksi
SHEAR STUD DIA. (1/2, 5/8, 3/4)	ϕ =	**3/4**	in
NUMBER OF STUD IN ONE RIB	N_r =	**1**	





Effective Section

(Total 160 - 3/4 x 4.5" Shear Studs Required)
(0 13/16 in camber suggested)

THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

CHECK DIMENSION REQUIREMENTS (AISC-ASD I5.1, page 5-61)

t_0 =	3.25	in	>	2	in	[Satisfactory]
h_r =	3	in	<	3	in	[Satisfactory]
ϕ =	3/4	in	<	3/4	in	[Satisfactory]
$H_s = h_r + 1.5$ =	4.5	in	>	3	in	[Satisfactory]
s =	4	in o.c.	<	36	in o.c.	[Satisfactory]
w_r =	6	in	>	2	in	[Satisfactory]

DETERMINE COMPOSITE PROPERTIES

$b = MIN\,(L/4\,,\,B)$ = 156 in, (AISC-ASD I1.1, page 5-56)

$$n = \frac{E}{E_C} = \quad 13.91$$, (ACI 8.5.1)

$A_{ctr} = b\,t_0\,/\,n$ = 36.5 in^2

$$y_b = \frac{A_{ctr}(d+h_r+0.5t_0)+0.5Ad}{A_{ctr}+A} = \quad 24.7$$ in, from steel bottom.

$$I_{tr} = I_x + A(y_b - 0.5d)^2 + \frac{A_{ctr}t_0^2}{12} + A_{ctr}(0.5t_0+h_r+d-y_b)^2 = \quad 13161$$ in^4

$$S_{tr} = \frac{I_{tr}}{y_b} = \quad 532$$ in^3, referred to steel bottom.

$$S_t = \frac{I_{tr}}{(d + h_r + t_0 - y_b)} = \quad 1112$$ in^3, referred to concrete top.

CHECK BENDING & SHEAR CAPACITIES

$w = w_s + w_{wt}$ = 246.00 + (43.50 + 8.27) = 297.77 lbs / ft^2 (total gravity loads)

$$M_{max} = \frac{wBL^2}{8} = \quad 1610$$ ft-kips, (changeable).

$$V_{max} = \frac{wBL}{2} = \quad 124$$ kips, (changeable per actual).

Bottom: $$f_b = \frac{M_{DL}}{S_x} + \frac{M_{LL}}{S_{tr}} = \quad 39$$ ksi < $0.9\,F_y$ = 45 ksi, (non-shored, AISC-ASD I2.2, page 5-57) [Satisfactory]

Top: $$f_c = \frac{M_{max}}{nS_t} = \quad 1.249$$ ksi < $0.45\,f_c'$ = 1.35 ksi, (AISC-ASD I2.2, page 5-57) [Satisfactory]

Shear: $$f_v = \frac{V_{max}}{d\,t_w} = \quad 6.648$$ ksi < $0.4F_y$ = 20 ksi [Satisfactory]
(neglecting concrete & steel deck capacity conservatively)

(cont'd)

CHECK SHEAR CONNECTOR CAPACITY

$$V_h = MIN\left(0.85 f'_c A_c / 2 \quad , \quad F_y A_s / 2\right) =$$ 646.43 kips, (AISC-ASD I4-1 & I4-2, page 5-58)

$S_{eff} = Min[M_{max} / (0.66 F_y), S_{tr}] =$ 532 in^3, referred to steel bottom.

$$V'_h = MAX\left[\left(\frac{S_{eff}-S_s}{S_{tr}-S_s}\right)^2, 0.25\right] V_h =$$ 646.43 kips, (AISC-ASD I2-1 & I4, page 5-57 & 58)

$$\rho = \begin{cases} \left(\frac{0.85}{\sqrt{N_r}}\right)\left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{h_r}-1.0\right) \ , \ for \ \ Perpendicular \\ 0.6\left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{h_r}-1.0\right) \ , \ for \ \ Parallel \ \ \& \ \left(\frac{w_r}{h_r}\right)<1.5 \end{cases} =$$ 0.850 < 1.0 , (AISC-ASD I5-1, page 5-61)

$q' = \rho \, q =$ 8.11 kips, (AISC-ASD I5.2, page 5-61)

Allowable Horizontal Shear Load for One Connector (q, kips)
(AISC-ASD Table I4.1 with coefficient Table I4.2, page 5-59)

Dia. ϕ (in)	min. H_s (in)	Concrete f_c'		
		3.0	3.5	4.0 or Larger
1/2	2	4.2	4.6	4.9
5/8	2 1/2	6.6	7.1	7.6
3/4	3	9.5	10.4	11.0

$2N_1' = 2V_h' / q' =$ 160 , total number on the beam for partical composite action.

$2N_1 = 2V_h / q' =$ 160 , total number on the beam for full composite action.

$n = MIN[MAX(2N_1', 2N_1/4), 2N_1] =$ 160 , total number required on the beam, (AISC-ASD I4, page 5-59)

CHECK INITIAL DEFLECTION / CAMBER AND STRESS ON NON-COMPOSITE (AISC-ASD I2.1, page 5-56)

$w_{DL} =$ 100% **Self Wt =** 0.83 kips / ft (100% self weight load only)

$$\Delta_{DL} = \frac{5 w_{DL} L^4}{384 E I_x} =$$ 0.81 **in, Camber Suggested**

$$f_b = \frac{w_{DL} L^2}{8 S_x} =$$ 9 ksi < $0.66 F_y =$ 33 ksi
[Satisfactory]

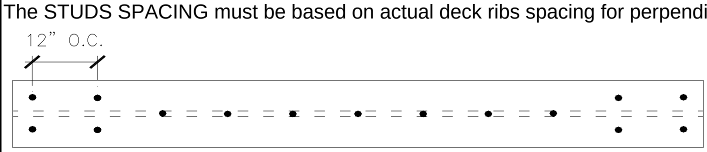
CHECK LIVE LOAD DEFLECTION ON COMPOSITE (AISC-ASD I2.1 page 5-56 & page 2-249)

$w_{LL} =$ 3.94 kips / ft (live load only)

$I_{eff} =$ 13161 in^4 (AISC-ASD, I4-4)

$$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_{eff}} =$$ 1.70 in < $L / 360 =$ 1.73 in
[Satisfactory]

Note:
The STUDS SPACING must be based on actual deck ribs spacing for perpendicular to beam. For the following total [15] studs, if ribs spacing 12" o. c., the minimum composte beam capacity is from 2 rows @ 12" o.c., not one row @ 8.57" o.c. See software ***CompositeFloorBeamWithCantilever.xls*** on the website www.Engineering-International.com for more information.



Technical References:
1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. Alan Williams Ph.D., S.E., C.Eng.: "Structural Steel Design - Volume 1: ASD", ICBO, 2001.

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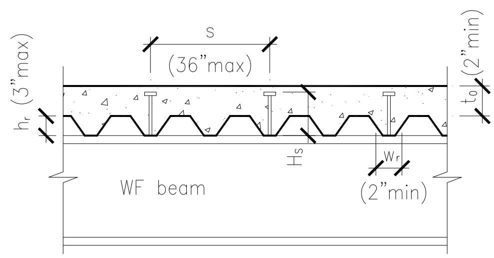
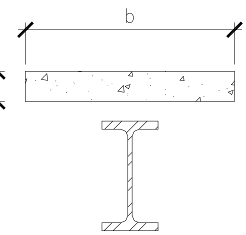
Composite Beam Design with Formed Steel Deck Based on AISC-ASD

INPUT DATA & DESIGN SUMMARY

						A	d	I_x	S_x
BEAM SECTION	=>	W21X62			=>				
BEAM SPAN	L =	40	ft			18.3	21.0	1330	127
BEAM SPACING (DECK SPAN)	B =	11	ft, o.c.						
DEAD LOAD	w_{DL} =	125	lbs / ft^2						
LIVE LOAD	w_{LL} =	80	lbs / ft^2						
RIBS PERPENDICULAR TO BEAM ?		yes	(perpendicular)						
BEAM YIELD STRESS	F_y =	50	ksi						
CONCRETE STRENGTH	f_c '=	4.5	ksi						
TOPPING CONCRETE THICK.	t_0 =	3	in						
SHEAR STUD DIA. (1/2, 5/8, 3/4)	ϕ =	5/8	in						
NOMINAL RIB HEIGHT	h_r =	2	in						
AVERAGE WIDTH OF RIB	w_r =	3	in						
NUMBER OF STUD IN ONE RIB	N_r =	1							





Effective Section

(Total 34 - 5/8 x 3.5" Shear Studs Required)

(2 1/16 in camber suggested)

THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

CHECK DIMENSION REQUIREMENTS (AISC-ASD I5.1, page 5-60)

t_0 =	3	in	>	2	in	[Satisfactory]
h_r =	2	in	<	3	in	[Satisfactory]
ϕ =	5/8	in	<	3/4	in	[Satisfactory]
$H_s = h_r + 1.5$ =	3.5	in	>	2.5	in	[Satisfactory]
s =	15	in o.c.	<	36	in o.c.	[Satisfactory]
w_r =	3	in	>	2	in	[Satisfactory]

DETERMINE COMPOSITE PROPERTIES

b = MIN (L / 4 , B) = 120 in, (AISC-ASD I1.1, page 5-56)

$$n = \frac{E}{E_C} = \frac{29000}{57\sqrt{f_c^{'}}} = \quad 7.58$$, (ACI 8.5.1)

$A_{ctr} = b\,t_0 / n =$ 47.5 in^2

$$y_b = \frac{A_{ctr}(d+h_r+0.5t_0)+0.5Ad}{A_{ctr}+A} = \quad 20.6$$ in, from steel bottom.

$$I_{tr} = I_x + A\left(y_b - 0.5d\right)^2 + \frac{A_{ctr}t_0^2}{12} + A_{ctr}\left(0.5t_0+h_r+d-y_b\right)^2 = \quad 3954$$ in^4

$$S_{tr} = \frac{I_{tr}}{y_b} = \quad 192$$ in^3, referred to steel bottom.

$$S_t = \frac{I_{tr}}{\left(d + h_r + t_0 - y_b\right)} = \quad 733$$ in^3, referred to concrete top.

CHECK BENDING & SHEAR CAPACITIES

w = 205 lbs / ft^2 (total gravity loads)

$$M_{max} = \frac{wBL^2}{8} = \quad 451$$ ft-kips, (changeable).

$$V_{max} = \frac{wBL}{2} = \quad 45$$ kips, (changeable per actual).

Bottom: $$f_b = \frac{M_{DL}}{S_x} + \frac{M_{LL}}{S_{tr}} = \quad 37$$ ksi < $0.9\,F_y$ = 45 ksi, (non-shored, AISC-ASD I2.2, page 5-57) [Satisfactory]

Top: $$f_c = \frac{M_{max}}{nS_t} = \quad 0.974$$ ksi < $0.45\,f_c$ ' = 2.025 ksi, (AISC-ASD I2.2, page 5-57) [Satisfactory]

Shear: $$f_v = \frac{V_{max}}{dt_w} = \quad 9.127$$ ksi < $0.4F_y$ = 20 ksi [Satisfactory]
(neglecting concrete & steel deck capacity conservatively)

(cont'd)

CHECK SHEAR CONNECTOR CAPACITY

$$V_h = MIN\left(0.85 f'_c A_c / 2 \quad , \quad F_y A_s / 2\right) =$$ 457.5 kips, (AISC-ASD I4-1 & I4-2, page 5-58)

S_{eff} = Min[M_{max} / (0.66 F_y) , S_{tr}] = 164 in^3, referred to steel bottom.

$$V'_h = MAX\left[\left(\frac{S_{eff}-S_s}{S_{tr}-S_s}\right)^2, 0.25\right] V_h =$$ 148.62 kips, (AISC-ASD I2-1 & I4, page 5-57 & 58)

$$\rho = \left(\frac{0.85}{\sqrt{N_r}}\right)\left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{h_r} - 1.0\right) =$$ 0.956 < 1.0 , (AISC-ASD I5-1, page 5-61)

q' = ρ q = 8.80 kips, (AISC-ASD I5.2, page 5-61)

Allowable Horizontal Shear Load for One Connector (q, kips), (AISC-ASD Table I4.1, page 5-59)

Dia. ϕ (in)	min. H_s (in)	Concrete f_c' 3.0	3.5	4.0 or Larger
1/2	2	5.1	5.5	5.9
5/8	2 1/2	8.0	8.6	9.2
3/4	3	11.5	12.5	13.3

2 N_1' = 2 V_h' / q' = 34 , total number on the beam for partical composite action.

2 N_1 = 2 V_h / q' = 104 , total number on the beam for full composite action.

n = MIN[MAX(2N_1' , 2N_1/4) , 2N_1] = 34 , total number required on the beam, (AISC-ASD I4, page 5-59)

CHECK INITIAL DEFLECTION / CAMBER AND STRESS ON NON-COMPOSITE (AISC-ASD I2.1, page 5-56)

w_{DL} = 1.38 kips / ft (dead load only)

$$\Delta_{DL} = \frac{5 w_{DL} L^4}{384 E I_X} =$$ 2.05 **in, Camber Suggested**

$$f_b = \frac{w_{DL} L^2}{8 S_x} =$$ 26 ksi **<** ***0.66 F_y =*** 33 ksi [Satisfactory]

CHECK LIVE LOAD DEFLECTION ON COMPOSITE (AISC-ASD I2.1 page 5-56 & page 2-249)

w_{LL} = 0.88 kips / ft (live load only)

I_{eff} = 2826 in^4 (AISC-ASD, I4-4)

$$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_{eff}} =$$ 0.62 in **<** ***L / 360 =*** 1.33 in [Satisfactory]

Technical References:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. Alan Williams Ph.D., S.E., C.Eng.: "Structural Steel Design - Volume 1: ASD", ICBO, 2001.

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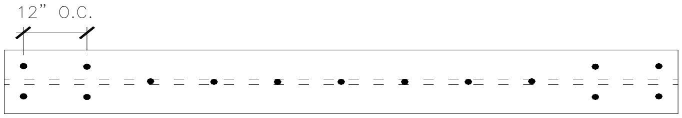
Composite Beam Design with Verco Floor Deck Based on AISC 360-05 / IBC 09

DESIGN CRITERIA

The input STUDS SPACING must be based on actual deck ribs spacing for perpendicular to beam. For the following total [15] studs, if ribs spacing 12" o. c., the minimum composte beam capacity is from 2 rows @ 12" o.c., not one row @ 8.57" o.c.

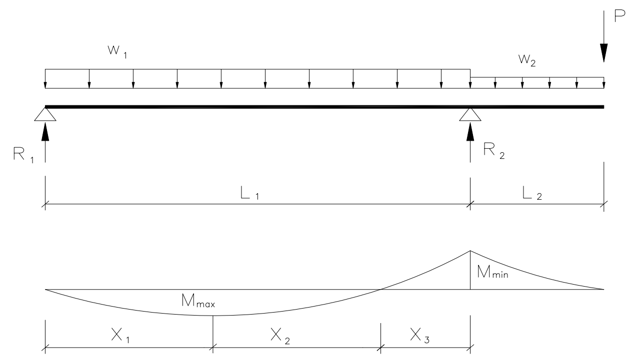


INPUT DATA & DESIGN SUMMARY

BEAM SECTION	= >	W18X50	= >	A	d	I_x	S_x	Z_x
				14.7	18.0	800	88.9	101
FLOOR DECK TYPE	= >	W3-6 1/4" LW						

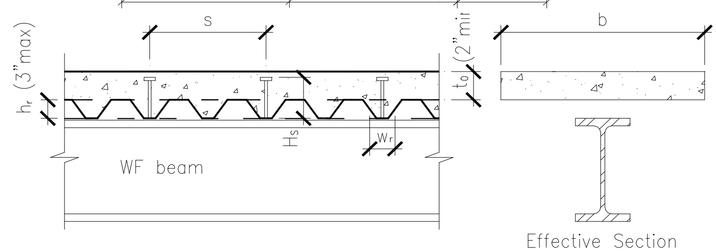
BEAM SPAN	L_1 =	42.425	ft
CANTILEVER (0 = no cantilever)	L_2 =	2.845	ft
BEAM SPACING (TRIB. WIDTH)	B =	9.67	ft, o.c.
SUPERIMPOSED LOAD, ASD	$w_{s,1}$ =	98.70	lbs / ft^2
	$w_{s,2}$ =	121.33	lbs / ft^2
CONCENTRATED END LOADS	P =	3.3845	kips
RIBS PERPENDICULAR TO BEAM ?		Yes	(perpendicular)
BEAM YIELD STRESS	F_y =	50	ksi
CONCRETE STRENGTH	f_c '=	3	ksi
SHEAR STUD DIA. (1/2, 5/8, 3/4)	ϕ =	3/4	in
STUDS SPACING 1	row @	12	in o.c.

(Total Studs 44 + 4)



THE BEAM DESIGN IS ADEQUATE.

USE C = 1 0/4" AT MID BEAM.



ANALYSIS

DETERMINE REACTIONS, MOMENTS & SHEARS

$w_1 = w_{s\,1} + w_{wt}$ = 98.6972148 + (43.50 + 5.17) = 147.37 lbs / ft^2
= 1.43 kips / ft (total gravity loads on span beam)

$w_2 = w_{s,\,2} + w_{wt}$ = 121.327215 + (43.50 + 5.17) = 170.00 lbs / ft^2
= 1.64 kips / ft (uniform gravity loads on cantilever)

P =	3.3845	kips			
X_1 =	20.94	ft	R_1 =	29.85	kips
X_2 =	20.94	ft	R_2 =	38.67	kips
X_3 =	0.54	ft	V_{max} =	34.00	kips, at R2 left.
M_{max} =	312.5	ft-kips	M_{min} =	-16.282	ft-kips

CHECK DIMENSION REQUIREMENTS

t_o =	3.25	in	>	2	in	[Satisfactory]	(AISC 360-05 I3.2c.1.c)
h_r =	3	in	<	3	in	[Satisfactory]	(AISC 360-05 I3.2c.1.a)
ϕ =	3/4	in	<	3/4	in	[Satisfactory]	(AISC 360-05 I3.2c.1.b)

$H_s = h_r + 1.5 =$ 4.5 in < $h_r + t_o - 0.5 =$ 5.75 in [Satisfactory] (AISC I3.2c.1.b)

s = 12 in o.c. < $MAX[\,8(h_r + t_o)\,,\,36\,] =$ 50 in o.c. [Satisfactory]
> $4\,\phi =$ 3 in o.c. [Satisfactory] (AISC 360-05 I3.2d.6)

w_r = 6 in > 2 in [Satisfactory] (AISC 360-05 I3.2c.1.a)

DETERMINE COMPOSITE PROPERTIES FOR PLASTIC DESIGN

$b = MIN\,(L/4\,,\,B) =$	116.04 in, (AISC 360-05 I3.1a)	$h = t_o + h_r + d =$	24.3 in, (total height)
$A_{ctr} = 0.85\,f_c'\,b\,t_o\,/\,F_y =$	19.2 in^2	$A_{total} = A_{ctr} + A =$	33.9 in^2
$A_{fill} = A - 2A_f - A_w =$	0.16 in^2	A_f =	4.28 in^2
t_w =	0.36 in	t_f =	0.57 in

(cont'd)

$$y_b = \begin{cases} h - \dfrac{AF_y}{0.85 f'_c b} & for \quad A_{total} \le 2 A_{ctr} \\ d - \dfrac{0.5 A_{total} - A_{ctr}}{b_f} & for \quad A_{total} \le 2\left(A_{ctr} + A_f\right) \\ t_f + \dfrac{A_{total} - A_f - 0.5 A_{fill}}{t_w} & for \quad A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases} = \quad 21.8 \text{ in, (plastic neutral axis to bottom)}$$

$$y = \begin{cases} 0.5\left(t_0 + h_r + y_b\right) & for \quad A_{total} \le 2 A_{ctr} \\ h - \dfrac{0.5 t_0 A_{ctr} + 0.5 dA + \left(0.5 A_{total} - A_{ctr}\right)\left(h - d - y_b\right)}{0.5 A_{total}} & for \quad A_{total} \le 2\left(A_{ctr} + A_f\right) \\ h - \dfrac{0.5 t_0 A_{total} + A_f\left(t_0 + h_r + t_f\right) + 0.5 A_{fill}\left(t_0 + h_r + 2t_f\right) + t_w\left(d - y_b - t_f\right)\left(h - 0.5d - 0.5y_b + 0.5t_f\right) + t_w\left(y_b - t_f\right)\left(0.5y_b + 0.5t_f\right)}{0.5 A_{total}} & for \quad A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases}$$

thus, ***y =*** 14.0 in, (moment arm between centroid of tensile force and the resultant compressive force.)

$Z_{tr} = 0.5\, y\, A_{total} =$ 238 in^3

DETERMINE COMPOSITE PROPERTIES FOR ELASTIC DESIGN

$n = \dfrac{E}{E_C} =$ 13.01 , (ACI 318-05 8.5.1)

$A_{ctr} = b\, t_0 / n =$ 29.0 in^2

$y_b = \dfrac{A_{ctr}\left(d + h_r + 0.5t_0\right) + 0.5Ad}{A_{ctr} + A} =$ 18.0 in, (elastic neutral axis to bottom)

$I_{tr} = I_x + A\left(y_b - 0.5d\right)^2 + \dfrac{A_{ctr} t_0^2}{12} + A_{ctr}\left(0.5t_0 + h_r + d - y_b\right)^2 =$ 2636 in^4

$S_{tr} = \dfrac{I_{tr}}{y_b} =$ 146 in^3, referred to steel bottom.

$S_t = \dfrac{I_{tr}}{\left(d + h_r + t_0 - y_b\right)} =$ 425 in^3, referred to concrete top.

CHECK BENDING & SHEAR CAPACITIES

Middle Bottom : $M_{max} = (Z_{tr} / Z_x)\, M_{DL} + M_{LL} =$ 452.2 ft-kips

< $M_n / \Omega_b = Z_{tr} F_y / \Omega_b =$ 593.0 ft-kips, (AISC 360 I3.2a) [Satisfactory]

where $\Omega_b =$ 1.67 (AISC 360-05 I3.2a)

$3.76(E / F_y)^{0.5} =$ 90.55 > $h / t_w =$ 50.70

Cantilever Top : $M_{min} =$ -16.28179 ft-kips < $M_n / \Omega_b =$ 252.0 ft-kips, (AISC 360 I3.2b) [Satisfactory]

where $\Omega_b =$ 1.67 (AISC 360-05 I3.2b)

$L_b = MAX(X_3, L_2) =$ 2.85 ft

$L_p = 1.76\, r_y\, (E / F_{yf})^{0.5} =$ 5.83 ft

$L_r = 1.95 r_{ts} \dfrac{E}{0.7F_y} \sqrt{\dfrac{Jc}{S_x h_0}} \sqrt{1 + \sqrt{1 + 6.76\left(\dfrac{0.7F_y}{E}\dfrac{S_x h_0}{Jc}\right)^2}} =$ 16.96 ft

$M_p = F_y Z_x =$ 420.8 ft-kips

$F_{cr} = \dfrac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \dfrac{Jc}{S_x h_0}\left(\dfrac{L_b}{r_{ts}}\right)^2} =$ 1265.34 ksi

$r_{ts} = [(I_y C_w)^{0.5} / S_x]^{0.5} =$ 1.98

$c =$ 1.00

$h_0 = d - t_f =$ 17.43 in

$C_b =$ 1.30 , (AISC 360-05 F1)

I_y	G	J	C_w
40	11200	1.24	3040
r_y	Z_x		
1.65	101		

$M_n =$

M_p	=	420.83	ft-kips, for L_b @ [0 , L_p]
MIN{C_b [M_p - (M_p - 0.75 F_y S_x) (L_b - L_p) / (L_r - L_p)] , M_p}	=	N/A	ft-kips, for L_b @ (L_p , L_r]
MIN(F_{cr} S_x , M_p)	=	N/A	ft-kips, for L_b @ (L_r , Larger)

(cont'd)

Shear : V_{max} = 34.00 kips < $V_n / \Omega_v = 0.6 F_y A_w C_v / \Omega_v =$ 114.79 kips, (AISC 360-05 I3.1b)

[Satisfactory]

where $2.24 (E / F_y)^{0.5}$ = 53.946

k_v = 5 (AISC 360-05 G2.1b) C_v = 1 (AISC 360-05 G2.1b)

$(k_v E / F_y)^{0.5}$ = 53.852 Ω_v = 1.67 (AISC 360-05 G1)

CHECK SHEAR CONNECTOR CAPACITY

M_{max} = 312.5 ft-kips > $M_n / \Omega_b = Z_x F_y / \Omega_b$ = 252.0 ft-kips **<== Shear Studs Required**

where Ω_b = 1.67 (AISC 360-05 F1 & F2-1)

$C_f = MIN(0.85 f_c' A_c , F_y A_s) =$ 735 kips, (AISC 360-05 C-I3.1)

$S_{eff} = Min[M_{max} / (0.66 F_y) , S_{tr}] =$ 114 in^3, refered to steel bottom.

$$V' = MAX\left[\left(\frac{S_{eff}-S_s}{S_{tr}-S_s}\right)^2, 0.25\right]C_f =$$ 183.75 kips, (AISC 360-05 C-I3-4)

$Q_n = MIN[0.5 A_{sc} (f_c' E_c)^{0.5} , R_g R_p A_{sc} F_u] =$ 15.37 kips, (AISC 360-05 I3.2d.3)

where w_c = 115 pcf

$E_c = w_c^{1.5}\ 33 (f_c')^{0.5}$ = 2229.1 ksi

A_{sc} = 0.44 in^2

F_u = 58 ksi

R_g = 1.00 (AISC 360-05 Table I3.2b.3)

R_p = 0.60 (AISC 360-05 Table I3.2b.3)

$\Sigma Q_n = Q_n N_r X_1 / s$ = 321.98 kips > V' [Satisfactory]

CHECK INITIAL DEFLECTION / CAMBER AND STRESS ON NON-COMPOSITE

DL = 75% *Self Weight*

$L = L_1$ = 42.43 ft $a = L_2$ = 2.85 ft

$P = P_{DL}$ = 2.54 kips

$w_1 = w_{DL,1}$ = 0.35 klf

$w_2 = w_{DL,2}$ = 0.35 klf

$$\Delta_{End} = \frac{Pa^2(L+a)}{3EI} - \frac{w_1 L^3 a}{24EI} + \frac{w_2 a^3(4L+3a)}{24EI} =$$ -0.21 in, uplift perpendicular to beam.

$$\Delta_{Mid} = -\frac{PaL^2}{16EI} + \frac{5w_1 L^4}{384EI} - \frac{w_2 L^2 a^2}{32EI} =$$ 1.06 in, downward perpendicular at middle of beam.

USE C = 1 0/4" AT MID BEAM.

M_{max} = ***77.4*** ft-kips < $M_n / \Omega_b = Z_x F_y / \Omega_b$ = ***252.0*** ft-kips [Satisfactory]

CHECK LIVE LOAD DEFLECTION ON COMPOSITE

P = 0.00 kips

$w_1 = w_{LL,1}$ = 0.95 klf

$w_2 = w_{LL,2}$ = 1.17 klf

I_{eff} = 2015 in^4 (AISC 360-05 C-I3-3)

$$\Delta_{End} = \left[\frac{Pa^2(L+a)}{3EI} - \frac{w_1 L^3 a}{24EI} + \frac{w_2 a^3(4L+3a)}{24EI}\right] =$$ -0.63 in, uplift.

< $2L_2 / 240$ = 0.28 in [Satisfactory]

$$\Delta_{Mid} = \left[-\frac{PaL^2}{16EI_{eff}} + \frac{5w_1 L^4}{384EI_{eff}} - \frac{w_2 L^2 a^2}{32EI_{eff}}\right] =$$ 0.92 in, downward.

< $L_1 / 360$ = 1.41 in [Satisfactory]

Technical References:

1. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.

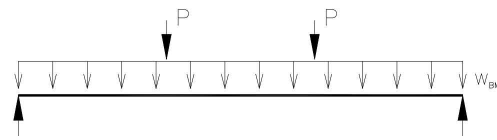
Composite Girder Design Based on AISC 360-05 / CBC 10 / IBC 09

INPUT DATA & DESIGN SUMMARY

GIRDER SECTION => **W21X44** =>

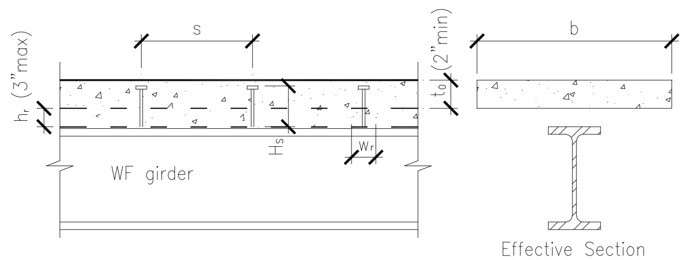
A	d	I_x	S_x	Z_x
13	20.7	843	81.6	95.4

FLOOR DECK TYPE => **W3-6 1/4" LW**



GIRDER SPAN	L =	28	ft
GIRDER SPACING (TRIB. WIDTH)	B =	21	ft, o.c.
GIRDER SELF WEIGHT, ASD	w_{BM} =	44.24	lbs / ft
NUMBER OF EQUAL POINT LOAD	N =	2	
EQUAL POINT LOAD, ASD	P =	30.417	kips @ 9.33" o.c.
RIBS PERPENDICULAR TO GIRDER ?		No	(parallel)
GIRDER YIELD STRESS	F_y =	50	ksi
CONCRETE STRENGTH	f_c'=	3	ksi
SHEAR STUD DIA. (1/2, 5/8, 3/4)	ϕ =	3/4	in
STUDS SPACING 1	row @	18	in o.c.

(Total Studs 20)



THE GIRDER DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE REACTIONS, MOMENTS & SHEARS

R = 0.5 (w_{BM} L + N P) = 31.04 kips

M_{max} = 0.5 R L - 0.125 $w_{BM} L^2$ - Σ(P D_i) = 288.2 ft-kips, at middle of girder

V_{max} = R = 31.04 kips

CHECK DIMENSION REQUIREMENTS

t_0 =	3.25	in	>	2	in	[Satisfactory]	(AISC 360-05 I3.2c.1.c)		
h_r =	3	in	<	3	in	[Satisfactory]	(AISC 360-05 I3.2c.1.a)		
ϕ =	3/4	in	<	3/4	in	[Satisfactory]	(AISC 360-05 I3.2c.1.b)		
H_s = h_r + 1.5 =	4.5	in	<	h_r + t_o - 0.5 =	5.75	in	[Satisfactory]	(AISC I3.2c.1.b)	
s =	18	in o.c.	<	MAX[8(h_r + t_o) , 36] =	50	in o.c.	[Satisfactory]		
			>	4 ϕ =	3	in o.c.	[Satisfactory]	(AISC 360-05 I3.2d.6)	
w_r =	6	in	>	2	in	[Satisfactory]	(AISC 360-05 I3.2c.1.a)		

DETERMINE COMPOSITE PROPERTIES FOR PLASTIC DESIGN

b = MIN (L / 4 , B) = 84 in, (AISC 360-05 I3.1a)

h = t_0 + h_r + d = 27.0 in, (total height)

A_{ctr} = 0.85 f_c' b t_0 / F_y = 13.9 in^2

A_{total} = A_{ctr} + A = 26.9 in^2

A_{fill} = A - 2A_f - A_w = 0.22 in^2

A_f = 2.93 in^2

t_w = 0.35 in

t_f = 0.45 in

$$y_b = \begin{cases} h - \dfrac{A F_y}{0.85 f_c' b} & for \quad A_{total} \le 2 A_{ctr} \\ d - \dfrac{0.5 A_{total} - A_{ctr}}{b_f} & for \quad A_{total} \le 2\left(A_{ctr} + A_f\right) \\ t_f + \dfrac{A_{total} - A_f - 0.5 A_{fill}}{t_w} & for \quad A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases} = \quad 23.9$$

in, (plastic neutral axis to bottom)

$$y = \begin{cases} 0.5\left(t_0 + h_r + y_b\right) & for \quad A_{total} \le 2 A_{ctr} \\ h - \dfrac{0.5 t_0 A_{ctr} + 0.5 d A + \left(0.5 A_{total} - A_{ctr}\right)\left(h - d - y_b\right)}{0.5 A_{total}} & for \quad A_{total} \le 2\left(A_{ctr} + A_f\right) \\ h - \dfrac{0.5 t_0 A_{total} + A_f\left(t_0 + h_r + t_f\right) + 0.5 A_{fill}\left(t_0 + h_r + 2 t_f\right) + t_w\left(d - y_b - t_f\right)\left(h - 0.5 d - 0.5 y_b + 0.5 t_f\right) + t_w\left(y_b - t_f\right)\left(0.5 y_b + 0.5 t_f\right)}{0.5 A_{total}} & for \quad A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases}$$

thus, y = 15.1 in, (moment arm between centroid of tensile force and the resultant compressive force.)

Z_{tr} = 0.5 y A_{total} = 203 in^3

DETERMINE COMPOSITE PROPERTIES FOR ELASTIC DESIGN

$$n = \frac{E}{E_c} = \quad 13.01$$

, (ACI 318-05 8.5.1)

(cont'd)

$A_{ctr} = b\,t_0 / n =$ 21.0 in^2

$$y_b = \frac{A_{ctr}(d+h_r+0.5t_0)+0.5Ad}{A_{ctr}+A} = \quad 19.6$$ in, (elastic neutral axis to bottom)

$$I_{tr} = I_x + A\left(y_b - 0.5d\right)^2 + \frac{A_{ctr}t_0^2}{12} + A_{ctr}\left(0.5t_0+h_r+d-y_b\right)^2 = \quad 2662$$ in^4

$$S_{tr} = \frac{I_{tr}}{y_b} = \quad 136$$ in^3, referred to steel bottom.

$$S_t = \frac{I_{tr}}{\left(d+h_r+t_0-y_b\right)} = \quad 362$$ in^3, referred to concrete top.

CHECK BENDING & SHEAR CAPACITIES

Middle Bottom : $M_{max} = (Z_{tr} / Z_x)\, M_{DL} + M_{LL} =$ 383.8 ft-kips

$< \quad M_n / \Omega_b = Z_{tr}\, F_y / \Omega_b =$ 506.6 ft-kips, (AISC 360 I3.2a) [Satisfactory]

where $\Omega_b =$ 1.67 (AISC 360-05 I3.2a)

$3.76(E / F_y)^{0.5} =$ 90.55 $>$ $h / t_w =$ 59.14

$w_{deck} =$ 43.50 lbs / ft^2

Shear : $V_{max} =$ 31.04 kips $<$ $V_n / \Omega_v = 0.6F_yA_wC_v / \Omega_v =$ 130.15 kips, (AISC 360-05 I3.1b)

[Satisfactory]

where $2.24\,(E / F_y)^{0.5} =$ 53.946

$k_v =$ 5 (AISC 360-05 G2.1b) $C_v =$ 1 (AISC 360-05 G2.1b)

$(k_v\, E / F_y)^{0.5} =$ 53.852 $\Omega_v =$ 1.67 (AISC 360-05 G1)

CHECK SHEAR CONNECTOR CAPACITY

$M_{max} =$ 288.2 ft-kips $>$ $M_n / \Omega_b = Z_x\, F_y / \Omega_b =$ 238.0 ft-kips **<== Shear Studs Required**

where $\Omega_b =$ 1.67 (AISC 360-05 F1 & F2-1)

$C_f = MIN\,(\,0.85\, f_c{}'A_c\,,\, F_yA_s) =$ 650 kips, (AISC 360-05 C-I3.1)

$S_{eff} = Min[\,M_{max} / (0.66\,F_y)\,,\, S_{tr}\,] =$ 105 in^3, refered to steel bottom.

$$V' = MAX\left[\left(\frac{S_{eff}-S_s}{S_{tr}-S_s}\right)^2, 0.25\right]C_f = \quad 162.5$$ kips, (AISC 360-05 C-I3-4)

$Q_n = MIN\,[\,0.5\,A_{sc}\,(f_c{}'E_c)^{0.5}\,,\, R_gR_pA_{sc}F_u\,] =$ 18.06 kips, (AISC 360-05 I3.2d.3)

where $w_c =$ 115 pcf

$E_c = w_c^{1.5}\,33\,(f_c{}')^{0.5} =$ 2229.1 ksi

$A_{sc} =$ 0.44 in^2

$F_u =$ 58 ksi

$R_g =$ 1.00 (AISC 360-05 Table I3.2b.3)

$R_p =$ 0.75 (AISC 360-05 Table I3.2b.3)

$\Sigma Q_n = Q_n\, N_r\, X_1 / s =$ 168.59 kips $>$ V' [Satisfactory]

CHECK INITIAL DEFLECTION / CAMBER AND STRESS ON NON-COMPOSITE

$DL =$ 75% ***Self Weight***

$w_{DL} =$ 33.18 lbs / ft

$P_{DL} =$ 6.70 kips @ 9.33" o.c.

$e =$ 0.036 $L =$ 28.00 ft

$$\Delta_{Mid} = \frac{eP_{DL}L^3}{EI} + \frac{5w_{DL}L^4}{384EI} = \quad 0.39$$ in, downward at middle of girder.

$M_{max} =$ ***66.9*** ft-kips $<$ $M_n / \Omega_b = Z_x\, F_y / \Omega_b =$ ***238.0*** ft-kips [Satisfactory]

CHECK LIVE LOAD DEFLECTION ON COMPOSITE

$P =$ 23.71 kips

$$\Delta_{Mid} = \frac{eP_{LL}L^3}{EI_{tr}} = \quad 0.42$$ in, downward at middle of girder.

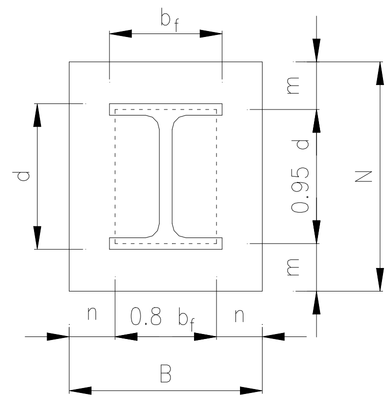
$< \quad L / 360 =$ 0.93 in [Satisfactory]

WF Base Plate Design Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

AXIAL LOAD OF COMPRESSION	P_a =	200	kips, ASD
STEEL PLATE YIELD STRESS	F_y =	60	ksi
CONCRETE STRENGTH	f_c' =	3	ksi
COLUMN SIZE	=>	W10X45	
BASE PLATE SIZE	N =	16	in
	B =	16	in
AREA OF CONCRETE SUPPORT	A_2 =	1156	in^2

(geometrically similar to and concentric with the loaded area.)



USE
16 x 16
1 in thick plate

ANALYSIS

CHECK BEARING PRESSURE (AISC 360-05 J8)

$$P_p/\Omega_c = \frac{f'_c A_1}{\Omega_c} MIN\left[0.85 MAX\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 1\right)\ ,\ 1.7\right] = \quad 522.24 \text{ kips} \quad > \quad P_a$$

Where A_1 = 256 in^2, actual area of base plate.

Ω_c = 2.50

[Satisfactory]

DETERMINE VALUES OF m, n, n', X , and λ (AISC 13th Page 14-5)

m = 0.5 (N - 0.95 d) =	3.20	in
n = 0.5 (B - 0.8 b_f) =	4.79	in
n' = 0.25 (d b_f)$^{0.5}$ =	2.25	in

$$X = MIN\left[\left(\frac{4 d b_f}{(d+b_f)^2}\right)\frac{\Omega_c P_a}{P_p}\ ,\ 1\right] = \quad 0.38$$

$$\lambda = MIN\left(\frac{2\sqrt{X}}{1+\sqrt{1-X}}\ ,\ 1\right) = \quad 0.69$$

Where	***d***	=	10.10	in, depth of column section.
	b_f	=	8.02	in, flange width of column section.

DETERMINE REQUIRED THICKNESS OF BASE PLATE (AISC 13th Page 14-6)

$$t_{min} = l\sqrt{\frac{3.33 P_a}{F_y B N}} = \quad 1.00 \text{ in}$$

Where ***l = MAX (m, n, λn') =*** 4.79 in

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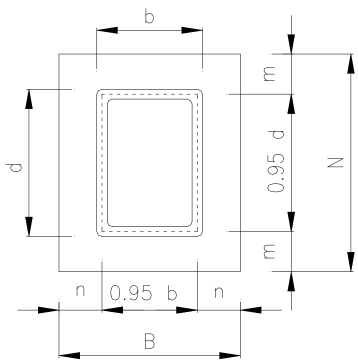
REVIEW BY :

Tube Base Plate Design Based on AISC Manual 13th Edition (AISC 360-05)



INPUT DATA & DESIGN SUMMARY

AXIAL LOAD OF COMPRESSION	P_a =	200	kips, ASD
STEEL PLATE YIELD STRESS	F_y =	60	ksi
CONCRETE STRENGTH	f_c' =	3	ksi
COLUMN SIZE	=>	HSS8X8X1/2	
BASE PLATE SIZE	N =	16	in
	B =	16	in
AREA OF CONCRETE SUPPORT	A_2 =	1156	in^2

(geometrically similar to and concentric with the loaded area.)

USE
16 x 16
7/8 in thick plate

ANALYSIS

CHECK BEARING PRESSURE (AISC 360-05 J8)

$$P_p/\Omega_c = \frac{f'_c A_1}{\Omega_c} MIN\left[0.85 MAX\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 1\right)\ ,\ 1.7\right] =$$ 522.24 kips **>** P_a

Where A_1 = 256 in^2, actual area of base plate. **[Satisfactory]**

Ω_c = 2.50

DETERMINE VALUES OF m, n, n', X , and λ (AISC 13th Page 14-5)

m = 0.5 (N - 0.95 d) = 4.20 in

n = 0.5 (B - 0.95 b) = 4.20 in

n' = 0.25 (d b)$^{0.5}$ = 2.00 in

$$X = MIN\left[\left(\frac{4db}{(d+b)^2}\right)\frac{\Omega_c P_a}{P_p}\ ,\ 1\right] = \quad 0.38$$

$$\lambda = MIN\left(\frac{2\sqrt{X}}{1+\sqrt{1-X}}\ ,\ 1\right) = \quad 0.69$$

Where ***d*** = 8.00 in, depth of column section.

b = 8.00 in, width of column section.

DETERMINE REQUIRED THICKNESS OF BASE PLATE (AISC 13th Page 14-6)

$$t_{min} = l\sqrt{\frac{3.33 P_a}{F_y BN}} = \quad 0.87 \text{ in}$$

Where l ***= MAX (m, n, λn') =*** 4.20 in

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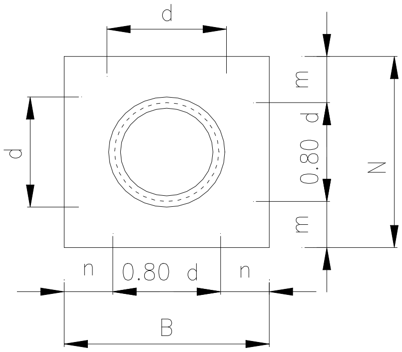
Pipe Base Plate Design Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

AXIAL LOAD OF COMPRESSION	P_a =	200	kips, ASD
STEEL PLATE YIELD STRESS	F_y =	60	ksi
CONCRETE STRENGTH	f_c' =	3	ksi
COLUMN SIZE	=>	HSS8.750X0.500	
BASE PLATE SIZE	N =	16	in
	B =	16	in
AREA OF CONCRETE SUPPORT	A_2 =	1156	in^2

(geometrically similar to and concentric with the loaded area.)



USE
16 x 16
1 in thick plate

ANALYSIS

CHECK BEARING PRESSURE (AISC 360-05 J8)

$$P_p/\Omega_c = \frac{f'_c A_1}{\Omega_c} MIN\left[0.85 MAX\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 1\right)\ ,\ 1.7\right] = \quad 522.24 \text{ kips} \quad > \quad P_a$$

Where A_1 = 256 in^2, actual area of base plate.

Ω_c = 2.50

[Satisfactory]

DETERMINE VALUES OF m, n, n', X , and λ (AISC 13th Page 14-5)

m = 0.5 (N - 0.80 d) = 4.50 in

n = 0.5 (B - 0.80 d) = 4.50 in

n' = 0.25 (d b)$^{0.5}$ = 2.19 in

$$X = MIN\left[\left(\frac{4db}{(d+b)^2}\right)\frac{\Omega_c P_a}{P_p}\ ,\ 1\right] = \quad 0.38$$

$$\lambda = MIN\left(\frac{2\sqrt{X}}{1+\sqrt{1-X}}\ ,\ 1\right) = \quad 0.69$$

Where ***d*** = 8.75 in, depth of column section.

b = 8.75 in, width of column section.

DETERMINE REQUIRED THICKNESS OF BASE PLATE (AISC 13th Page 14-6)

$$t_{min} = l\sqrt{\frac{3.33 P_a}{F_y BN}} = \quad 0.94 \text{ in}$$

Where l ***= MAX (m, n, λn') =*** 4.50 in

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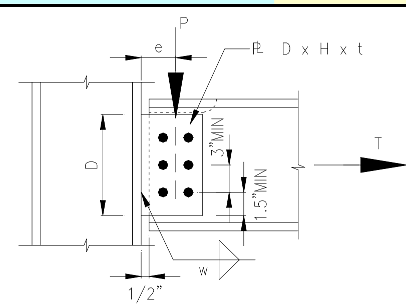
Beam Connection of Conventional Configuration Based on AISC 360-2010



INPUT DATA & DESIGN SUMMARY

WF BEAM SECTION	= >	W21X50	
GRAVITY SERVICE LOAD	P =	47	kips
LATERAL TENSION LOAD, ASD	T =	15	kips
PLATE THICKNESS	t =	0.75	in
PLATE STEEL YIELD STRESS	F_y =	36	ksi
TRIAL WELD SIZE	w =	0.5	in (1/2 in)
BOLT DIAMETER	ϕ =	1	in (1 in)
BOLT MATERIAL (A307, A325, A490)	ASTM =	A325	
HOLE TYPE (STD, NSL, OVS, SSL, LSL)	= >	STD	

STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

USE PLATE 8.5" x 3.0" x 3/4" WITH WELD 1/2" EACH SIDE TO COLUMN AND 1 ROW OF TOTAL (3) - 1" BOLTS AT BEAM END.

CONNECTION TYPE (SC, N, X) = > **N**

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

TRY BOLT NUMBERS **1** *row &* **3** *bolts per row, (total 3 bolts.)*
IS TOP FLANGE COPED ? (1=Yes, 0=No,) = > **0** No

ANALYSIS

SECTION PROPERTIES (AISC Manual Table 1)

d	t_w	t_f	k
20.8	0.38	0.535	1.04

CHECK CAPACITY OF BOLTS (AISC 360-10 J3)

Allow shear per bolt = 18.8 kips / bolt, (R_n / Ω_v, AISC Manual Table 7)

$(P^2 + T^2)^{0.5}$ = 49 kips

No. of bolts required	=	2.6		Number of bolts used	=	3	bolts	[Satisfactory]
Bolt spacing required	=	3.00	in	Bolt spacing used	=	3.00	in	[Satisfactory]
Edge spacing required	=	1.25	in, (Tab J3.4)	Edge spacing used	=	1.25	in	[Satisfactory]
Number of rows required	=	1	rows	Number of rows used	=	1	rows	[Satisfactory]
Bolt group capacity	=	57	kips	> $(P^2 + T^2)^{0.5}$	=	49	kips	
				> P =		47	kips	[Satisfactory]

CHECK CAPACITY OF WELDING (AISC 360-10 J2)

e	=	1.75	in, (AISC 360-10, Table J3.4)	
Plate thickness	=	0.75	in	
Weld size,w	=	0.50	in	
Min allowable weld	=	0.25	in	[Satisfactory]
Max allowable weld	=	0.69	in	[Satisfactory]
t_e	=	0.35	in	
D	=	8.5	in	
I = 2 (t_e D^3 / 12)	=	36.2	in^4	
Vertical shear = P / A_w = P / 2 D t_e	=	7.8	ksi	
Bending stress = 0.5 P e D / I	=	9.7	ksi	
Tension stress = T / A_w = T / 2 D t_e	=	2.5	ksi	
Resultant Stress = $[(P/A_w)^2 + (0.5\, P\, e\, D / I + T/A_w)^2]^{0.5}$	=	14.5	ksi	
Allow shear F_w / Ω = F_w / 2.0	=	28.6	ksi	
	>	14.5	ksi	[Satisfactory]

θ =	17.7	deg, (AISC 360-10, J2-5)
Δ_u =	0.0694	in
Δ_m =	0.0403	in
f(p) =	1.2563	, (AISC 360-10, J2-9)
F_w =	57.188	ksi, (AISC 360-10, J2-8)

CHECK PLATE FOR SHEAR CAPACITY (AISC 360-05 G2)

P / A = 7.4 ksi < 0.6 F_y C_v / Ω_v = 0.6 F_y 1.0 / 1.5 = 14.4 ksi [Satisfactory]

CHECK PLATE FOR TENSION CAPACITY (AISC 365-05 D)

T / A = 2.4 ksi < F_y / Ω_t = F_y / 1.67 = 21.56 ksi [Satisfactory]

CHECK NET SHEAR FRACTURE (AISC 360-10 J4.2)

F_u = 58 ksi (AISC Manual, Pg. 2-39)

P_{allow} = 0.6 F_u / Ω [D - n (d_s + 1/8)] t = 67 kips > 47 kips [Satisfactory]

CHECK NET TENSION FRACTURE (AISC 360-10 J4.1)

F_u = 58 ksi

T_{allow} = F_u / Ω [D - n (d_s + 1/8)] t = 111 kips > 15 kips [Satisfactory]

CHECK BLOCK SHEAR (WEB TEAR-OUT, AISC 360-10 J4) <== Applicable only for top flange coped.

l_h = 0.8 in

l_v = 4.8 in

F_u = 65 ksi (for WF, AISC Manual, Pg. 2-39)

$R_{bs,P}$ = 0.6 A_v F_u / Ω + A_t F_u / Ω = (0.3 l_v + 0.5 l_h) t_w F_u

[Satisfactory]

$R_{bs,T}$ = (0.5 l_v + 2 x 0.3 l_h) t_w F_u = 70 > T = 15 kips

[Satisfactory]

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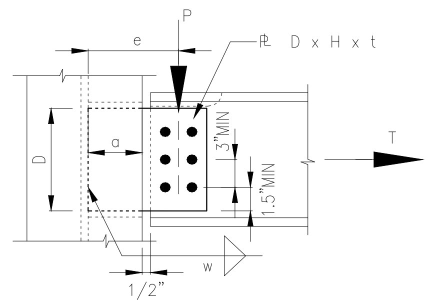
Beam Connection of Extended Configuration Based on AISC 360-2010

INPUT DATA & DESIGN SUMMARY



WF BEAM SECTION	=>	W24X94	
GRAVITY SERVICE LOAD	P =	43	kips
LATERAL TENSION LOAD, ASD	T =	155	kips
PLATE THICKNESS	t =	0.625	in
PLATE STEEL YIELD STRESS	F_y =	50	ksi
TRIAL WELD SIZE	w =	0.5	in (1/2 in)
BOLT DIAMETER	ϕ =	1	in (1 in)
BOLT MATERIAL (A307, A325, A490)	ASTM =	A325	
HOLE TYPE (STD, NSL, OVS, SSL, LSL)	=>	STD	

STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

CONNECTION TYPE (SC, N, X) => SC

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

USE PLATE 20.5" x 13.1" x 5/8" WITH WELD 1/2" EACH SIDE TO COLUMN AND 2 ROW OF TOTAL (14) - 1" BOLTS AT BEAM END.

TRY BOLT NUMBERS 2 *rows &* 7 *bolts per row, (total 14 bolts.)*
IS TOP FLANGE COPED ? (1=Yes, 0=No,) => 0 No
EXTENDED DIMENSION a = 7.11 in

ANALYSIS

SECTION PROPERTIES (AISC Manual Table 1)

d	t_w	t_f	k
24.3	0.515	0.875	1.38

CHECK CAPACITY OF BOLTS (AISC 360-10 J3)

Allow shear per bolt = 11.5 kips / bolt, (R_n / Ω_v, AISC Manual Table 7)
$(P^2 + T^2)^{0.5}$ = 161 kips

No. of bolts required	=	14.0		Number of bolts used	=	14	bolts	[Satisfactory]
Bolt spacing required	=	3.00	in	Bolt spacing used	=	3.00	in	[Satisfactory]
Edge spacing required	=	1.25	in, (Tab J3.4)	Edge spacing used	=	1.25	in	[Satisfactory]
Number of rows required	=	2	rows	Number of rows used	=	2	rows	[Satisfactory]

Bolt group capacity = 161 kips > $(P^2 + T^2)^{0.5}$ = 161 kips
> P = 43 kips [Satisfactory]

CHECK CAPACITY OF WELDING (AISC 360-10 J2)

e, (including ***a***)	=	10.36	in, (AISC 360-10, Table J3.4)	
Plate thickness	=	0.63	in	
Weld size,w	=	0.50	in	
Min allowable weld	=	0.25	in	[Satisfactory]
Max allowable weld	=	0.56	in	[Satisfactory]
t_e	=	0.35	in	
D	=	20.5	in	

$I = 2\ (t_e\ D^3\ /\ 12)$	=	507.6	in^4	θ =	74.495	deg, (AISC 360-10, J2-5)
Vertical shear = $P / A_w = P / 2\,D\,t_e$	=	3.0	ksi	Δ_u =	0.0314	in
Bending stress = $0.5\,P\,e\,D / I$	=	9.0	ksi	Δ_m =	0.0261	in
Tension stress = $T / A_w = T / 2\,D\,t_e$	=	10.7	ksi	f(p) =	1.1321	, (AISC 360-10, J2-9)
Resultant Stress = $[\ (P/A_w)^2 + (0.5\,P\,e\,D / I + T/A_w)^2\ ]^{0.5}$ =		19.9	ksi	F_w =	70.035	ksi, (AISC 360-10, J2-8)

Allow shear $F_w / \Omega = F_w / 2.0$ = 35.0 ksi
> 19.9 ksi [Satisfactory]

CHECK PLATE FLEXURE CAPACITY WITH VON-MISES REDUCTION (AISC Manual, page 10-103)

$f_v = [(P/A)^2 + (T/A + 6Pe / tD^2)^2]^{0.5}$ = 22.5 ksi
$F_{cr} = (F_y^2 - 3 f_v^2)^{0.5}$ = 31.3 ksi
M = Pe = 37.1 ft-k < $F_{cr}\,Z / \Omega$ = 102.5 ft-k [Satisfactory]

CHECK PLATE FOR SHEAR CAPACITY (AISC 360-05 G2)

P / A = 3.4 ksi < $0.6\,F_y\,C_v / \Omega_v = 0.6\,F_y\,1.0 / 1.5$ = 20 ksi [Satisfactory]

CHECK PLATE FOR TENSION CAPACITY (AISC 365-05 D)

T / A = 12.1 ksi < $F_y / \Omega_t = F_y / 1.67$ = 29.94 ksi [Satisfactory]

CHECK NET SHEAR FRACTURE (AISC 360-10 J4.2)

F_u = 70 ksi (AISC Manual, Pg. 2-39)
$P_{allow} = 0.6\,F_u / \Omega\ [\ D - n\ (d_s + 1/8\ )\ ]\ t$ = 166 kips > 43 kips [Satisfactory]

CHECK NET TENSION FRACTURE (AISC 360-10 J4.1)

F_u = 70 ksi
$T_{allow} = F_u / \Omega\ [\ D - n\ (d_s + 1/8\ )\ ]\ t$ = 276 kips > 155 kips [Satisfactory]

CHECK BLOCK SHEAR (WEB TEAR-OUT, AISC 360-10 J4) <== Applicable only for top flange coped.

l_h = 2.8 in l_v = 12.8 in
F_u = 65 ksi (for WF, AISC Manual, Pg. 2-39)
$R_{bs,P} = 0.6\,A_v\,F_u / \Omega + A_t\,F_u / \Omega = (0.3\,l_v + 0.5\,l_h)\,t_w\,F_u$
[Satisfactory]
$R_{bs,T} = (0.5\,l_v + 2 \times 0.3\,l_h)\,t_w\,F_u$ = 269 > T = 155 kips [Satisfactory]

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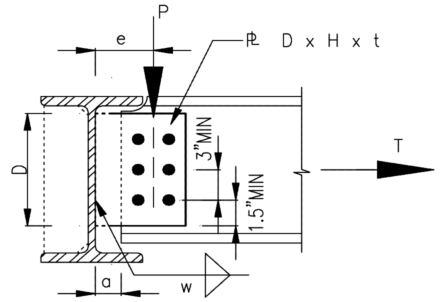
Beam to Girder Connection Design Based on AISC 360-10

INPUT DATA & DESIGN SUMMARY

WF BEAM SECTION	=>	**W24X94**	
GRAVITY SERVICE LOAD	P =	**43**	kips
LATERAL TENSION LOAD, ASD	T =	**155**	kips
PLATE THICKNESS	t =	**0.625**	in
PLATE STEEL YIELD STRESS	F_y =	**50**	ksi
TRIAL WELD SIZE	w =	**0.5**	in (1/2 in)
BOLT DIAMETER	ϕ =	**1**	in (1 in)
BOLT MATERIAL (A307, A325, A490)	ASTM =	**A325**	
HOLE TYPE (STD, NSL, OVS, SSL, LSL)	=>	**STD**	



STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

USE PLATE 20.5" x 7.5" x 5/8" WITH WELD 1/2" EACH SIDE TO GIRDER WEB AND 2 ROW OF TOTAL (14) - 1" BOLTS AT BEAM END.

CONNECTION TYPE (SC, N, X) => **SC**

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

TRY BOLT NUMBERS **2** *rows &* **7** *bolts per row, (total 14 bolts.)*

IS TOP FLANGE COPED ? (1=Yes, 0=No,) => **1** Yes

EXTENDED DIMENSION a = **2** in

ANALYSIS

SECTION PROPERTIES (AISC Manual Table 1)

d	t_w	t_f	k
24.3	0.515	0.875	1.38

CHECK CAPACITY OF BOLTS (AISC 360-10 J3)

Allow shear per bolt = 11.5 kips / bolt, (R_n / Ω_v, AISC Manual Table 7)

$(P^2 + T^2)^{0.5}$ = 161 kips

No. of bolts required	=	14.0		Number of bolts used	=	14	bolts	[Satisfactory]
Bolt spacing required	=	3.00	in	Bolt spacing used	=	3.00	in	[Satisfactory]
Edge spacing required	=	1.25	in, (Tab J3.4)	Edge spacing used	=	1.25	in	[Satisfactory]
Number of rows required	=	2	rows	Number of rows used	=	2	rows	[Satisfactory]

Bolt group capacity = 161 kips > $(P^2 + T^2)^{0.5}$ = 161 kips
> P = 43 kips [Satisfactory]

CHECK CAPACITY OF WELDING (AISC 360-10 J2)

e, (including ***a***)	=	4.75	in, (AISC 360-10, Table J3.4)	
Plate thickness	=	0.63	in	
Weld size,w	=	0.50	in	
Min allowable weld	=	0.25	in	[Satisfactory]
Max allowable weld	=	0.56	in	[Satisfactory]
t_e	=	0.35	in	
D	=	20.5	in	
$I = 2\ (t_e\ D^3\ / 12)$	=	507.6	in^4	
Vertical shear = $P / A_w = P / 2\ D\ t_e$	=	3.0	ksi	
Bending stress = $0.5\ P\ e\ D / I$	=	4.1	ksi	
Tension stress = $T / A_w = T / 2\ D\ t_e$	=	10.7	ksi	

θ =	74.495	deg, (AISC 360-10, J2-5)
Δ_u =	0.0314	in
Δ_m =	0.0261	in
f(p) =	1.1321	, (AISC 360-10, J2-9)
F_w =	70.035	ksi, (AISC 360-10, J2-8)

Resultant Stress = $[\ (P/A_w)^2 + (0.5\ P\ e\ D / I + T/A_w)^2\ ]^{0.5}$ = 15.1 ksi

Allow shear $F_w / \Omega = F_w / 2.0$ = 35.0 ksi
> 15.1 ksi [Satisfactory]

CHECK PLATE FLEXURE CAPACITY WITH VON-MISES REDUCTION (AISC Manual, page 10-103)

$f_v = [(P/A)^2 + (T/A + 6Pe / tD^2)^2]^{0.5}$ = 17.1 ksi

$F_{cr} = (F_y^2 - 3\ f_v^2)^{0.5}$ = 40.3 ksi

M = Pe = 17.0 ft-k < $F_{cr}\ Z / \Omega$ = 132.0 ft-k [Satisfactory]

CHECK PLATE FOR SHEAR CAPACITY (AISC 360-05 G2)

P / A = 3.4 ksi < $0.6\ F_y\ C_v / \Omega_v = 0.6\ F_y\ 1.0 / 1.5$ = 20 ksi [Satisfactory]

CHECK PLATE FOR TENSION CAPACITY (AISC 365-05 D)

T / A = 12.1 ksi < $F_y / \Omega_t = F_y / 1.67$ = 29.94 ksi [Satisfactory]

CHECK NET SHEAR FRACTURE (AISC 360-10 J4.2)

F_u = 70 ksi (AISC Manual, Pg. 2-39)

$P_{allow} = 0.6\ F_u / \Omega\ [\ D - n\ (d_s + 1/8\ )\ ]\ t$ = 166 kips > 43 kips [Satisfactory]

CHECK NET TENSION FRACTURE (AISC 360-10 J4.1)

F_u = 70 ksi

$T_{allow} = F_u / \Omega\ [\ D - n\ (d_s + 1/8\ )\ ]\ t$ = 276 kips > 155 kips [Satisfactory]

CHECK BLOCK SHEAR (WEB TEAR-OUT, AISC 360-10 J4) <== Applicable only for top flange coped.

l_h = 2.8 in l_v = 12.8 in

F_u = 65 ksi (for WF, AISC Manual, Pg. 2-39)

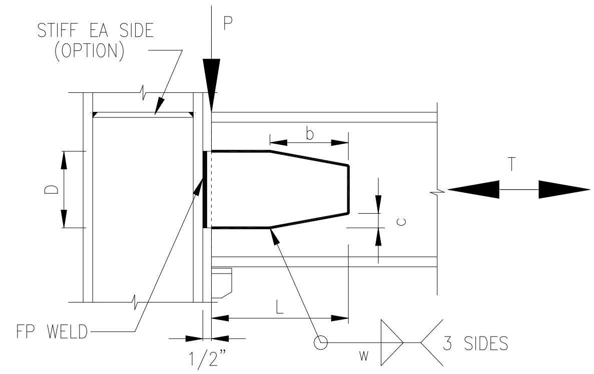
$R_{bs,P} = 0.6\ A_v\ F_u / \Omega + A_t\ F_u / \Omega = (0.3\ l_v + 0.5\ l_h)\ t_w\ F_u$ = 174 kips
> P = 43.00 kips [Satisfactory]

$R_{bs,T} = (0.5\ l_v + 2 \times 0.3\ l_h)\ t_w\ F_u$ = 269 > T = 155 kips [Satisfactory]

Drag Connection Based on AISC 360-05 & AISC 341-05

INPUT DATA & DESIGN SUMMARY

DRAG BEAM SECTION	=>	W27X94	
COLUMN SECTION	=>	W27X217	
GRAVITY SERVICE LOAD	P =	71	kips
LATERAL DRAG LOAD, ASD	T =	245	kips
PLATE THICKNESS	t_p =	0.625	in
STIFF. THICKNESS (OPTION)	t_{st} =	0	in
PLATE STEEL YIELD STRESS	F_y =	50	ksi
WELD SIZE	w =	0.5	in
DIMENSIONS	L =	15	in
	D =	23	in
	c =	5	in
	b =	2	in



THE DESIGN IS ADEQUATE

ANALYSIS

BEAM SECTION PROPERTIES (AISC 13th Table 1)

d	t_w	k	A
26.9	0.49	1.34	27.7

CHECK PLATE CAPACITY (AISC 360-05, D2 & G2)

$T_n / \Omega_t = F_y\, t_p\, D / 1.67 =$ 430 kips > T [Satisfactory]

$V_n / \Omega_v = 0.6\, C_v\, F_y\, t_p\, D / 1.5 =$ 288 kips > P [Satisfactory]

Where $C_v =$ 1.0

CHECK DIMENSION "***D***" LIMITATION

$D =$ 23 in < $d - 2k - 2w =$ 23.22 in [Satisfactory]

CHECK FILLET WELD TO WEB (AISC 360-05, J2.2b)

$w =$ 0.5 in < $w_{max} =$ 0.56 in [Satisfactory]

> $w_{min} =$ 0.25 in [Satisfactory]

$e =$ 10.84 in, eccentricity from column face.

$I_x =$ 4311 in^4 / in $I_y =$ 731 in^4 / in

$f_x = T / A + P\, e\, (0.5\, D) / (I_x + I_y) =$ 18.9 ksi

$f_y = P / A + P\, e\, (e - 0.5) / (I_x + I_y) =$ 8.5 ksi

$f_v = (f_x + f_y)^{0.5} =$ 20.7 ksi < $F_v = 0.6\, F_{EXX} / \Omega =$ 21.0 ksi [Satisfactory]

CHECK BLOCK SHEAR (WEB TEAR-OUT, AISC 360-05 J4)

$F_u =$ 65 ksi (for WF, AISC Manual 13th Edition, Pg. 2-39)

$R_n / \Omega = (0.5\, L_v + 2 \times 0.3\, L_h)\, t_w\, F_u =$ 557 kips > T [Satisfactory]

CHECK BEAM TENSION CAPACITY (AISC 360-05, D2)

$T_n / \Omega_t = (50\ ksi)\, A / 1.67 =$ 829 kips > T [Satisfactory]

CHECK STIFF PLATE COMPRESSION CAPACITY (AISC 358-05, E)

$P_{n,st} / \Omega_c = F_{cr}\, A / 1.67 =$ 357.3 kips > T [Satisfactory]

Where $K =$ 0.75

$b_{st} =$ 6 in, one side stiff width

$h_{st} = d_c - 2k =$ 23.82 in, stiff depth

$I = t_{st}\,(2b_{st} + t_{wc})^3 / 12 + D\, t_{wc}^3 / 12 =$ 1 in^4

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 17 in^2

$r_{st} = (I / A)^{0.5} =$ 0.25 in

$K\, h_{st} / r_{st} < 200$ (AISC 360, E2) [Satisfactory]

$F_e = \pi^2 E / (K\, h_{st} / r_{st})^2 =$ 57.066 ksi, (AISC 360, E3)

$\lambda_c = (K\, h_{st} / r_{st})\, (F_y / E)^{0.5} =$ 2.94

$F_{cr} = \begin{cases} (0.658^{(Fy/Fe)})F_y = 34.65, & \lambda_c \le 4.71 \\ 0.877\, F_e = \text{N/A} & \text{kis, for } \lambda_c > 4.71 \end{cases}$

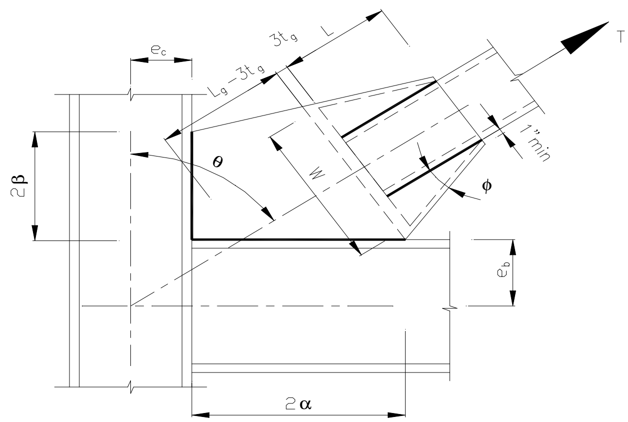
(Only top stiffeners used conservatively)

Corner Bracing Connection Capacity Based on AISC Manual 13th Edition (AISC 360-05)

DESIGN CRITERIA

For a new ***TYPICAL*** detail of bracing connection or existing bracing connection, the interface dimensions of α and β may not satisfy the basic relationship of the original Uniform Force Method: $\alpha - \beta \tan\theta = e_b \tan\theta - e_c$. This software can determine corner gusset dimensions based on geometry and can check the gusset interface weld capacities with moment loads.

INPUT DATA & DESIGN SUMMARY



BRACE AXIAL LOAD AT SERVICE LEVEL — T = **250** kips

ANGLE BETWEEN BRACE & COLUMN — θ = **25** ^{0}

ANGLE BTW BRACE & GUSSET EDGE — ϕ = **20** ^{0}

BRACE SECTION (Tube or Pipe) => **HSS8X8X5/8**

Tube	A	r_{min}	t	b	h
	16.40	2.98	0.63	8.00	8.00

COLUMN SECTION => **W12X96**

ORIENTATION = **x-x** , e_c = **6.35** in

BEAM SECTION => **W16X67**

ORIENTATION = **x-x** , e_b = **8.15** in

THE CONNECTION DESIGN IS ADEQUATE.

(1" Gusset Plate with 5/8" Fillet Weld, 4 leg x 8" Length at Brace, and 2 leg x 27" at Column Interface, 2 leg x 15" at Beam Interface.)

ANALYSIS

DETERMINE BEST FILLET WELD SIZE PER BRACE THICKNESS (AISC 360-05 J2.2b)

w = 0.625 in > w_{MIN} = 0.25 in

(USE w = 0.625 in) < w_{MAX} = $(\phi\ 0.6\ F_u\ t) / (\phi\ 0.707\ F_{EXX})$ = (0.75 x 0.6 x 58 ksi) t / (0.75 x 0.707 x 70 ksi)

[Satisfactory] = 1.1795 t = 0.74 in

DETERMINE REQUIRED WELD LENGTH AT BRACE (AISC 360-05 J2.4)

$L = \Omega\ T / [(4)\ (0.6)\ F_{EXX}\ (0.707\ w)]$

= (2.0) (250.00) / [(4) (0.6) (70) (0.707 x 5/8)] = 6.74 in **(USE L = 8.00 in)**

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE PER $T_{t,rup,brace}$ ABOVE (AISC 360-05 Tab. J2.4)

t_g = 1 in **(USE t_g = 1 in)**

DETERMINE GUSSET DIMENSIONS BASED ON GEOMETRY

L_g = 22.3 in 2β = 26.8 in 2α = 14.6 in

DETERMINE CONNECTION INTERFACE FORCES (AISC Manual 13th Edition, Page 13-10)

β = 13.39 in

α = 7.32 in > $(e_b + \beta) \tan\theta - e_c$ = 3.69 in

[The original Uniform Force Method may not apply]

$K = e_b \tan\theta - e_c$ = -2.55 in

$D = \tan^2\theta + (\alpha / \beta)^2$ = 0.5166

$K' = \alpha\ (\tan\theta + \alpha / \beta)$ = 7.4191

$\alpha_{Ideal} = (K' \tan\theta + K\ (\alpha / \beta)^2) / D$ = 5.22 in

$\beta_{Ideal} = (\alpha_{Ideal} - K) / \tan\theta$ = 16.66 in

$r = [(e_b + \beta_{Ideal})^2 + (e_c + \alpha_{Ideal})^2]^{0.5}$ = 27.38 in

$V_c = (\beta_{Ideal} / r)\ T$ = 152.2 kips

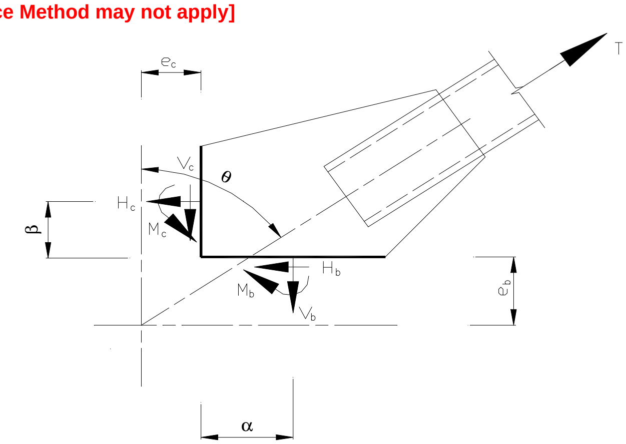
$H_c = (e_c / r)\ T$ = 58.0 kips

$M_c = H_c\ [\ \beta_{Ideal} - \beta\ ]$ = 15.8 ft-kips

$V_b = (e_b / r)\ T$ = 74.4 kips

$H_b = (\alpha_{Ideal} / r)\ T$ = 47.7 kips

$M_b = V_b\ [\ \alpha_{Ideal} - \alpha\ ]$ = -13.0 ft-kips



CHECK WELD CAPACITY AT INTERFACES (AISC 360-05 J2.4)

$f_{Vc} = V_c / (4\ \beta\ 0.707\ w)$ = 6.43 ksi

(cont'd)

$f_{Hc} = H_c / (4\ \beta\ 0.707\ w) =$ 2.45 ksi

$f_{Mc} = 3\ M_c / (4\ \beta^2\ 0.707\ w) =$ 1.80 ksi

$f_{Vb} = V_b / (4\ \alpha\ 0.707\ w) =$ 5.75 ksi

$f_{Hb} = H_b / (4\ \alpha\ 0.707\ w) =$ 3.68 ksi

$f_{Mb} = 3\ M_b / (4\ \alpha^2\ 0.707\ w) =$ 4.95 ksi

$\Omega =$ 2.0

$f_{v,c} = [(f_{vc})^2 + (f_{Hc} + f_{Mc})^2]^{0.5} =$ 7.71 ksi $<$ $0.6\ F_{EXX} / \Omega =$ 21.00 ksi **[Satisfactory]**

$f_{v,b} = [(f_{vb})^2 + (f_{Hb} + f_{Mb})^2]^{0.5} =$ 10.38 ksi $<$ $0.6\ F_{EXX} / \Omega =$ 21.00 ksi **[Satisfactory]**

CHECK SHEAR RUPTURE CAPACITY OF SLOTTED BRACE (AISC 360-05 J4.2)

$R_{n,rup,brace} / \Omega = (0.6F_u)A_{nu} / \Omega =$ 348 kips $>$ T **[Satisfactory]**

Where $F_u =$ 58 ksi (AISC Manual 13th Edition, Pg. 2-39)

$A_{nu} = 4\ t\ L = 4 \times 0.625 \times 8 =$ 20.00 in^2

$\Omega =$ 2.0

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 J4.2)

$R_{n,rup,gusset} / \Omega = (0.6F_u)A_{nv} / \Omega =$ 278.4 kips $>$ T **[Satisfactory]**

Where $F_u =$ 58 ksi (A36 Steel)

$A_{nv} = 2\ t_g\ L = 2 \times 1 \times 8 =$ 16.00 in^2

$\Omega =$ 2.0

CHECK TENSION CAPACITY AT SLOTTED BRACE (AISC 360-05 J4.1)

$R_t\ R_n / \Omega = R_t\ F_u\ A_e / \Omega =$ 353.29 kips $>$ T **[Satisfactory]**

Where $\Omega =$ 2

$F_u =$ 58 ksi (AISC Manual 13th Edition, Pg. 2-335)

$x = B^2 + 2BH / 4(B+H) =$ 3.00 , for Tube (AISC Tab. D3.1)

$D / \pi =$ 2.55 , for Pipe (AISC 360 Tab. D3.1)

$U = MIN(1 - x / L, 0.9) =$ 0.63 ,(AISC Tab D3.1)

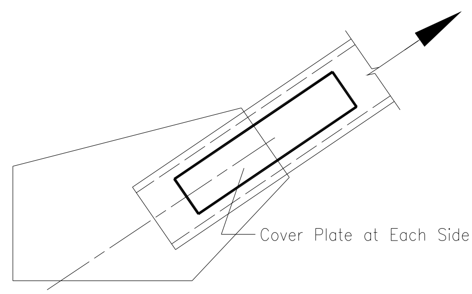
$A_n = A_g - 2\ (t_g + 1/8)\ t =$ 14.99 in^2

$A_e = U\ A_n =$ 9.37 in^2

$R_t =$ 1.3 (AISC 341-05 6.2)

Try Cover Plate 0 x 7 , at Each Sides.

(0 for no cover required)



Region	x	0.5 A_n	x A
HSS	3.00	7.50	22.49
Cover Plate	4.00	0.00	0.00
Σ		7.50	22.49

x $=$ 22.49 / 7.50 = 3.00

U $=$ MIN(1 - x / L , 0.9) = 0.63

$\mathbf{A_n} =$ 14.99 + 0.00 = 14.99 in^2

$\mathbf{A_e} = U\ A_n =$ 9.37 in^2

Thus, $R_n / \Omega = R_t\ F_u\ A_e / \Omega =$ 353.29 kips $>$ T **[Satisfactory]**

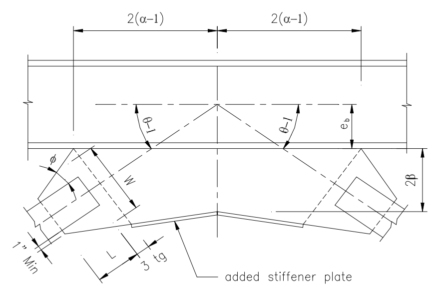
CHECK GUSSET BLOCK SHEAR CAPACITY (AISC 360-05 J4.3)

$R_{n,guss} / \Omega = Min\ [0.6F_uA_{nv}, 0.6F_yA_{gv}] / \Omega + U_{bs}F_uA_{nt} / \Omega =$ 278.4 $+ U_{bs}F_uA_{nt} / \Omega$

$>$ T = 250.0 **[Satisfactory]**

Bracing Connection Capacity at Middle of Beam Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY



Case A

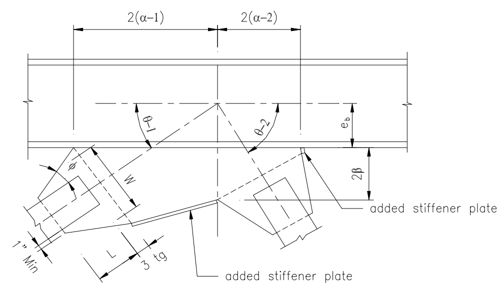
BRACE AXIAL LOAD AT SERVICE LEVEL T-1 = **250** kips
T-2 = **-230** kips

ANGLE BETWEEN BRACE & BEAM θ - 1 = **35** ^{0}
θ - 2 = **65** ^{0}

ANGLE BTW BRACE & GUSSET EDGE ϕ = **20** ^{0}

BRACE SECTION (Tube or Pipe) = > **HSS8X8X5/8**

Tube	A	r_{min}	t	b	h
	16.40	2.98	0.63	8.00	8.00



Case B

BEAM SECTION = > **W16X67**

ORIENTATION = **x-x** , e_b = **8.15** in

(1" Gusset Plate with 5/8" Fillet Weld, 4 leg x 8" Length at Brace,
and 2 leg x 42" at Beam Interface.)

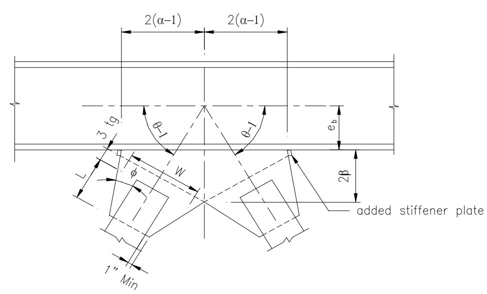
THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE GUSSET DIMENSIONS BASED ON GEOMETRY

<u>**Case B**</u>

2β = 10.19 in
$2(\alpha - 1)$ = 27.34 in
$2(\alpha - 2)$ = 13.73 in
L_g = 15.86 in, the max buckling length
W = 13.82 in, the Whitmore width



Case C

DETERMINE BEST FILLET WELD SIZE PER BRACE THICKNESS
(AISC 360-05 J2.2b)

w = 0.625 in
(USE w = 0.625 in) **>** w_{MIN} = 0.25 in
< w_{MAX} = (ϕ 0.6 F_u t) / (ϕ 0.707 F_{EXX}) = (0.75 x 0.6 x 58 ksi) t / (0.75 x 0.707 x 70 ksi)
[Satisfactory] = 1.1795 t = 0.74 in

DETERMINE REQUIRED WELD LENGTH AT BRACE (AISC 360-05 J2.4)

L = Ω T / [(4) (0.6) F_{EXX} (0.707 w)]
= (2.0) (250.00) / [(4) (0.6) (70) (0.707 x 5/8)] = 6.74 in **(USE L = 8.00 in)**

DETERMINE REQUIRED THICKNESS OF GUSSET PLATE PER $T_{t,rup,brace}$ ABOVE (AISC 360-05 Tab. J2.4)

t_g = 1 in **(USE t_g = 1 in)**

DETERMINE CONNECTION INTERFACE FORCES (AISC Manual 13th Edition, Page 13-10)

β = 5.10 in e_c = 0 in
α - 1 = 13.67 in α - 2 = 6.87 in

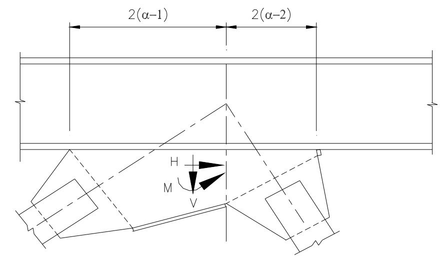
<u>**For T-1**</u>			<u>**For T-2**</u>		
K = e_b tanθ - e_c =	5.71	in	K = e_b tanθ - e_c =	17.48	in
D = tan$^2\theta$ + (α / β)^2 =	7.6824		D = tan$^2\theta$ + (α / β)^2 =	6.4145	
K' = α (tanθ + α / β) =	46.227		K' = α (tanθ + α / β) =	23.981	
α_{Ideal} = (K' tanθ + K (α / β)^2) / D =	9.56	in	α_{Ideal} = (K' tanθ + K (α / β)^2) / D =	12.96	in
β_{Ideal} = (α_{Ideal} - K) / tanθ =	5.50	in	β_{Ideal} = (α_{Ideal} - K) / tanθ =	-2.10	in
r = [(e_b + β_{Ideal})^2 + (e_c + α_{Ideal})^2] $^{0.5}$ =	16.66	in	r = [(e_b + β_{Ideal})^2 + (e_c + α_{Ideal})^2] $^{0.5}$ =	14.30	in
V_c = (β_{Ideal} / r) T =	82.5	kips	V_c = (β_{Ideal} / r) T =	33.8	kips
H_c = (e_c / r) T =	0.0	kips	H_c = (e_c / r) T =	0.0	kips
M_c = H_c [β_{Ideal} - β] =	0.0	ft-kips	M_c = H_c [β_{Ideal} - β] =	0.0	ft-kips

(cont'd)

$V_b = (e_b / r)\ T =$ 122.3 kips $V_b = (e_b / r)\ T =$ -131.0 kips

$H_b = (\alpha_{Ideal} / r)\ T =$ 143.4 kips $H_b = (\alpha_{Ideal} / r)\ T =$ -208.5 kips

$M_b = V_b\ [\ \alpha_{Ideal} - \alpha\ ] =$ -41.9 ft-kips $M_b = V_b\ [\ \alpha_{Ideal} - \alpha\ ] =$ -66.6 ft-kips



CHECK WELD CAPACITY AT INTERFACES (AISC 360-05 J2.4)

$V =$ 8.7 kips

$H =$ 351.8 kips

$M =$ 27.1 ft-kips

$f_{Vb} = V / (4\ \alpha\ 0.707\ w) =$ 0.36 ksi

$f_{Hb} = H / (4\ \alpha\ 0.707\ w) =$ 14.56 ksi

$f_{Mb} = 3\ M / (4\ \alpha^2\ 0.707\ w) =$ 2.96 ksi

$\Omega =$ 2.0

$f_{v,b} = [(f_{vb})^2 + (f_{Hb} + f_{Mb})^2]^{0.5} =$ 17.53 ksi $<$ $0.6\ F_{EXX} / \Omega =$ 21.00 ksi **[Satisfactory]**

CHECK SHEAR RUPTURE CAPACITY OF SLOTTED BRACE (AISC 360-05 J4.2)

$R_{n,rup,brace} / \ \Omega = (0.6F_u)A_{nu} / \Omega =$ 348 kips $>$ T

[Satisfactory]

Where $F_u =$ 58 ksi (AISC Manual 13th Edition, Pg. 2-39)

$A_{nu} = 4\ t\ L = 4 \times 0.625 \times 8 =$ 20.00 in^2

$\Omega =$ 2.0

CHECK SHEAR RUPTURE CAPACITY OF GUSSET PLATE (AISC 360-05 J4.2)

$R_{n,rup,gusset} / \Omega = (0.6F_u)A_{nv} / \Omega =$ 278.4 kips $>$ T **[Satisfactory]**

Where $F_u =$ 58 ksi (A36 Steel)

$A_{nv} = 2\ t_g\ L = 2 \times 1 \times 8 =$ 16.00 in^2

$\Omega =$ 2.0

CHECK TENSION CAPACITY AT SLOTTED BRACE (AISC 360-05 J4.1)

$R_t\ R_n / \Omega = R_t\ F_u\ A_e / \Omega =$ 353.29 kips $>$ T **[Satisfactory]**

Where $\Omega =$ 2

$F_u =$ 58 ksi (AISC Manual 13th Edition, Pg. 2-335)

$x = B^2 + 2BH / 4(B+H) =$ 3.00 , for Tube (AISC Tab. D3.1)

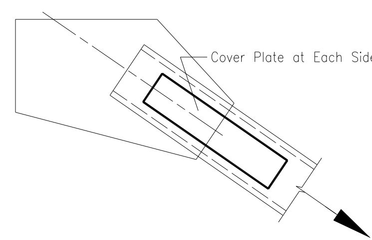
$D / \pi =$ 2.55 , for Pipe (AISC 360 Tab. D3.1)

$U = MIN(\ 1 - x\ / L\ , 0.9\ ) =$ 0.63 ,(AISC Tab D3.1)

$A_n = A_g - 2\ (t_g + 1/8)\ t =$ 14.99 in^2

$A_e = U\ A_n =$ 9.37 in^2

$R_t =$ 1.3 (AISC 341-05 6.2)



Try Cover Plate 0 x 7 , at Each Sides.

(0 for no cover required)

Region	**x**	**0.5 A_n**	**x A**
HSS	3.00	7.50	22.49
Cover Plate	4.00	0.00	0.00
Σ		7.50	22.49

x $=$ 22.49 / 7.50 = 3.00

U $= MIN(\ 1 - x\ / L\ , 0.9\ ) =$ 0.63

$\mathbf{A_n} =$ 14.99 + 0.00 = 14.99 in^2

$\mathbf{A_e} = U\ A_n =$ 9.37 in^2

Thus, $R_n / \Omega = R_t\ F_u\ A_e / \Omega =$ 353.29 kips $>$ T **[Satisfactory]**

CHECK GUSSET BLOCK SHEAR CAPACITY (AISC 360-05 J4.3)

$R_{n,guss} / \Omega = Min\ [0.6F_uA_{nv}\ ,\ 0.6F_yA_{gv}] / \Omega + U_{bs}F_uA_{nt} / \Omega =$ 278.4 $+ U_{bs}F_uA_{nt} / \Omega$

$>$ T = 250.0 **[Satisfactory]**

Daniel T. Li
Engineering International

PROJECT :
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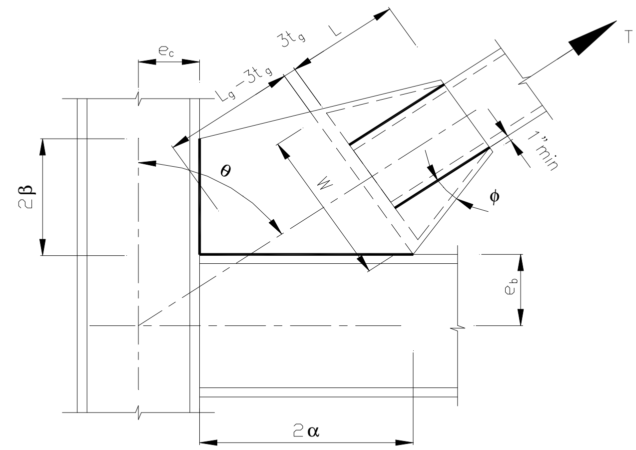
Corner Gusset Plate Dimensions Generator



INPUT DATA

ANGLE BETWEEN BRACE & COLUMN	θ =	**44.72**	0
ANGLE BTW BRACE & GUSSET EDGE	ϕ =	**30**	0
BRACE SECTION (Tube or Pipe)	=>	**HSS8X8X5/8**	
Tube	h =	8.00	in
	(USE h =	**8.00**	**in)**
COLUMN SECTION	=>	**W12X96**	
ORIENTATION = **y-y**	, e_c =	0.275	in
	(USE e_c =	**0.28**	**in)**
BEAM SECTION	=>	**W18X65**	
ORIENTATION = **x-x**	, e_b =	9.2	in
	(USE e_b =	**9.20**	**in)**
THICKNESS OF GUSSET PLATE	t_g =	**1**	in
WELD LENGTH AT BRACE	L =	**15**	in

ANALYSIS

2β = 19.31 in, the interface dimension between gusset and column
2α = 30.50 in, the interface dimension between gusset and beam
L_g = 18.24 in, the average buckling length
W = 25.32 in, the Whitmore width

β = 9.65 in
α = 15.25 in **<** $(e_b + \beta)\tan\theta - e_c$ = 18.39 in, (AISC Manual 13th Edition, Page 13-10)

[The original Uniform Force Method may not apply]

Daniel T. Li
Engineering International

PROJECT :		PAGE :	
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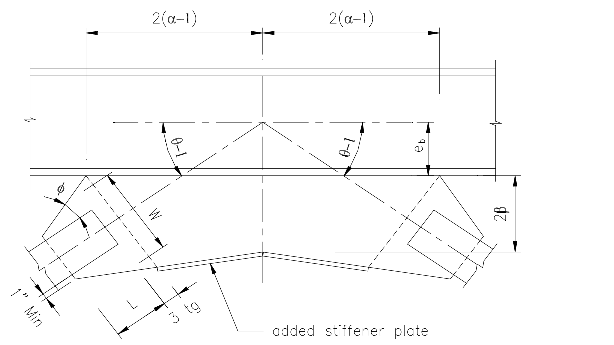
JOB NO. :	DATE :	REVIEW BY :	

Middle of Beam Gusset Plate Dimensions Generator

INPUT DATA

ANGLE BETWEEN BRACE & BEAM	θ - 1 =	35	^{0}
	θ - 2 =	65	^{0}
ANGLE BTW BRACE & GUSSET EDGE	ϕ =	25	^{0}
BRACE SECTION (Tube or Pipe)	= >	HSS8X8X5/8	
Tube	h =	8.00	in
	(USE h =	**8.00**	**in)**
BEAM SECTION	= >	W21X68	
ORIENTATION = x-x	, e_b =	10.55	in
	(USE e_b =	**10.55**	**in)**
THICKNESS OF GUSSET PLATE	t_g =	1	in
WELD LENGTH AT BRACE	L =	10	in

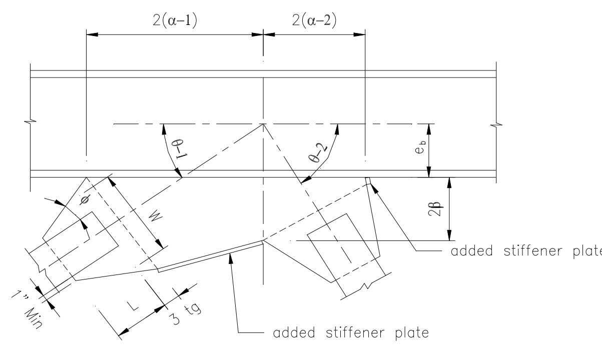


Case A

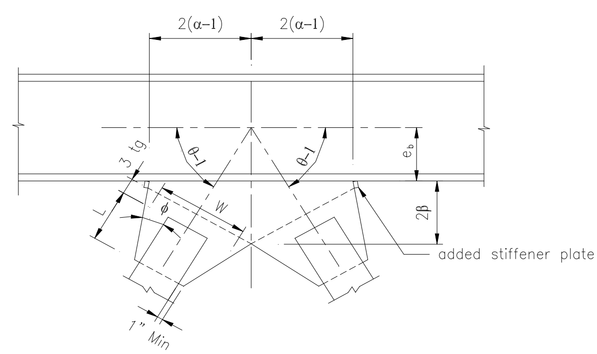
ANALYSIS

Case B

2 β =	11.81	in
2 (α - 1) =	34.35	in
2 (α - 2) =	17.13	in
L_g =	18.80	in, the max buckling length
W =	17.33	in, the Whitmore width



Case B



Case C

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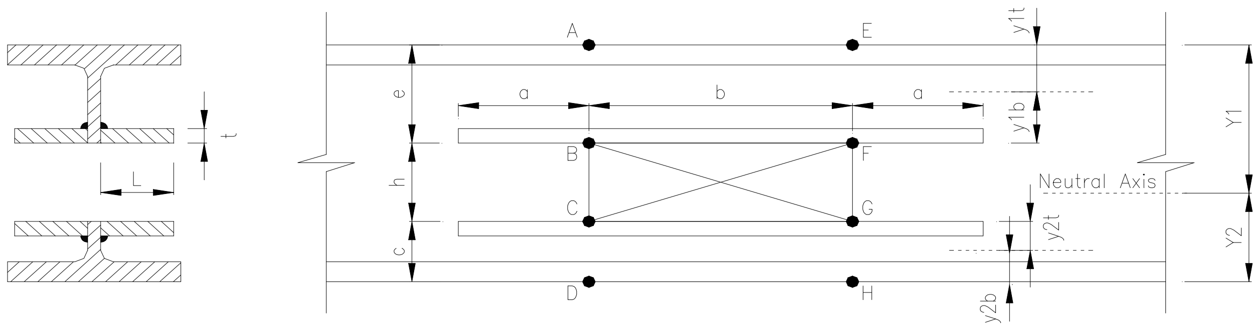
PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Check Capacity of WF Beam at Opening Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY

WF SECTION		= >	**W24x76**		OPENING DIMENSIONS	b	=	**48**	in
MOMENT @ ABCD SECTION	M_{ABCD}	=	**90**	ft-k		h	=	**10**	in
MOMENT @ EFGH SECTION	M_{EFGH}	=	**75**	ft-k	OPENING LOCATION	e	=	**8**	in
MAX SHEAR @ OPENING	V	=	**60**	kips	PLATE SIZE @ EACH SIDE	t	=	**0.75**	in
BEAM YIELD STRESS	F_y	=	**50**	ksi		L	=	**4.25**	in
TRIAL WELD SIZE	w	=	**1/4**	in					

USE (4) - 3/4" x 4-1/4" x 7' -2" PLATES, WITH WELD 1/4" AT EACH SIDES, TOP & BOTTOM.



ANALYSIS

DATA FOR ROLLED SECTION CHOSEN

A	d	t_w	b_f	t_f	S_x
22.4	23.9	0.44	8.99	0.68	176

$F_b = F_y / \Omega_b = F_y / 1.67 =$ 29.94 ksi, (AISC 360-05, F1 & F5)

$F_v = 0.6\ F_y / \Omega_v = 0.6\ F_y / 1.67 =$ 17.96 ksi, (AISC 360-05, G2)

PROPERTIES OF OPENING SECTION

t	L	y_{1t}	y_{1b}	y_{2t}	y_{2b}	A_{top}	A_{bott}	Y_1	Y_2	I_{top}	I_{bott}	I_{total}
0.75	4.25	4.12	3.88	2.87	3.03	15.7	14.8	12.2	11.7	173	89	2,398

CHECK BENDING STRESSES

V_{top}	V_{bott}	$M_{s,top}$	$M_{s,bott}$
34.53	25.47	69.1	50.9

MAIN BENDING STRESSES			SECONDARY BENDING STRESSES			TOTAL BENDING STRESSES		
$\sigma1_A$ =	-5.51	ksi	$\sigma2_A$ =	-19.17	ksi	$(\sigma1+\sigma2)$ A =	-24.68	ksi
$\sigma1_B$ =	-1.91	ksi	$\sigma2_B$ =	19.17	ksi	$(\sigma1+\sigma2)$ B =	17.26	ksi
$\sigma1_C$ =	2.59	ksi	$\sigma2_C$ =	-20.34	ksi	$(\sigma1+\sigma2)$ C =	-17.74	ksi
$\sigma1_D$ =	5.25	ksi	$\sigma2_D$ =	20.34	ksi	$(\sigma1+\sigma2)$ D =	25.59	ksi
$\sigma1_E$ =	-4.59	ksi	$\sigma2_E$ =	19.17	ksi	$(\sigma1+\sigma2)$ E =	14.57	ksi
$\sigma1_F$ =	-1.59	ksi	$\sigma2_F$ =	-19.17	ksi	$(\sigma1+\sigma2)$ F =	-20.76	ksi
$\sigma1_G$ =	2.16	ksi	$\sigma2_G$ =	20.34	ksi	$(\sigma1+\sigma2)$ G =	22.50	ksi
$\sigma1_H$ =	4.38	ksi	$\sigma2_H$ =	-20.34	ksi	$(\sigma1+\sigma2)$ H =	-15.96	ksi

Max f_b = Max $(\sigma1+\sigma2)$ = 25.59 ksi < 29.94 ksi [Satisfactory]

DETERMINE STIFFENER EXTENSIONS

Max bending stress f_b @ stiffener = 22.5 ksi

Force, F = 8.5 x 0.75 x 22.50 = 143.4 k

Allow stress in web = 17.96 ksi

Extension = 143.4 / (0.44 x 17.96) = 18.1 in

Say => 19.0 in

CHECK WELDING

Weld width, w = 0.25 in   Min weld = 0.25 in   Max weld = 0.69 in

V_{top}	Q_{top}	I_{top}	q_{top}
34.53	23.5	173	4.70

q critical = 6.40 k / in

$t_e = 0.707w$ = 0.18 in

$q / 4\ t_e$ = 9.05 ksi

V_{bott}	Q_{bott}	I_{bott}	q_{bott}
25.47	22.3	89	6.40

< $F_v = 0.6\ F_u / \Omega = 0.6 \times 70\ ksi / 2.0 =$ 21.00 ksi

[Satisfactory] (AISC 360-05, J4.2)

Purchaser's LOGO
(Please email us your pdf LOGO at purchase.)

PROJECT :
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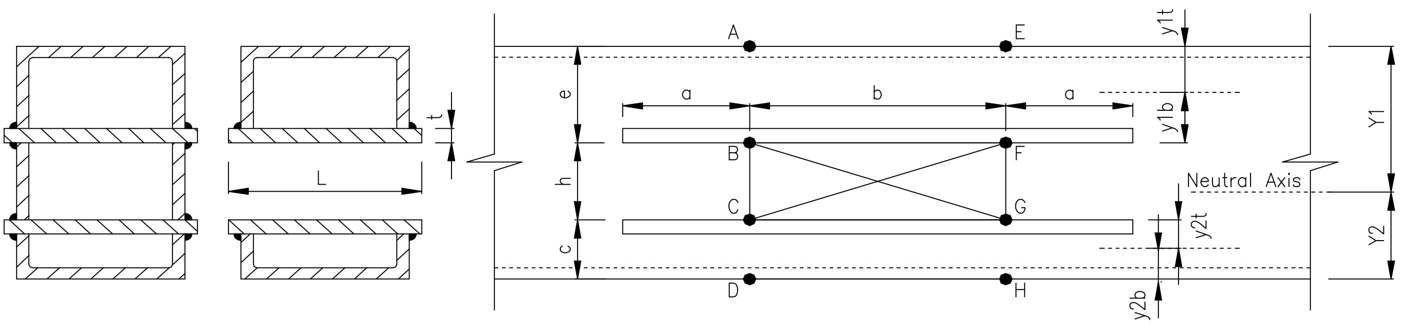
Check Capacity of HSS Tube Beam at Opening Based on AISC 360-16

INPUT DATA & DESIGN SUMMARY

HSS SECTION		= >	**HSS20X12X1/2**		OPENING DIMENSIONS	b =	**48**	*in (1219 mm)*
MOMENT @ ABCD SECTION	M_{ABCD}	=	**65**	*ft-kips, (88 kN-m)*		h =	**6**	*in (152 mm)*
MOMENT @ EFGH SECTION	M_{EFGH}	=	**55**	*ft-kips, (75 kN-m)*	OPENING LOCATION	e =	**8**	*in (203 mm)*
MAX SHEAR @ OPENING	V	=	**60**	*kips, (267 kN)*		c =	6.0	*in (152 mm)*
BEAM YIELD STRESS	F_y	=	**50**	*ksi, (345 MPa)*	PLATE SIZE @ EACH SIDE	t =	**0.75**	*in (19 mm)*
TRIAL WELD SIZE	w	=	**1/4**	*in (6 mm)*		L =	13.5	*in (343 mm)*
						a =	10.0	*in (254 mm)*

USE (2) - 3/4" x 13.5" x 5' -8" PLATES, WITH WELD 1/4" AT EACH SIDES, TOP & BOTTOM.



ANALYSIS

DATA FOR ROLLED SECTION CHOSEN

A	d	2 t_w	b_f	t_f	S_x
28.3	20	1	12	0.5	155

$F_b = F_y / \Omega_b = F_y / 1.67 =$ 29.94 ksi, (AISC 360 F1 & F5)

$F_v = 0.6\ F_y / \Omega_v = 0.6\ F_y / 1.67 =$ 17.96 ksi, (AISC 360 G4)

PROPERTIES OF OPENING SECTION

t	$0.5(L-2t_w)$	y_{1t}	y_{1b}	y_{2t}	y_{2b}	A_{top}	A_{bott}	Y_1	Y_2	I_{top}	I_{bott}	I_{total}
0.75	6.25	4.58	3.42	2.55	3.45	22.9	20.9	10.3	9.7	201	114	1,877

CHECK BENDING STRESSES

V_{top}	V_{bott}	$M_{s,top}$	$M_{s,bott}$
34.29	25.71	68.6	51.4

MAIN BENDING STRESSES				SECONDARY BENDING STRESSES				TOTAL BENDING STRESSES				
σ1 A	=	-4.28	ksi	σ2 A	=	-16.35	ksi	(σ1+σ2)	A	=	-20.62	ksi
σ1 B	=	-0.95	ksi	σ2 B	=	16.35	ksi	(σ1+σ2)	B	=	15.39	ksi
σ1 C	=	1.54	ksi	σ2 C	=	-16.22	ksi	(σ1+σ2)	C	=	-14.68	ksi
σ1 D	=	4.03	ksi	σ2 D	=	16.22	ksi	(σ1+σ2)	D	=	20.26	ksi
σ1 E	=	-3.62	ksi	σ2 E	=	16.35	ksi	(σ1+σ2)	E	=	12.73	ksi
σ1 F	=	-0.81	ksi	σ2 F	=	-16.35	ksi	(σ1+σ2)	F	=	-17.15	ksi
σ1 G	=	1.30	ksi	σ2 G	=	16.22	ksi	(σ1+σ2)	G	=	17.53	ksi
σ1 H	=	3.41	ksi	σ2 H	=	-16.22	ksi	(σ1+σ2)	H	=	-12.81	ksi

Max f_b = Max (σ1+σ2) **=** 20.62 ksi **<** 29.94 ksi [Satisfactory]

DETERMINE STIFFENER EXTENSIONS

Max bending stress f_b @ stiffener						=	17.5	ksi
Force, F =	12.5	x	0.75	x	17.53	=	164.3	k
Allow stress in web						=	17.96	ksi
Extension =	164.3	/ (	1	x	17.96)	=	9.1	in
					Say =>	***a* =**	10.0	in

CHECK WELDING

Weld width, w = 0.25 in   Min weld = 0.25 in   Max weld = 0.69 in

V_{top}	Q_{top}	I_{top}	q_{top}
34.29	30.8	201	5.24

V_{bott}	Q_{bott}	I_{bott}	q_{bott}
25.71	29.3	114	6.60

q critical **=** 6.60 k / in

$t_e = 0.707w$ **=** 0.18 in

$q / 4\ t_e$ **=** 9.34 ksi

< $F_v = 0.6\ F_u / \Omega = 0.6 \times 70\ \text{ksi} / 2.0 =$ 21.00 ksi

[Satisfactory] (AISC 360 J4.2)

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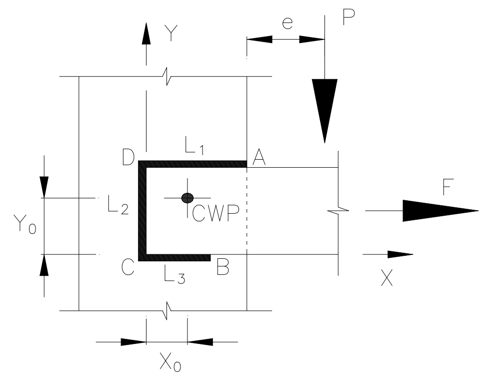
Weld Capacity of Eccentric Connection Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY

THICKER PART JOINTED	t =	**0.75**	in
WELD SIZE	w =	**0.25**	in
GRAVITY LOAD, ASD	P =	**10.7**	kips
ECCENTRICITY TO EDGE	e =	**30**	in
LATERAL LOAD, ASD	F =	**5**	kips
WELD LENGTH, DA	L_1 =	**4**	in
WELD LENGTH, DC	L_2 =	**18**	in
WELD LENGTH, CB	L_3 =	**2**	in



THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

MIN WELD SIZE (AISC 360-05 Tab. J2.4)	w_{min} =	0.25 in			
MAX WELD SIZE (AISC 360-05 J2.2b)	w_{max} =	0.69 in			
EFFECTIVE THROAT THICKNESS	t_e =	$0.707\ w$	=	0.18	in
CENTROID OF WELD GROUP	X_0 =	$\Sigma X_i A_i / \Sigma A_i$	=	0.42	in
	Y_0 =	$\Sigma Y_i A_i / \Sigma A_i$	=	9.75	in
CENTRODIAL MOMENT OF INERTIA	I_x =	$\Sigma (b h^3 / 12 + A d^2)$	=	169	in^4
	I_y =	$\Sigma (b h^3 / 12 + A d^2)$	=	4	in^4
TOTAL ECCENTRICITY	$e_{total,\ P}$ =	$e + L_1 - X_0$	=	33.6	in
	$e_{total,\ F}$ =	$0.5\ L_2 - Y_0$	=	-0.8	in
ALLOWABLE STRESS (AISC 360-05 J2.4)	F_v =	$0.6\ F_{EXX} / \Omega$	=	21.0	ksi

DETERMINE SHEAR STRESS @ POINTS A, B, C, D

Point A :	$f_x = F / A_w + \Sigma M\ y_A / (I_x + I_y)$ =	18.1 ksi	$f_y = -P / A_w - \Sigma M\ x_A / (I_x + I_y)$ =	-9.9 ksi
Point B :	$f_x = F / A_w - \Sigma M\ y_B / (I_x + I_y)$ =	-18.9 ksi	$f_y = -P / A_w - \Sigma M\ x_B / (I_x + I_y)$ =	-5.8 ksi
Point C :	$f_x = F / A_w - \Sigma M\ y_C / (I_x + I_y)$ =	-18.9 ksi	$f_y = -P / A_w + \Sigma M\ x_C / (I_x + I_y)$ =	-1.7 ksi
Point D :	$f_x = F / A_w + \Sigma M\ y_D / (I_x + I_y)$ =	18.1 ksi	$f_y = -P / A_w + \Sigma M\ x_D / (I_x + I_y)$ =	-1.7 ksi

CHECK SHEAR CAPACITY @ POINTS A, B, C, D

Point A :	$f_v = (f_x^2 + f_y^2)^{0.5}$ =	20.7 ksi	**<**	F_v =	21.0 ksi	**[Satisfactory]**
Point B :	$f_v = (f_x^2 + f_y^2)^{0.5}$ =	19.7 ksi	**<**	F_v =	21.0 ksi	**[Satisfactory]**
Point C :	$f_v = (f_x^2 + f_y^2)^{0.5}$ =	18.9 ksi	**<**	F_v =	21.0 ksi	**[Satisfactory]**
Point D :	$f_v = (f_x^2 + f_y^2)^{0.5}$ =	18.2 ksi	**<**	F_v =	21.0 ksi	**[Satisfactory]**

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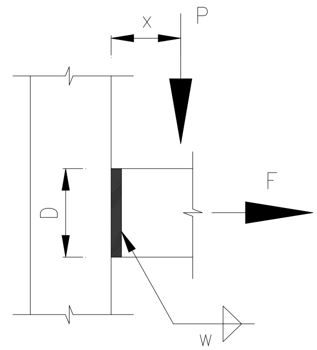
Weld Capacity of Eccentric Connection Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY

THICKER PART JOINTED	t =	**0.75**	in
WELD SIZE	w =	**0.375**	in
ECCENTRICITY TO EDGE	x =	**6**	in
WELD LENGTH	D =	**10**	in
GRAVITY LOAD, ASD	P =	**29**	kips
LATERAL LOAD, ASD	F =	**5**	kips



THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

MIN WELD SIZE (AISC 360-05 Tab. J2.4) w_{min} = 0.25 in

MAX WELD SIZE (AISC 360-05 J2.2b) w_{max} = 0.69 in

EFFECTIVE THROAT THICKNESS	t_e =	0.707 w	=	0.27	in
CENTRODIAL MOMENT OF INERTIA	I_x =	$2\,(t_e\,D^3 / 12)$	=	44.2	in^4
DIRECT SHEAR STRESS	f_x =	$F / 2\,D\,t_e$	=	0.9	ksi
	f_y =	$P / 2\,D\,t_e$	=	5.5	ksi
BENDING STRESS	f_x =	$D\,P\,x / 2\,I_x$	=	19.7	ksi
RESULTANT STRESS	f_v =	$[(\Sigma f_x)^2 + f_y^{\,2})^{0.5}$	=	20.7	ksi

ALLOWABLE STRESS (AISC 360-05 J2.4) F_v = $0.6\,F_{EXX} / \Omega$ = 21.0 ksi

> f_v = 20.7 ksi **[Satisfactory]**

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Design of 1 1/2" Type "B" Roof Deck Based on ICBO ER-2078P

INPUT DATA & DESIGN SUMMARY

NO. OF SPANS (1,2 or 3)	n =	2		**USE:**
DECK VERT. SPAN LENGTH	l =	9	ft	1 1/2" x 20 GA. VERCO PLB-36/HSB-36 GALVANIZED ROOF DECK
GAGE (22,20,18,16) ? =>	== >	20	GA	(2 SPANS MINS)
DEAD LOAD	DL =	20	psf	5 -1/2 "Ø PUDDLE WELDS PER SHEET, EACH SUPPORT.
LIVE LOAD	LL =	20	psf	1/2"Ø PUDDLE WELD @ 12" O.C. EACH PARALLEL SUPPORT.
DIAPHRAGM HORIZ SPAN	L =	150	ft	SIDELAP TOP SEAM WELD (TSW) @ 12" O.C.
DIAPHRAGM HORIZ DEPTH	d =	50	ft	
THE MAX DIAPHRAGM SHEAR	v =	680	plf	**(THE DIAPHRAGM DEFLECTION, 0.69 in, AT MIDDLE SPAN.)**
NO. OF SUPPORT WELD (4, 5 or 7)	== >	5	per sheet	
SPACING OF PUDDLE WELD	== >	12	in o.c.	
SIDE LAP TYPE (0=BP, 1=TSW)		1	Top Seam Weld	
SPACING OF SIDELAP CONNECTION		12	in o.c.	

ANALYSIS

PLB & HSB SECTION PROPERTIES (ER-2078P, Table 4, page 3)

GAGE	thk, in	I, in^4/ft	+S, in^3/ft	-S, in^3/ft	Wt, psf
16	0.0598	0.377	0.411	0.417	3.5
18	0.0478	0.302	0.322	0.335	2.9
20	0.0359	0.216	0.235	0.248	2.3
22	0.0299	0.175	0.187	0.198	1.9

PUDDLE WELDS ALLOWABLE DIAPHRAGM SHEAR (ER-2078P, Table 1, page 2)

GAGE	6" o.c.	9" o.c.	12" o.c.	18" o.c.
16	4186	2791	2093	1395
18	3346	2231	1673	1115
20	2513	1675	1257	838
22	2093	1395	1047	698

HSB-36 ALLOWABLE DIAPHRAGM SHEAR, q (plf), AND FLEXIBILITY FACTORS, F (ER-2078P, Table 20, page 40-47)

SUPPORT	GAGE	BP		4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
36/5	20	24	q	690	675	590	516	447	405	361	334	`
36/5	20	24	F	4.0+91R	5.1+73R	6.5+61R	8.2+52R	10.4+45R	12.8+40R	15.9+36R	19.0+33R	23.1+30R
36/5	20	12	q	762	733	656	568	501	450	408	374	346
36/5	20	12	F	3.9+91R	4.9+73R	6.1+61R	7.7+52R	9.5+45R	11.7+40R	14.3+36R	17.2+33R	20.6+30R
		TSW										
36/5	20	24	q	991	872	730	679	598	572	518	504	465
36/5	20	24	F	20.5+43R	18.2+34R	23.7+28R	21.5+24R	26.2+21R	24.1+19R	28.3+17R	26.3+16R	30.2+14R
36/5	20	18	q	1084	941	788	727	680	608	583	561	518
36/5	20	18	F	13.4+43R	13.4+34R	17.2+28R	16.6+24R	16.3+21R	19.4+19R	18.8+17R	18.5+16R	21.1+14R
36/5	20	12	q	1169	1006	895	816	756	709	672	641	615
36/5	20	12	F	10.0+43R	10.6+34R	11.0+28R	11.4+24R	11.8+21R	12.1+19R	12.4+17R	12.6+16R	12.8+14R
36/5	20	6	q	1469	1293	1174	1088	1023	972	931	818	688
36/5	20	6	F	5.4+43R	5.5+34R	5.6+28R	5.7+24R	5.8+21R	5.9+19R	5.9+17R	6.0+16R	6.0+14R

CHECK VERTICAL BENDING CAPACITY

$$f_b = \begin{cases} \dfrac{0.125wl^2}{+S}\ , & for\ \ Simple\ \ Span \\ \dfrac{-0.125wl^2}{-S}\ , & for\ \ Double\ \ Spans \\ \dfrac{-0.1wl^2}{-S}\ , & for\ \ Triple\ \ Spans \end{cases} \quad = \quad 20.72$$

ksi, (Vero PunchLok Book, page 5.)

< F_b [Satisfactory]

Where w = (DL + Wt) + LL = 42 psf

F_b = 22.8 ksi, (Vero PunchLok Book, page 4.)

CHECK VERTICAL DEFLECTION

$$\Delta_{LL} = \begin{cases} \dfrac{0.013w_{LL}l^4C_d}{EI}\ , & for\ \ Simple\ \ Span \\ \dfrac{0.0054w_{LL}l^4C_d}{EI}\ , & for\ \ Double\ \ Spans \\ \dfrac{0.0069w_{LL}l^4C_d}{EI}\ , & for\ \ Triple\ \ Spans \end{cases} \quad = \quad 0.19$$

in, (Vero PunchLok Book, page 5.)

< l / 240 = 0.45 in [Satisfactory]

Where w_{LL} = 20 psf, as given

C_d = 1728 , (Vero PunchLok Book, page 5.)

E = 29500 ksi, , (from ER-2078P, page 6)

CHECK HORIZONTAL DIAPHRAGM SHEAR CAPACITY

v = 680 plf, as given < v_{allow} = 709 plf, (for 36/5 & TSW from table above.) [Satisfactory]

< v_{allow} = 1257 plf, (for Puddle Weld from table above.) [Satisfactory]

DETERMINE HORIZONTAL DIAPHRAGM DEFLECTION

$$\Delta = \Delta_f + \Delta_w = \frac{5wL^4(12)^3}{384EI} + \frac{wL^2F}{8\times(10)^6 d} = 0.0600 + 0.6316 = 0.6916$$

in, (from ER-2078P, page 6)

Where w = 2 d v / L = 453 plf

R = (4 - n) / 3 = 0.667 , (from ER-2078P, page 39 footnotes.)

F = 12.1+19R = 24.77 , for TSW connection.

A = 16.2 in^2, steel chord member area.

I = 0.5 A d^2 = 2916000 in^2, (ER-2078 page 6)

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DEPRESSED FLOOR DECK CAPACITY USING STEEL PROPERTIES ONLY (NON COMPOSITE)

DEAD LOAD	DL =	70	psf (including partitions)
LIVE LOAD	LL =	50	psf (non-reducible)
SPAN LENGTH	L =	8	ft
GAGE (22, 20, 18, 16)	GA =	18	
DECK TYPE (B, W2, W3, N)	TYPE =	W2	
DEFLECTION LIMIT OF LIVE LOAD	Δ_{LL} = L /	240	

THE DEPRESSED FLOOR DECK IS ADEQUATE.

SECTION PROPERTIES & ALLOWABLE REACTIONS (See Tables below)

Type	Gage	I (in^4/ft)	+S (in^3/ft)	-S (in^3/ft)	F_b	2" end (lbs/ft)	3" end	4" end	3" mid	4" mid (lbs/ft)
W2	18	0.555	0.51	0.511	22.8	533	613	693	1587	1807

For one span,

M_{max} = 0.125 (DL+LL) L^2 = 0.960 ft-kips/ft $M_{allowable}$ = F_b (+S) = 0.969 ft-kips/ft **[Satisfactory]**

$R_{end,max}$ = 480 lbs/ft (2 in bearing length required at end)

$\Delta_{max,DL+LL}$ = 5 (DL+LL) L^4 / (384 EI) = 0.7 in (L / 140)

$\Delta_{max,LL}$ = 5 (LL) L^4 / (384 EI) = 0.3 in (L / 335) **[Satisfactory]**

For two spans, uniform LL on one span

M_{max} = 0.0070 (DL) L^2 + 0.096 (LL) L^2 = 0.621 $M_{allowable}$ = F_b (+S) = 0.969 ft-kips/ft **[Satisfactory]**

$R_{end,max}$ = 385 lbs/ft (2 in bearing length required at end) $R_{mid,max}$ = 950 lbs/ft (3 in bearing length required at mid)

$\Delta_{max,DL+LL}$ = (0.00541DL + 0.0092LL) L^4 / (EI) = 0.4 in (L / 260)

$\Delta_{max,LL}$ = (0.0092LL) L^4 / (EI) = 0.2 in (L / 475) **[Satisfactory]**

For two spans, all spans loaded

$-M_{max}$ = -0.125 (DL+LL) L^2 = -0.960 ft-kips/ft $-M_{allowable}$ = F_b (-S) = -0.969 ft-kips/ft **[Satisfactory]**

$R_{end,max}$ = 360 lbs/ft (2 in bearing length required at end) $R_{mid,max}$ = 1200 lbs/ft (3 in bearing length required at mid)

$\Delta_{max,DL+LL}$ = 0.00541(DL + LL) L^4 / (EI) = 0.3 in (L / 336)

$\Delta_{max,LL}$ = (0.0054LL) L^4 / (EI) = 0.1 in (L / 807) **[Satisfactory]**

For three spans, LL partial loaded

$- M_{max}$ = - (0.100 DL+0.117 LL) L^2 = -0.822 ft-kips/ft $- M_{allowable}$ = F_b (-S) = -0.969 ft-kips/ft **[Satisfactory]**

$R_{end,max}$ = 384 lbs/ft (2 in bearing length required at end) $R_{mid,max}$ = 592 lbs/ft (3 in bearing length required at mid)

$\Delta_{max,DL+LL}$ = (0.0069DL + 0.0099LL) L^4 / (EI) = 0.4 in (L / 223)

$\Delta_{max,LL}$ = 0.0099 (LL) L^4 / (EI) = 0.2 in (L / 441) **[Satisfactory]**

B SECTION PROPERTIES (ER-2078P, Table 4)

Gage	I (in^4/ft)	+S (in^3/ft)	-S (in^3/ft)
16	0.377	0.411	0.417
18	0.302	0.322	0.335
20	0.216	0.235	0.248
22	0.175	0.187	0.198

N SECTION PROPERTIES (ER-2078P, Table 4)

Gage	I (in^4/ft)	+S (in^3/ft)	-S (in^3/ft)
16	1.647	0.950	1.005
18	1.223	0.731	0.776
20	0.837	0.508	0.562
22	0.655	0.394	0.454

W2 SECTION PROPERTIES (ER-2078P, Table 4)

Gage	I (in^4/ft)	+S (in^3/ft)	-S (in^3/ft)
16	0.694	0.639	0.639
18	0.555	0.510	0.511
20	0.423	0.361	0.370
22	0.340	0.283	0.287

W3 SECTION PROPERTIES (ER-2078P, Table 4)

Gage	I (in^4/ft)	+S (in^3/ft)	-S (in^3/ft)
16	1.509	0.960	0.960
18	1.203	0.767	0.767
20	0.896	0.534	0.564
22	0.718	0.418	0.444

B ALLOWABLE REACTION, lbs/ft (ER-2078P, Table 5)

Gage	2" end	3" end	4" end	3" mid	4" mid
16	2208	2484	2761	4789	5268
18	1226	1407	1588	3062	3551
20	665	784	903	1790	2118
22	487	585	683	1250	1498

N ALLOWABLE REACTION, lbs/ft (ER-2078P, Table 5)

Gage	2" end	3" end	4" end	3" mid	4" mid
16	1670	1879	2088	4033	4532
18	916	1051	1186	2695	3066
20	486	572	659	1582	1828
22	349	419	489	1104	1287

W2 ALLOWABLE REACTION, lbs/ft (ER-2078P, Table 5)

Gage	2" end	3" end	4" end	3" mid	4" mid
16	981	1105	1229	2380	2677
18	533	613	693	1587	1807
20	301	355	409	972	1123
22	219	263	306	684	797

W3 ALLOWABLE REACTION, lbs/ft (ER-2078P, Table 5)

Gage	2" end	3" end	4" end	3" mid	4" mid
16	942	1062	1181	2309	2597
18	507	583	659	1528	1740
20	268	316	365	877	1014
22	190	228	267	601	701

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8-Bolted Stiffened End Plate for SMF Based on AISC 341-10, 358-10, 360-10 & FEMA-350

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W14X211**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
62	15.7	0.98	15.80	1.56	338	2660	6.55	4.08	390	2.16

BEAM SECTION => **W21X62**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
18.3	21.0	0.40	8.24	0.62	127	1330	8.53	1.77	144	1.12

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

THE FACTOR GRAVITY LOAD ON THE BEAM w_u = **4.2** klf

THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **800** kips

BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft

AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **12** ft

THE SMRF DESIGN IS ADEQUATE.

(Continuity column stiffeners 5/8 x 7 with 7/16" fillet weld to web & CP to flanges. A doubler plate is not required.)

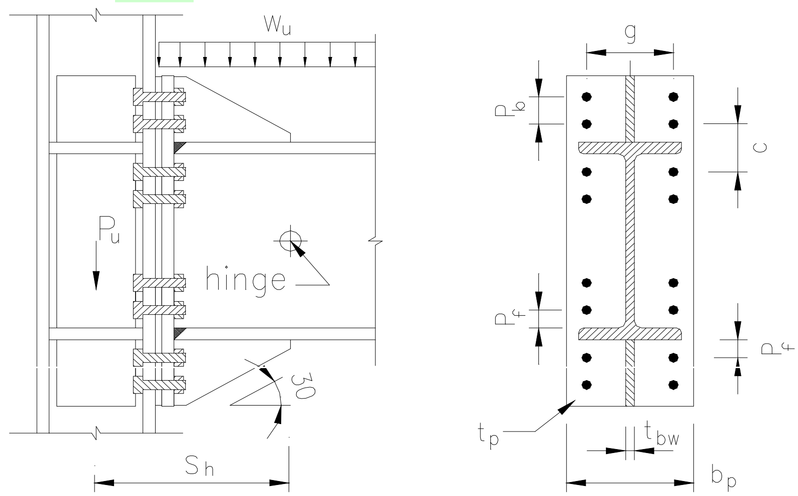
BOLTS ϕ = **1 5/8** in

GRADES (A325 or A490) **A325**

PLATE & SHIM t_p = **1 5/8** in

NUMBER COLUMNS N_c = **2** (Top & Bot)

NUMBER BEAM N_b = **1** (One Side Only)



ANALYSIS

$g = \mathrm{Max}(b_{bf} - \phi\ ,\ t_w + 3\ \phi) =$ 6.00 in

$P_b = 3\ \phi =$ 3.75 in (AISC 358 Tab 6.1)

$P_f = 1.5\ \phi =$ 2.00 in (AISC 358 Tab 6.1)

$S_h = d_c / 2 + t_p + 1" + (2P_f + P_b - 1")\ \tan^{-1} 30^o$

= 22.17 in

$c = 2\ P_f + t_{bf} =$ 4.62 in

$b_p = g + 3\ \phi =$ 10.88 in

< b_{cf} **[Satisfactory]**

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 6.70 < $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

Where E_s = 29000 ksi

$h / t_w =$ 46.90 < $2.45\ (E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 5.06 < $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

$h / t_w =$ 11.61 <
- $3.76(E_s/F_y)^{0.5}(1\text{-}2.75P_u/\phi_bP_y) =$ N/A , for $P_u/\phi_bP_y \le 0.125$
- $1.12(E_s/F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y) =$ 55.11 , for $P_u/\phi_bP_y > 0.125$

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 3100 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10 Sec. E3.4a)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*) =$ 2.59 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c\ Z_c\ (F_{yc} - P_u / A_g) =$ 2411 ft-kips

$\Sigma M_{pb}^* = N_b\ (M_{hinge} + M_v) =$ 930 ft-kips, at center of column

$M_v = V_{hinge}\ S_h = [2M_{hinge} /(L\text{-}2S_h)+w_u(L\text{-}2S_h)/2]\ S_h =$ 204 ft-kips

$M_{hinge} = C_{pr}R_yF_{yb}\ Z_b =$ 726 ft-kips

R_y = 1.1 (AISC 341-10 Tab. A3.1)

C_{pr} = 1.1 (FEMA Sec. 3.5.5.1)

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.2.1.2)

$M_f = M_{hinge} + [2M_{hinge} / (L - 2S_h) + w_u(L - 2S_h)/2]\ (S_h - d_c /2)$

= 858 ft-kips < $3.4\ T_{ub}\ (d_0 + d_i) =$ 1205 ft-kips **[Satisfactory]**

Where $d_0 = d_b + P_f - 0.5\ t_{bf} =$ 22.69 in T_b = 103 kips, (AISC 360-10, Tab. J3.1)

$d_i = d_0 - c - P_b =$ 14.33 in A_{bt} = 1.28 in^2 / bolt

$F_{fu} = M_f / (d_b - t_{bf}) =$ 504.93 kips

T_{ub} = 114.9 kips, (FEMA Sec. 3.6.2.1.2)

> $(0.00002305\ P_f^{0.591}\ F_{fu}^{2.583} / (t_p^{0.895}\ d_{bt}^{1.909}\ t_{bw}^{0.327}\ b_p^{0.965}) + T_b =$ 114.5 kips

(Cont'd)

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.2.1.3)

A_b = 1.28 in^2 **>** $[2 M_f / (L - d_c) + V_g] / 6F_v$ = 0.56 **[Satisfactory]**

Where $V_g = w_u (L - d_c) / 2$ = 60.3 kips

$F_v = \phi F_{nv}$ = 36 ksi, (AISC 360-10, Tab. J3.2)

CHECK END PLATE THICKNESS (FEMA Sec. 3.6.2.1.4)

t_p = 1.625 in **>**

$\mathrm{Max}[\, 0.00609\, P_f^{0.9}\, g^{0.6}\, F_{fu}^{\ 9} / (d_{bt}^{\ 0.9}\, t_{bw}^{\ 0.1}\, b_p^{\ 0.7})\ ,\ 0.00413\, P_f^{0.25}\, g^{0.15}\, F_{fu} / (d_{bt}^{\ 0.7}\, t_{bw}^{\ 0.15}\, b_p^{\ 0.3})]$ = 1.29518 in

[Satisfactory]

CHECK CONTINUITY PLATE REQUIREMENT (FEMA Sec. 3.6.2.1 & 3.3.3.1)

$t_{cf,\ reqD} = \{\alpha_m F_{fu} C_3 / [0.9\ F_{yc}\ (3.5\ p_b + c)]\}^{0.5}$ = 0.48 in **<** $t_{cf,\ actual}$

$t_{cw,\ reqD} = M_f\ / [(\ d_b - t_{bf}\ )(\ 6\ k_c + 2\ t_p + t_{bf})\ F_{yc}]$ = 0.60 in **<** $t_{cw,\ actual}$

(The continuity plates may not be required.)

Where C_a = 1.45 , (FEMA Sec. 3.6.2.1.5)

$C_3 = g / 2 - d_{bt} / 4 - k_c$ = 0.4338 in

$\alpha_m = C_a\ (A_f / A_w\ )^{1/3}\ C_3 / d_{bt}^{\ 1/4}$ = 0.8309 , (FEMA Sec. 3.6.2.1.5)

$t_{st} = t_{bf}$ for interior connection, or (t_{bf} /2) for exterior connection = 0.62 in, USE 0.63 in, (5/8 in)

b_{st} = 7 in **<** $0.56\ (E / F_{yst}\ )^{0.5}\ t_{st}$ = 9.93 in, (AISC 358-10 Eq 6.10-10)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr}\ A$ = 413.0 kips

Where ϕ_c = 0.9 ,(AISC 360 E1) $h_{st} = d_c - 2k_c$ = 11.38 in

K = 0.75 $K\ h_{st} / r_{st}$ < 200 (AISC 360 E2) **[Satisfactory]**

$I = t_{st}\ (2b_{st} + t_{wc})^{\ 3} / 12$ = 156 in^4 F_e = 47926 ksi (AISC 360 E3)

$A = 2b_{st}t_{st} + 25(t_{wc})^{\ 2}$ = 13 in^2 F_{cr} = 35.99 ksi (AISC 360 E3)

$r_{st} = (\ I / A\ )^{0.5}$ = 3.49 in F_{yst} = 36 kips, plate yield stress

$P_{u,st} = R_{yb}\ F_{yb}\ b_{fb}\ t_{fb}$ = 278.7 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 Sec.J2.2b)

w = 7/16 in
> w_{MIN} = 0.25 in
< w_{MAX} = 0.5625 in

[Satisfactory]

The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6t_{st}L_{nst}F_y / [(2)\ \phi\ F_w\ (0.707\ w)]$ = (0.625 x 8.4) x 36 / [(2) 0.75 (0.6x70)(0.707x7/16)] = 4.42 in

Where $L_{net} = d_c - 2(k_c + 1.5)$ = 8.4 **<** $2(L_{net} - 0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341 Sec. E3.6e & FEMA Sec. 3.3.3.2)

$t_{ReqD} = \mathrm{MAX}\ (t_1, t_2)$ = 0.80 in

$t_1 = C_y\ M_c\ (h - d_b\ ) / [0.9\ (0.6)\ F_{yc}\ R_{yc}\ d_c\ (d_b - t_{fb})\ h]$ = 0.80 in

Where $C_y = S_b / (C_{pr}\ Z_{hing})$ = 0.80

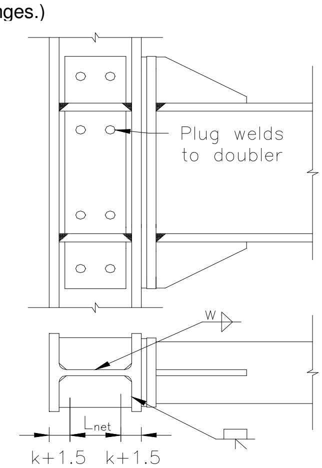
$S_b = 2I_b / d_b$ = 127 in^2

$I_b = I_x$ = 1330 in^4

$M_c = \Sigma M_{pb}^*$ = 930 ft-kips

$t_2 = (d_z + w_z\ ) / 90 = (d_b - 2t_{st} + d_c - 2k_c) / 90$ = 0.35 in

Since t_{wc} = 0.98 in **>** t_{ReqD} , a doubler plate is not required.



Technical References:

1. AISC 341-10: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, 2010.
2. AISC 358-10: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, 2010.
3. AISC 360-10: "Specification for Structural Steel Buildings", American Institute of Steel Construction, 2010.
4. FEMA 350: "Recommended Seismic Design Criteria for New Steel Moment-frame Buildings.", SAC Joint Venture, 2000.

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4-Bolted Stiffened End Plate for SMF Based on AISC 341-10, 358-10, 360-10 & FEMA-350

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W12X106**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
31.2	12.9	0.61	12.20	0.99	145	933	5.47	3.11	164	1.59

BEAM SECTION => **W18X50**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.7	18.0	0.36	7.50	0.57	88.9	800	7.38	1.65	101	0.97

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

THE FACTOR GRAVITY LOAD ON THE BEAM w_u = **4.2** klf

THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **800** kips

BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft

AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **12** ft

THE SMRF DESIGN IS ADEQUATE.
(Continuity column stiffeners 5/8 x 6
with 5/16" fillet weld to web & CP to flanges.
A doubler plate is required with thickness of 3/16 in.)

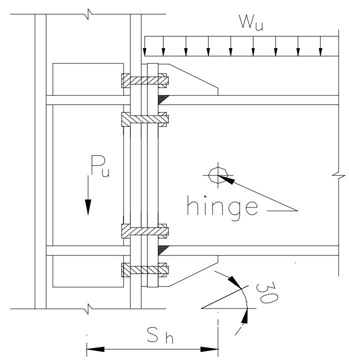
BOLTS ϕ = **1 3/4** in

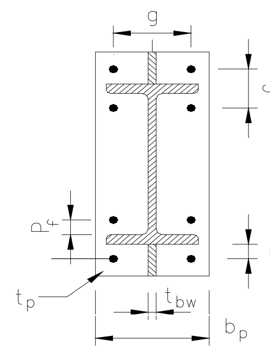
GRADES (A325 or A490) **A325**

PLATE & SHIM t_p = **3/4** in

NUMBER COLUMNS N_c = **2**
(Top & Bot)

NUMBER BEAM N_b = **1**
(One Side Only)





ANALYSIS

g =Max(b_{bf} - ϕ , t_w + 3 ϕ) = 5.86 in

P_f = 1.5 ϕ = 2.00 in (AISC 358 Tab 6.1)

$S_h = d_c / 2 + t_p + 1" + (2P_f - 1") \tan^{-1} 30^o$
= 13.40 in

c = 2 P_f + t_{bf} = 4.57 in

b_p = g + 3 ϕ = 11.11 in
< b_{cf} **[Satisfactory]**

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f)$ = 6.58 < $0.3 (E_s / F_y)^{0.5}$ = 7.22 **[Satisfactory]**
Where E_s = 29000 ksi

h / t_w = 45.23 < $2.45 (E_s / F_y)^{0.5}$ = 59.00 **[Satisfactory]**

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f)$ = 6.16 < $0.3 (E_s / F_y)^{0.5}$ = 7.22 **[Satisfactory]**

h / t_w = 15.93 <
$3.76(E_s/F_y)^{0.5}(1-2.75P_u/\phi_bP_y)$ = N/A , for $P_u/\phi_bP_y \le 0.125$
$1.12(E_s/F_y)^{0.5}(2.33-P_u/\phi_bP_y)$ = 47.48 , for $P_u/\phi_bP_y > 0.125$

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 1560 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10 Sec. E3.4a)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*)$ = 1.08 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c Z_c (F_{yc} - P_u / A_g)$ = 666 ft-kips

$\Sigma M_{pb}^* = N_b (M_{hinge} + M_v)$ = 615 ft-kips, at center of column

$M_v = V_{hinge} S_h = [2M_{hinge} /(L-2S_h)+w_u(L-2S_h)/2] S_h$= 106 ft-kips

$M_{hinge} = C_{pr}R_yF_{yb} Z_b$ = 509 ft-kips

R_y = 1.1 (AISC 341-10 Tab. A3.1)

C_{pr} = 1.1 (FEMA Sec. 3.5.5.1)

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.2)

$M_f = M_{hinge} + [2M_{hinge} / (L - 2S_h) + w_u(L - 2S_h)/2] (S_h - d_c /2)$

= 564 ft-kips < $2 T_{ub} (d_0 + d_i)$ = 719 ft-kips **[Satisfactory]**

Where $d_0 = d_b + P_f - 0.5 t_{bf}$ = 19.72 in T_b = 103 kips, (AISC 360-10, Tab. J3.1)

$d_i = d_0 - c$ = 15.15 in A_{bt} = 1.37 in^2 / bolt

$F_{fu} = M_f / (d_b - t_{bf})$ = 388.43 kips

T_{ub} = 123.7 kips, (FEMA Sec. 3.6.1.1 & 3.6.2.1.2)

> $(0.00002305\ P_f^{0.591}\ F_{fu}^{2.583} / (t_p^{0.895}\ d_{bt}^{1.909}\ t_{bw}^{0.327}\ b_p^{0.965}) + T_b$ = 113.3 kips

(Cont'd)

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.3)

$A_b =$ 1.37 in^2 **>** $[2 M_f / (L - d_c) + V_g] / 3F_v =$ 0.92 **[Satisfactory]**

Where $V_g = w_u (L - d_c) / 2 =$ 60.7 kips

$F_v = \phi F_{nv} =$ 36 ksi, (AISC 360-10, Tab. J3.2)

CHECK END PLATE THICKNESS (AISC 358-10 Eq 6.10-13)

$t_p =$ 0.75 in **>** $[1.11 M_f / \phi_d F_{yp} Y_p)]^{0.5} =$ 0.41 in **[Satisfactory]**

Where $Y_p =$ 1258 in, (AISC 358-10 Tab. 6.3 Case 1)

$F_{yp} =$ 36 ksi $\phi_d =$ 1.0

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-10 Eq 6.10-13, FEMA Sec 3.3.3.1)

$t_{cf,\ reqD} = [1.11 M_f / \phi_d F_{yc} Y_c)]^{0.5} =$ 0.31 in **<** $t_{cf,\ actual}$

Where $Y_c =$ 1595 in, (AISC 358-10 Tab. 6.5 Stiffened)

$t_{cw,\ reqD} = M_f / [(d_b - t_{bf})(6 k_c + 2 t_p + t_{bf}) F_{yc}] =$ 0.67 in **>** $t_{cw,\ actual}$

(The continuity plates required.)

$t_{st} = t_{bf}$ for interior connection, or $(t_{bf} /2)$ for exterior connection = 0.57 in, USE 0.63 in, (5/8 in)

$b_{st} =$ 6 in **<** $0.56 (E / F_{yst})^{0.5} t_{st} =$ 9.93 in, (AISC 358-10 Eq 6.10-10)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr} A =$ 345.0 kips

Where $\phi_c =$ 0.9 ,(AISC 360 E1) $h_{st} = d_c - 2k_c =$ 9.72 in

$K =$ 0.75 $K h_{st} / r_{st} < 200$ (AISC 360 E2) **[Satisfactory]**

$I = t_{st} (2b_{st} + t_{wc})^3 / 12 =$ 98 in^4 $F_e =$ 49670 ksi (AISC 360 E3)

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 11 in^2 $F_{cr} =$ 35.99 ksi (AISC 360 E3)

$r_{st} = (I / A)^{0.5} =$ 3.04 in $F_{yst} =$ 36 kips, plate yield stress

$P_{u,st} = R_{yb} F_{yb} b_{fb} t_{fb} =$ 235.1 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 Sec.J2.2b)

$w =$ 5/16 in $> w_{MIN} =$ 0.1875 in

$< w_{MAX} =$ 0.4375 in

[Satisfactory]

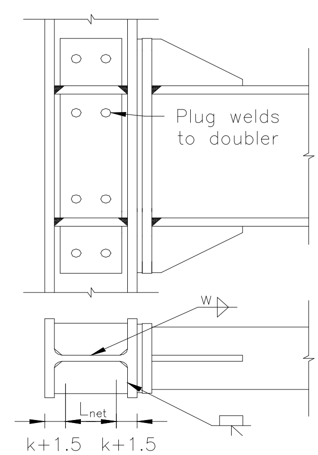
The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6t_{st}L_{nst}F_y / [(2) \phi F_w (0.707 w)]$ = (0.625 x 6.7) x 36 / [(2) 0.75 (0.6x70)(0.707x5/16)] = 4.61 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 6.7 **<** $2(L_{net} -0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341 Sec. E3.6e & FEMA Sec. 3.3.3.2)



$t_{ReqD} = MAX (t_1, t_2) =$ 0.77 in

$t_1 = C_y M_c (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h] =$ 0.77 in

Where $C_y = S_b / (C_{pr} Z_{hing}) =$ 0.80

$S_b = 2I_b / d_b =$ 89 in^2

$I_b = I_x =$ 800 in^4

$M_c = \Sigma M_{pb}^* =$ 615 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b -2t_{st} + d_c - 2k_c) / 90 =$ 0.29 in

Since $t_{wc} =$ 0.61 in **<** t_{ReqD} , a doubler plate is required with thickness of 3/16 in.

Technical References:

1. AISC 341-10: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, 2010.
2. AISC 358-10: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, 2010.
3. AISC 360-10: "Specification for Structural Steel Buildings", American Institute of Steel Construction, 2010.
4. FEMA 350: "Recommended Seismic Design Criteria for New Steel Moment-frame Buildings.", SAC Joint Venture, 2000.

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4-Bolted Unstiffened End Plate for SMF Based on AISC 341-10, 358-10, 360-10 & FEMA-350

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W12X96**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
28.2	12.7	0.55	12.20	0.90	131	833	5.43	3.09	147	1.50

BEAM SECTION => **W18X35** **<== Err. See AISC 385-10 Table 6.1, no section work!**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
10.3	17.7	0.30	6.00	0.43	57.6	510	7.04	1.22	67	0.83

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

THE FACTOR GRAVITY LOAD ON THE BEAM w_u = **4.2** klf

THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **800** kips

BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft

AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **12** ft

THE SMRF DESIGN IS ADEQUATE.

(Continuity column stiffeners 7/16 x 6 with 1/4" fillet weld to web & CP to flanges. A doubler plate is not required.)

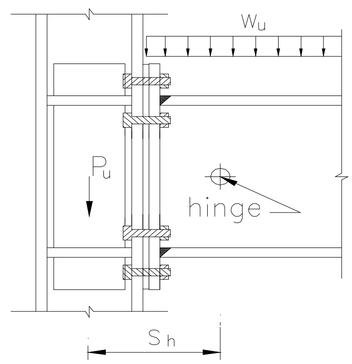
BOLTS ϕ = **1 1/16** in

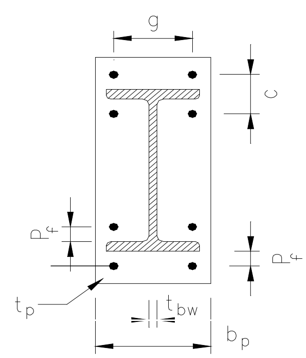
GRADES (A325 or A490) **A325**

PLATE & SHIM t_p = **3/4** in

NUMBER COLUMNS N_c = **2** (Top & Bot)

NUMBER BEAM N_b = **1** (One Side Only)





ANALYSIS

$g = \text{Max}(b_{bf} - \phi\ ,\ t_w + 3\ \phi\ ) =$ 5.00 in

$P_f = 1.5\ \phi =$ 1.75 in (AISC 358 Tab 6.1)

$S_h = d_c / 2 + t_p + d_b / 3$ = 13.00 in

$c = 2\ P_f + t_{bf} =$ 3.93 in

$b_p = g + 3\ \phi =$ 9.00 in

< b_{cf} **[Satisfactory]**

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 7.06 < $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

Where E_s = 29000 ksi

$h / t_w =$ 53.49 < $2.45\ (E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 6.78 < $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

$h / t_w =$ 17.64 <

$3.76(E_s/F_y)^{0.5}(1\text{-}2.75P_u/\phi_bP_y) =$ N/A , for $P_u/\phi_bP_y \le 0.125$

$1.12(E_s/F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y) =$ 45.84 , for $P_u/\phi_bP_y > 0.125$

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 1410 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10 Sec. E3.4a)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*) =$ 1.25 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c\ Z_c\ (F_{yc} - P_u / A_g) =$ 530 ft-kips

$\Sigma M_{pb}^* = N_b\ (M_{hinge} + M_v) =$ 425 ft-kips, at center of column

$M_v = V_{hinge}\ S_h = [2M_{hinge} /(L\text{-}2S_h)+w_u(L\text{-}2S_h)/2]\ S_h =$ 89 ft-kips

$M_{hinge} = C_{pr}R_yF_{yb}\ Z_b =$ 335 ft-kips

R_y = 1.1 (AISC 341-10 Tab. A3.1)

C_{pr} = 1.1 (FEMA Sec. 3.5.5.1)

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.2)

$M_f = M_{hinge} + [2M_{hinge} / (L - 2S_h) + w_u(L - 2S_h)/2]\ (S_h - d_c /2)$

= 381 ft-kips < $2\ T_{ub}\ (d_0 + d_i\ ) =$ 432 ft-kips **[Satisfactory]**

Where $d_0 = d_b + P_f - 0.5\ t_{bf} =$ 19.24 in T_b = 51 kips, (AISC 360-10, Tab. J3.1)

$d_i = d_0 - c =$ 15.31 in A_{bt} = 0.83 in^2 / bolt

$F_{fu} = M_f / (\ d_b - t_{bf}\ ) =$ 264.67 kips

T_{ub} = 75.1 kips, (FEMA Sec. 3.6.1.1 & 3.6.2.1.2)

> $(0.00002305\ P_f^{\ 0.591}\ F_{fu}^{\ 2.583} / (t_p^{\ 0.895}\ d_{bt}^{\ 1.909}\ t_{bw}^{\ 0.327}\ b_p^{\ 0.965}) + T_b =$ 62.9 kips

(Cont'd)

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.3)

$A_b =$ 0.83 in^2 **>** $[2 M_f / (L - d_c) + V_g] / 3F_v =$ 0.81 **[Satisfactory]**

Where $V_g = w_u (L - d_c) / 2 =$ 60.8 kips

$F_v = \phi F_{nv} =$ 36 ksi, (AISC 360-10, Tab. J3.2)

CHECK END PLATE THICKNESS (AISC 358-10 Eq 6.10-13)

$t_p =$ 0.75 in **>** $[1.11 M_f / \phi_d F_{yp} Y_p)]^{0.5} =$ 0.40 in **[Satisfactory]**

Where $Y_p =$ 864 in, (AISC 358-10 Tab. 6.2)

$F_{yp} =$ 36 ksi $\phi_d =$ 1.0

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-10 Eq 6.10-13, FEMA Sec 3.3.3.1)

$t_{cf,\ reqD} = [1.11 M_f / \phi_d F_{yc} Y_c)]^{0.5} =$ 0.41 in **<** $t_{cf,\ actual}$

Where $Y_c =$ 594 in, (AISC 358-10 Tab. 6.5 Unsiffened)

$t_{cw,\ reqD} = M_f / [(d_b - t_{bf})(6 k_c + 2 t_p + t_{bf}) F_{yc}] =$ 0.48 in **<** $t_{cw,\ actual}$

(The continuity plates may not be required.)

$t_{st} = t_{bf}$ for interior connection, or $(t_{bf} /2)$ for exterior connection = 0.43 in, USE 0.44 in, (7/16 in)

$b_{st} =$ 6 in **<** $0.56 (E / F_{yst})^{0.5} t_{st} =$ 6.95 in, (AISC 358-10 Eq 6.10-10)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr} A =$ 242.9 kips

Where $\phi_c =$ 0.9 ,(AISC 360 E1) $h_{st} = d_c - 2k_c =$ 9.7 in

$K =$ 0.75 $K h_{st} / r_{st} < 200$ (AISC 360 E2) **[Satisfactory]**

$I = t_{st} (2b_{st} + t_{wc})^3 / 12 =$ 68 in^4 $F_e =$ 48920 ksi (AISC 360 E3)

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 8 in^2 $F_{cr} =$ 35.99 ksi (AISC 360 E3)

$r_{st} = (I / A)^{0.5} =$ 3.01 in $F_{yst} =$ 36 kips, plate yield stress

$P_{u,st} = R_{yb} F_{yb} b_{fb} t_{fb} =$ 140.3 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 Sec.J2.2b)

w = 1/4 in
> $w_{MIN} =$ 0.1875 in
< $w_{MAX} =$ 0.3125 in

[Satisfactory]

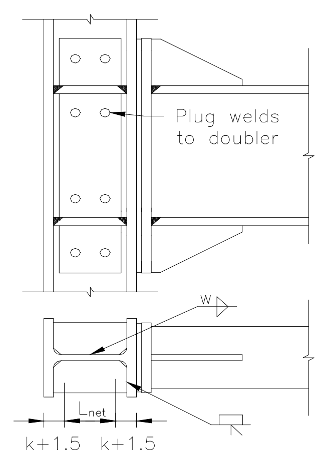
The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6t_{st}L_{nst}F_y / [(2) \phi F_w (0.707 w)]$ = (0.4375 x 6.7) x 36 / [(2) 0.75 (0.6x70)(0.707x1/4)] = 4.02 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 6.7 **<** $2(L_{net} -0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341 Sec. E3.6e & FEMA Sec. 3.3.3.2)



$t_{ReqD} = MAX (t_1, t_2) =$ 0.54 in

$t_1 = C_y M_c (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h] =$ 0.54 in

Where $C_y = S_b / (C_{pr} Z_{hing}) =$ 0.79

$S_b = 2I_b / d_b =$ 58 in^2

$I_b = I_x =$ 510 in^4

$M_c = \Sigma M_{pb}^* =$ 425 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b -2t_{st} + d_c - 2k_c) / 90 =$ 0.29 in

Since $t_{wc} =$ 0.55 in **>** t_{ReqD} , a doubler plate is not required.

Technical References:

1. AISC 341-10: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, 2010.
2. AISC 358-10: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, 2010.
3. AISC 360-10: "Specification for Structural Steel Buildings", American Institute of Steel Construction, 2010.
4. FEMA 350: "Recommended Seismic Design Criteria for New Steel Moment-frame Buildings.", SAC Joint Venture, 2000.

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8-Bolted Moment Connection Based on AISC 341-10, 358-10, 360-10 & FEMA-350

DESIGN CRITERIA

THE NON-SEISMIC MOMENTION CONNECTION HAS RELEASED BEAM & COLUMN SECTION LIMITS, BEAM-COLUMN RATIO REQUIREMENT, AND BENDING MOMENT AT THE COLUMN FACE FROM MEMBER CAPACITY TO ACTUAL BEAM END FORCE.

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W14X211**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
62	15.7	0.98	15.80	1.56	338	2660	6.55	4.08	390	2.16

BEAM SECTION => **W21X62**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
18.3	21.0	0.40	8.24	0.62	127	1330	8.53	1.77	144	1.12

BENDING MOMENT AT THE COLUMN FACE M_f = **850** ft-kips, SD level

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

THE FACTOR GRAVITY LOAD ON THE BEAM w_u = **4.2** klf

THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **800** kips

BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft

AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **12** ft

THE DESIGN IS ADEQUATE.
(Continuity column stiffeners 5/8 x 7
with 7/16" fillet weld to web & CP to flanges.
A doubler plate is not required.)

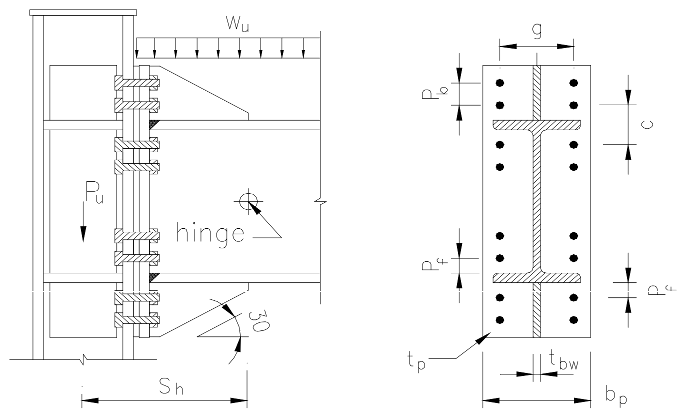
BOLTS ϕ = **1 5/8** in

GRADES (A325 or A490) **A325**

PLATE & SHIM t_p = **1 5/8** in

NUMBER COLUMNS N_c = **2**
(Top & Bot)

NUMBER BEAM N_b = **1**
(One Side Only)



ANALYSIS

$g = \text{Max}(b_{bf} - \phi\ ,\ t_w + 3\ \phi\ ) =$ 6.00 in

$P_b = 3\ \phi =$ 3.75 in (AISC 358 Tab 6.1)

$P_f = 1.5\ \phi =$ 2.00 in (AISC 358 Tab 6.1)

$S_h = d_c / 2 + t_p + 1" + (2P_f + P_b - 1")\ \tan^{-1} 30^o$
= 22.17 in

$c = 2\ P_f + t_{bf} =$ 4.62 in

$b_p = g + 3\ \phi =$ 10.88 in
< b_{cf} **[Satisfactory]**

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 6.70 < $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**
Where E_s = 29000 ksi

$h / t_w =$ 46.90 < $2.45\ (E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 5.06 < $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

$h / t_w =$ 11.61 <
$3.76(E_s/F_y)^{0.5}(1\text{-}2.75P_u/\phi_bP_y) =$ N/A , for $P_u/\phi_bP_y \le 0.125$
$1.12(E_s/F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y) =$ 55.11 , for $P_u/\phi_bP_y > 0.125$

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 3100 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10 Sec. E3.4a)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*) =$ 2.59 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c\ Z_c\ (F_{yc} - P_u / A_g) =$ 2411 ft-kips

$\Sigma M_{pb}^* = N_b\ (M_{hinge} + M_v) =$ 930 ft-kips, at center of column

$M_v = V_{hinge}\ S_h = [2M_{hinge}/(L\text{-}2S_h)+w_u(L\text{-}2S_h)/2]\ S_h =$ 204 ft-kips

$M_{hinge} = C_{pr}R_yF_{yb}\ Z_b =$ 726 ft-kips

R_y = 1.1 (AISC 341-10 Tab. A3.1)

C_{pr} = 1.1 (FEMA Sec. 3.5.5.1)

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.2.1.2)

$M_f = M_{hinge} + [2M_{hinge} / (L - 2S_h) + w_u(L - 2S_h)/2]\ (S_h - d_c /2) =$ 858 ft-kips = M_f , input value for non-seismic

= 850 ft-kips < $3.4\ T_{ub}\ (d_0 + d_i\ ) =$ 1205 ft-kips **[Satisfactory]**

Where $d_0 = d_b + P_f - 0.5\ t_{bf} =$ 22.69 in T_b = 103 kips, (AISC 360-10, Tab. J3.1)

$d_i = d_0 - c - P_b =$ 14.33 in A_{bt} = 1.28 in^2 / bolt

$F_{fu} = M_f / (\ d_b - t_{bf}\ ) =$ 500.37 kips

T_{ub} = 114.9 kips, (FEMA Sec. 3.6.2.1.2)

> $(0.00002305\ P_f^{0.591}\ F_{fu}^{2.583} / (t_p^{0.895}\ d_{bt}^{1.909}\ t_{bw}^{0.327}\ b_p^{0.965}) + T_b =$ 114.3 kips

[Satisfactory]

(Cont'd)

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.2.1.3)

A_b = 1.28 in^2 **>** $[2 M_f / (L - d_c) + V_g] / 6F_v$ = 0.55 **[Satisfactory]**

Where $V_g = w_u (L - d_c) / 2$ = 60.3 kips

$F_v = \phi F_{nv}$ = 36 ksi, (AISC 360-10, Tab. J3.2)

CHECK END PLATE THICKNESS (FEMA Sec. 3.6.2.1.4)

t_p = 1.625 in **>**

$Max[0.00609 P_f^{0.9} g^{0.6} F_{fu}^{9} / (d_{bt}^{0.9} t_{bw}^{0.1} b_p^{0.7}) , 0.00413 P_f^{0.25} g^{0.15} F_{fu} / (d_{bt}^{0.7} t_{bw}^{0.15} b_p^{0.3})]$ = 1.28347 in **[Satisfactory]**

CHECK CONTINUITY PLATE REQUIREMENT (FEMA Sec. 3.6.2.1 & 3.3.3.1)

$t_{cf, reqD} = \{\alpha_m F_{fu} C_3 / [0.9 F_{yc} (3.5 p_b + c)]\}^{0.5}$ = 0.48 in **<** $t_{cf, actual}$

$t_{cw, reqD} = M_f / [(d_b - t_{bf})(6 k_c + 2 t_p + t_{bf}) F_{yc}]$ = 0.59 in **<** $t_{cw, actual}$

(The continuity plates may not be required.)

Where C_a = 1.45 , (FEMA Sec. 3.6.2.1.5)

$C_3 = g / 2 - d_{bt} / 4 - k_c$ = 0.4338 in

$\alpha_m = C_a (A_f / A_w)^{1/3} C_3 / d_{bt}^{1/4}$ = 0.8309 , (FEMA Sec. 3.6.2.1.5)

$t_{st} = t_{bf}$ for interior connection, or (t_{bf} /2) for exterior connection = 0.62 in, USE 0.63 in, (5/8 in)

b_{st} = 7 in **<** $0.56 (E / F_{yst})^{0.5} t_{st}$ = 9.93 in, (AISC 358-10 Eq 6.10-10) **[Satisfactory]**

$\phi_c P_{n,st} = \phi_c F_{cr} A$ = 413.0 kips

Where ϕ_c =	0.9 ,(AISC 360 E1)	$h_{st} = d_c - 2k_c$ =	11.38 in
K =	0.75	$K h_{st} / r_{st}$ < 200	(AISC 360 E2) **[Satisfactory]**
$I = t_{st} (2b_{st} + t_{wc})^3 / 12$ =	156 in^4	F_e =	47926 ksi (AISC 360 E3)
$A = 2b_{st}t_{st} + 25(t_{wc})^2$ =	13 in^2	F_{cr} =	35.99 ksi (AISC 360 E3)
$r_{st} = (I / A)^{0.5}$ =	3.49 in	F_{yst} =	36 kips, plate yield stress

$P_{u,st} = R_{yb} F_{yb} b_{fb} t_{fb}$ = 278.7 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 Sec.J2.2b)

w = 7/16 in > w_{MIN} = 0.25 in

< w_{MAX} = 0.5625 in

[Satisfactory]

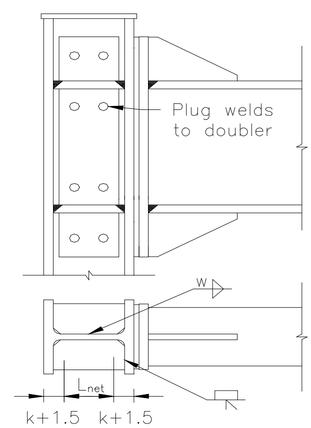
The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6 t_{st} L_{nst} F_y / [(2) \phi F_w (0.707 w)]$ = (0.625 x 8.4) x 36 / [(2) 0.75 (0.6x70)(0.707x7/16)] = 4.42 in

Where $L_{net} = d_c - 2(k_c + 1.5)$ = 8.4 **<** $2(L_{net} - 0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341 Sec. E3.6e & FEMA Sec. 3.3.3.2)



$t_{ReqD} = MAX (t_1, t_2)$ = 0.80 in

$t_1 = C_y M_c (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h]$ = 0.80 in

Where $C_y = S_b / (C_{pr} Z_{hing})$ = 0.80

$S_b = 2I_b / d_b$ = 127 in^2

$I_b = I_x$ = 1330 in^4

$M_c = \Sigma M_{pb}^*$ = 930 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b - 2t_{st} + d_c - 2k_c) / 90$ = 0.35 in

Since t_{wc} = 0.98 in **>** t_{ReqD} , a doubler plate is not required.

Technical References:

1. AISC 341-10: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, 2010.
2. AISC 358-10: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, 2010.
3. AISC 360-10: "Specification for Structural Steel Buildings", American Institute of Steel Construction, 2010.
4. FEMA 350: "Recommended Seismic Design Criteria for New Steel Moment-frame Buildings.", SAC Joint Venture, 2000.

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4-Bolted Stiffened Moment Connection Based on AISC 341-10, 358-10, 360-10 & FEMA-350

DESIGN CRITERIA
THE NON-SEISMIC MOMENTION CONNECTION HAS RELEASED BEAM & COLUMN SECTION LIMITS, BEAM-COLUMN RATIO REQUIREMENT, AND BENDING MOMENT AT THE COLUMN FACE FROM MEMBER CAPACITY TO ACTUAL BEAM END FORCE.

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION = > **W12X106**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
31.2	12.9	0.61	12.20	0.99	145	933	5.47	3.11	164	1.59

BEAM SECTION = > **W18X50**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.7	18.0	0.36	7.50	0.57	88.9	800	7.38	1.65	101	0.97

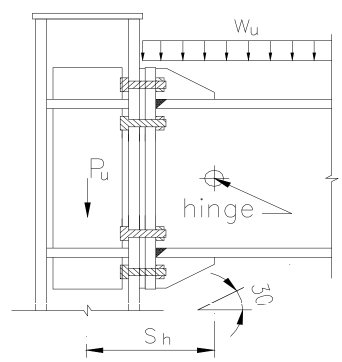
BENDING MOMENT AT THE COLUMN FACE M_f = **550** ft-kips, SD level
STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

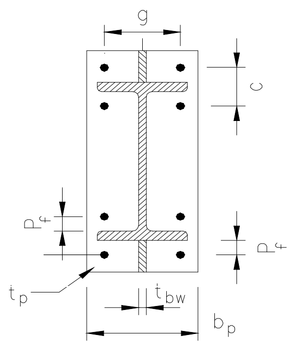
THE DESIGN IS ADEQUATE.
(Continuity column stiffeners 5/8 x 6
with 5/16" fillet weld to web & CP to flanges.
A doubler plate is required with thickness of 3/16 in.)

THE FACTOR GRAVITY LOAD ON THE BEAM w_u = **4.2** klf
THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **800** kips
BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft
AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **12** ft
BOLTS ϕ = **1 3/4** in
GRADES (A325 or A490) **A325**
PLATE & SHIM t_p = **3/4** in

NUMBER COLUMNS N_c = **2**
(Top & Bot)

NUMBER BEAM N_b = **1**
(One Side Only)





ANALYSIS

$g = \text{Max}(b_{bf} - \phi\ ,\ t_w + 3\ \phi\ ) =$ 5.86 in

$P_f = 1.5\ \phi =$ 2.00 in (AISC 358 Tab 6.1)

$S_h = d_c / 2 + t_p + 1" + (2P_f - 1")\ \tan^{-1} 30^o$
= 13.40 in

$c = 2\ P_f + t_{bf} =$ 4.57 in

$b_p = g + 3\ \phi =$ 11.11 in
< b_{cf} **[Satisfactory]**

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 6.58 **<** $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**
Where E_s = 29000 ksi

$h / t_w =$ 45.23 **<** $2.45\ (E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 6.16 **<** $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

$h / t_w =$ 15.93 <
$3.76(E_s/F_y)^{0.5}(1\text{-}2.75P_u/\phi_bP_y) =$ N/A , for $P_u/\phi_bP_y \le 0.125$
$1.12(E_s/F_y)^{0.5}(2.33\text{-}P_u/\phi_bP_y) =$ 47.48 , for $P_u/\phi_bP_y > 0.125$

[Satisfactory] Where ϕ_b = 0.9 , $P_y = F_yA$ = 1560 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10 Sec. E3.4a)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*) =$ 1.08 **>** 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c\ Z_c\ (F_{yc} - P_u / A_g\ ) =$ 666 ft-kips
$\Sigma M_{pb}^* = N_b\ (M_{hinge} + M_v) =$ 615 ft-kips, at center of column
$M_v = V_{hinge}\ S_h = [2M_{hinge} /(L\text{-}2S_h)+w_u(L\text{-}2S_h)/2]\ S_h =$ 106 ft-kips
$M_{hinge} = C_{pr}R_yF_{yb}\ Z_b =$ 509 ft-kips
R_y = 1.1 (AISC 341-10 Tab. A3.1)
C_{pr} = 1.1 (FEMA Sec. 3.5.5.1)

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.2)

$M_f = M_{hinge} + [2M_{hinge} / (L - 2S_h) + w_u(L - 2S_h)/2]\ (S_h - d_c /2) =$ 564 ft-kips = M_f , input value for non-seismic
= 550 ft-kips **<** $2\ T_{ub}\ (d_0 + d_i\ ) =$ 719 ft-kips **[Satisfactory]**

Where $d_0\ = d_b + P_f - 0.5\ t_{bf} =$ 19.72 in T_b = 103 kips, (AISC 360-10, Tab. J3.1)
$d_i\ = d_0 - c =$ 15.15 in A_{bt} = 1.37 in^2 / bolt
$F_{fu}\ = M_f / (\ d_b - t_{bf}\ ) =$ 378.66 kips
T_{ub} = 123.7 kips, (FEMA Sec. 3.6.1.1 & 3.6.2.1.2)
> $(0.00002305\ P_f^{0.591}\ F_{fu}^{2.583} / (t_p^{0.895}\ d_{bt}^{1.909}\ t_{bw}^{0.327}\ b_p^{0.965}) + T_b =$ 112.7 kips
[Satisfactory]

(Cont'd)

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.3)

$A_b =$ 1.37 in^2 **>** $[2 M_f / (L - d_c) + V_g] / 3F_v =$ 0.91 **[Satisfactory]**

Where $V_g = w_u (L - d_c) / 2 =$ 60.7 kips

$F_v = \phi F_{nv} =$ 36 ksi, (AISC 360-10, Tab. J3.2)

CHECK END PLATE THICKNESS (AISC 358-10 Eq 6.10-13)

$t_p =$ 0.75 in **>** $[1.11 M_f / \phi_d F_{yp} Y_p)]^{0.5} =$ 0.40 in **[Satisfactory]**

Where $Y_p =$ 1258 in, (AISC 358-10 Tab. 6.3 Case 1)

$F_{yp} =$ 36 ksi $\phi_d =$ 1.0

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-10 Eq 6.10-13, FEMA Sec 3.3.3.1)

$t_{cf,\ reqD} = [1.11 M_f / \phi_d F_{yc} Y_c)]^{0.5} =$ 0.30 in **<** $t_{cf,\ actual}$

Where $Y_c =$ 1595 in, (AISC 358-10 Tab. 6.5 Stiffened)

$t_{cw,\ reqD} = M_f / [(d_b - t_{bf})(6 k_c + 2 t_p + t_{bf}) F_{yc}] =$ 0.65 in **>** $t_{cw,\ actual}$

(The continuity plates required.)

$t_{st} = t_{bf}$ for interior connection, or $(t_{bf} /2)$ for exterior connection = 0.57 in, USE 0.63 in, (5/8 in)

$b_{st} =$ 6 in **<** $0.56 (E / F_{yst})^{0.5} t_{st} =$ 9.93 in, (AISC 358-10 Eq 6.10-10)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr} A =$ 345.0 kips

Where $\phi_c =$ 0.9 ,(AISC 360 E1) $h_{st} = d_c - 2k_c =$ 9.72 in

$K =$ 0.75 $K h_{st} / r_{st} < 200$ (AISC 360 E2) **[Satisfactory]**

$I = t_{st} (2b_{st} + t_{wc})^3 / 12 =$ 98 in^4 $F_e =$ 49670 ksi (AISC 360 E3)

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 11 in^2 $F_{cr} =$ 35.99 ksi (AISC 360 E3)

$r_{st} = (I / A)^{0.5} =$ 3.04 in $F_{yst} =$ 36 kips, plate yield stress

$P_{u,st} = R_{yb} F_{yb} b_{fb} t_{fb} =$ 235.1 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 Sec.J2.2b)

$w =$ 5/16 in $> w_{MIN} =$ 0.1875 in

$< w_{MAX} =$ 0.4375 in

[Satisfactory]

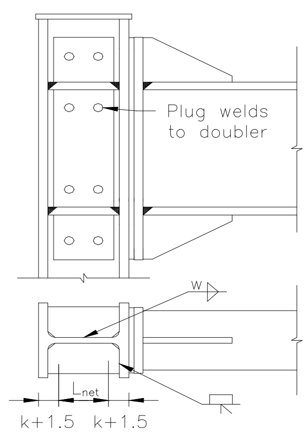
The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6t_{st}L_{nst}F_y / [(2) \phi F_w (0.707 w)]$ = (0.625 x 6.7) x 36 / [(2) 0.75 (0.6x70)(0.707x5/16)] = 4.61 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 6.7 **<** $2(L_{net} -0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341 Sec. E3.6e & FEMA Sec. 3.3.3.2)



$t_{ReqD} = MAX\ (t_1, t_2) =$ 0.77 in

$t_1 = C_y M_c (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h] =$ 0.77 in

Where $C_y = S_b / (C_{pr} Z_{hing}) =$ 0.80

$S_b = 2I_b / d_b =$ 89 in^2

$I_b = I_x =$ 800 in^4

$M_c = \Sigma M_{pb}^* =$ 615 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b -2t_{st} + d_c - 2k_c) / 90 =$ 0.29 in

Since $t_{wc} =$ 0.61 in **<** t_{ReqD} , a doubler plate is required with thickness of 3/16 in.

Technical References:

1. AISC 341-10: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, 2010.
2. AISC 358-10: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, 2010.
3. AISC 360-10: "Specification for Structural Steel Buildings", American Institute of Steel Construction, 2010.
4. FEMA 350: "Recommended Seismic Design Criteria for New Steel Moment-frame Buildings.", SAC Joint Venture, 2000.

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PROJECT :
CLIENT :
JOB NO. : DATE :

PAGE :
DESIGN BY :
REVIEW BY :

4-Bolted Unstiffened Moment Connection Based on AISC 341-10, 358-10, 360-10 & FEMA-350

DESIGN CRITERIA
THE NON-SEISMIC MOMENTION CONNECTION HAS RELEASED BEAM & COLUMN SECTION LIMITS, BEAM-COLUMN RATIO REQUIREMENT, AND BENDING MOMENT AT THE COLUMN FACE FROM MEMBER CAPACITY TO ACTUAL BEAM END FORCE.

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W12X96**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
28.2	12.7	0.55	12.20	0.90	131	833	5.43	3.09	147	1.50

BEAM SECTION => **W18X35**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
10.3	17.7	0.30	6.00	0.43	57.6	510	7.04	1.22	67	0.83

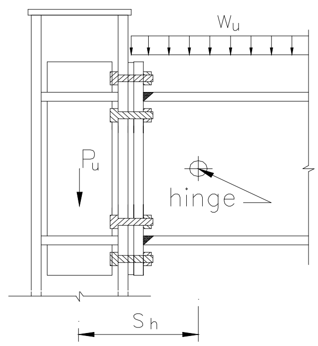
BENDING MOMENT AT THE COLUMN FACE M_f = **380** ft-kips, SD level
STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi
THE FACTOR GRAVITY LOAD ON THE BEAM w_u = **4.2** klf
THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **800** kips
BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft
AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **12** ft

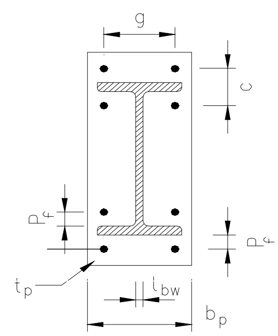
THE DESIGN IS ADEQUATE.
(Continuity column stiffeners 7/16 x 6 with 1/4" fillet weld to web & CP to flanges. A doubler plate is not required.)

BOLTS ϕ = **1 1/16** in
GRADES (A325 or A490) **A325**
PLATE & SHIM t_p = **3/4** in

NUMBER COLUMNS N_c = **2**
(Top & Bot)

NUMBER BEAM N_b = **1**
(One Side Only)





ANALYSIS

g =Max(b_{bf} - ϕ , t_w + 3 ϕ) = 5.00 in

P_f = 1.5 ϕ = 1.75 in (AISC 358 Tab 6.1)

S_h = d_c / 2 + t_p + d_b / 3
= 13.00 in

c = 2 P_f + t_{bf} = 3.93 in

b_p = g + 3 ϕ = 9.00 in
< b_{cf} **[Satisfactory]**

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)
b_f / (2t_f) = 7.06 < 0.3 (E_s / F_y)$^{0.5}$ = 7.22 **[Satisfactory]**
Where E_s = 29000 ksi
h / t_w = 53.49 < 2.45 (E_s / F_y)$^{0.5}$ = 59.00 **[Satisfactory]**

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 341-10 Tab. D1.1)
b_f / (2t_f) = 6.78 < 0.3 (E_s / F_y)$^{0.5}$ = 7.22 **[Satisfactory]**
h / t_w = 17.64 <
3.76(E_s/F_y)$^{0.5}$(1-2.75P_u/$\phi_b P_y$) = N/A , for $P_u/\phi_b P_y \le 0.125$
1.12(E_s/F_y)$^{0.5}$(2.33-P_u/$\phi_b P_y$) = 45.84 , for $P_u/\phi_b P_y > 0.125$
[Satisfactory] Where ϕ_b = 0.9 , P_y = $F_y A$ = 1410 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10 Sec. E3.4a)
ΣM_{pc}^* / (ΣM_{pb}^*) = 1.25 > 1.00 **[Satisfactory]**
Where ΣM_{pc}^* = N_c Z_c (F_{yc} - P_u / A_g) = 530 ft-kips
ΣM_{pb}^* = N_b (M_{hinge} + M_v) = 425 ft-kips, at center of column
M_v = V_{hinge} S_h = [2M_{hinge} /(L-2S_h)+w_u(L-2S_h)/2] S_h= 89 ft-kips
M_{hinge} = $C_{pr} R_y F_{yb} Z_b$ = 335 ft-kips
R_y = 1.1 (AISC 341-10 Tab. A3.1)
C_{pr} = 1.1 (FEMA Sec. 3.5.5.1)

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.2)
M_f = M_{hinge} + [2M_{hinge} / (L - 2S_h) + w_u(L - 2S_h)/2] (S_h - d_c /2) = 381 ft-kips = M_f , input value for non-seismic
= 380 ft-kips < 2 T_{ub} (d_0 + d_i) = 432 ft-kips **[Satisfactory]**
Where d_0 = d_b + P_f - 0.5 t_{bf} = 19.24 in T_b = 51 kips, (AISC 360-10, Tab. J3.1)
d_i = d_0 - c = 15.31 in A_{bt} = 0.83 in^2 / bolt
F_{fu} = M_f / (d_b - t_{bf}) = 263.97 kips
T_{ub} = 75.1 kips, (FEMA Sec. 3.6.1.1 & 3.6.2.1.2)
> (0.00002305 $P_f^{0.591}$ $F_{fu}^{2.583}$ / ($t_p^{0.895}$ $d_{bt}^{1.909}$ $t_{bw}^{0.327}$ $b_p^{0.965}$) + T_b = 62.8 kips
[Satisfactory]

(Cont'd)

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.3)

A_b = 0.83 in^2 **>** $[2 M_f / (L - d_c) + V_g] / 3F_v$ = 0.81 **[Satisfactory]**

Where $V_g = w_u (L - d_c) / 2$ = 60.8 kips

$F_v = \phi F_{nv}$ = 36 ksi, (AISC 360-10, Tab. J3.2)

CHECK END PLATE THICKNESS (AISC 358-10 Eq 6.10-13)

t_p = 0.75 in **>** $[1.11 M_f / \phi_d F_{yp} Y_p)]^{0.5}$ = 0.40 in **[Satisfactory]**

Where Y_p = 864 in, (AISC 358-10 Tab. 6.2)

F_{yp} = 36 ksi ϕ_d = 1.0

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-10 Eq 6.10-13, FEMA Sec 3.3.3.1)

$t_{cf,\ reqD} = [1.11 M_f / \phi_d F_{yc} Y_c)]^{0.5}$ = 0.41 in **<** $t_{cf,\ actual}$

Where Y_c = 594 in, (AISC 358-10 Tab. 6.5 Unsiffened)

$t_{cw,\ reqD} = M_f / [(d_b - t_{bf})(6 k_c + 2 t_p + t_{bf}) F_{yc}]$ = 0.48 in **<** $t_{cw,\ actual}$

(The continuity plates may not be required.)

$t_{st} = t_{bf}$ for interior connection, or $(t_{bf} /2)$ for exterior connection = 0.43 in, USE 0.44 in, (7/16 in)

b_{st} = 6 in **<** $0.56 (E / F_{yst})^{0.5} t_{st}$ = 6.95 in, (AISC 358-10 Eq 6.10-10)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr} A$ = 242.9 kips

Where ϕ_c = 0.9 ,(AISC 360 E1) $h_{st} = d_c - 2k_c$ = 9.7 in

K = 0.75 $K\ h_{st} / r_{st} < 200$ (AISC 360 E2) **[Satisfactory]**

$I = t_{st} (2b_{st} + t_{wc})^3 / 12$ = 68 in^4 F_e = 48920 ksi (AISC 360 E3)

$A = 2b_{st}t_{st} + 25(t_{wc})^2$ = 8 in^2 F_{cr} = 35.99 ksi (AISC 360 E3)

$r_{st} = (I / A)^{0.5}$ = 3.01 in F_{yst} = 36 kips, plate yield stress

$P_{u,st} = R_{yb} F_{yb} b_{fb} t_{fb}$ = 140.3 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 Sec.J2.2b)

w = 1/4 in > w_{MIN} = 0.1875 in

< w_{MAX} = 0.3125 in

[Satisfactory]

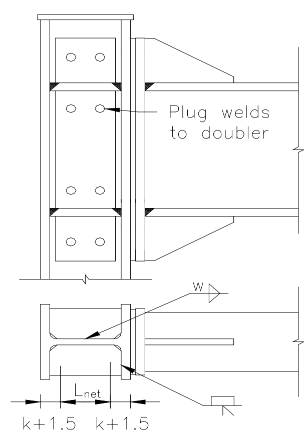
The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6t_{st}L_{nst}F_y / [(2) \phi F_w (0.707 w)]$ = (0.4375 x 6.7) x 36 / [(2) 0.75 (0.6x70)(0.707x1/4)] = 4.02 in

Where $L_{net} = d_c - 2(k_c + 1.5)$ = 6.7 **<** $2(L_{net} -0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341 Sec. E3.6e & FEMA Sec. 3.3.3.2)



$t_{ReqD} = MAX (t_1, t_2)$ = 0.54 in

$t_1 = C_y M_c (h - d_b) / [0.9 (0.6) F_{yc} R_{yc} d_c (d_b - t_{fb}) h]$ = 0.54 in

Where $C_y = S_b / (C_{pr} Z_{hing})$ = 0.79

$S_b = 2I_b / d_b$ = 58 in^2

$I_b = I_x$ = 510 in^4

$M_c = \Sigma M_{pb}^*$ = 425 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b -2t_{st} + d_c - 2k_c) / 90$ = 0.29 in

Since t_{wc} = 0.55 in **>** t_{ReqD} , a doubler plate is not required.

Technical References:

1. AISC 341-10: "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, 2010.
2. AISC 358-10: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications", American Institute of Steel Construction, 2010.
3. AISC 360-10: "Specification for Structural Steel Buildings", American Institute of Steel Construction, 2010.
4. FEMA 350: "Recommended Seismic Design Criteria for New Steel Moment-frame Buildings.", SAC Joint Venture, 2000.

Steel Stair Design Based on AISC 360-10/16

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION (Tube or Pipe) => **HSS4X4X1/4**

Tube	A	r_{min}	t	h
	3.37	1.52	0.23	4.00

FLOOR BEAM - 1 => **W16X26**

	A	d	I_x	S_x	Z_x
	7.68	15.70	301.00	38.40	44.20

STRINGER - 1 (Channel or Tube) => **MC8X8.5**

Channel	A	d	I_x	S_x	Z_x
	2.50	8.00	23.30	5.82	6.95

STRINGER - 2 (Channel or Tube) => **C12X20.7**

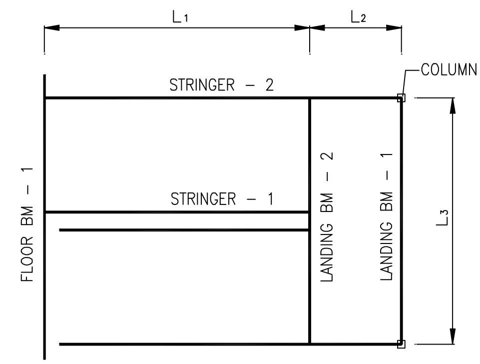
Channel	A	d	I_x	S_x	Z_x
	6.08	12.00	129.00	21.50	25.60

LANDING BEAM - 1 (Channel or Tube) => **C8X18.75**

Channel	A	d	I_x	S_x	Z_x
	5.51	8.00	43.90	11.00	13.90

LANDING BEAM - 2 (Channel or Tube) => **C8X18.75**

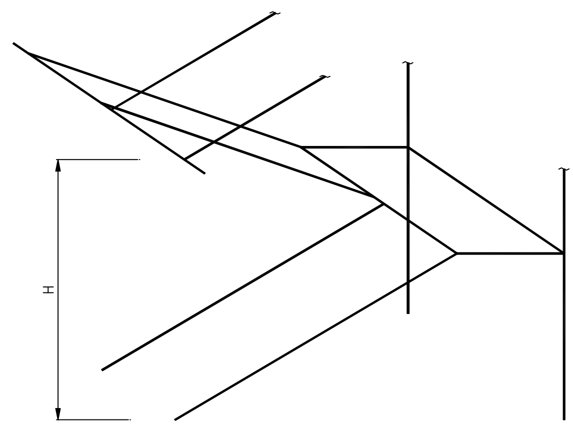
Channel	A	d	I_x	S_x	Z_x
	5.51	8.00	43.90	11.00	13.90



DIMENSIONS
H = **16** ft, story Ht
L_1 = **12** ft
L_2 = **6** ft
L_3 = **10** ft

NUMBER OF STORIES n = **2**

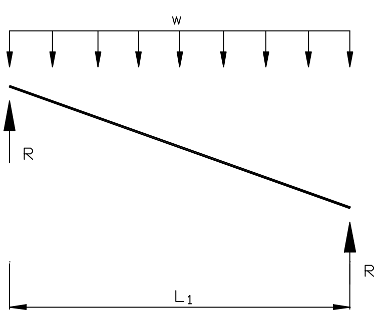
GRAVITY LOAD
DL = **50** psf
LL = **100** psf



THE STAIR DESIGN IS ADEQUATE.

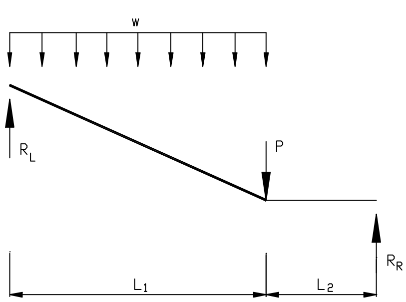
ANALYSIS

STRINGER - 1



θ = 33.69 deg, from horizontal
$w = 0.25\ (DL / \cos\theta + LL)\ L_3$ = 400 plf , projected
$R = 0.5\ w\ L_1$ = 2.40 kips
$M = w\ L_1^2 / 8$ = 7.20 ft-kips
F_y = 36.00 ksi
$M_n / \Omega_b = F_y Z_x / 1.67$ = 12.49 ft-kips
\> M **[Satisfactory]**
E = 29000 ksi
$\Delta_{LL} = 5\ (w_{LL} \cos\theta)\ (L_1 / \cos\theta)^4 / (384\ E\ I)$ = 0.46 in
< $(L_1 / \cos\theta) / 240$ = 0.72 in **[Satisfactory]**

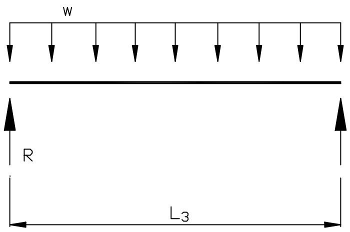
STRINGER - 2



w = 400 plf , projected, from STRINGER - 1
P = 6.00 kips, from LANDING BEAM - 2
$R_L = [w\ L_1\ (0.5\ L_1 + L_2) + P\ L_2] / (L_1 + L_2)$ = 5.20 kips
$R_R = [w\ L_1\ (0.5\ L_1) + P\ L_1] / (L_1 + L_2)$ = 5.60 kips
$X = R_L / w$ = 12.00 ft, from left
$M_{max} = R_L X - (0.5\ w\ X^2)$ = 33.61 ft-kips
F_y = 36.00 ksi
$M_n / \Omega_b = F_y Z_x / 1.67$ = 45.99 ft-kips
\> M **[Satisfactory]**
$\Delta_{LL} = 5\ w_{LL}\ (L_1 + L_2)^4 / (384\ E\ I) + P_{LL}\ (L_1 + L_2)^3 / (48\ E\ I)$ = 0.29
< $(L_1 + L_2) / 240$ = 0.90 in **[Satisfactory]**

(cont'd)

LANDING BEAM - 1



$w = 0.5\,(DL + LL)\,L_2 =$ 450 plf

$R = 0.5\,w\,L_3 =$ 2.25 kips

$M = w\,L_3^2 / 8 =$ 5.63 ft-kips

$F_y =$ 36.00 ksi

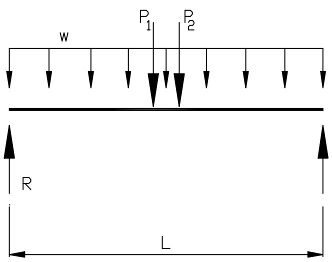
$M_n / \Omega_b = F_y Z_x / 1.67 =$ 24.97 ft-kips

$>$ M **[Satisfactory]**

$\Delta_{LL} = 5\,w_{LL}\,L_3^4 / (384\,E\,I) =$ 0.05 in

$<$ $L_3 / 240 =$ 0.50 in **[Satisfactory]**

LANDING BEAM - 2



$L =$ 12 ft

$w =$ 600 plf, floor gravity load

$P = P1 + P2 =$ 4.80 kips, total point loads, from STRINGER - 1

$R = 0.5\,w\,L + 0.5\,P =$ 6.00 kips

$M = w\,L^2 / 8 + P\,L_3 / 4 =$ 25.21 ft-kips

$F_y =$ 50.00 ksi

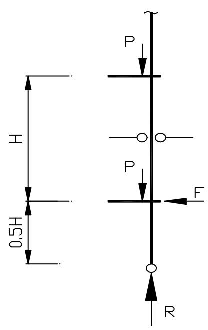
$M_n / \Omega_b = F_y Z_x / 1.67 =$ 110.28 ft-kips

$>$ M **[Satisfactory]**

$\Delta_{LL} = 5\,w_{LL}\,L^4 / (384\,E\,I) + P_{LL}\,L^3 / (48\,E\,I) =$ 0.04 in

$<$ $L / 240 =$ 0.60 in **[Satisfactory]**

COLUMN



$P =$ 7.85 kips

$R = n\,P =$ 15.70 kips

$KL = H =$ 16 ft

$K =$ 1.0

$F_y =$ 46.00 ksi

$K\,l / r =$ 126

$F_e =$ 18 ksi

$F_{cr} =$ 16 ksi

$P_n / \Omega_c = F_{cr} A_g / 1.67 =$ 31.80 kips

$>$ R **[Satisfactory]**

$M_c = M_n / \Omega_b =$ 17.98 / 1.67 = 10.77 ft-kips, (AISC 360 F)

$F =$ **0.18** W = 0.18 x (**50** psf x **45.0** ft^2) = **0.41** kips, ASD

(If no landing seismic load, F shall be zero.)

$M_r = F\,H / 4 =$ 1.62 ft-kips

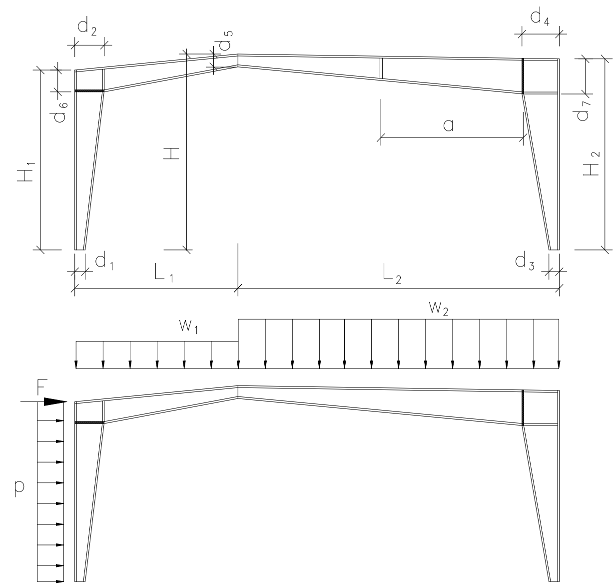
$$\begin{cases} \dfrac{P_r}{P_c} + \dfrac{8}{9}\left(\dfrac{M_r}{M_c}\right), & for\ \dfrac{P_r}{P_c} \geq 0.2 \\ \dfrac{P_r}{2P_c} + \left(\dfrac{M_r}{M_c}\right), & for\ \dfrac{P_r}{P_c} < 0.2 \end{cases} = 0.63 < 1.0 \quad \textbf{[Satisfactory]}$$

$\Delta_F = F\,H^3 / (48\,E\,I) =$ 0.26 in

$<$ $H / 240 =$ 0.80 in **[Satisfactory]**

Web Tapered Frame Design Based on AISC-ASD 9th, Appendix F

INPUT DATA & DESIGN SUMMARY



STEEL YIELD STRESS	F_y =	36	ksi
LEFT COLUMN DIMENSIONS	b_f =	5	in
	t_f =	0.1875	in
	t_w =	0.1345	in
	d_1 =	12	in
	d_2 =	17	in
	H_1 =	12	ft
RIGHT COLUMN DIMENSIONS	b_f =	5	in
	t_f =	0.25	in
	t_w =	0.25	in
	d_3 =	12	in
	d_4 =	34	in
	H_2 =	14	ft
LEFT BEAM DIMENSIONS	b_f =	5	in
	t_f =	0.25	in
	t_w =	0.3125	in
	d_5 =	9	in
	d_6 =	13	in
	L_1 =	15	ft
RIGHT BEAM DIMENSIONS	b_f =	5	in
	t_f =	0.25	in
	t_w =	0.3125	in
	d_5 =	9	in
	d_7 =	38	in
	L_2 =	26	ft

RIDGE HEIGHT	H =	18	ft
GRAVITY LOAD	w_1 =	0.24	kips / ft (" - " for wind uplift)
	w_2 =	0.5	kips / ft (" - " for wind uplift)
LATERAL LOAD	F =	3.6	kips (" - " to left direction)
	p =	0.5	kips / ft (" - " to left direction)
BEAM STIFFENER SPACING	a_{bm} =	10	ft
COLUMN STIFFENER SPACING	a_{col} =	5	ft
UNBRACED LENGTH / PURLIN SPACING	$L_{b,top}$ =	5	ft
UNBRACED LENGTH AT BOTTOM FLANGE	$L_{b,bot}$ =	10	ft

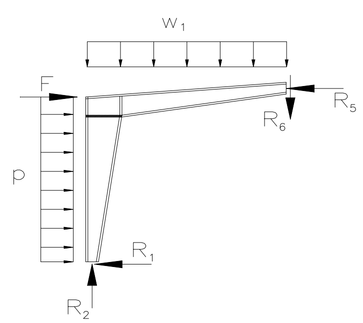
(Diaphragm is not bracing member. ***L*** is different with " l " in F1.3, pg 5-47)

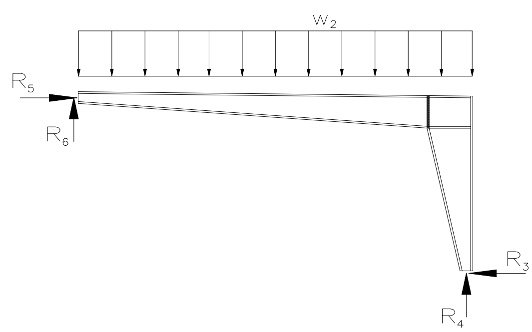
THE FRAME DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE REACTIONS

R_1 =	0.4	kips
R_2 =	7.6	kips
R_3 =	3.6	kips
R_4 =	9.0	kips
R_5 =	3.6	kips
R_6 =	4.0	kips





DETERMINE ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , & for \ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , & for \ F_{b\gamma} \le F_y/3 \end{cases}$$

where $A_f = t_f\, b_f$

$A_{To} = t_f\, b_f + d_0\, t_w / 6$

γ = MIN[$(d_L - d_0) / d_0$, 0.268 L/d_0, 6.0]

$I_{To} = (t_f\, b_f^3 + d_0\, t_w^3 / 6) / 12$

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}}$$

$$F_{s\gamma} = \frac{12000}{h_s L d_o / A_f}$$

$$h_s = 1.0 + 0.0230\gamma\sqrt{\frac{L d_o}{A_f}}$$

$$F_{w\gamma} = \frac{170000}{(h_w L / r_{To})^2}$$

(cont'd)

$$h_w = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}} \qquad\qquad B = \frac{1.75}{1.0+0.25\sqrt{\gamma}}$$

	Length	γ	A_{To}	I_{To}	r_{To}	H_s	H_w	$F_{s\gamma}$	$F_{w\gamma}$	B	$F_{b\gamma}$
Left Col	11.5	0.42	1.21	2	1.27	1.40	1.02	4.85	14.03	1.51	**17.56**
Right Col	12.5	1.83	1.75	3	1.22	2.60	1.08	3.21	9.68	1.31	**13.20**
L. Bm (+)	15.6	0.44	1.72	3	1.23	1.21	1.01	22.91	69.97	1.50	**21.60**
(-)	15.6	0.44	1.72	3	1.23	1.30	1.02	10.68	17.32	1.50	**19.28**
R. Bm (+)	25.1	1.79	1.72	3	1.23	1.85	1.05	14.98	65.24	1.31	**21.60**
(-)	25.1	3.22	1.72	3	1.23	3.18	1.12	4.37	14.22	1.21	**15.99**

DETERMINE ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y\ ,\ for\ h/t_w \le 380\sqrt{F_y} \\ \dfrac{C_v F_y}{2.89} \le 0.4F_y\ ,\ for\ h/t_w > 380\sqrt{F_y} \end{cases}$$

where $h = d_L - 2\,t_f$

$$k_v = \begin{cases} 4.0+\dfrac{5.34}{(a/h)^2}\ ,\ for\ a/h \le 1.0 \\ 5.34+\dfrac{4.0}{(a/h)^2}\ ,\ for\ a/h > 1.0 \end{cases} \qquad C_v = \begin{cases} \dfrac{45000k_v}{F_y(h/t_w)^2}\ ,\ for\ C_v \le 0.8 \\ \dfrac{190}{h/t_w}\sqrt{\dfrac{k_v}{F_y}}\ ,\ for\ C_v > 0.8 \end{cases}$$

	a	h	h/t_w	$380/F_y^{0.5}$	K_v	C_v	$F_{b\gamma}$
Left Col	10.0	16.63	124	63	5.42	0.44	**5.52**
Right Col	10.0	33.50	134	63	5.65	0.39	**4.90**
Left Bm	5.0	12.50	40	63	5.51	1.86	**14.40**
Right Bm	5.0	37.50	120	63	6.90	0.60	**7.46**

DETERMINE ALLOWABLE COMPRESSIVE STRESS (APP. F7.3, pg 5-102)

$$F_{a\gamma} = \begin{cases} \dfrac{\left(1.0-\dfrac{S^2}{2C_c^2}\right)F_y}{\dfrac{5}{3}+\dfrac{3S}{8C_c}-\dfrac{S^3}{8C_c^3}}\ ,\ for\ S \le C_c \\ \dfrac{12\pi^2 E}{23S^2}\ ,\ for\ S > C_c \end{cases}$$

where K_γ = (effective length factor by an analysis)

$S = K_\gamma\, l / r_{ox}$

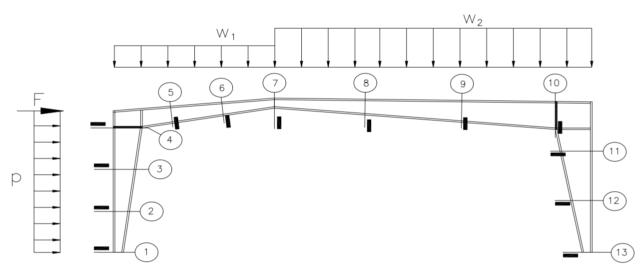
$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

E = 29000 ksi

	l	K_γ	C_c	I_x	A	r_{ox}	S	$F_{a\gamma}$
Left Col	11.5	2.0	126	53	3.49	3.90	70.61	**16.37**
Right Col	12.5	2.0	126	81	5.50	3.84	78.16	**15.56**
Left Bm	15.6	2.5	126	44	5.31	2.89	161.71	**5.71**
Right Bm	25.1	2.5	126	44	5.31	2.89	261.07	**2.19**

CHECK EACH SECTION CAPACITIES

	d_0	d_L	t_f	b_f	t_w	A_f
Left Col	12	17	0.1875	5	0.1345	0.9375
Right Col	12	34	0.25	5	0.25	1.25
Left Bm	9	13	0.25	5	0.3125	1.25
Right Bm	9	38	0.25	5	0.3125	1.25



Section	1	2	3	4	5	6	7	8	9	10	11	12	13
d (in)	12	14	15	17	12	10	9	19	28	38	27	19	12
I (in^4)	87	116	151	191	126	95	70	387	1094	2331	840	384	126
A_w (in^2)	1.61	1.84	2.06	2.29	3.65	3.23	2.81	5.83	8.85	11.88	6.67	4.83	3.00
N (kips)	7.6	7.6	7.6	7.6	3.6	3.6	3.6	3.6	3.6	3.6	9.0	9.0	9.0
V (kips)	0.36	-1.64	-3.64	-5.10	6.18	5.09	4.00	0.14	-3.72	-7.58	3.60	3.60	3.60
M (ft-k)	0.0	-2.0	-10.6	-25.8	-31.7	-13.4	0.0	20.8	11.8	-27.0	33.6	16.8	0.0
f_a (ksi)	2.18	2.05	1.93	1.83	0.59	0.63	0.68	0.43	0.32	0.25	0.98	1.23	1.64
F_a (ksi)	16.37	16.37	16.37	16.37	5.71	5.71	5.71	2.19	2.19	2.19	15.56	15.56	15.56
f_v (ksi)	0.22	0.89	1.76	2.23	1.69	1.58	1.42	0.02	0.42	0.64	0.54	0.75	1.20
F_v (ksi)	5.52	5.52	5.52	5.52	14.40	14.40	14.40	7.46	7.46	7.46	4.90	4.90	4.90
f_b (ksi)	0.00	1.41	6.48	13.83	17.55	8.69	0.00	6.02	1.84	2.64	6.41	5.08	0.00
F_b (ksi)	17.56	17.56	17.56	17.56	19.28	19.28	21.60	21.60	21.60	15.99	13.20	13.20	13.20

$f_a < F_a$ **[Satisfactory]** $\qquad f_v < F_v$ **[Satisfactory]** $\qquad f_b < F_b$ **[Satisfactory]**

(cont'd)

CHECK COMBINED FLEXURE AND AXIAL FORCE (APP. F7.4, pg 5-104)

$$(f_a \ , \ f_b) = \begin{cases} \dfrac{f_{a0}}{F_{a\gamma}} + \dfrac{f_{bl}}{F_{b\gamma}} \ , \quad \mathit{for} \quad \dfrac{f_{a0}}{F_{a\gamma}} \le 0.15 \\ Larger \quad of \quad \begin{pmatrix} \dfrac{f_{a0}}{F_{a\gamma}} + \dfrac{C_m' f_{bl}}{\left(1 - \dfrac{f_{a0}}{F'_{e\gamma}}\right) F_{b\gamma}} \\ \dfrac{f_{a0}}{0.6 F_y} + \dfrac{f_{bl}}{F_{b\gamma}} \end{pmatrix} \ , \quad \mathit{for} \quad \dfrac{f_{a0}}{F_{a\gamma}} > 0.15 \end{cases} \le 1.3$$

	f_{a0}	f_{bl}	$F_{a\gamma}$	$F_{b\gamma}$	$F_{e\gamma}'$	C_m'	(f_a, f_b)
Left Col	2.18	13.83	16.37	17.56	29.95	0.94	**0.92**
Right Col	1.64	6.41	15.56	13.20	24.45	0.94	**0.59**
Left Bm	0.68	17.55	5.71	19.28	5.71	0.90	**1.03**
Right Bm	0.68	6.41	2.19	15.99	2.19	0.78	**0.76**

[Satisfactory]

Technical Reference:

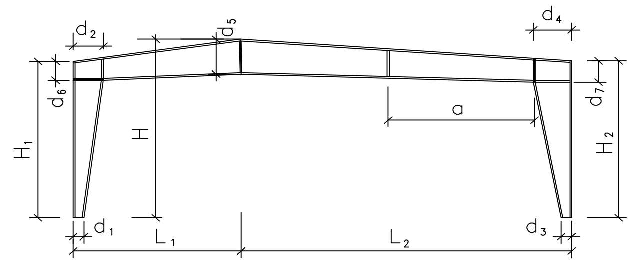
1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.

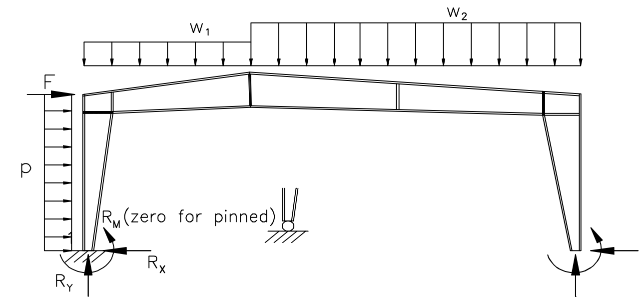
Purchaser's LOGO

PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Web Tapered Portal Design Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25

INPUT DATA & DESIGN SUMMARY





STEEL YIELD STRESS F_y = **50** ksi

LEFT COLUMN DIMENSIONS
- b_f = **8** in
- t_f = **0.1875** in
- t_w = **0.25** in
- d_1 = **12** in
- d_2 = **17** in
- H_1 = **12** ft

RIGHT COLUMN DIMENSIONS
- b_f = **8** in
- t_f = **0.25** in
- t_w = **0.25** in
- d_3 = **22** in
- d_4 = **34** in
- H_2 = **14** ft

LEFT BEAM DIMENSIONS
- b_f = **8** in
- t_f = **0.375** in
- t_w = **0.25** in
- d_5 = **48** in
- d_6 = **13** in
- L_1 = **30** ft

RIGHT BEAM DIMENSIONS
- b_f = **8** in
- t_f = **0.375** in
- t_w = **0.25** in
- d_5 = **48** in
- d_7 = **22** in
- L_2 = **41** ft

RIDGE HEIGHT H = **18** ft

GRAVITY LOAD w_1 = **0.24** kips / ft (" - " for wind uplift)

w_2 = **0.5** kips / ft (" - " for wind uplift)

LATERAL LOAD F = **18** kips (" - " to left direction)

p = **0.5** kips / ft (" - " to left direction)

BEAM STIFFENER SPACING a_{bm} = **10** ft

COLUMN STIFFENER SPACING a_{col} = **5** ft

UNBRACED LENGTH / PURLIN SPACING $L_{b,top}$ = **5** ft

UNBRACED LENGTH AT BOTTOM FLANGE $L_{b,bot}$ = **10** ft

(Diaphragm is not bracing member. L is different with " l " in F1.3, pg 5-47)

BASE (d_1 & d_3) PINNED ? **No** , (fixed)

THE FRAME DESIGN IS ADEQUATE.

HORIZONTAL DRIFT Δ_H = **0.10** in, (horiz. to right)

BEAM DEFLECTION Δ_{max} / L = **1 / 487**

ANALYSIS

DETERMINE REACTIONS

R_X =	-3.6 kips (Left)	R_X =	24.6 kips (Right)
R_Y =	9.5 kips (Left)	R_Y =	18.2 kips (Right)
R_M =	-18.6 ft-kips (Left)	R_M =	-108.6 ft-kips (Right)

DETERMINE ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \ for \ F_{b\gamma} > F_y / 3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \ for \ F_{b\gamma} \le F_y / 3 \end{cases}$$

where $A_f = t_f\, b_f$

$A_{To} = t_f\, b_f + d_0\, t_w / 6$

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}}$$

$$h_s = 1.0 + 0.0230\gamma\sqrt{\frac{L d_o}{A_f}}$$

$$h_w = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}}$$

γ = MIN[(d_L - d_0) / d_0 , 0.268 L/d_0, 6.0]

I_{To} = ($t_f\, b_f^3$ + $d_0\, t_w^3$ / 6) / 12

$$F_{s\gamma} = \frac{12000}{h_s L d_o / A_f}$$

$$F_{w\gamma} = \frac{170000}{(h_w L / r_{To})^2}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}}$$

(cont'd)

	Length	γ	A_{To}	I_{To}	r_{To}	H_s	H_w	$F_{s\gamma}$	$F_{w\gamma}$	B	$\boldsymbol{F_{b\gamma}}$
Left Col	11.5	0.42	2.00	8	2.00	1.32	1.01	8.26	34.91	1.51	**28.19**
Right Col	13.2	0.55	2.92	11	1.91	1.52	1.02	4.54	24.02	1.48	**25.64**
L. Bm (+)	29.6	1.24	3.54	16	2.13	1.46	1.03	31.64	202.97	1.37	**30.00**
(-)	29.6	2.47	3.54	16	2.13	2.30	1.07	10.04	46.46	1.26	**28.68**
R. Bm (+)	39.7	0.73	3.92	16	2.02	1.35	1.02	20.16	187.18	1.44	**30.00**
(-)	39.7	1.18	3.92	16	2.02	1.81	1.04	7.55	45.03	1.38	**28.91**

DETERMINE ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ for \ h/t_w \le 380\sqrt{F_y} \\ \dfrac{C_v F_y}{2.89} \le 0.4F_y \ , \ for \ h/t_w > 380\sqrt{F_y} \end{cases}$$

where $h = d_L - 2\,t_f$

$$k_v = \begin{cases} 4.0 + \dfrac{5.34}{(a/h)^2} \ , \ for \ a/h \le 1.0 \\ 5.34 + \dfrac{4.0}{(a/h)^2} \ , \ for \ a/h > 1.0 \end{cases}$$

$$C_v = \begin{cases} \dfrac{45000 k_v}{F_y (h/t_w)^2} \ , \ for \ C_v \le 0.8 \\ \dfrac{190}{h/t_w}\sqrt{\dfrac{k_v}{F_y}} \ , \ for \ C_v > 0.8 \end{cases}$$

	a	h	h/t_w	$380/F_y^{0.5}$	K_v	C_v	$\boldsymbol{F_{b\gamma}}$
Left Col	10.0	16.63	67	54	5.42	0.94	**16.27**
Right Col	10.0	33.50	134	54	5.65	0.28	**4.90**
Left Bm	5.0	47.25	189	54	7.82	0.20	**3.41**
Right Bm	5.0	47.25	189	54	7.82	0.20	**3.41**

DETERMINE ALLOWABLE COMPRESSIVE STRESS (APP. F7.3, pg 5-102)

$$F_{a\gamma} = \begin{cases} \dfrac{\left(1.0 - \dfrac{S^2}{2C_c^2}\right)F_y}{\dfrac{5}{3} + \dfrac{3S}{8C_c} - \dfrac{S^3}{8C_c^3}} \ , \ for \ S \le C_c \\ \dfrac{12\pi^2 E}{23 S^2} \ , \ for \ S > C_c \end{cases}$$

where $K_\gamma =$ (effective length factor by an analysis)

$S = K_\gamma\, l / r_{ox}$

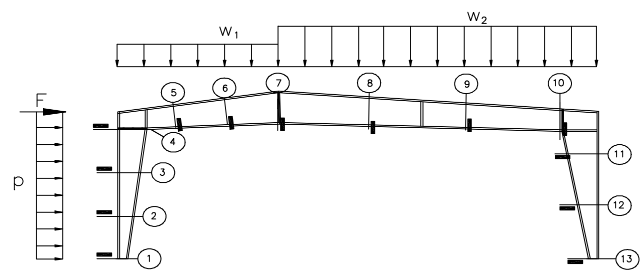
$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

$E =$ 29000 ksi

	l	K_γ	C_c	I_x	A	r_{ox}	S	$\boldsymbol{F_{a\gamma}}$
Left Col	11.5	2.0	107	90	6.00	3.87	71.14	**20.73**
Right Col	13.2	2.0	107	464	9.50	6.99	45.20	**25.08**
Left Bm	29.6	2.5	107	173	9.25	4.32	205.91	**3.52**
Right Bm	39.7	2.5	107	585	11.50	7.13	166.97	**5.36**

CHECK EACH SECTION CAPACITIES

	d_0	d_L	t_f	b_f	t_w	A_f
Left Col	12	17	0.1875	8	0.25	1.5
Right Col	22	34	0.25	8	0.25	2
Left Bm	13	48	0.375	8	0.25	3
Right Bm	22	48	0.375	8	0.25	3



Section	1	2	3	4	5	6	7	8	9	10	11	12	13
d (in)	12	14	15	17	25	36	48	39	31	22	30	26	22
I (in^4)	144	193	251	319	1225	2979	5760	3588	2012	948	1463	1042	706
A_w (in^2)	3.00	3.42	3.83	4.25	6.17	9.08	12.00	9.83	7.67	5.50	7.50	6.50	5.50
N (kips)	9.5	9.5	9.5	9.5	24.6	24.6	24.6	24.6	24.6	24.6	18.2	18.2	18.2
V (kips)	-3.55	-5.55	-7.55	-9.01	6.84	4.56	2.86	-3.50	-9.87	-16.23	24.55	24.55	24.55
M (ft-k)	18.6	-29.8	-44.7	-63.0	120.0	130.8	119.8	148.4	96.0	-37.2	120.6	6.0	108.6
$\boldsymbol{f_a}$ (ksi)	1.58	1.48	1.39	1.31	2.02	1.63	1.36	1.55	1.80	2.13	1.59	1.74	1.92
$\boldsymbol{F_a}$ (ksi)	20.73	20.73	20.73	20.73	3.52	3.52	3.52	5.36	5.36	5.36	25.08	25.08	25.08
$\boldsymbol{f_v}$ (ksi)	1.18	1.62	1.97	2.12	1.11	0.50	0.24	0.36	1.29	2.95	3.27	3.78	4.46
$\boldsymbol{F_v}$ (ksi)	16.27	16.27	16.27	16.27	3.41	3.41	3.41	3.41	3.41	3.41	4.90	4.90	4.90
$\boldsymbol{f_b}$ (ksi)	9.30	12.66	16.34	20.14	14.49	9.57	5.99	9.76	8.79	5.18	14.84	0.90	20.30
$\boldsymbol{F_b}$ (ksi)	28.19	28.19	28.19	28.19	30.00	30.00	30.00	30.00	30.00	28.91	25.64	25.64	25.64

$\boldsymbol{f_a} < F_a$ **[Satisfactory]** $\boldsymbol{f_v} < F_v$ **[Satisfactory]** $\boldsymbol{f_b} < F_b$ **[Satisfactory]**

(cont'd)

CHECK COMBINED FLEXURE AND AXIAL FORCE (APP. F7.4, pg 5-104)

$$\left(f_a \quad f_b\right) = \begin{cases} \dfrac{f_{a0}}{F_{a\gamma}} + \dfrac{f_{bl}}{F_{b\gamma}} & \text{for } \dfrac{f_{a0}}{F_{a\gamma}} \le 0.15 \\ \text{Larger of } \begin{pmatrix} \dfrac{f_{a0}}{F_{a\gamma}} + \dfrac{C_m\, f_{bl}}{\left(1 - \dfrac{f_{a0}}{F_{e\gamma}}\right) F_{b\gamma}} \\ \dfrac{f_{a0}}{0.\, F_y} + \dfrac{f_{bl}}{F_{b\gamma}} \end{pmatrix} & \text{for } \dfrac{f_{a0}}{F_{a\gamma}} > 0.15 \end{cases} \le 1.3$$

	f_{a0}	f_{bl}	$F_{a\gamma}$	$F_{b\gamma}$	$F_{e\gamma}'$	C_m	(f_a, f_b)
Left Col	1.58	20.14	20.73	28.19	29.51	0.95	**0.79**
Right Col	1.92	14.84	25.08	25.64	73.09	0.98	**0.66**
Left Bm	1.36	20.14	3.52	28.68	3.52	0.74	**1.24**
Right Bm	1.36	14.84	5.36	28.91	5.36	0.81	**0.81**

[Satisfactory]

Technical Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. AISC: "Design Guide 25: Frame Design Using Web-Tapered Members, 2010.

Daniel T. Li
Engineering International

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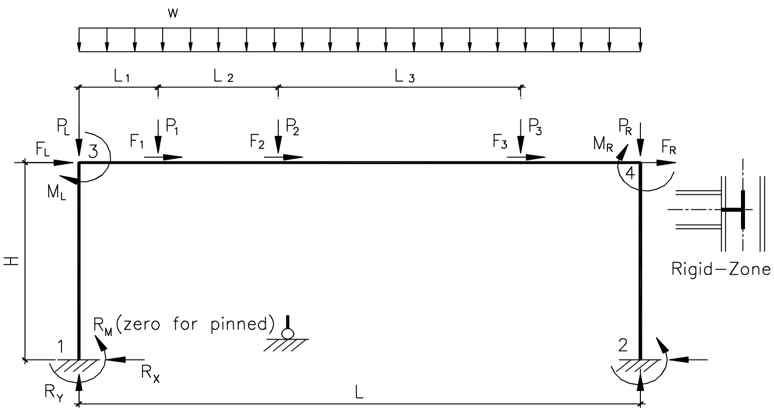
DESIGN BY :
REVIEW BY :

Portal Frame Analysis using Finite Element Method



INPUT DATA & SUMMARY

COLUMN SIZE (WF, Tube, or Pipe)

1 - 3 = **W18x234** WF
ORIENTATION = **x-x**
2 - 4 = **W18x234** WF
ORIENTATION = **x-x**

BEAM SIZE (WF, Tube, or Pipe)

3 - 4 = **W24X250** WF
ORIENTATION = **x-x**

DIMENSIONS

H = **15** ft
L = **40** ft
L_1 = **10** ft
L_2 = **10** ft
L_3 = **10** ft

LOADS

P_L = **81** kips, (downward)
F_L = **10** kips, (horiz. to right)
M_L = **100** ft-kips, (clockwise)
P_1 = **0**
F_1 = **0**
P_2 = **79** kips, (downward)
F_2 = **13.5** kips, (horiz. to right)
P_3 = **145** kips, (downward)
F_3 = **-77** kips, (horiz. to left)
P_R = **0**
F_R = **0**
M_R = **-150** ft-kips, (anticlockwise)
w = **0.2501** kips / ft, (downward)

BASE (1 & 2) PINNED ? **Yes** , (pinned)
RIGID-ZONE LENGTH FACTOR **75** % of (d / 2)

DESIGN SECTION FORCES

Column	N_{13} =	-183.1	k,(compre.)	N_{24} =	-131.9	k,(compression)
	V_{13} =	79.7	kips	V_{24} =	26.2	kips
	M_{13} =	1130.5	ft-kips	M_{24} =	371.9	ft-kips
Beam	N_{34} =	-103.2	k,(compression)			
	V_{34} =	130.7	kips			
	$M_{34, max}$ =	1065.5	ft-kips			

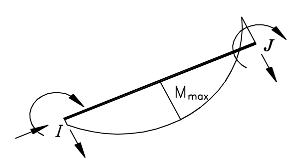
HORIZONTAL DRIFT

$\Delta_{H,3}$ = -0.8007 in, (horiz. to left)
$\Delta_{H,4}$ = -0.8178 in, (horiz. to left)

BEAM DEFLECTION Δ_{max} / L = 1 / 587

ANALYSIS

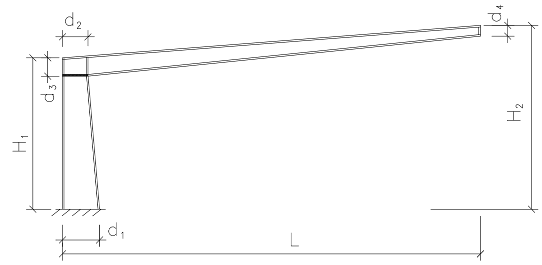
	Coordinates (ft)		Reaction (k, ft-k)			Joint J Deflection (in,deg)		
Joint	X	Y	R_X	R_Y	R_M	X	Y	θ
1	0	0	-79.73	183.06	0.00	0.000	0.000	-0.421
2	40	0	26.23	131.94	0.00	0.000	0.000	-0.206
3	0	15				0.801	-0.016	0.061
4	40	15				0.818	-0.011	-0.364
P_1/F_1	10	15				0.805	-0.392	0.242
P_2/F_2	20	15				0.811	-0.831	0.106
P_3/F_3	30	15				0.816	-0.722	-0.223



TYPICAL POSITIVE FORCES

End Joint		A	I	E	Length	Moment (ft-k)			Axial	Shear (k)	
I	J	(in^2)	(in^4)	(ksi)	(ft)	I	Mid	J	(k)	I	J
1	3	68.8	4900	29000	14.18	0.00	565.23	1130.45	183.06	-79.73	79.73
2	4	68.8	4900	29000	14.18	0.00	-185.96	-371.92	131.94	26.23	-26.23
3	P_1/F_1	73.5	8490	29000	9.34	-1027.42	559.32	85.76	89.73	-100.81	100.81
P_1/F_1	P_2/F_2	73.5	8490	29000	10.00	-85.76	-402.68	-897.37	89.73	-98.31	98.31
P_2/F_2	P_3/F_3	73.5	8490	29000	10.00	897.37	-978.31	-1065.49	103.23	-16.81	16.81
P_3/F_3	4	73.5	8490	29000	9.34	1065.49	-452.41	155.23	26.23	130.69	-130.69

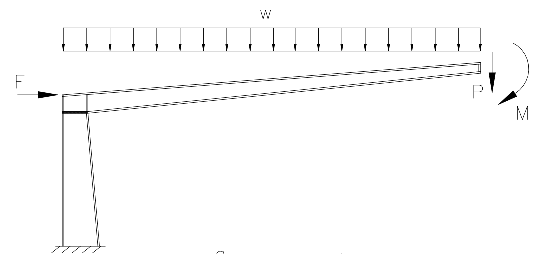
Web-Tapered Cantilever Frame Design Based on AISC-ASD 9th, Appendix F

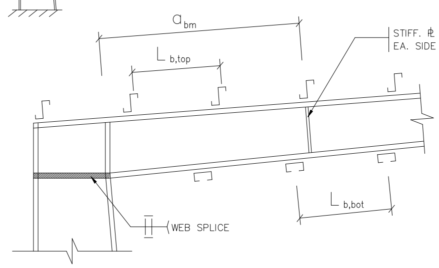


DESIGN CRITERIA

1. IN ORDER TO QUALIFY UNDER THIS DESIGN, THE FLANGES SHALL BE OF EQUAL AND CONSTANT AREA. (APP. F7.1.b, page 5-102)
2. DIAPHRAGM IS NOT BRACING MEMBER, SINCE ***L*** IS DIFFERENT WITH *ℓ* in F1.3, page 5-47.
3. TOP END FORCES, P & M, SHOULD INCLUDE IMPACT FACTOR, 1.25, IF THEY ARE FROM MOVABLE COVER. (A4.2, page 5-29)

INPUT DATA & DESIGN SUMMARY





COLUMN DIMENSIONS	b_f =	96	in (2438 mm)
	t_f =	1	in (25 mm)
	t_w =	0.75	in (19 mm)
	d_1 =	780	in (19812 mm)
	d_2 =	700	in (17780 mm)
	H_1 =	200	ft (61.0 m)
BEAM DIMENSIONS	b_f =	96	in (2438 mm)
	t_f =	1	in (25 mm)
	t_w =	0.75	in (19 mm)
	d_3 =	550	in (13970 mm)
	d_4 =	240	in (6096 mm)
	L =	323.7	ft (98.7 m)
END HEIGHT	H_2 =	224.6	ft (68.5 m)
BEAM STIFFENER SPACING	a_{bm} =	8	ft (2.4 m)
COLUMN STIFFENER SPACING	a_{col} =	8	ft (2.4 m)
UNBRACED LENGTH / PURLIN SPACING	$L_{b,top}$ =	36	ft (11.0 m)
UNBRACED LENGTH AT BOTTOM FLANGE	$L_{b,bot}$ =	36	ft (11.0 m)
UNBRACED LENGTH AT OUTSIDE OF COLUMN FLANGE	$L_{c,outside}$ =	36	ft (11.0 m)
FRAME SPACING	S =	120	ft (36.6 m)
STEEL YIELD STRESS	F_y =	50	ksi (345 N / mm^2)
TOTAL GRAVITY LOAD	w =	36	psf (" - " for wind uplift)
	=	4.32	kips / ft (63.0 kN / m)
AVAILABLE LIVE ROAD	w_{LL} =	21.4	psf (104 kg / m^2)
LATERAL LOAD	F =	377.6	kips (1679.4 kN, " - " to left direction)
END LOADS	P =	874.0	kips (3887.5 kN, " - " to uplift)
	M =	70727.6	ft-kips (314596.5 kN-m, " - " to uplift)
STEEL COST	*Weight* =	31.1	psf (152 kg / m^2)

THE FRAME DESIGN IS ADEQUATE.

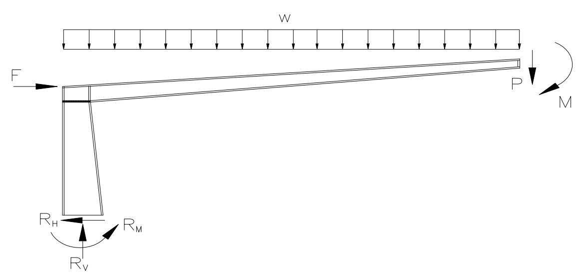
ANALYSIS

DETERMINE REACTIONS

R_H = 377.6 kips

R_V = 2272.4 kips

R_M = 372568.8 ft-kips



DETERMINE ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \dfrac{2}{3}\left[1.0 - \dfrac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \quad for \ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \quad for \ F_{b\gamma} \le F_y/3 \end{cases}$$

where $A_f = t_f\, b_f$

$A_{To} = t_f\, b_f + d_0\, t_w / 6$

$\gamma = \mathrm{MIN}[(d_L - d_0) / d_0 \ , \ 0.268\ L/d_0, \ 6.0]$

$I_{To} = (t_f\, b_f^3 + d_0\, t_w^3 / 6) / 12$

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}}$$

$$F_{s\gamma} = \frac{12000}{h_s L d_o / A_f}$$

$$h_S = 1.0 + 0.0230\gamma\sqrt{\frac{L d_o}{A_f}}$$

$$F_{w\gamma} = \frac{170000}{(h_w L / r_{To})^2}$$

(cont'd)

$$h_w = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}} \qquad B = \frac{1.75}{1.0+0.25\sqrt{\gamma}}$$

	Length	γ	A_{To}	I_{To}	r_{To}	H_s	H_w	$F_{s\gamma}$	$F_{w\gamma}$	B	$F_{b\gamma}$
Column	177.1	0.11	183.50	73732	20.05	1.15	1.00	3.32	364.527	1.61	**30.00**
Beam (+)	296.9	0.48	126.00	73729	24.19	1.36	1.01	8.14	524.76	1.49	**30.00**
Beam (-)	296.9	0.48	126.00	73729	24.19	1.36	1.01	8.14	524.76	1.49	**30.00**

DETERMINE ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ for \ h/t_w \le 380\sqrt{F_y} \\ \frac{C_v F_y}{2.89} \le 0.4F_y \ , \ for \ h/t_w > 380\sqrt{F_y} \end{cases}$$

where $h = d_L - 2\,t_f$

$$k_v = \begin{cases} 4.0 + \frac{5.34}{(a/h)^2} \ , \ for \ a/h \le 1.0 \\ 5.34 + \frac{4.0}{(a/h)^2} \ , \ for \ a/h > 1.0 \end{cases} \qquad C_v = \begin{cases} \frac{45000 k_v}{F_y (h/t_w)^2} \ , \ for \ C_v \le 0.8 \\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \ , \ for \ C_v > 0.8 \end{cases}$$

	a	h	h/t_w	$380/F_y^{0.5}$	K_v	C_v	$F_{b\gamma}$
Column	8.0	778.00	1037	54	354.72	0.30	**5.13**
Beam	8.0	548.00	731	54	178.00	0.30	**5.19**

DETERMINE ALLOWABLE COMPRESSIVE STRESS (APP. F7.3, pg 5-102)

$$F_{a\gamma} = \begin{cases} \frac{\left(1.0 - \frac{S^2}{2C_c^2}\right)F_y}{\frac{5}{3} + \frac{3S}{8C_c} - \frac{S^3}{8C_c^3}} \ , \ for \ S \le C_c \\ \frac{12\pi^2 E}{23 S^2} \ , \ for \ S > C_c \end{cases}$$

where $K_\gamma =$ (effective length factor by an analysis)

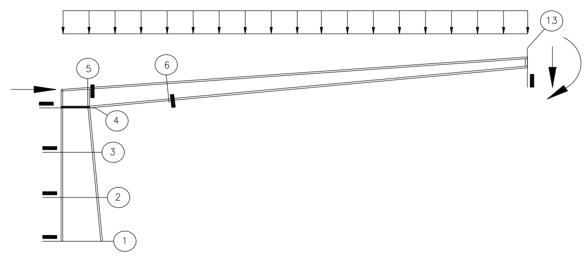
$S = K_\gamma \, l / r_{ox}$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

$E =$ 29000 ksi

	l	K_γ	C_c	I_x	A	r_{ox}	S	$F_{a\gamma}$
Column	36.0	2.0	107	3.3E+07	717.00	215.18	4.02	**29.73**
Beam	36.0	2.5	107	2246400	372.00	77.71	13.90	**28.91**

CHECK EACH SECTION CAPACITIES



	d_0	d_L	t_f	b_f	t_w	A_f
Column	700	780	1	96	0.75	96
Beam	240	550	1	96	0.75	96

Section	1	2	3	4	5	6	7	8	9	10	11	12	13
d (in)	780	753	727	700	550	511	473	434	395	356	318	279	240
I (in^4)	58862700	53960848	49328151.85	4.5E+07	2.5E+07	2.1E+07	1.7E+07	1.4E+07	1.1E+07	8917696	6839074	5083382	3628800
A_w (in^2)	585.00	565.00	545.00	525.00	412.50	383.44	354.38	325.31	296.25	267.19	238.13	209.06	180.00
N (kips)	2272.4	2272.4	2272.4	2272.4	172.2	158.9	145.7	132.5	119.2	106.0	92.7	79.5	66.2
V (kips)	2272.4	2272.4	2272.4	2272.4	2020.4	1876.8	1733.1	1589.5	1445.9	1302.3	1158.7	1015.1	871.5
M (ft-k)	-372569	-353166	-333764	-314361	-222834	-187184	-156287	-130144	-108754	-92118	-80234	-73104	-70728
f_a (ksi)	2.92	3.00	3.08	3.17	0.28	0.28	0.27	0.26	0.24	0.23	0.22	0.20	0.18
F_a (ksi)	29.73	29.73	29.73	29.73	28.91	28.91	28.91	28.91	28.91	28.91	28.91	28.91	28.91
f_v (ksi)	3.88	4.02	4.17	4.33	4.90	4.89	4.89	4.89	4.88	4.87	4.87	4.86	4.84
F_v (ksi)	5.13	5.13	5.13	5.13	5.19	5.19	5.19	5.19	5.19	5.19	5.19	5.19	5.19
f_b (ksi)	29.6	29.6	29.5	29.4	29.5	27.5	25.6	24.0	22.7	22.1	22.3	24.1	28.1
F_b (ksi)	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00

$f_a < F_a$ **[Satisfactory]** $f_v < F_v$ **[Satisfactory]** $f_b < F_b$ **[Satisfactory]**

CHECK COMBINED FLEXURE AND AXIAL FORCE (APP. F7.4, pg 5-104)

$$(f_a \ , \ f_b) = \begin{cases} \frac{f_{a0}}{F_{a\gamma}} + \frac{f_{bl}}{F_{b\gamma}} \ , \ for \ \frac{f_{a0}}{F_{a\gamma}} \le 0.15 \\ Larger \ of \ \begin{pmatrix} \frac{f_{a0}}{F_{a\gamma}} + \frac{C_m' f_{bl}}{\left(1 - \frac{f_{a0}}{F'_{e\gamma}}\right)F_{b\gamma}} \\ \frac{f_{a0}}{0.6F_y} + \frac{f_{bl}}{F_{b\gamma}} \end{pmatrix} \ , \ for \ \frac{f_{a0}}{F_{a\gamma}} > 0.15 \end{cases} \le 1.3$$

(cont'd)

	f_{a0}	f_{bl}	$F_{a\gamma}$	$F_{b\gamma}$	$F_{e\gamma}'$	C_m'	(f_a, f_b)
Column	2.92	29.62	29.73	30.00	9262.11	1.00	**1.09**
Beam	0.28	29.51	28.91	30.00	773.12	1.00	**0.99**

[Satisfactory]

DESIGN STIFFENERS

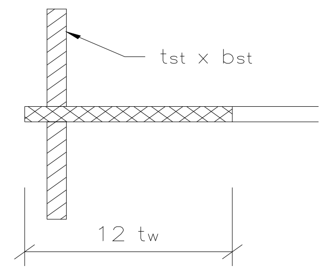
1. ***BEARING STIFFENERS ARE REQUIRED AT EACH END SUPPORT.*** (K1.8, page 5-82)

2. DETERMINE STIFFENER SIZE.

t_w = 1 1/2 in , b_{st} = 9 in

b_{st} / t_w = 6.00 < $95 / F_y^{0.5}$, AISC-ASD, B5.1
[Satisfactory]

A_{eff} = 33.75 in^2 , I = 927 in^4



f_a = 25.9 ksi

E_s = 29000 ksi

$K l / r = 0.75\, h / (I / A_{eff})^{0.5}$ = 34.1

$C_c = (2\pi^2 E_s / F_y)^{0.5}$ = 107

$$F_a = \begin{cases} \dfrac{\left[1 - \dfrac{(kl/r)^2}{2C_c^2}\right] F_y}{\dfrac{5}{3} + \dfrac{3(kl/r)}{8C_c} - \dfrac{(kl/r)^3}{8C_c^3}} , & for\ \dfrac{kl}{r} \le C_c \\ \dfrac{12\pi^2 E}{23(kl/r)^2} , & for\ \dfrac{kl}{r} > C_c \end{cases} = 26.6 \text{ ksi, (AISC-ASD, E2, page 5-42)}$$

> f_a **[Satisfactory]**

DETERMINE STEEL COST AND AVAILABLE ROOF LIVE LOAD

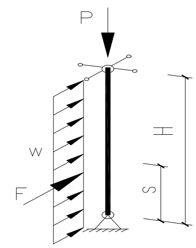
	SW (pcf)	Flanges (ft^3)	Web (ft^3)	Stiffers (ft^3)	Misc.(ft^3)	TA (ft^2)	Weight (lb / ft^2)
Column	490	298.3	682.6	277.3	50	38844	16.5
Beam	490	250.0	610.8	248.2	50	38844	14.6
Σ							*31.1*

Available Roof Live Load = 36.0 psf - 14.6 psf = 21.4 psf

Technical Reference:
1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.

Tube / Pipe Column Design Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY



COLUMN SECTION (Tube or Pipe) **HSS12X12X1/2** **Tube**

COLUMN YIELD STRESS F_y = **46** ksi
DIMENSION H = **40** ft

AXIAL LOAD, ASD P = **87** kips

STRONG AXIS BENDING ? (1=Yes, 0=No) => **0** no, weak axis, y-y, bending.
UNIFORM LATERAL LOAD, ASD w = **0.06** k / ft
CONCENTRATED LATERAL LOAD, ASD F = **12.5** kips at **29** ft from bottom

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) & , \ for \ \dfrac{P_r}{P_c}\geq 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) & , \ for \ \dfrac{P_r}{P_c}< 0.2 \end{cases} = 0.78 < 1.0$$ **[Satisfactory]**

Where KL_x = 40 ft, for x-x axial load. KL_y = 40 ft, for y-y axial load.

$(KL / r)_{max}$ = 103 < 200 **[Satisfactory]**

P_r = 87 kips

M_{rx} = 0.00

M_{ry} = 109.26 ft-kips, at 29.00 ft from bottom

$P_c = P_n / \Omega_c$ = 473 / 1.67 = 283.38 kips, (AISC 360-05 Chapter E)

> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 343.47 / 1.67 = 205.67 ft-kips, (AISC 360-05 Chapter F)

> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 343.47 / 1.67 = 205.67 ft-kips, (AISC 360-05 Chapter F)

> M_{ry} **[Satisfactory]**

CHECK LATERAL DEFLECTION

Δ_{max} = 1.89 in, at 21.80 ft from bottom

< L / 240 = 2.00 in **[Satisfactory]**

Where E_s = 29000 ksi

I_x = 457 in^4

I_y = 457 in^4

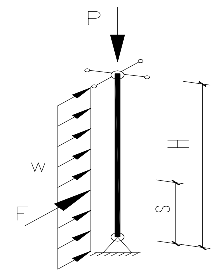
Daniel T. Li
Engineering International

PROJECT :			PAGE :
CLIENT :			DESIGN BY :
JOB NO. :	DATE :		REVIEW BY :

WF Column Design Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION **W10X49**



COLUMN YIELD STRESS	F_y =	50	ksi
DIMENSIONS	H =	14	ft
AXIAL LOAD, ASD	P =	35	kips
STRONG AXIS BENDING ? (1=Yes, 0=No)	=>	0	no, weak axis, y-y, bending.
UNIFORM LATERAL LOAD, ASD	w =	0.75	k / ft
CONCENTRATED LATERAL LOAD, ASD	F =	20	kips at 11 ft from bottom

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) \;,\; for \;\frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) \;,\; for \;\frac{P_r}{P_c}< 0.2 \end{cases} = 0.90 < 1.0$$ **[Satisfactory]**

Where KL_x = 14 ft, for x-x axial load. KL_y = 14 ft, for y-y axial load.

$(KL / r)_{max}$ = 66 < 200 **[Satisfactory]**

P_r = 35 kips

M_{rx} = 0.00

M_{ry} = 59.52 ft-kips, at 11.00 ft from bottom

$P_c = P_n / \Omega_c$ = 524 / 1.67 = 313.64 kips, (AISC 360-05 Chapter E)

> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 231.22 / 1.67 = 138.46 ft-kips, (AISC 360-05 Chapter F)

> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 117.92 / 1.67 = 70.609 ft-kips, (AISC 360-05 Chapter F)

> M_{ry} **[Satisfactory]**

CHECK LATERAL DEFLECTION

Δ_{max} = 0.68 in, at 7.40 ft from bottom

< L / 240 = 0.70 in **[Satisfactory]**

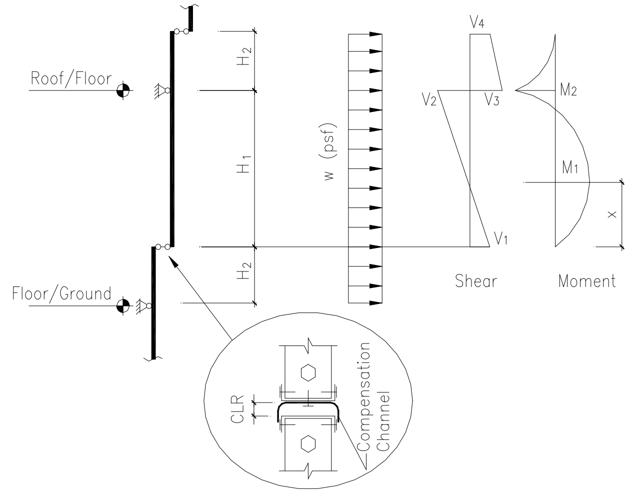
Where E_s = 29000 ksi

I_x = 272 in^4

I_y = 93.4 in^4

Wall Stud & Compensation Channel Design Based on AISI S100-2007 & ER-4943P

INPUT DATA & DESIGN SUMMARY



STUD SIZE & SPACING	600S162-54	@	12	in o.c
			(50 ksi)	
WALL DIMENSION		H_1 =	14	ft
(H_1 or H_2 = 0 for stud flr to flr, w/ Bearing or Hanging)		H_2 =	1	ft
STORY HEIGHT		H =	15	ft
SERVICE LATERAL LOAD		w =	28.48	psf
WALL GRAVITY LOAD		W_t =	32	psf
DEFLECTION LIMITS (IBC 06 / CBC 07, Tab. 1604.3)			1	H /240

(1= H /240, 2= H /360, 3= H /180, 4= H /480)

COMPENSATION CHANNEL 600T200-68 (14 GA , 50 ksi)
CLEAR DISTANCE CLR = 0.75 in, (for story drift & floor vertical deflection)

THE DESIGN IS ADEQUATE.

ANALYSIS

SECTION PROPERTIES OF EACH METAL STUD (SSMA page 7 & 8)

thk = 0.0566 in	F_y = 50 ksi	I_{xx} = 2.86 in^4	M_n/Ω_b = 27.76 in-kips
t = 6 in	Wt = 1.89 lb/ft	S_{xx} = 0.927 in^3	V_n/Ω_v = 2708 lbs
A = 0.556 in^2	r_x = 2.267 in	r_y = 0.57 in	x_o = -1.072 in
J = 0.000594 in^4	C_w = 1.318 in^6		

SECTION PROPERTIES OF COMPENSATION CHANNEL (SSMA page 10 & 11)

t = 2 in, leg length > CLR + 1" **[Satisfactory]**
F_y = 50 ksi
thk = 0.0713 in, metal thickness
wall = 6 in, wall thickness < channel width **[Satisfactory]**

DETERMINE SECTION FORCES

V_1 = 0.19 kips/stud	V_4 = 0.19 kips/stud	x = 6.50 ft
V_2 = -0.21 kips/stud	M_1 = 0.60 ft-kips/stud	
V_3 = 0.21 kips/stud	M_2 = 0.20 ft-kips/stud	
C = 0.03 kips, (the max axial compression)		T = 0.45 kips, (the max axial tension)

CHECK WALL STUD MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)

t / (thk) = 106.01 < 200 **[Satisfactory]**

CHECK WALL STUD FLEXURAL CAPACITY (AISI C3.1)

M_n/Ω_b = 27.76 in-kips / stud **>** **M** **[Satisfactory]**
Where M = (1/1.4) MAX(M1 , M2) = 5.16 in-kips / stud, (1/1.4 for wind/seismic, from AISI App. A4.1.2, typical)

CHECK WALL STUD SHEAR CAPACITY (AISI C3.2)

V_n/Ω_V = 2708 lbs / stud **>** **V** **[Satisfactory]**
Where V = (1/1.4) MAX(V_1 , V_2 , V_3, V_4) = 153 lbs / stud

CHECK WALL STUD CAPACITY COMBINED BENDING & SHEAR (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2} = 0.1941 < 1.0$$ **[Satisfactory]**

Where M = 5.16 in-kips / stud, (1/1.4 included)
V = 153 lbs / stud, (1/1.4 included)
V_n/Ω_v = 2708 lbs / stud, from SSMA page 7 & 8.
M_n/Ω_b = 27.76 in-kips / stud, from SSMA page 7 & 8.

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.19 < 0.5 \qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.06 < 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right)+\left(\frac{\Omega_v V}{V_n}\right) = 0.1678 < 1.3$$ **[Satisfactory]**

(cont'd)

CHECK WALL STUD COMPRESSION CAPACITY WITH, AT LEAST, ONE FLANGE THROUGH-FASTENED TO SHEATHING (AISI D6.1.3)

P_n/Ω_c = 4.99 kips / stud **>** **P** **[Satisfactory]**

Where Ω_c = 1.8

$P_n = C_1C_2C_3$ AE / 29500 = 8.97 kips / stud

C_1 = (0.79 x + 0.54) = 0.949

C_2 = (1.17 α t + 0.93) = 0.996

C_3 = α (2.5b - 1.63d) + 22.8 = 17.070

E = 29500 ksi (AISI pg xiv)

P = 0.03 kips / stud

CHECK WALL STUD CAPACITY COMBINED AXIAL LOAD & BENDING (AISI C5.2.1)

$$\frac{\Omega_c P}{P_n}+\frac{\Omega_b C_m M}{M_n \alpha}= \quad 0.19 \quad < \quad 1.0$$ **[Satisfactory]**

Where M = 5.16 in-kips / stud, (1/1.4 included)

P = 0.03 kips / stud

P_n/Ω_c = 4.99 kips / stud

M_n/Ω_b = 27.76 in-kips / stud

C_m = 1.0

$$P_{Ex}=\frac{\pi^2 E I_x}{(K_x L_x)^2}= \quad 5782.63 \text{ kips / stud}$$

$$\alpha = 1-\frac{\Omega_c P}{P_{Ex}}= \quad 1.000$$

CHECK WALL STUD DEFLECTION

$$\Delta_{wall}=\left|\left(\frac{-Rb}{6EIL}\right)\left[\left(\frac{L}{b}\right)(x-a)^3+\left(L^2-b^2\right)x-x^3\right]+\left(\frac{w}{24EI}\right)\left(L^3x-2Lx^3+x^4\right)\right|= \quad 0.26 \text{ in}$$

< H /240 = 0.75 in **[Satisfactory]**

Where x = 8.00 ft, (from max deflection point to short stud end.)

R = 0.43 kips, (total floor horizontal reaction.)

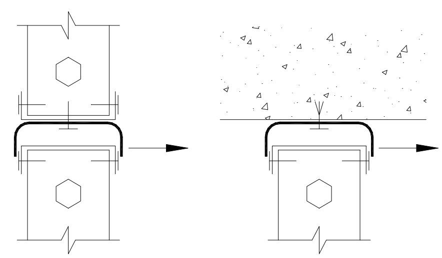
a = 1.00 ft, (from floor to short stud end.)

b = 14.00 ft, (from floor to far stud end.)

CHECK LEG BENDING CAPACITY OF COMPENSATION CANNEL

M_n/Ω_b = 0.36 in-kips / 12"o.c. **>** **M** **[Satisfactory]**

Where M = (1/1.4) V_1(CLR + 0.5") = 0.17 in-kips / 12"o.c.



Technical References:

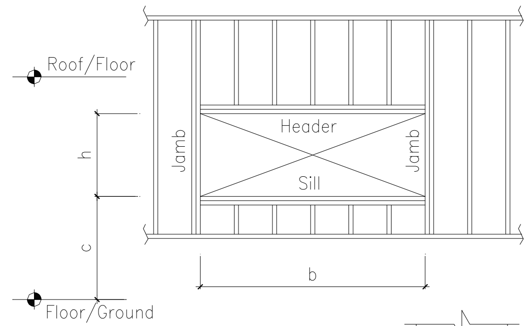
1. AISI STANDARD, S100-2007 Edition. American Iron and Steel Institute.
2. SSMA, Product Technical Information, ICBO ER-4943P, Steel Stud Manufactures Association, 2001.

Daniel T. Li
Engineering International

PROJECT :		PAGE :	
CLIENT :		DESIGN BY :	
JOB NO. :	DATE:	REVIEW BY :	

Header, Sill & Jamb Design Based on AISI 2001 & ICBO ER-4943P

INPUT DATA & DESIGN SUMMARY



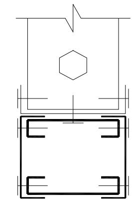
OPENING SIZE & LOCATION
b = **12** ft
h = **5** ft
c = **3** ft
(See Wall Design for Story Height, Lateral Load and More Information.)

HEADER

2 **x** **600S162-54** (flat studs for lateral load)
(50 ksi studs)

2 **x** **600T200-54** (vert tracks for gravity load)
(50 ksi tracks)

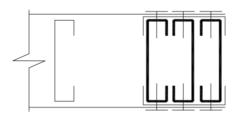
(TOTAL SECTION: 6 x 6)



w_{header} = **171** plf, (total lateral load on header)
$W_{t,\ header}$ = **0** plf, (total gravity load on header, 0 for vertical studs tension above)

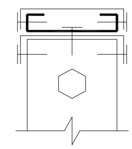
SILL **1** **x** **600S162-54** (flat stud for lateral load)
(50 ksi stud)

w_{sill} = **100** plf, (total lateral load on sill)



JAMB **3** **x** **600S162-97** (vert end studs for lateral load)
(50 ksi studs)

w_{jamb} = **185** plf, (total lateral load on jamb)
P_{jamb} = **208** lbs, (total axial load)



Jamb End Track ==> **2** **x** **600T125-54** (bent outside track 9.3" min)
(50 ksi tracks)

THE DESIGN IS ADEQUATE.

ANALYSIS

Header Design

SECTION PROPERTIES OF EACH FLAT STUD (SSMA page 7 & 8)

t =	0.0566	in	F_y =	50	ksi	I_{xx} =	2.86	in^4	M_n/Ω_b =	27.76	in-kips
h =	6	in	Wt =	1.89	lb/ft	S_{xx} =	0.927	in^3	V_n/Ω_v =	2708	lbs
A =	0.556	in^2	r_x =	2.267	in	r_y =	0.57	in	x_o =	-1.072	in
J =	0.000594	in^4	C_w =	1.318	in^6	L =	12	ft, (header span)			

CHECK MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)
h / t = 106.01 **<** 200 **[Satisfactory]**

CHECK HORIZONTAL FLEXURAL CAPACITY (AISI C3.1)
$\mathbf{M_n/\Omega_b}$ = 4.63 ft-kips **>** **M** **[Satisfactory]**
Where $M = w_{header}\ L^2 / 8$ = 3.08 ft-kips

CHECK HORIZONTAL SHEAR CAPACITY (AISI C3.2)
$\mathbf{V_n/\Omega_V}$ = 5.42 kips **>** **V** **[Satisfactory]**
Where $V = w_{header}\ L / 2$ = 1.03 kips

CHECK HORIZONTAL LATERAL-TORSIONAL BUCKLING (AISI C3.1.2.2)
Compression Flange Supported ? (0=No, 1=Yes) ==> **1** vertical diaphragm on both sides

$$L_u = \frac{0.36 C_b \pi}{F_y S_f}\sqrt{E G J I_y} = \quad 1.09 \text{ ft} \quad < \quad L$$

Where C_b = 1.0
S_f = 1.91 in^3 (total vertical studs, SSMA page 7 & 8.)
E = 29500 ksi (AISI page 18)
G = 11300 ksi (AISI page 21)
I_y = 3.077 in^4 (neglecting top & bottom tracks conservatively.)
J = 0.001 in^4

$$F_e = \frac{C_b \pi}{K_y L_y S_f}\sqrt{E G J I_y} = \quad 12.6 \text{ ksi} \quad < \quad 2.78\,F_y = 139.0 \text{ ksi}$$
$$< \quad 0.56\,F_y = 28.0 \text{ ksi}$$

Where K_y = 1.0
L_y = 144 in

(cont'd)

$$F_c = \begin{cases} F_y\ , & for\ \ F_e \geq 2.78F_e \\ \frac{10}{9}F_y\left(1-\frac{10F_y}{36F_e}\right)\ , & for\ \ 2.78 > F_e \geq 0.56F_e \\ F_e\ , & for\ \ F_e \leq 0.56F_e \end{cases} \quad = \quad 12.6 \quad \text{ksi}$$

M_n/Ω_b = 1.20 ft-kips **<** **M** **[Satisfactory]** **<== Does not apply.**

Where S_c = 1.91 in^3 (total vertical studs, SSMA page 7 & 8.)

Ω_b = 1.67

M_n = S_cF_c = 24.08 in-kips

M = 3.08 ft-kips

CHECK HORIZONTAL CAPACITY COMBINED BENDING & SHEAR AT ANY SAME SECTION (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2} \quad = \quad 0.6912 \quad < \quad 1.0 \quad \textbf{[Satisfactory]}$$

Where M = 3.08 ft-kips

V = 1.03 kips

V_n/Ω_v = 5.42 kips

Ω_b = 1.67

M_n = MIN(Bending , Buckling) = 7.73 ft-kips

M_n/Ω_b = 4.63 ft-kips, for bending only.

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.66 \quad > \quad 0.5 \qquad\qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.19 \quad < \quad 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right)+\left(\frac{\Omega_v V}{V_n}\right) \quad = \quad 0.5882 \quad < \quad 1.3 \quad \textbf{[Satisfactory]}$$

CHECK HORIZONTAL DEFLECTION

$$\Delta_{Lateral} = \frac{5(w_{header})L^4}{384EI} = \quad 0.47 \text{ in} \quad < \quad L/240 = \quad 0.60 \text{ in} \quad \textbf{[Satisfactory]}$$

CHECK VERTICAL DEFLECTION

$$\Delta_{Vertical} = \frac{5(W_{t,header})L^4}{384EI} = \quad 0.00 \text{ in} \quad < \quad L/240 = \quad 0.60 \text{ in} \quad \textbf{[Satisfactory]}$$

Where I = 6.29 in^4 , for (2) - 600T200-54

Sill Design

CHECK HORIZONTAL DEFLECTION

$$\Delta_{Lateral} = \frac{5(w_{Sill})L^4}{384EI} = \quad 0.55 \text{ in} \quad < \quad L/240 = \quad 0.60 \text{ in} \quad \textbf{[Satisfactory]}$$

Where I = 2.86 in^4 , for (1) - 600S162-54

Jamb Design

SECTION PROPERTIES OF EACH VERTICAL STUD (SSMA page 7 & 8)

thk =	0.1017	in	F_y =	50	ksi	I_{xx} =	4.797	in^4	M_n/Ω_b =	56.73	in-kips
t =	6	in	Wt =	3.29	lb/ft	S_{xx} =	1.599	in^3	V_n/Ω_v =	11124	lbs
A =	0.966	in^2	r_x =	2.229	in	r_y =	0.541	in	x_o =	-1.039	in
J =	0.003329	in^4	C_w =	2.093	in^6	h =	15	ft, (jamb height)			

CHECK MAX WEB DEPTH-TO-THICKNESS RATIO (AISI B1.2)

t / (thk) = 59.00 **<** 200 [Satisfactory]

CHECK FLEXURAL CAPACITY (AISI C3.1)

M_n/Ω_b = 14.18 ft-kips **>** **M** **[Satisfactory]**

Where M = (1/1.4) M_{max} = 2.00 ft-kips, (1/1.4 for wind/seismic, from AISI App. A4.1.2, typical)

CHECK SHEAR CAPACITY (AISI C3.2)

V_n/Ω_V = 33.37 kips **>** **V** **[Satisfactory]**

Where V = (1/1.4) V_{max} = 0.71 kips, (see Wall Design diagram for M_{max} & V_{max})

(cont'd)

CHECK COMPRESSION CAPACITY WITH, AT LEAST, ONE FLANGE THROUGH-FASTENED TO SHEATHING (AISI D6.1.3)

P_n/Ω_c = 25.88 kips **>** **P** **[Satisfactory]**

Where Ω_c = 1.8

$P_n = C_1C_2C_3\, AE / 29500 =$ 46.59 kips

$C_1 = (0.79\, x + 0.54) =$ 0.949

$C_2 = (1.17\, \alpha\, t + 0.93) =$ 1.049

$C_3 = \alpha\, (2.5b - 1.63d) + 22.8 =$ 16.145

E = 29500 ksi (AISI pg xiv)

P = 0.21 kips

CHECK CAPACITY COMBINED BENDING & SHEAR AT ANY SAME SECTION (AISI C3.3.1)

$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2} = 0.1016 < 1.0$$ **[Satisfactory]**

Where M = 1.43 ft-kips, (1/1.4 included)

V = 0.51 kips, (1/1.4 included)

V_n/Ω_v = 33.37 kips

M_n/Ω_b = 14.18 ft-kips

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.10 < 0.5 \qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.02 < 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right)+\left(\frac{\Omega_v V}{V_n}\right) = 0.0755 < 1.3$$ **[Satisfactory]**

CHECK CAPACITY COMBINED AXIAL LOAD & BENDING (AISI C5.2.1)

$$\frac{\Omega_c P}{P_n}+\frac{\Omega_b C_m M}{M_n \alpha}= 0.11 < 1.0$$ **[Satisfactory]**

Where M = 1.43 ft-kips, (1/1.4 included)

P = 0.21 kips

P_n/Ω_c = 25.88 kips

M_n/Ω_b = 14.18 in-kips

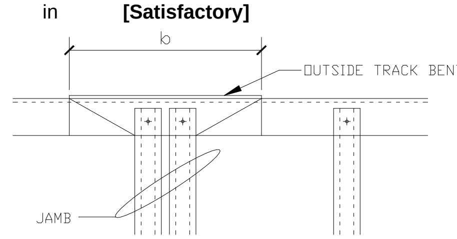
C_m = 1.0

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x H_2)^2} = 29097.14 \text{ kips}$$

$$\alpha = 1 - \frac{\Omega_c P}{P_{Ex}} = 1.000$$

CHECK DEFLECTION

$$\Delta_{jamb} = \Delta_{wall}\left(\frac{I}{w}\right)_{wall}\left(\frac{w}{I}\right)_{jamb} = 0.34 \text{ in} < H/240 = 0.75 \text{ in}$$ **[Satisfactory]**



CHECK BENDING CAPACITY OF TRACK LEG

t = 1.25 in, leg length

F_y = 50 ksi

thk = 0.1132 in, metal thickness

wall = 6 in, wall thickness < track width

[Satisfactory]

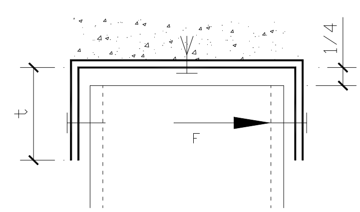
d = 5 in, jamb width

b = d + 2 t (tan 60°) = 9.3 in, effective width

F = 0.86 kips, (factor 1/1.4 included, AISI App. A4.1.2)

M = F (t + 1/4") / 2 = 0.6 in-kips

S = b (thk)^2 / 6 = 0.0199 in^3

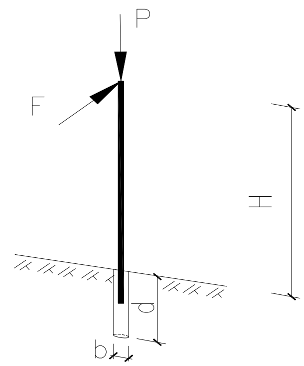


M_n/Ω_b = 0.71 in-kips **>** **M** **[Satisfactory]**

Cantilever Column & Footing Design Based on AISC 360-05, ACI 318-08, and IBC 09 1807.3

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION (Tube, Pipe, or WF)		HSS8X8X5/8		**Tube**
COLUMN YIELD STRESS	F_y =	46	ksi	
CANTILEVER HEIGHT	H =	15	ft	
COLUMN TOP LATERAL LOAD (Strong Axis Bending only)	F =	3.2	kips, ASD	
COLUMN TOP GRAVITY LOAD	P =	10	kips, ASD	
DIAMETER OF POLE FOOTING	b =	3	ft	
ALLOW SOIL PRESSURE	Q_a =	2	ksf	
LATERAL SOIL CAPACITY	P_P =	0.35	ksf / ft	
RESTRAINED @ GRADE ?(1=yes,0=no)		1	Yes	



Use 3 ft dia x 4.59 ft deep footing restrained @ ground level

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY OF COLUMN (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for \quad \frac{P_r}{P_c}\ge 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for \quad \frac{P_r}{P_c}< 0.2 \end{cases} = 0.50 < 1.0$$ **[Satisfactory]**

Where
P_r = 10.00 kips
M_{rx} = 48.00 ft-kips
M_{ry} = 0 ft-kips
KL_y = 30 ft, weak axis unbraced axial length
$P_c = P_n / \Omega_c$ = 283 / 1.67 = 169.33 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**
$M_{cx} = M_n / \Omega_b$ = 171.35 / 1.67 = 102.60 ft-kips, (AISC 360-05 Chapter F)
> M_{rx} **[Satisfactory]**
$M_{cy} = M_n / \Omega_b$ = 171.35 / 1.67 = 102.60 ft-kips, (AISC 360-05 Chapter F)
> M_{ry} **[Satisfactory]**

DESIGN POLE FOOTING (IBC 09 1807.3)
By trials, use pole depth, d = 4.588 ft
Lateral bearing @ bottom, $S_3 = 2 P_P \, Min(d , 12') =$ 3.21 ksf
Lateral bearing @ $d / 3$, $S_1 = 2 P_P \, Min(d / 3 , 12') =$ 1.07 ksf
Require Depth is given by

$$d = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & for \quad nonconstrained \\ \sqrt{\frac{4.25Ph}{bS_3}} & for \quad constrained \end{cases} = 4.588 \text{ ft}$$ **[Satisfactory]**

Where
$P = F =$ 3.20 kips
$A = 2.34 P / (b S_1) =$ 1.72
$h = M_{max} / F =$ 15.00 ft

CHECK VERTICAL SOIL BEARING CAPACITY (ACI, Sec. 15.2.2)
$q_{soil} = P / (\pi b^2 / 4) =$ 1.41 ksf, (net weight of pole footing included.)
< Q_a **[Satisfactory]**

CHECK STRONG AXIS LATERAL DEFLECTION

$$\Delta = \frac{FH^3}{3EI} = 1.47 \text{ in} < 2H / 240 = 1.50 \text{ in}$$ **[Satisfactory]**

Drag / Collector Forces for Brace Frame

DESIGN CRITERIA

1. NEGLECTING DRAG / COLLECTOR AXIAL DEFLECTIONS, SO THE AXIAL FORCE DIAGRAM AS FOLLOWS IS THE SAME FOR BOTH RIGID AND FLEXIBLE DIAPHRAGM.
2. ASSUMING THAT DIAPHRAGM SHEAR STRESS ALONG DRAG ARE UNIFORM AND EQUAL, SINCE THE DIAPHRAGM THICKNESS SAME AND NAILING OR SHEAR STUD DISTRIBUTED CONTINUOUSLY.

INPUT DATA & DRAG / COLLECTOR AXIAL FORCES

NUMBER OF SEGMENTS / SPAN $n =$ 10

Segment	Length (ft)	Joint	Distance (ft)	Horiz. Force F_{brace} (k)	Drag Axial Force (kips) Left	Right
		1	0			0.00
Span 1	20					
		2	20		-16.44	-16.44
Span 2	15					
		3	35		-28.77	-28.77
Span 3	15					
		4	50	46.5	-41.10	5.40
Span 4	18					
		5	68	46.5	-9.40	37.10
Span 5	25					
		6	93		16.55	16.55
Span 6	20					
		7	113		0.12	0.12
Span 7	20					
		8	133	32	-16.32	15.68
Span 8	20					
		9	153	32	-0.76	31.24
Span 9	18					
		10	171		16.44	16.44
Span 10	20					
		11	191		0.00	

TOTAL DRAG LENGTH

$L_{drag} =$ 191 ft

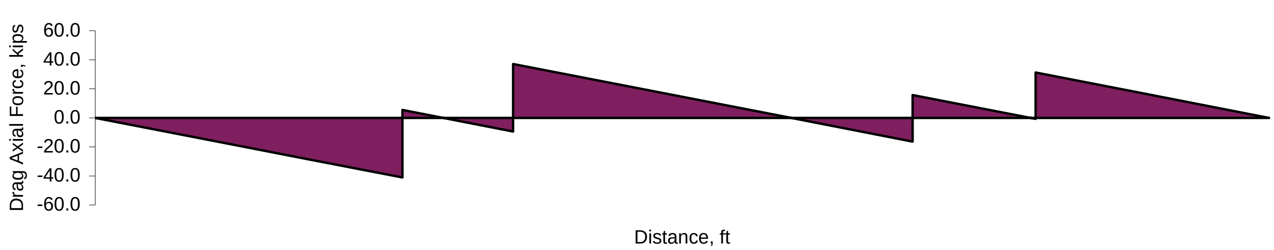
TOTAL LATERAL FORCE

$\Sigma F_{brace} =$ 157 kips

DIAPHRAGM SHEAR STRESS

$v_{diaphragm} = \Sigma F_{brace} / L_{drag}$

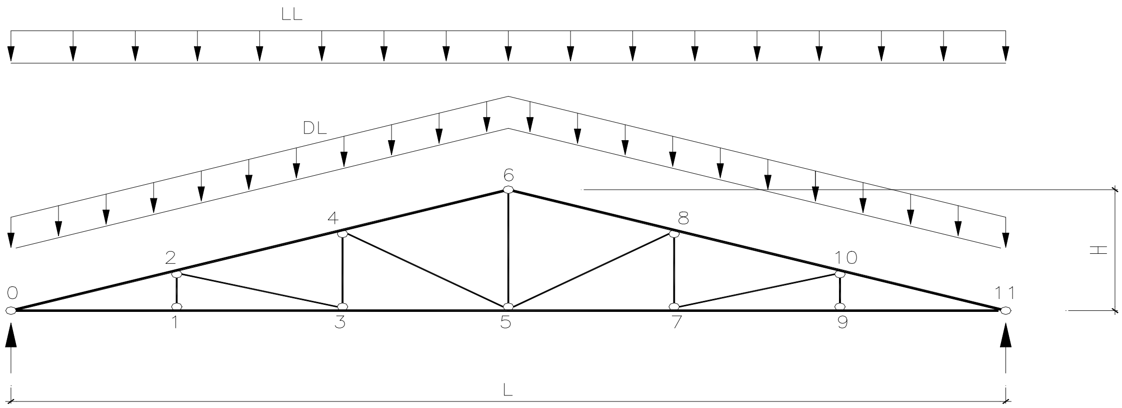
= 822 plf



DRAG / COLLECTOR FORCE DIAGRAM

Light Gage Truss Design Based on AISI S100-2007 & ER-4943P

INPUT DATA & DESIGN SUMMARY



CONTINUED TOP CHORD (from 0 to 6 & from 6 to 11) 2 x 1200S250-97 < == (Total Section: 12 in Ht. x 5 in Thk.)

CONTINUED BOTTOM CHORD (from 0 to 11) 2 x 600S250-54 < == (Total Section: 6 in Ht. x 5 in Thk.)

ALL WEB MEMBER SIZE 600S162-54

SLOPED DEAD LOADS DL = 300 lbs / ft
PROJECTED LIVE LOADS LL = 240 lbs / ft

TRUSS SPAN LENGTH L = 24 ft
TRUSS DEPTH H = 4 ft

THE TRUSS DESIGN IS ADEQUATE.

ANALYSIS

DESIGN CRITERIA

1. The intermediate finite element analysis include shear deformations using E & G equal to 29500 ksi & 11300 ksi.
2. There are different rotations at same joint of web to chord, since web pinned to continued chord.
3. Top chord shall be fastened to sheathing before vertical laod added.

DETERMINE JOINT DEFLECTIONS BY FINITE ELEMENT ANALYSIS

	DL Deflection (in, deg)					LL Deflection (in, deg)				
Joint	X	Y	θ (cont.)	θ (pin-1)	θ (pin-2)	X	Y	θ (cont.)	θ (pin-1)	θ (pin-2)
0	0	0		-0.1050	-0.1046	0	0		-0.0797	-0.0794
1	-0.0158	-0.0775	-0.0658	-0.0532		-0.0120	-0.0588	-0.0499	-0.0404	
2	-0.0010	-0.0777	-0.0642	-0.0532	-0.0324	-0.0007	-0.0590	-0.0488	-0.0404	-0.0246
3	-0.0083	-0.1054	-0.0087	-0.0324	-0.0235	-0.0063	-0.0800	-0.0066	-0.0246	-0.0178
4	0.0049	-0.1058	-0.0075	-0.0235	-0.0019	0.0037	-0.0803	-0.0057	-0.0178	-0.0015
5	0	-0.1049	0	-0.0019	0.0019	0	-0.0796	7.22E-17	-0.0015	0.0015
6	0	-0.1002		0.0161	-0.0161	0	-0.0761		0.0122	-0.0122
7	0.0083	-0.1054	0.0087	0.0324	0.0235	0.0063	-0.0800	0.0066	0.0246	0.0178
8	-0.0049	-0.1058	0.0075	0.0235	0.0019	-0.0037	-0.0803	0.0057	0.0178	0.0015
9	0.0158	-0.0775	0.0658	0.0532		0.0120	-0.0588	0.0499	0.0404	
10	0.0010	-0.0777	0.0642	0.0532	0.0324	0.0007	-0.0590	0.0488	0.0404	0.0246
11	0	0		0.1050	0.1046	0	0		0.0797	0.0794

CHECK MAXIMUM LIVE LOAD DEFLECTION

D_{LL} = 0.08 in **<** L / 240 = 1.20 in **[Satisfactory]**

DETERMINE REQUIRED CAMBER AT 1.0 TIMES MAXIMUM DEAD LOAD DEFLECTION

1.0 D_{DL} = 0.11 in

DETERMINE MEMBER FORCES UNDER LOAD COMBINATION **(DL + LL)**

	Joints	I end section			J end section		
Member	I - J	N (kips)	V (kips)	M(ft-kips)	N (kips)	V (kips)	M(ft-kips)
1	0 - 1	-10.952	0.232	0.000	10.952	-0.232	0.928
2	0 - 2	12.428	2.649	0.000	-11.724	-0.538	6.718
3	1 - 2	0.359	0.000	0.000	-0.359	0.000	0.000
4	1 - 3	-10.952	-0.127	-0.928	10.952	0.127	0.422
5	2 - 3	-1.053	0.000	0.000	1.053	0.000	0.000
6	2 - 4	12.680	0.246	-6.718	-11.976	1.864	3.307
7	3 - 4	0.366	0.000	0.000	-0.366	0.000	0.000

REACTIONS:
$R_0 = R_{11}$ = 6.67 kips, upward

(cont'd)

8	3 - 5	-11.951	-0.160	-0.422	11.951	0.160	-0.216
9	4 - 5	2.265	0.000	0.000	-2.265	0.000	0.000
10	4 - 6	10.701	0.271	-3.307	-9.997	1.840	0.000
11	5 - 6	-2.832	0.000	0.000	2.832	0.000	0.000
12	5 - 7	-11.951	0.160	0.216	11.951	-0.160	0.422
13	5 - 8	2.265	0.000	0.000	-2.265	0.000	0.000
14	6 - 8	9.997	1.840	0.000	-10.701	0.271	3.307
15	7 - 8	0.366	0.000	0.000	-0.366	0.000	0.000
16	7 - 9	-10.952	0.127	-0.422	10.952	-0.127	0.928
17	7 - 10	-1.053	0.000	0.000	1.053	0.000	0.000
18	8 - 10	11.976	1.864	-3.307	-12.680	0.246	6.718
19	9 - 10	0.359	0.000	0.000	-0.359	0.000	0.000
20	9 - 11	-10.952	-0.232	-0.928	10.952	0.232	0.000
21	10 - 11	11.724	-0.538	-6.718	-12.428	2.649	0.000

CHECK LIGHT GAGE MEMBERS CAPACITIES

Member	Max. Section Force		
	N (kips)	V (kips)	M(ft-kips)
Top Chord	-12.680	2.649	6.718

M_n/Ω_b = 30.53 ft-kips

V_n/Ω_v = 15.63 kips

P_n/Ω_c = 18.54 kips

[Satisfactory]

Member	Max. Section Force		
	N (kips)	V (kips)	M(ft-kips)
Bot Chord	11.951	0.232	0.928

M_n/Ω_b = 5.33 ft-kips

V_n/Ω_v = 5.42 kips

T_n/Ω_t = 33.50 kips

[Satisfactory]

Compression Web Member	Max. Section Force		
	N (kips)	V (kips)	M(ft-kips)
9 & 13	-2.265		

P_n/Ω_c = 17.88 kips

[Satisfactory]

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Engineering International

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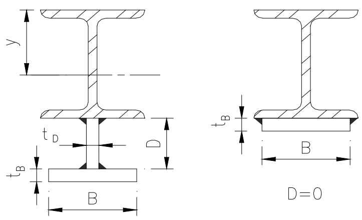
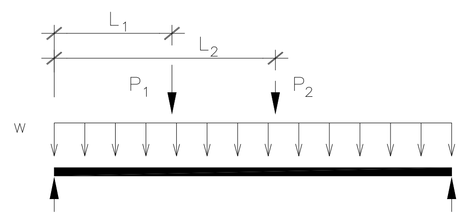
Enhanced Steel Beam Design Based on AISC 14th (AISC 360-10)

INPUT DATA & DESIGN SUMMARY

WF BEAM SECTION	=>	**W18X143**		=>	A	d	r_x	r_y	I_x	S_x
WF BEAM YIELD STRESS	F_y =	**50**	ksi		42.1	19.5	8.08	2.72	2750	282
ENHANCING PLATE SIZE	B =	**12**	in		I_y	S_y	λ	t_w	b_f	t_f
	t_B =	**1**	in		311	55.5	0.017	0.73	11.20	1.32
	D =	**0**	in							
	t_D =	**0**	in							
	F_y =	**36**	ksi		A	d	y	I_x	E	
				Total = =>	54.1	20.5	12.0	3732	29000	
					y_z					
					15.02					



BEAM SPAN	L =	**25.6**	ft
UNIFORMLY DISTRIBUTED DEAD LOAD	w_D =	**0**	kips / ft
UNIFORMLY DISTRIBUTED LIVE LOAD	w_L =	**3.3**	kips / ft
CONCENTRATED LOADS	$P_{1,D}$ =	**0**	kips
(0 for no concentrated load)	$P_{1,L}$ =	**53.2**	kips
	L_1 =	**14.25**	ft
	$P_{2,D}$ =	**0**	kips
	$P_{2,L}$ =	**0**	kips
	L_2 =	**0**	ft
DEFLECTION LIMIT OF LIVE LOAD	Δ_L = L /	**360**	
VERTICAL BENDING UNBRACED LENGTH	L_b =	**0**	ft



THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE REACTIONS, MOMENT, SHEAR, AND CHECK CAPACITIES

$w_{Self\ Wt}$ = 0.184 kips / ft R_{Left} = 68.18 kips R_{Right} = 74.21 kips

V_{Max} = 74.21 kips, at from right end M_{Max} = 617.86 ft-kips, at 14.25 ft from left end

< $V_{allowable}$ = 268.83 kips, (from following analysis)   < $M_{allowable}$ = 957.97 ft-kips

[Satisfactory]   **[Satisfactory]**   (from following analysis)

CHECK LIVE LOAD DEFLECTION

Δ_{Max} = 0.59 inch, at 13.09 ft, from left end

< Δ_L = L / 360 = 0.85 inch **[Satisfactory]**

CHECK LIMITING WIDTH-THICKNESS RATIOS FOR WEB (AISC 360-10 Table B4.1)

$h_c / Min(t_w , t_D)$ = 23.10 < λ_r = 137.27

< λ_p = 44.37

Compact Web

where $\lambda_r = 5.7 (E / F_y)^{0.5}$ = 137.27

$\lambda_p = (h_c / h_p) (E / F_y)^{0.5} / (0.54 M_p / M_y - 0.09)^2$ = 44.37 ,for $A_{f,top} \neq A_{f,bot}$

$\lambda_p = 3.76 (E / F_y)^{0.5}$ = 90.55 ,for $A_{f,top} = A_{f,bot}$

h_c = 16.86 in h_p = 27.40 in

M_y = 1293.3 ft-kips M_p = 1599.8 ft-kips, ($F_{y,WF}$ & $F_{y,Plate}$ may different)

CHECK LIMITING WIDTH-THICKNESS RATIOS FOR FLANGES (AISC 360-10 Table B4.1)

$0.5\ b_{f,top} / t_{f,top}$ = 4.24 < λ_r = 25.09

< λ_p = 9.15

Compact Flanges

where $\lambda_r = 1.0 (k_c\ E / F_L)^{0.5}$ = 25.09

$\lambda_p = 0.38 (E / F_y)^{0.5}$ = 9.15

$k_c = Min [0.76 , Max (0.35 , 4 / (h / t_w)^{0.5})]$ = 0.76

S_{xt} = 440 in^3 S_{xc} = 310 in^3

F_L = 35 ksi, (AISC 360-10 Table note B4.1 & Eq F4-6)

(cont'd)

DETERMINE ALLOWABLE FLEXURAL STRENGTH , M_n / Ω_b , BASED ON AISC 360-10 Chapter F4

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = \quad 7.08 \text{ ft}$$

$$L_r = 1.95 r_t \frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_0}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_L}{E}\frac{S_{xc}h_0}{J}\right)^2}} = \quad 38.32 \text{ ft}$$

where $a_w = h_c\, t_w / (b_{fc}\, t_{fc}) =$ 0.83 , $h_0 =$ 18.70 in , $J =$ 19.2 in^4 (use WF only conservatively)

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h_0}{d}+\frac{1}{6}a_w\frac{h^2}{h_0 d}\right)}} = \quad 3.21 \text{ in}$$

$C_b =$ 1.0 , (AISC Manual 14th Table 3-1)

$M_p = Min\,[M_p\ , 1.6 S_{xc}\, F_y\,] =$ 1599.8 ft-kips

$M_{yc} = S_{xc}\, F_y =$ 1293.3 ft-kips $\qquad M_{yt} = S_{xt}\, F_y =$ 1320.9 ft-kips

$\lambda = h_c / t_w =$ 23.10

$\lambda_{pw} = \lambda_p =$ 44.37 $\qquad \lambda_{rw} = \lambda_r =$ 137.27

$\lambda = b_f / (2\, t_f) =$ 4.24

$\lambda_{pf} = \lambda_p =$ 9.15 $\qquad \lambda_{rf} = \lambda_r =$ 25.09

$$R_{pc} = \begin{cases} \frac{M_p}{M_{yc}} \ , \quad for \quad h_c/t_w \le \lambda_{pw} \\ Min\left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}}-1\right)\left(\frac{\lambda-\lambda_{pw}}{\lambda_{rw}-\lambda_{pw}}\right) \ , \ \frac{M_p}{M_{yc}}\right] \ , \quad for \quad h_c/t_w > \lambda_{pw} \end{cases} = \quad 1.237$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1+0.078\frac{J}{S_{xc}h_0}\left(\frac{L_b}{r_t}\right)^2} = \quad 2E+08 \text{ ksi, (for } I_{yc} / I_y > 0.23\text{, AISC 360-10 F4-5)}$$

$$M_{n,F4.2} = \begin{cases} R_{pc}M_{yc} \ , \quad for \quad L_b \le L_p \\ Min\left(C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \ , \ R_{pc}M_{yc}\right) \ , \quad for \quad L_p < L_b \le L_r \\ Min\left(F_{cr}S_{xc} \ , \ R_{pc}M_{yc}\right) \ , \quad for \quad L_r \le L_b \end{cases} = \quad 1599.8 \text{ ft-kips}$$

$$M_{n,F5.3} = \begin{cases} R_{pc}M_{yc} \ , \quad for \quad Compact \quad Flanges \\ \left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)\right] \ , \quad for \quad Noncompact \quad Flanges \\ \frac{0.9 E k_c S_{xc}}{\lambda^2} \ , \quad for \quad Slender \quad Flanges \end{cases} = \quad 1599.8 \text{ ft-kips}$$

$$R_{pt} = \begin{cases} \frac{M_p}{M_{yt}} \ , \quad for \quad h_c/t_w \le \lambda_{pw} \\ Min\left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}}-1\right)\left(\frac{\lambda-\lambda_{pw}}{\lambda_{rw}-\lambda_{pw}}\right) \ , \ \frac{M_p}{M_{yt}}\right] \ , \quad for \quad h_c/t_w > \lambda_{pw} \end{cases} = \quad 1.2112$$

$\Omega_b =$ 1.67 , (AISC 360-10 F1)

$M_{allowable,\,F4} = Min(\,M_{n,F4.2}\ , M_{n,F4.3}\,, R_{pt}\,M_{yt}\,) / \Omega_b =$ **958.0** ft-kips

(cont'd)

DETERMINE ALLOWABLE FLEXURAL STRENGTH , M_n / Ω_b , BASED ON AISC 360-10 Chapter F5

<== Not Applicable.

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} = \quad 24.18 \text{ ft}$$

$$F_{cr,F5.2} = \begin{cases} F_y \ , \ for \ \ L_b \le L_p \\ Min\left(C_b\left[F_y - (0.3F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \ , \ F_y \right) \ , \ for \ \ L_p < L_b \le L_r \\ Min\left(\frac{C_b \pi^2 E}{\left(\frac{Lb}{rt}\right)^2} \ , \ F_y \right) \ , \ for \ \ L_r \le L_b \end{cases} = \quad 50 \text{ ksi}$$

$$F_{cr,F5.3} = \begin{cases} F_y \ , \ for \ \ Compact \ \ Flanges \\ \left[F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \ , \ for \ \ Noncompact \ \ Flanges \\ \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2} \ , \ for \ \ Slender \ \ Flanges \end{cases} = \quad 50 \text{ ksi}$$

$$R_{pg} = Min\left(1 - \frac{Min(a_w \ , \ 10)}{1200 + 300Min(a_w \ , \ 10)}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \ , \ 1.0 \right) = \quad 1$$

***$M_{allowable, F5}$* = *Min($R_{pg} F_y S_{xc}$, $R_{pg} F_{cr,F5.2} S_{xc}$, $R_{pg} F_{cr,F5.3} S_{xc}$, $F_y S_{xt}$) / Ω_b* =** **774.4** ft-kips

DETERMINE ALLOWABLE SHEAR STRENGTH , V_n / Ω_v , BASED ON AISC 360-10 Chapter G2

***h* = *d* - *t*$_{f,top}$ - *t*$_{f,bot}$ =** 16.86 in , $h / t_w =$ 23 , ***A_w* =** 14.97 in^2 ,

$$k_v = \begin{cases} 5 + \frac{5}{(a/h)^2} \ , \ for \ \ a/h \le 3 \\ 5 \ , \ for \ \ a/h > 3 \end{cases} = \quad 5.00$$

$$C_v = \begin{cases} 1.0 \ , \ for \ \ h/t_w \le 1.10\sqrt{\frac{k_v E}{F_y}} \\ \frac{1.10}{h/t_w}\sqrt{\frac{k_v E}{F_y}} \ , \ for \ \ 1.10\sqrt{\frac{k_v E}{F_y}} < h/t_w \le 1.37\sqrt{\frac{k_v E}{F_y}} \\ \frac{1.51E}{(h/t_w)^2}\frac{k_v}{F_y} \ , \ for \ \ 1.37\sqrt{\frac{k_v E}{F_y}} < h/t_w \end{cases} = \quad 1.000$$

***V_n* = *0.6 $F_y A_w C_v$* =** 448.95 kips

***$V_{allowable}$* = *V_n / Ω_v* =** **268.83** kips   ***Ω_v* =** 1.67 , (AISC 360-10 G1)

Daniel T. Li Engineering International	PROJECT : CLIENT : JOB NO. :	DATE :	PAGE : DESIGN BY : REVIEW BY :	

Enhanced Composite Beam Design Based on AISC 360-05 / IBC 09 / CBC 10

INPUT DATA & DESIGN SUMMARY

FLOOR DECK TYPE

W3-6 1/4" LW

CONCRETE STRENGTH

$f_c' =$ **3** ksi

SHEAR STUD DIAMETER (1/2, 5/8, 3/4)

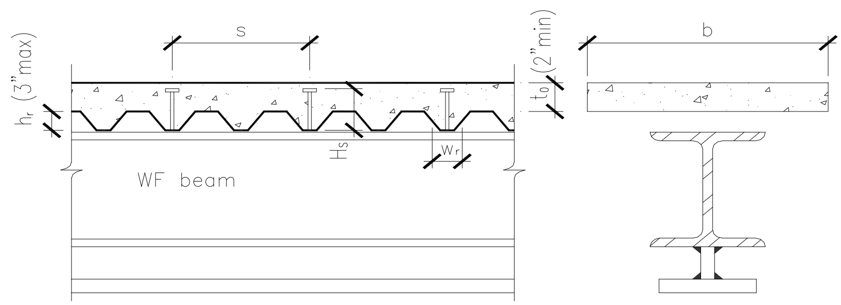
$\phi =$ **3/4** in

STUDS SPACING

1 *row @* **12** in o.c.

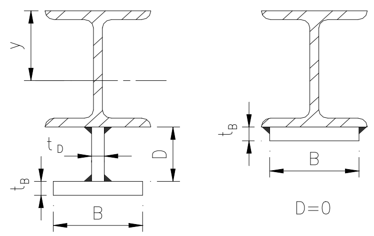
RIBS PERPENDICULAR TO BEAM ?

No (parallel)

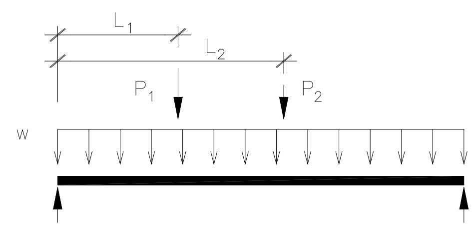


Effective Section

WF BEAM SECTION	=>	**W24X62**	=>	A	d	r_x	r_y	I_x	S_x
WF BEAM YIELD STRESS	F_y =	**50** ksi		18.3	23.7	9.23	1.37	1560	132
ENHANCING PLATE SIZE	B =	**12** in		I_y	S_y	λ	t_w	b_f	t_f
	t_B =	**1** in		35	9.8	0.0122	0.43	7.04	0.59
	D =	**5** in							
	t_D =	**0.5** in							
	$F_{y,\ plate}$ =	**36** ksi		A	d	y	I_x	E	y_z
			Total Steel = =>	32.8	29.7	19.3	3877	29000	23.11
				S_x	Z_x				
				201	263				



BEAM SPAN	L =	**30**	ft
BEAM SPACING (TRIB. WIDTH)	S =	**30**	ft, o.c.
SUPERIMPOSED DEAD LOAD, ASD	w_D =	**20**	lbs / ft^2
LIVE LOAD	w_L =	**100**	lbs / ft^2
CONCENTRATED LIVE LOADS ON BEAM	$P_{1,L}$ =	**10**	kips
(0 for no concentrated load)	L_1 =	**10**	ft
	$P_{2,L}$ =	**30**	kips
	L_2 =	**20**	ft
TO ENHANCE EXISTING BEAM? ==>		**No**	,new construction



THE BEAM DESIGN IS ADEQUATE.

(Camber = 1/4".)

ANALYSIS

DETERMINE REACTIONS, MOMENTS & SHEARS

$w = w_{D+L} + w_{wt} =$ 120 + (43.50 + 3.72) = 167.22 lbs / ft^2

= 5.02 kips / ft (total gravity loads on span beam)

R_{Left} = 91.92 kips R_{Right} = 98.58 kips

M_{max} = 781.16 ft-kips, at 16.50 ft from left end V_{max} = 98.58 kips, at from right end

CHECK DIMENSION REQUIREMENTS

t_o =	3.25	in	>	2	in	[Satisfactory]	(AISC 360-05 I3.2c.1.c)
h_r =	3	in	<	3	in	[Satisfactory]	(AISC 360-05 I3.2c.1.a)
ϕ =	3/4	in	<	3/4	in	[Satisfactory]	(AISC 360-05 I3.2c.1.b)
$H_s = h_r + 1.5$ =	4.5	in	<	$h_r + t_o - 0.5$ = 5.75	in	[Satisfactory]	(AISC I3.2c.1.b)
s =	12	in o.c.	<	$MAX[\ 8(h_r + t_o)\ ,\ 36\ ]$ = 50	in o.c.	[Satisfactory]	
			>	$4\ \phi$ = 3	in o.c.	[Satisfactory]	(AISC 360-05 I3.2d.6)
w_r =	6	in	>	2	in	[Satisfactory]	(AISC 360-05 I3.2c.1.a)

DETERMINE COMPOSITE PROPERTIES FOR PLASTIC DESIGN

$b = MIN\ (L\ /\ 4\ ,\ B) =$ 90 in, (AISC 360-05 I3.1a) $h = t_o + h_r + d =$ 36.0 in, (total height)

$A_{ctr} = 0.85\ f_c'\ b\ t_o\ /\ F_y =$ 14.9 in^2 $A_{total} = A_{ctr} + A =$ 47.7 in^2

$A_{fill} = A - 2A_f - A_w =$ 12.23 in^2 $A_f =$ 4.15 in^2

$t_w =$ 0.43 in $t_f =$ 0.59 in

(cont'd)

$$y_b = \begin{cases} h - \dfrac{AF_y}{0.85 f'_c b} & for \quad A_{total} \le 2A_{ctr} \\ d - \dfrac{0.5A_{total} - A_{ctr}}{b_f} & for \quad A_{total} \le 2\left(A_{ctr} + A_f\right) \\ t_f + \dfrac{A_{total} - A_f - 0.5A_{fill}}{t_w} & for \quad A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases} =$$ 32.2 in, (plastic neutral axis to bottom)

$$y = \begin{cases} 0.5\left(t_0 + h_r + y_b\right) & for \quad A_{total} \le 2A_{ctr} \\ h - \dfrac{0.5t_0 A_{ctr} + 0.5dA + \left(0.5A_{total} - A_{ctr}\right)\left(h - d - y_b\right)}{0.5A_{total}} & for \quad A_{total} \le 2\left(A_{ctr} + A_f\right) \\ h - \dfrac{0.5t_0 A_{total} + A_f\left(t_0 + h_r + t_f\right) + 0.5A_{fill}\left(t_0 + h_r + 2t_f\right) + t_w\left(d - y_b - t_f\right)\left(h - 0.5d - 0.5y_b + 0.5t_f\right) + t_w\left(y_b - t_f\right)\left(0.5y_b + 0.5t_f\right)}{0.5A_{total}} & for \quad A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases}$$

thus, $y =$ 22.8 in, (moment arm between centroid of tensile force and the resultant compressive force.)

$Z_{tr} = 0.5\, y\, A_{total} =$ 544 in^3

DETERMINE COMPOSITE PROPERTIES FOR ELASTIC DESIGN

$n = \dfrac{E}{E_c} =$ 13.01 , (ACI 318-05 8.5.1)

$A_{ctr} = b\, t_0 / n =$ 22.5 in^2

$y_b = \dfrac{A_{ctr}\left(d + h_r + 0.5t_0\right) + 0.5Ad}{A_{ctr} + A} =$ 22.8 in, (elastic neutral axis to bottom)

$I_{tr} = I_x + A\left(y_b - 0.5d\right)^2 + \dfrac{A_{ctr} t_0^2}{12} + A_{ctr}\left(0.5t_0 + h_r + d - y_b\right)^2 =$ 8956 in^4

$S_{tr} = \dfrac{I_{tr}}{y_b} =$ 393 in^3, referred to steel bottom.

$S_t = \dfrac{I_{tr}}{\left(d + h_r + t_0 - y_b\right)} =$ 680 in^3, referred to concrete top.

CHECK BENDING & SHEAR CAPACITIES

Moment : $M_{max} = (Z_{tr} / Z_{x,\ WF})\, M_{DL} + M_{LL} =$ 1339.6 ft-kips

$< \quad M_n / \Omega_b = Z_{tr} F_y / \Omega_b =$ 1357.0 ft-kips, (AISC 360 I3.2a) [Satisfactory]

where $Z_x =$ 154 in^3

$\Omega_b =$ 1.67 (AISC 360-05 I3.2a)

$3.76(E / F_y)^{0.5} =$ 90.55 $>$ $h / t_w =$ 69.07

Shear : $V_{max} =$ 98.58 kips $<$ $V_n / \Omega_v = 0.6F_y A_w C_v / \Omega_v =$ 196.76 kips, (AISC 360-05 I3.1b)

[Satisfactory]

where $2.24\,(E / F_y)^{0.5} =$ 53.946

$k_v =$ 5 (AISC 360-05 G2.1b) $C_v =$ 0.8576 (AISC 360-05 G2.1b)

$(k_v\, E / F_y)^{0.5} =$ 53.852 $\Omega_v =$ 1.67 (AISC 360-05 G1)

CHECK SHEAR CONNECTOR CAPACITY

$M_{max} =$ 781.2 ft-kips $>$ $M_n / \Omega_b = Z_x F_y / \Omega_b =$ 472.8 ft-kips **<== Shear Studs Required**

where $\Omega_b =$ 1.67 (AISC 360-05 F1 & F2-1)

$C_f = MIN\,(0.85\, f_c' A_c\,,\, F_y A_s) =$ 745.88 kips, (AISC 360-05 C-I3.1)

$S_{eff} = Min[\, M_{max} / (0.66\, F_y)\,,\, S_{tr}\,] =$ 284 in^3, refered to steel bottom.

$V' = MAX\left[\left(\dfrac{S_{eff} - S_s}{S_{tr} - S_s}\right)^2, 0.25\right] C_f =$ 186.47 kips, (AISC 360-05 C-I3-4)

$Q_n = MIN\,[\,0.5\, A_{sc}\, (f_c' E_c)^{0.5}\,,\, R_g R_p A_{sc} F_u\,] =$ 18.06 kips, (AISC 360-05 I3.2d.3)

where $w_c =$ 115 pcf

$X_1 =$ 15.50 ft

(cont'd)

$E_c = w_c^{1.5}\ 33\ (f_c')^{0.5}$ = 2229.1 ksi

A_{sc} = 0.44 in^2

F_u = 58 ksi

R_g = 1.00 (AISC 360-05 Table I3.2b.3)

R_p = 0.75 (AISC 360-05 Table I3.2b.3)

$\Sigma Q_n = Q_n\ N_r\ X_1\ /\ s$ = 279.99 kips > V' [Satisfactory]

CHECK LIVE LOAD DEFLECTION ON COMPOSITE

Δ_{Max} = 0.35 inch, at 15.5 ft, from left end

< Δ_L = L / 360 = 1.00 inch [Satisfactory]

DETERMINE DEAD LOAD DEFLECTION ON NON-COMPOSITE

$$\Delta_{Mid} = \frac{5 w_{DL} L^4}{384 EI} =$$ 0.31 in

where w_{DL} = 1.91 kips / ft

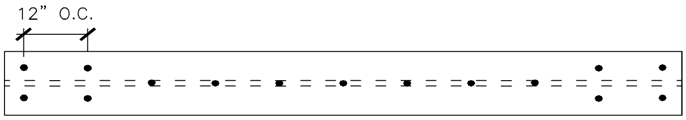
I = 3877 in^4

Composite Collector Beam Design with Seismic Loads Based on AISC 360-05 / CBC 10 / IBC 09

DESIGN CRITERIA

1. When welded shear connectors are used for transfer of lateral shear loads to drags/collectors, the allowable shear strength shall be 75% of the available strength for both gravity and lateral design (CBC 10 2204A.1.3), to consider concrete craked at lateral loads.

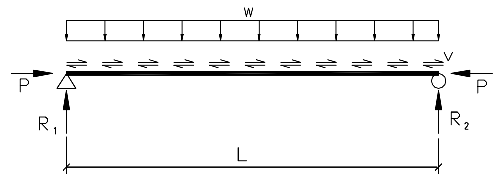
2. The input STUDS SPACING must be based on actual deck ribs spacing for perpendicular to beam. For the following total [15] studs, if ribs spacing 12" o. c., the minimum compose beam capacity is from 2 rows @ 12" o.c., not one row @ 8.57" o.c.



INPUT DATA & DESIGN SUMMARY

BEAM SECTION	=>	W18X46	=>	A	d	I_x	S_x	Z_x
				13.5	18.1	712	78.8	90.7

FLOOR DECK TYPE => W3-7 1/2" NW

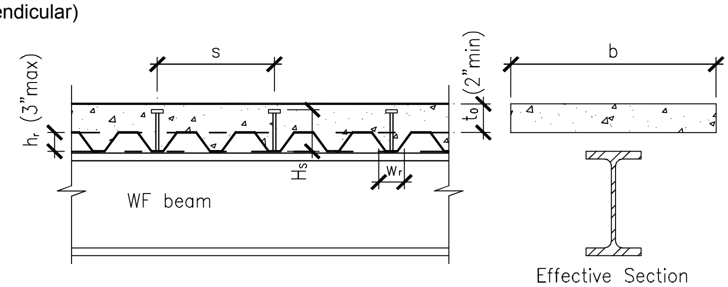


BEAM SPAN	$L =$	30	ft
BEAM SPACING (TRIB. WIDTH)	$B =$	10	ft, o.c.
SUPERIMPOSED LOAD, ASD	$w_s =$	106	lbs / ft^2

LATERAL SHEAR STRESS BETWEEN BEAM & FLOOR DECK

$v =$ 720 lbs / ft, (*ASD level, no* Ω_0)

COLLECTOR / DRAG AXIAL LOAD	$P =$	106	kips, (*ASD level, including* Ω_0)
WEAK AXIS EFFECTIVE LENGTH	$KL_y =$	10	ft
RIBS PERPENDICULAR TO BEAM ?		Yes	(perpendicular)
BEAM YIELD STRESS	$F_y =$	50	ksi
CONCRETE STRENGTH	$f_c' =$	4	ksi
SHEAR STUD DIA. (1/2, 5/8, 3/4)	$\phi =$	3/4	in
STUDS SPACING 2	rows @	12	in o.c.



THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE REACTIONS, MOMENTS & SHEARS

$w = w_s + w_{wt} =$ 106 + (72.50 + 4.59) = 183.09 lbs / ft^2

= 1.83 kips / ft (total gravity loads on span beam)

$M_{max} =$ 206.0 ft-kips $R_1 =$ 27.46 kips

$V_{max} =$ 27.46 kips, at R1 right. $R_2 =$ 27.46 kips

CHECK DIMENSION REQUIREMENTS

$t_o =$ 4.5 in > 2 in [Satisfactory] (AISC 360-05 I3.2c.1.c)

$h_r =$ 3 in < 3 in [Satisfactory] (AISC 360-05 I3.2c.1.a)

$\phi =$ 3/4 in < 3/4 in [Satisfactory] (AISC 360-05 I3.2c.1.b)

$H_s = h_r + 1.5 =$ 4.5 in < $h_r + t_o - 0.5 =$ 7 in [Satisfactory] (AISC I3.2c.1.b)

$s =$ 12 in o.c. < $MAX[\,8(h_r + t_o)\,,\,36\,] =$ 60 in o.c. [Satisfactory]

> $4\,\phi =$ 3 in o.c. [Satisfactory] (AISC 360-05 I3.2d.6)

$w_r =$ 6 in > 2 in [Satisfactory] (AISC 360-05 I3.2c.1.a)

DETERMINE COMPOSITE PROPERTIES FOR PLASTIC DESIGN

$b = MIN\,(L / 4\,,\,B) =$ 90 in, (AISC 360-05 I3.1a)

$h = t_o + h_r + d =$ 25.6 in, (total height)

$A_{ctr} = 0.85\,f_c'\,b\,t_o / F_y =$ 27.5 in^2

$A_{total} = A_{ctr} + A =$ 41.0 in^2

$A_{fill} = A - 2A_f - A_w =$ 0.09 in^2

$A_f =$ 3.67 in^2

$t_w =$ 0.36 in

$t_f =$ 0.61 in

$$y_b = \begin{cases} h - \dfrac{A F_y}{0.85 f_c' b} & for \quad A_{total} \le 2 A_{ctr} \\ d - \dfrac{0.5 A_{total} - A_{ctr}}{b_f} & for \quad A_{total} \le 2\left(A_{ctr} + A_f\right) \\ t_f + \dfrac{A_{total} - A_f - 0.5 A_{fill}}{t_w} & for \quad A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases} = \quad 23.4 \text{ in, (plastic neutral axis to bottom)}$$

(cont'd)

$$y = \begin{cases} 0.5\left(t_0+h_r+y_b\right) \text{ for } A_{total} \le 2A_{ctr} \\ h-\dfrac{0.5t_0A_{ctr}+0.5dA+\left(0.5A_{total}-A_{ctr}\right)\left(h-d-y_b\right)}{0.5A_{total}} \text{ for } A_{total} \le 2\left(A_{ctr}+A_f\right) \\ h-\dfrac{0.5t_0A_{total}+A_f\left(t_0+h_r+t_f\right)+0.5A_{fill}\left(t_0+h_r+2t_f\right)+t_w\left(d-y_b-t_f\right)\left(h-0.5d-0.5y_b+0.5t_f\right)+t_w\left(y_b-t_f\right)\left(0.5y_b+0.5t_f\right)}{0.5A_{total}} \text{ for } A_{total} > 2\left(A_{ctr}+A_f\right) \end{cases}$$

$Z_{tr} = 0.5\, y\, A_{total}$ = 317 in^3

DETERMINE COMPOSITE PROPERTIES FOR ELASTIC DESIGN

$$n = \frac{E}{E_c} = \quad 7.56 \quad \text{, (ACI 318-05 8.5.1)}$$

$A_{ctr} = b\, t_0 / n$ = 53.5 in^2

$$y_b = \frac{A_{ctr}\left(d+h_r+0.5t_0\right)+0.5Ad}{A_{ctr}+A} = \quad 20.5$$ in, (elastic neutral axis to bottom)

$$I_{tr} = I_x + A\left(y_b-0.5d\right)^2 + \frac{A_{ctr}t_0^2}{12} + A_{ctr}\left(0.5t_0+h_r+d-y_b\right)^2 = \quad 3007$$ in^4

$$S_{tr} = \frac{I_{tr}}{y_b} = \quad 147$$ in^3, referred to steel bottom.

$$S_t = \frac{I_{tr}}{\left(d+h_r+t_0-y_b\right)} = \quad 586$$ in^3, referred to concrete top.

CHECK BENDING & SHEAR CAPACITIES

Middle Bottom : $M_{max} = (Z_{tr} / Z_x)\, M_{DL} + M_{LL}$ = 422.4 ft-kips

< $M_n / \Omega_b = Z_{tr}\, F_y / \Omega_b$ = 790.9 ft-kips, (AISC 360 I3.2a) [Satisfactory]

where Ω_b = 1.67 (AISC 360-05 I3.2a)

$3.76(E / F_y)^{0.5}$ = 90.55 > h / t_w = 50.28

Shear : V_{max} = 27.46 kips < $V_n / \Omega_v = 0.6F_yA_wC_v / \Omega_v$ = 117.05 kips, (AISC 360-05 I3.1b)

[Satisfactory]

where $2.24\,(E / F_y)^{0.5}$ = 53.946

k_v = 5 (AISC 360-05 G2.1b) C_v = 1 (AISC 360-05 G2.1b)

$(k_v\, E / F_y)^{0.5}$ = 53.852 Ω_v = 1.67 (AISC 360-05 G1)

CHECK SHEAR CONNECTOR CAPACITY FOR GRAVITY

M_{max} = 206.0 ft-kips < $M_n / \Omega_b = Z_x\, F_y / \Omega_b$ = 226.3 ft-kips **<== No Shear Stud Required**

where Ω_b = 1.67 (AISC 360-05 F1 & F2-1)

$C_f = MIN\,(0.85 f_c' A_c\, ,\, F_y A_s)$ = 675 kips, (AISC 360-05 C-I3.1)

$S_{eff} = Min[\, M_{max} / (0.66\, F_y)\, ,\, S_{tr}\,]$ = 75 in^3, refered to steel bottom.

$$V' = MAX\left[\left(\frac{S_{eff}-S_s}{S_{tr}-S_s}\right)^2, 0.25\right]C_f = \quad 168.75$$ kips, (AISC 360-05 C-I3-4)

$Q_n = (3/4)\, MIN\,[\, 0.5\, A_{sc}\, (f_c' E_c)^{0.5}\, ,\, R_g R_p A_{sc} F_u\,]$ = 9.80 kips, (AISC 360-05 I3.2d.3 & CBC 07 2204A.1.3)

where w_c = 150 pcf

$E_c = w_c^{\,1.5}\; 33\,(f_c')^{0.5}$ = 3834.3 ksi

A_{sc} = 0.44 in^2

F_u = 58 ksi

R_g = 0.85 (AISC 360-05 Table I3.2b.3)

R_p = 0.60 (AISC 360-05 Table I3.2b.3)

$\Sigma Q_n = Q_n\, N_r\, X_1 / s$ = 294.03 kips > V' [Satisfactory]

CHECK SHEAR CONNECTOR CAPACITY FOR LATERAL LOADS

$\Sigma Q_n = Q_n\, N_r / s$ = 1634 lbs / ft > v = 720 lbs / ft [Satisfactory]

(cont'd)

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right), & for \ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}\right), & for \ \frac{P_r}{P_c}<0.2 \end{cases} = 0.97 < 1.33$$ [Satisfactory]

Where $P_c = P_n / \Omega_c =$ 359 / 1.67 = 214.90 kips, (WF steel only, AISC 360-05 Chapter E)

> $P_r =$ 106.00 kips [Satisfactory]

$M_{cx} = M_n / \Omega_b =$ 790.85 ft-kips, (AISC 360 I3.2a)

> $M_{rx} =$ 422.35 ft-kips [Satisfactory]

CHECK INITIAL DEFLECTION / CAMBER AND STRESS ON NON-COMPOSITE

$w_{DL} =$ 75% *Self Wt* = 0.58 kips / ft (75% self weight load only)

$$\Delta_{Mid} = \frac{5wL^4}{384EI} = 0.51$$ in, downward perpendicular at middle of beam.

$M_{max} =$ *65.0* ft-kips < $M_n / \Omega_b = Z_x F_y / \Omega_b =$ *226.3* ft-kips [Satisfactory]

CHECK LIVE LOAD DEFLECTION ON COMPOSITE

$w = w_{LL} =$ 1.06 klf

$$\Delta_{Mid} = \left[\frac{5wL^4}{384EI_{eff}}\right] = 0.22$$ in, downward. < *L / 360* = 1.00 in [Satisfactory]

Technical References:

1. AISC 360-05: "Specification for Structural Steel Buildings", American Institute of Steel Construction, March 9, 2005.
2. CBC 2010: "California Building Code, Volume 2 of 2", International Building Code, January 1, 2011.

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Tube, Pipe, or WF Member Capacity Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY

MEMBER SHAPE (Tube, Pipe, or WF) & SIZE		HSS2X2X1/4	< == **Tube**
STEEL YIELD STRESS	F_y =	46	ksi
AXIAL COMPRESSION FORCE	P_r =	1	kips, ASD
STRONG AXIS EFFECTIVE LENGTH	kL_x =	8	ft
WEAK AXIS EFFECTIVE LENGTH	kL_y =	8	ft
STRONG AXIS BENDING MOMENT	M_{rx} =	0.8	ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH	L_b =	8	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	1	kips
WEAK AXIS BENDING MOMENT	M_{ry} =	0.1	ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	1	kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) & , \quad for \quad \dfrac{P_r}{P_c}\geq 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) & , \quad for \quad \dfrac{P_r}{P_c}< 0.2 \end{cases} = 0.45 \quad < \quad 1.0$$ **[Satisfactory]**

Where $P_c = P_n / \Omega_c$ = 20 / 1.67 = 12.18 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 3.70 / 1.67 = 2.21 ft-kips, (AISC 360-05 Chapter F)
> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 3.70 / 1.67 = 2.21 ft-kips, (AISC 360-05 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong} / \Omega_v$ = 18.4 / 1.67 = 11.0 kips > V_{strong} = 1.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 18.4 / 1.67 = 11.0 kips > V_{weak} = 1.0 kips **[Satisfactory]**

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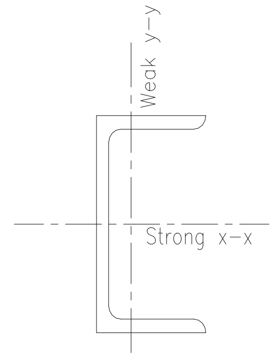
Channel Steel Member Capacity Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY



CHANNEL SIZE		C12X30	
STEEL YIELD STRESS	F_y =	36	ksi
AXIAL COMPRESSION FORCE	P =	10	kips, ASD
STRONG AXIS EFFECTIVE LENGTH	kL_x =	20	ft
WEAK AXIS EFFECTIVE LENGTH	kL_y =	10	ft
STRONG AXIS BENDING MOMENT	M_{rx} =	13	ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH	L_b =	20	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	77	kips
WEAK AXIS BENDING MOMENT	M_{ry} =	2	ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	35	kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) & , \quad for \quad \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) & , \quad for \quad \frac{P_r}{P_c}< 0.2 \end{cases} = 0.94 < 1.0 \quad \textbf{[Satisfactory]}$$

Where $P_c = P_n / \Omega_c$ = 89 / 1.67 = 53.44 kips, (AISC 360-05 Chapter E)

> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 42.79 / 1.67 = 25.62 ft-kips, (AISC 360-05 Chapter F)

> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 9.84 / 1.67 = 5.89 ft-kips, (AISC 360-05 Chapter F)

> M_{ry} **[Satisfactory]**

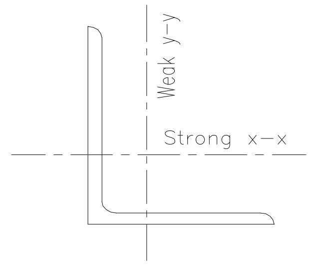
CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong} / \Omega_v$ = 132.2 / 1.67 = 79.2 kips > V_{strong} = 77.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 68.6 / 1.67 = 41.1 kips > V_{weak} = 35.0 kips **[Satisfactory]**

Angle Steel Member Capacity Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY



ANGLE SIZE		L8X6X1	
STEEL YIELD STRESS	F_y =	36	ksi
AXIAL COMPRESSION FORCE	P =	10	kips, ASD
STRONG GEOMETRIC AXIS EFFECTIVE LENGTH	kL_x =	20	ft
WEAK GEOMETRIC AXIS EFFECTIVE LENGTH	kL_y =	10	ft
STRONG GEOMETRIC AXIS BENDING MOMENT	M_{rx} =	5	ft-kips, ASD
STRONG GEOMETRIC AXIS BENDING UNBRACED LENGTH	L_{bx} =	20	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	77	kips
WEAK GEOMETRIC AXIS BENDING MOMENT	M_{ry} =	6	ft-kips, ASD
WEAK GEOMETRIC AXIS BENDING UNBRACED LENGTH	L_{by} =	10	ft
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	60	kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{r,\max}}{M_{c,max}}+\dfrac{M_{r,\min}}{M_{c,\min}}\right) & ,\ for\ \dfrac{P_r}{P_c}\ge 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{r,\max}}{M_{c,max}}+\dfrac{M_{r,\min}}{M_{c,\min}}\right) & ,\ for\ \dfrac{P_r}{P_c}< 0.2 \end{cases} = 0.91 < 1.0$$

[Satisfactory]

Where $P_c = P_n\ /\ \Omega_c$ = 136 / 1.67 = 81.28 kips, (AISC 360-05 Chapter E5)
> P_r **[Satisfactory]**

$M_{r,max}$ = 7.78 ft-kips or 0.71 ft-kips ,(major principal axis bending)

$M_{r,min}$ = 0.71 ft-kips or 7.78 ft-kips ,(minor principal axis bending)

$M_{c,max} = M_n\ /\ \Omega_b$ = 42.24 / 1.67 = 25.29 ft-kips, (smaller equal-leg section used, AISC 360-05 Chapter F10-2)
> $M_{r,max}$ **[Satisfactory]**

$M_{c,min} = M_n\ /\ \Omega_b$ = 15.89 / 1.67 = 9.51 ft-kips
> $M_{r,min}$ **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong}\ /\ \Omega_v$ = 172.8 / 1.67 = 103.5 kips > V_{strong} = 77.0 kips **[Satisfactory]**

$V_{n,weak}\ /\ \Omega_v$ = 129.6 / 1.67 = 77.6 kips > V_{weak} = 60.0 kips **[Satisfactory]**

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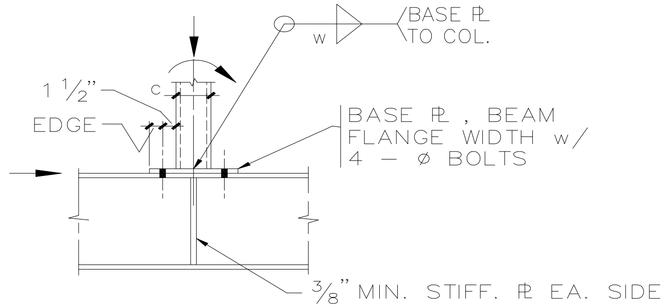
Connection Design for Column above Beam, Based on AISC Manual & AISC 360-05

INPUT DATA & DESIGN SUMMARY



BEAM SIZE W16X40

COLUMN SIZE ***c*** = 6 in
d = 6 in

BASE PLATE THICKNESS ***t*** = 3/4 in
COLUMN TO PLATE WELD ***w*** = 1/4 in
BOLT SIZE ϕ = 3/4 in
BOLT MATERIAL (A307, A325, A490) ASTM = A325

COLUMN BASE SERVICE LOADS
Axial ***P*** = 10 kips
Shear ***V*** = 25 kips
Moment ***M*** = 10 ft-kips

Plate Size : 7 in x 10.5 in x 0.75 in

THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS
CHECK WELD OF COLUMN TO BASE PLATE (AISC 360-05 J2.4)

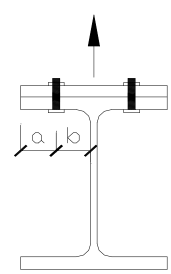
$f_x = V / A_w =$ 5.7 ksi
$f_y = P / A_w + M / S_w =$ 20.1 ksi
$f_v = (f_x^2 + f_y^2)^{0.5} =$ 20.9 ksi **<** $F_v = 0.6\ F_{EXX} / \Omega =$ 21.0 ksi **[Satisfactory]**
Where ***A_w*** = 4.4 in^2
S_w = 6.7 in^3

CHECK SHEAR BOLTS CAPACITY
$R_{nv} / \Omega =$ 42.4 kips, (AISC Manual 13th, Table 7-1)
> ***V*** **[Satisfactory]**

CHECK TENSION BOLTS FLANGE CAPACITY
$d' = \phi + 1/16 =$ 0.813 in $R_{nt} / \Omega =$ 19.9 kips / bolt, (AISC Manual 13th, Table 7-2)
$b' = b - 0.5\ \phi =$ 1.299 in
$a' = a + 0.5\ \phi =$ 2.049 in $\delta = 1 - \frac{d'}{S} =$ 0.910
S = 9.000 in, (AISC Manual 9th, Page 4-90)

$$t_c = \sqrt{\frac{8Bb'}{SF_y}} = \quad 0.678$$

$$\alpha' = \frac{1}{\delta\left(1+\frac{b'}{a'}\right)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \quad 0.538$$



$F_y =$ 50 ksi, for WF

$$R_n/\Omega = R_{nt}/\Omega \times Min\left(1\ ,\ 1.3 - \Omega\frac{f_v}{F_{nv}}\right) = \quad 14.1$$ kips / bolt, (AISC 360-05, J3.7)

$$T_{allow} = \begin{cases} R_n/\Omega\left(\frac{t}{t_c}\right)^2(1+\delta)\ ,\ for\ \ \alpha' > 1 \\ R_n/\Omega\left(\frac{t}{t_c}\right)^2(1+\delta\alpha')\ ,\ for\ \ 0 \le \alpha' \le 1 \\ R_n/\Omega\ ,\ for\ \ \alpha' < 0 \end{cases} = \quad 11.7$$ kips / bolt, (AISC Manual 9th, Page 4-89 to 4-95)

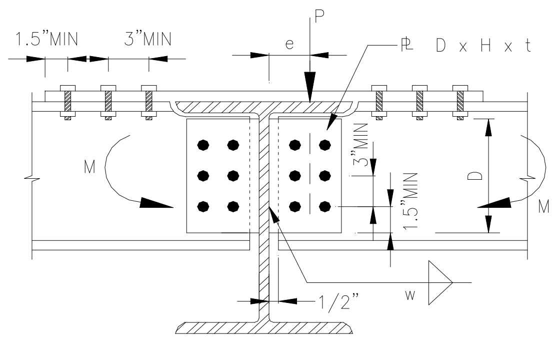
> T = - P / 4 + 0.5 M / (1.5 + c) = 5.5 kips / bolt
[Satisfactory]

CHECK BASE PLATE BENDING CAPACITY
Edge = 1.500 in, (AISC 360-05, Tab J3.4 & J3.5)
Base Plate Size = 7 in x 10.5 in x 3/4 in

M_n / Ω_b = 1.48 ft-kips, (AISC 360-05 Chapter F) **>** ***M =*** 1.38 ft-kips
$F_y =$ 36 ksi, for Base Plate **[Satisfactory]**

Design for Fully Restrained Moment Connection across Girder Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY



WF BEAM SECTION	=>	W21X50	
GRAVITY SERVICE VERTICAL LOAD	***P*** =	47	kips
BALANCED SERVICE MOMENT	***M*** =	26	ft-kips
VERTICAL PLATE THICKNESS	t_V =	0.75	in
HORIZONTAL PLATE THICKNESS	t_H =	0.375	in
PLATE STEEL YIELD STRESS	F_y =	36	ksi
TRIAL WELD SIZE	***w*** =	0.5	in (1/2 in)
BOLT DIAMETER	ϕ =	1	in (1 in)
BOLT MATERIAL (A307, A325, A490)	***ASTM*** =	A325	
HOLE TYPE (STD, NSL, OVS, SSL, LSL)	=>	STD	

STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

CONNECTION TYPE (SC, N, X) => N

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

USE VERT. PLATE 9.5" x 4.0" x 3/4" WITH WELD 1/2" EA. SIDE TO GIRDER WEB & 1 ROW OF TOTAL (3) - 1" BOLTS AT BEAM END.
HORIZ. TOP PLATE THK. 3/8" WITH 2 ROW 2 BOLTS PER ROW AT ONE END.

TRY VERTICAL BOLTS 1 ***row &*** 3 ***bolts per row, (total 3 bolts.)***
IS TOP FLANGE COPED ? (1=Yes, 0=No,) => 1 Yes
HORIZONTAL BOLTS 2 ***rows &*** 2 ***bolts per row, (total 4 bolts at one end.)***

ANALYSIS

BEAM SECTION PROPERTIES (AISC 13th Table 1)

d	t_w	t_f	**k**
20.8	0.38	0.535	1.04

CHECK CAPACITY OF VERTICAL BOLTS (AISC 360-05 J3)

Allow shear per bolt	=	18.8	kips / bolt, (R_n / Ω_v, AISC 13th Table 7)					
T = 2 M / d = coupling force	=	30	kips	$(P^2 + T^2)^{0.5}$ =	=	56	kips	
No. of bolts required	=	3.0		Number of bolts used	=	3	bolts	[Satisfactory]
Bolt spacing required	=	3.00	in	Bolt spacing used	=	3.00	in	[Satisfactory]
Edge spacing required	=	1.75	in, (Tab J3.4)	Edge spacing used	=	1.75	in	[Satisfactory]
Number of rows required	=	1	rows	Number of rows used	=	1	rows	[Satisfactory]
Bolt group capacity	=	57	kips >	$(P^2 + T^2)^{0.5}$ =		56	kips	
			>	P =		47	kips	[Satisfactory]

CHECK CAPACITY OF VERTICAL WELDING (AISC 360-05 J2)

e	=	2.25	in, (AISC 360-05, Table J3.4)	
Plate thickness	=	0.75	in	
Weld size,w	=	0.50	in	
Min allowable weld	=	0.25	in	[Satisfactory]
Max allowable weld	=	0.69	in	[Satisfactory]
t_e	=	0.35	in	
D	=	9.5	in	
I = 2 (t_e D^3 / 12)	=	50.5	in^4	
Vertical shear = P / A_w = P / 2 D t_e	=	7.0	ksi	
Bending stress = 0.5 P e D / I	=	9.9	ksi	
Tension stress = T / A_w = T / 2 D t_e	=	4.5	ksi	

Resultant Stress = $[(P/A_w)^2 + (0.5\ P\ e\ D / I + T/A_w)^2]^{0.5}$ = 16.0 ksi

Allow shear F_w / Ω = (0.6 x 70 ksi) / 2.0 = = 21.0 ksi > 16.0 ksi [Satisfactory]

CHECK VERTICAL PLATE FOR SHEAR CAPACITY (AISC 365-05 G2)

P / A = 6.6 ksi < 0.6 F_y C_v / Ω_v = 0.6 F_y 1.0 / 1.5 = 14.4 ksi [Satisfactory]

CHECK VERTICAL PLATE FOR TENSION CAPACITY (AISC 365-05 D)

T / A = 4.2 ksi < F_y / Ω_t = F_y / 1.67 = 21.56 ksi [Satisfactory]

CHECK VERTICAL NET SHEAR FRACTURE (AISC 360-05 J4.2)

F_u = 58 ksi (AISC Manual 13th Edition, Pg. 2-39)

P_{allow} = 0.6 F_u / Ω [D - n (d_s + 1/8)] t_V = 80 kips > 47 kips [Satisfactory]

CHECK VERTICAL NET TENSION FRACTURE (AISC 360-05 J4.1)

F_u = 58 ksi

T_{allow} = F_u / Ω [D - n (d_s + 1/8)] t_V = 133 kips > 30 kips [Satisfactory]

(cont'd)

CHECK BLOCK SHEAR (WEB TEAR-OUT, AISC 360-05 J4) <== Applicable only for top flange coped.

l_h = 1.3 in

l_v = 5.3 in

F_u = 65 ksi (for WF, AISC Manual 13th Edition, Pg. 2-39)

$R_{bs,P}$ = $0.6\ A_v\ F_u\ /\ \Omega + A_t\ F_u\ /\ \Omega = (0.3\ l_v + 0.5\ l_h)\ t_w\ F_u$ = 54 kips > P = 47.00 kips [Satisfactory]

$R_{bs,T}$ = $(0.5\ l_v + 2 \times 0.3\ l_h)\ t_w\ F_u$ = 83 > T = 30 kips [Satisfactory]

CHECK CAPACITY OF HORIZONTAL BOLTS (AISC 360-05 J3)

Allow shear per bolt = 18.8 kips / bolt, ($R_n\ /\ \Omega_v$, AISC 13th Table 7)

Bolt group capacity = 75 kips > T = 30 kips [Satisfactory]

CHECK HORIZONTAL PLATE FOR TENSION CAPACITY (AISC 365-05 D)

T / A = 13.3 ksi < $F_y\ /\ \Omega_t = F_y\ /\ 1.67$ = 21.56 ksi [Satisfactory]

CHECK HORIZONTAL NET TENSION FRACTURE (AISC 360-05 J4.1)

F_u = 58 ksi

T_{allow} = $F_u\ /\ \Omega\ [\ 6" - 2\ (d_s + 1/8\ )\ ]\ t_H$ = 41 kips > 30 kips [Satisfactory]

CHECK BLOCK SHEAR (TOP FLANGE TEAR-OUT, AISC 360-05 J4)

l_t = 1.9 in

l_v = 5.6 in

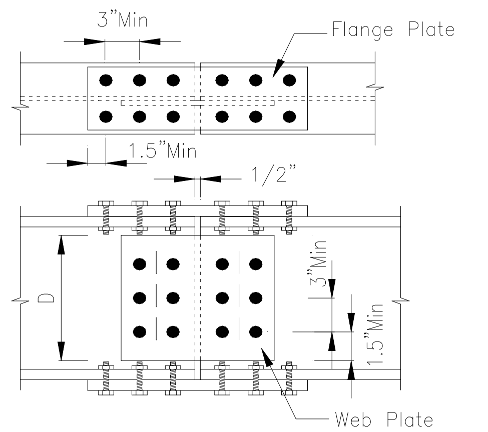
F_u = 65 ksi (for WF, AISC Manual 13th Edition, Pg. 2-39)

$R_{bs,T}$ = $(0.5\ l_t + 0.3\ l_v)\ t_f\ F_u$ = 91 > T = 30 kips [Satisfactory]

Beam Bolted Splice Design Based on AISC Manual 13th Edition (AISC 360-05)

INPUT DATA & DESIGN SUMMARY

(Flange Plate 5.71 in x 19.5 in x 1 in)



(Web Plate 9.5 in x 13.5 in x 0.75 in)

WF BEAM SECTION => W21X48

WF STEEL YIELD STRESS F_y = 50 ksi

PLATE STEEL YIELD STRESS F_{yp} = 36 ksi

FLANGE PLATE THICKNESS t_{fp} = 1 in

WEB PLATE THICKNESS t_{wp} = 0.75 in

BEAM AXIAL LOAD, ASD T = 70 kips, (20% WF's P_n / Ω_t)

VERTICAL SHEAR FORCE, ASD V = 100 kips, (69% WF's V_n / Ω_v)

MOMENT AT SPLICE, ASD M = 80 ft-k, (30% WF's M_n / Ω_b)

BOLT DIAMETER ϕ = 1 in (1 in)

BOLT MATERIAL (A307, A325, A490) ASTM = A325

HOLE TYPE (STD, NSL, OVS, SSL, LSL) => STD

STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

CONNECTION TYPE (SC, N, X) => N

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

THE DESIGN IS ADEQUATE.

FLANGE BOLTS 2 rows 3 bolts on each row (total 6 bolts at each end)

WEB BOLTS 2 vert rows 3 bolts on each row (total 6 bolts at each end)

ANALYSIS

SECTION PROPERTIES (AISC 13th Table 1)

d	t_w	t_f	k	b_f	A	Z_x
20.6	0.35	0.43	0.93	8.14	14.1	107

DETERMINE PLATE DIMENSIONS

Bolt spacing required = 3.00 in, (Tab J3.3) Bolt spacing used = 3.00 in [Satisfactory]

Edge spacing required = 1.75 in, (Tab J3.4) Edge spacing used = 1.75 in [Satisfactory]

Flange Plate B = 5.71 in < b_f [Satisfactory]

L_{fp} = 19.5 in

Web Plate D = 9.5 in < $d - 2k - 2t_f$ [Satisfactory]

L_{wp} = 13.5 in

CHECK CAPACITY OF BOLTS (AISC 360-05 J3)

Allowable shear capacity R_n / Ω_v = 18.8 kips / bolt, (AISC 13th Table 7)

Flange bolt shear $v = T / A_T + M / S =$ 11.7 kips / bolt < R_n / Ω_v [Satisfactory]

Web bolt shear $v = [(T / A_T)^2 + (V / A_V)^2]^{0.5} =$ 17.1 k / bolt < R_n / Ω_v [Satisfactory]

where A_T = 18 bolts, (total one end bolts)

S = 123.6 in-bolts, (flange bolts only)

A_V = 6 bolts, (one end web bolts only)

CHECK WEB PLATE FOR SHEAR CAPACITY (AISC 365-05 G2)

$V / (D\, t_{wp})$ = 14.0 ksi < $0.6\, F_{yp}\, C_v / \Omega_v = 0.6\, F_{yp}\, 1.0 / 1.5 =$ 14.4 ksi [Satisfactory]

CHECK FLANGE PLATE FOR TENSION CAPACITY (AISC 365-05 D)

$T / (2\, B\, t_{fp} + D\, t_{wp}) + M / (d\, B\, t_{fp}) =$ 11.9 ksi < $F_{yp} / \Omega_t = F_{yp} / 1.67 =$ 21.56 ksi [Satisfactory]

CHECK FLANGE NET TENSION FRACTURE (AISC 360-05 J4.1)

F_{up} = 58 ksi (for plate, AISC Manual 13th Edition, page. 2-39)

$0.5\, T + M / d =$ 81.60 kips < $F_{up} / \Omega\, [B - n\,(d_s + 1/8)]\, t_{fp} =$ 100 kips [Satisfactory]

CHECK FLANGE NET SHEAR FRACTURE (AISC 360-05 J4.2)

F_u = 65 ksi (for WF, AISC Manual 13th Edition, page. 2-39)

$0.5\, T + M / d =$ 81.60 kips < $0.6\, Min(F_u\, t_f\, , F_{up}\, t_{fp}) / \Omega\, (\Sigma\ \text{shear length}) =$ 82.80 kips [Satisfactory]

CHECK WEB PLATE BLOCK SHEAR (AISC 360-05 J4)

l_h = 4.9 in

l_v = 1.2 in

V = 100.00 kips < $0.6\, A_v\, F_{up} / \Omega + A_t\, F_{up} / \Omega = (0.3\, l_v + 0.5\, l_h)\, t_{wp}\, F_{up} =$ 122.89 kips [Satisfactory]

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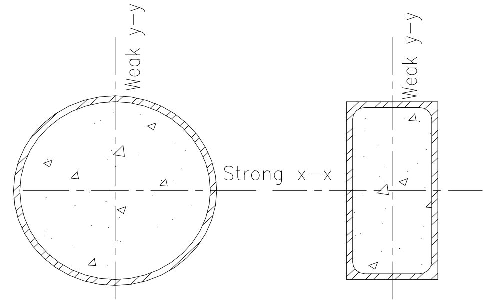
Filled Composite Column Design Based on AISC 360-05 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

STEEL SHAPE (Tube or Pipe) & SIZE		HSS10X8X1/2	< == Tube
STEEL YIELD STRESS	F_y =	46	ksi
CONCRETE STRENGTH	f_c' =	4	ksi
AXIAL COMPRESSION FORCE	P_r =	100	kips, ASD
STRONG AXIS EFFECTIVE LENGTH	kL_x =	40	ft
WEAK AXIS EFFECTIVE LENGTH	kL_y =	3	ft
STRONG AXIS BENDING MOMENT	M_{rx} =	15	ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH	L_b =	40	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	50	kips
WEAK AXIS BENDING MOMENT	M_{ry} =	20	ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	30	kips



THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK FILLED COMPOSITE COLUMN LIMITATIONS (AISC 360-05, I2.2a)

A_{steel} / A_{total} = 0.20 > 1.0% **[Satisfactory]**

Where A_{steel} = 15.3 in^2

A_{total} = 78.3 in^2

b / t = 16.00 < $2.26\,(E / F_y)^{0.5}$ = 56.75 **[Satisfactory]**

D / t = N/A $0.15\,E / F_y$ = 94.57

Where b = 8.0 in

D = N/A E = 29000 ksi

t = 0.5 in

CHECK COMPRESSION CAPACITY (AISC 360-05, I2.2b)

$P_c = P_n / \Omega_c$ = 142.77 kips > P_r **[Satisfactory]**

Where Ω_c = 2.0 P_n = 285.54 kips

C_2 = 0.85 P_0 = 918 kips

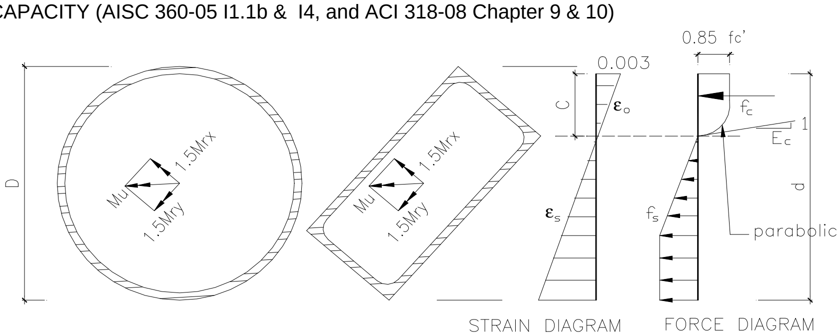
C_3 = 0.90 P_e = 325.59 kips

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05 I1.1b & I4, and ACI 318-08 Chapter 9 & 10)

$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c}\ ,\quad E_c = 57\sqrt{f'_c}\ ,\quad E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right], & for\ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c\ , & for\ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ , & for\ \ \varepsilon_s \le \varepsilon_t \\ f_y\ , & for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$



C = 5.45 in

$P_c = P_n / \Omega_b$ = 100 kips

$M_c = M_n / \Omega_b$ = 86 ft-kips > $M_u / (\Omega_b / \phi_b) = (M_{rx}^2 + M_{ry}^2)^{0.5}$ = 25.0 ft-kips

Where Ω_b = 1.67 P_n = 167 kips **[Satisfactory]**

ϕ_b = 0.9 M_n = 143.1 ft-kips

CHECK SHEAR CAPACITY (AISC 360-05, I2.2d & G2)

$V_{n,strong} / \Omega_v$ = 184.0 / 1.67 = 110.2 kips > V_{strong} = 50.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 147.2 / 1.67 = 88.1 kips > V_{weak} = 30.0 kips **[Satisfactory]**

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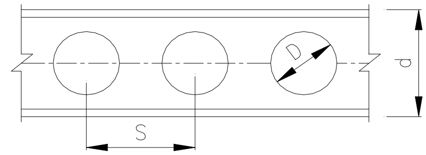
Cellular Beam Design Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY

BEAM SIZE		W21X83	
CELL DIAMETER	D =	6	in
CELL PITCH ***(1.25 D to 1.5 D)***	S =	9	in
STEEL YIELD STRESS	F_y =	50	ksi
AXIAL COMPRESSION FORCE	P_r =	60	kips, ASD
STRONG AXIS EFFECTIVE LENGTH	kL_x =	28	ft
WEAK AXIS EFFECTIVE LENGTH	kL_y =	8	ft
STRONG AXIS BENDING MOMENT	M_{rx} =	50	ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH	L_b =	8	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	25	kips
WEAK AXIS BENDING MOMENT	M_{ry} =	10	ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	200	kips



(12% steel weight saved.)

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK FULL SECTION CAPACITY, AT WEB OPENING, USING STRAIN-COMPATIBILITY METHOD (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for\ \frac{P_r}{P_c}\ge 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for\ \frac{P_r}{P_c}< 0.2 \end{cases} = 0.31 < 1.0 \quad \textbf{[Satisfactory]}$$

Where $P_c = P_n / \Omega_c$ = 867 / 1.67 = 519.32 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 682.82 / 1.67 = 408.88 ft-kips, (AISC 360-05 Chapter F)
> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 125.43 / 1.67 = 75.11 ft-kips, (AISC 360-05 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK T-SHAPE CAPACITY AT WEB OPENINGS (AISC 360-05, H1)

$0.5 (b_f - t_w) / t_f$ = 4.70 < $1.0 (E / F_y)^{0.5}$ = 24.08 **[Satisfactory]**
d_T / t_w = 14.95 < $0.75 (E / F_y)^{0.5}$ = 18.06 **[Satisfactory]**
(AISC 360-05, Table B4.1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{r,x}}{M_{c,x}}+\frac{M_{r,y}}{M_{c,y}}\right) , & for\ \frac{P_r}{P_c}\ge 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{r,x}}{M_{c,x}}+\frac{M_{r,y}}{M_{c,y}}\right) , & for\ \frac{P_r}{P_c}< 0.2 \end{cases} = 0.97 < 1.0 \quad \textbf{[Satisfactory]}$$

Where $P_c = P_n / \Omega_c$ = 341 / 1.67 = 203.99 kips, (AISC 360-05 Chapter E4a or D2)
> $0.5 P_r$ = 30.00 kips **[Satisfactory]**

$M_{c,x} = M_n / \Omega_b$ = 54.64 / 1.67 = 32.72 ft-kips, (AISC 360-05, F9)
> $0.5 M_{r,x}$ = 25.00 ft-kips **[Satisfactory]**

$M_{c,y} = M_n / \Omega_b$ = 62.69 / 1.67 = 37.54 ft-kips, (AISC 360-05, F6)
> $0.5 M_{r,y}$ = 5.00 ft-kips **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong} / \Omega_v$ = 237.9 / 1.67 = 142.5 kips > V_{strong} = 25.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 418.8 / 1.67 = 250.8 kips > V_{weak} = 200.0 kips **[Satisfactory]**

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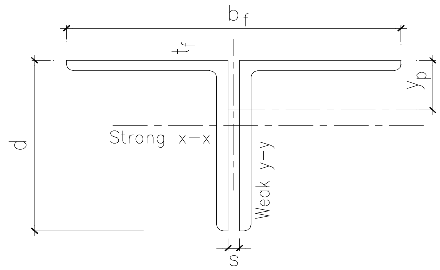
Double Angle Capacity Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY



DOUBLE ANGLE SIZE 2 - L8X6X1
LONG LEGS BACK TO BACK ? Yes (LLBB)
GAP DIMENSION s = 0.375 in
STITCHES SPACING kL_z = 4 ft, o.c.

STEEL YIELD STRESS F_y = 36 ksi

AXIAL FORCE (plus sign for compression) P = 80 kips, ASD
STRONG GEOMETRIC AXIS EFFECTIVE LENGTH kL_x = 20 ft
WEAK GEOMETRIC AXIS EFFECTIVE LENGTH kL_y = 10 ft

b_f =	12.4	in	t_f =	1.0	in
d =	8.0	in	y_p =	1.50	in
A =	22.0	in^2	I_x =	162	in^4
y =	3.14	in	I_y =	179	in^4

STRONG GEOMETRIC AXIS BENDING MOMEN' M_{rx} = 35 ft-kips, ASD,
DOUBLE LEGS TENSION ? Yes (back to back legs tension)
STRONG GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{bx} = 20 ft, (AISC 360-05 F9.2 & F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD V_{strong} = 200 kips

THE DESIGN IS ADEQUATE.

WEAK GEOMETRIC AXIS BENDING MOMENT M_{ry} = 15 ft-kips, ASD
WEAK GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{by} = 10 ft
WEAK DIRECTION SHEAR LOAD, ASD V_{weak} = 150 kips

ANALYSIS

CHECK LIMITING WIDTH-THICKNESS RATIOS (AISC 360-05, Table B4.1)

$0.5\,(b_f - s - t_w)/t_f$ = 5.00 < $1.0\,(E/F_y)^{0.5}$ = 28.38 **[Satisfactory]**

d/t_w = 4.00 < $0.75\,(E/F_y)^{0.5}$ = 21.29 **[Satisfactory]**

CHECK COMBINED COMPRESSION AND BENDING CAPACITY FOR DOUBLE ANGLES (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{r,x}}{M_{c,x}}+\frac{M_{r,y}}{M_{c,y}}\right) & , \ for \ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{r,x}}{M_{c,x}}+\frac{M_{r,y}}{M_{c,y}}\right) & , \ for \ \frac{P_r}{P_c}<0.2 \end{cases} = 0.99 < 1.0$$ **[Satisfactory]**

Where $P_c = P_n\,/\,\Omega_c$ = 270 / 1.67 = 161.41 kips, (AISC 360-05 Chapter E4a or D2)

> P_r = 80.00 kips **[Satisfactory]**

$M_{c,x} = M_n\,/\,\Omega_b$ = 159.68 / 1.67 = 95.62 ft-kips, (AISC 360-05, F9) > $M_{r,x}$ **[Satisfactory]**

$M_{c,y} = M_n\,/\,\Omega_b$ = 129.00 / 1.67 = 77.25 ft-kips, (AISC 360-05, F6) > $M_{r,y}$ **[Satisfactory]**

CHECK COMBINED COMPRESSION AND BENDING CAPACITY FOR SINGLE ANGLE BETWEEN STITCHES (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{r,\max}}{M_{c,max}}+\frac{M_{r,\min}}{M_{c,\min}}\right) & , \ for \ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{r,\max}}{M_{c,max}}+\frac{M_{r,\min}}{M_{c,\min}}\right) & , \ for \ \frac{P_r}{P_c}<0.2 \end{cases} = 0.96 < 1.0$$ **[Satisfactory]**

Where $P_c = P_n\,/\,\Omega_c$ = 256 / 1.67 = 153.09 kips, (AISC 360-05 Chapter E5)

> P_r = 43.50 kips **[Satisfactory]**

(Conservatively, P_r is single angle possible maximum axial load, which may be non-concurrent with moments.)

$M_{r,max}$ = 5.30 ft-kips or 5.30 ft-kips ,(major principal axis bending)

$M_{r,min}$ = 5.30 ft-kips or 5.30 ft-kips ,(minor principal axis bending)

$M_{c,max} = M_n\,/\,\Omega_b$ = 44.40 / 1.67 = 26.58 ft-kips, (smaller equal-leg section used, AISC 360-05 Chapter F10-2)

> $M_{r,max}$ **[Satisfactory]**

$M_{c,min} = M_n\,/\,\Omega_b$ = 15.89 / 1.67 = 9.51 ft-kips > $M_{r,min}$ **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong}\,/\,\Omega_v$ = 345.6 / 1.67 = 206.9 kips > V_{strong} = 200.0 kips **[Satisfactory]**

$V_{n,weak}\,/\,\Omega_v$ = 259.2 / 1.67 = 155.2 kips > V_{weak} = 150.0 kips **[Satisfactory]**

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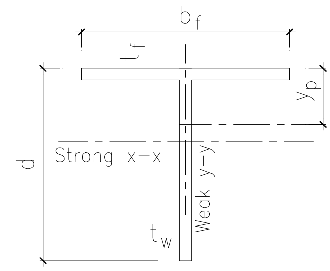
T-Shape Member Capacity Based on AISC 360-05

INPUT DATA & DESIGN SUMMARY



T-SHAPE SIZE	b_f =	12	in	t_f =	2.74	in	A =	45.7 in^2
	d =	11.2	in	t_w =	1.52	in	y =	2.94 in
							y_p =	1.91 in
STEEL YIELD STRESS	F_y =	50	ksi				I_x =	387 in^4
							I_y =	397 in^4

AXIAL FORCE (plus sign for compression) P = **10** kips, ASD
STRONG GEOMETRIC AXIS EFFECTIVE LENGTH kL_x = **20** ft
WEAK GEOMETRIC AXIS EFFECTIVE LENGTH kL_y = **10** ft

STRONG GEOMETRIC AXIS BENDING MOMEN M_{rx} = **70** ft-kips, ASD, THE STEM IN TENSION ? **Yes** (stem tension)
STRONG GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{bx} = **20** ft, (AISC 360-05 F9.2 & F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD V_{strong} = **300** kips

WEAK GEOMETRIC AXIS BENDING MOMENT M_{ry} = **150** ft-kips, ASD
WEAK GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{by} = **10** ft
WEAK DIRECTION SHEAR LOAD, ASD V_{weak} = **500** kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK LIMITING WIDTH-THICKNESS RATIOS (AISC 360-05, Table B4.1)

$0.5\,(b_f - t_w)/t_f$ = 1.91 < $1.0\,(E/F_y)^{0.5}$ = 24.08 **[Satisfactory]**

d/t_w = 7.37 < $0.75\,(E/F_y)^{0.5}$ = 18.06 **[Satisfactory]**

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{r,x}}{M_{c,x}}+\dfrac{M_{r,y}}{M_{c,y}}\right), & for\ \dfrac{P_r}{P_c}\ge 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{r,x}}{M_{c,x}}+\dfrac{M_{r,y}}{M_{c,y}}\right), & for\ \dfrac{P_r}{P_c}< 0.2 \end{cases} = 0.96 < 1.0$$ **[Satisfactory]**

Where $P_c = P_n / \Omega_c$ = 1386 / 1.67 = 829.85 kips, (AISC 360-05 Chapter E4a or D2)
> P_r = 10.00 kips **[Satisfactory]**

$M_{c,x} = M_n / \Omega_b$ = 312.64 / 1.67 = 187.21 ft-kips, (AISC 360-05, F9)
> $M_{r,x}$ **[Satisfactory]**

$M_{c,y} = M_n / \Omega_b$ = 431.36 / 1.67 = 258.30 ft-kips, (AISC 360-05, F6)
> $M_{r,y}$ **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong} / \Omega_v$ = 510.7 / 1.67 = 305.8 kips > V_{strong} = 300.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 986.4 / 1.67 = 590.7 kips > V_{weak} = 500.0 kips **[Satisfactory]**

Daniel T. Li
Engineering International

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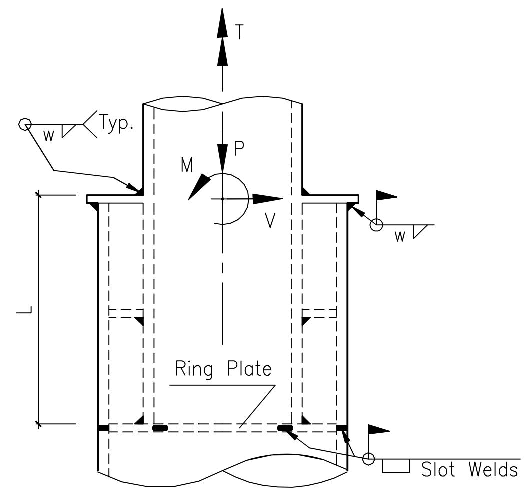
Sleeve Joint Connection Design, for Steel Cell Tower / Sign, Based on AISC 360-10

INPUT DATA & DESIGN SUMMARY

	D (in)	F_y (ksi)	t (in)	
SMALL PIPE	20	42	0.25	
BIG PIPE	24	42	0.25	
RING PLATE (Top, Middle, & Bottom)			t (in) 0.75	F_y (ksi) 36



SLEEVE LENGTH L = **30** in

JOINT HORIZONTAL SECTION FORCES

P = **15** kips, (Axial)
V = **39** kips, (Shear)
M = **50** ft-kips, (Bending)
T = **30** ft-kips, (Torsion)

SIZE OF FILLET WELD ALL AROUND

w = **0.1875** in

SLOT WELDS **6** - 5/8 in x **3** in
(Side Welds @ 60 deg. O.C.)

THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

CHECK FILLET WELD LIMITATIONS (AISC 360-10 J2.2b)

w = 0.1875 in > w_{MIN} = 0.125 in

< w_{MAX} = (ϕ 0.6 F_u t) / (ϕ 0.707 F_{EXX}) = (0.75 x 0.6 x 58 ksi) t / (0.75 x 0.707 x 70 ksi)

[Satisfactory] = 1.1795 t = 0.295 in

CHECK FILLET WELD CAPACITY (AISC 360-10 J2.4)

$f_x = V / A_w + T / (0.5\ D\ A_w)$ = 9.00 ksi

$f_y = P / A_w + M / S_w$ = 16.11 ksi

$f = (f_x^2 + f_y^2)^{0.5}$ = 18.46 ksi < 0.6 F_{EXX} / Ω = 21.00 ksi **[Satisfactory]**

Where D = 20 in, use small pipe conservatively
t_e = 0.707 w = 0.1326 in
A_w = D π t_e = 8.3291 in^2
S_w = $\pi\ [(D + 2\ t_e)^4 - D^4] / 32(D + 2\ t_e)$ = 41.92541 in^3
Ω = 2.0

CHECK SLOT WELD CAPACITY (AISC 360-10 J2.4)

$F_x = V / n + T / (0.5\ n\ D) + M / (n\ L)$ = 14.83 kips / slot weld

$F_y = P / n$ = 2.50 kips / slot weld

$f = (F_x^2 + F_y^2)^{0.5} / A_w$ = 8.02 ksi < 0.6 F_{EXX} / Ω = 21.00 ksi **[Satisfactory]**

Where n = 6 , number of slot welds
D = 24 in, big pipe diameter
A_w = 5/8 in x 3 in = 1.875 in^2 / slot weld

CHECK SHEAR RUPTURE CAPACITY OF RING PLATE (AISC 360-10 J4.2)

$f = (F_x^2 + F_y^2)^{0.5} / A_w$ = 8.02 ksi < 0.6 F_u / Ω = 17.40 ksi **[Satisfactory]**

Where F_u = 58 ksi, (AISC Manual 14th Edition, Tab. 2-4)
Ω = 2.0

CHECK BLOCK SHEAR CAPACITY OF SLOTTED PIPE (AISC 360-10 J4.3)

$f = (F_x^2 + F_y^2)^{0.5} / A_w$ = 8.30 ksi < Min (0.6 F_u , U F_u) / Ω = 14.50 ksi **[Satisfactory]**

Where F_u = 58 ksi, (AISC Manual 14th Edition, Tab. 2-4)
U = 0.5 ,not 1.0 conservatively
A_w = 2 t (5/8 in + 3 in) = 1.813 in^2 / slot weld

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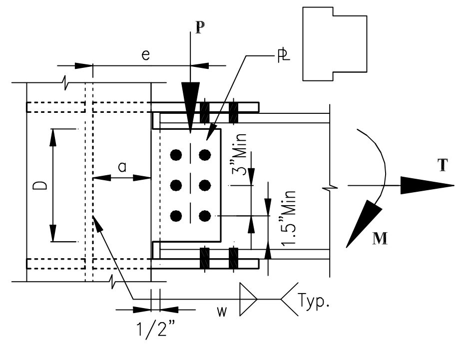
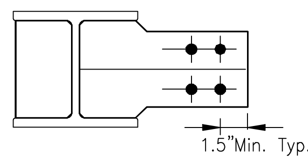
Moment Connection Design for Beam to Weak Axis Column Based on AISC 360-10

DESIGN CRITERIA

1. The moment load, M, is supported by top and bottom flanges, and the vertical load, P, by beam web. But the axial load, T, may be supported by flanges/web, or by both, so the design conservatively double accounts that the axial load fully by both flanges and web.
2. The additional thickness of connection plates and/or shims may be used to accommodate tolerances for fabrication.





INPUT DATA & DESIGN SUMMARY

WF BEAM SECTION	=>	W24X94	
MOMENT LOAD, ASD	M =	75	ft-kips
VERTICAL SERVICE LOAD, ASD	P =	43	kips
HORIZONTAL TENSION LOAD, ASD	T =	60	kips
PLATE THICKNESS	t =	0.625	in
PLATE STEEL YIELD STRESS	F_y =	50	ksi
WELD SIZE	w =	0.5	in (1/2 in)
BOLT DIAMETER	ϕ =	1	in (1 in)
BOLT MATERIAL (A307, A325, A490)	ASTM =	A325	
HOLE TYPE (STD, NSL, OVS, SSL, LSL)	=>	STD	

STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

CONNECTION TYPE (SC, N, X) => **SC**

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

THE DESIGN IS ADEQUATE.

WEB BOLT NO. **2** *rows &* **7** *bolts per row, (total 14 bolts.)*
WEB PLATE EXTENDED DIMENSION *a* = **7.11** in
EACH FLANGE BOLT NO. 2 *rows &* **3** *bolts per row, (total 6 bolts.)*

ANALYSIS

BEAM SECTION PROPERTIES (AISC Manual Table 1)

d	t_w	t_f	k	b_f
24.3	0.515	0.875	1.38	9.07

CHECK CAPACITY OF WEB BOLTS (AISC 360-10 J3)

Allow shear per bolt = 11.5 kips / bolt, (R_n / Ω_v, AISC Manual Table 7)

$(P^2 + T^2)^{0.5}$ = 74 kips

No. of bolts required	=	6.4		Number of bolts used	=	14	bolts	[Satisfactory]
Bolt spacing required	=	3.00	in	Bolt spacing used	=	3.00	in	[Satisfactory]
Edge spacing required	=	1.25	in, (Tab J3.4)	Edge spacing used	=	1.25	in	[Satisfactory]
Number of rows required	=	2	rows	Number of rows used	=	2	rows	[Satisfactory]
Bolt group capacity	=	161	kips	> $(P^2 + T^2)^{0.5}$	=	74	kips	
				> P	=	43	kips	[Satisfactory]

CHECK CAPACITY OF WEB PLATE WELDING (AISC 360-10 J2)

e, (including ***a***)	=	10.36	in, (AISC 360-10, Table J3.4)	
Plate thickness	=	0.63	in	
Weld size,w	=	0.50	in	
Min allowable weld	=	0.25	in	[Satisfactory]
Max allowable weld	=	0.56	in	[Satisfactory]
t_e	=	0.35	in	
$I = 2\ (t_e\ d^3\ / 12)$	=	845.4	in^4	
Vertical shear = $P / A_w = P / 2\ d\ t_e$	=	2.5	ksi	
Bending stress = $0.5\ P\ e\ d / I$	=	6.4	ksi	
Tension stress = $T / A_w = T / 2\ d\ t_e$	=	3.5	ksi	
Resultant Stress = $[\ (P/A_w)^2 + (0.5\ P\ e\ d / I + T/A_w)^2\ ]^{0.5}$	=	10.2	ksi	
Allow shear $F_w / \Omega = F_w / 2.0$	=	34.1	ksi	
	>	10.2	ksi	[Satisfactory]

θ =	54.372	deg, (AISC 360-10, J2-5)
Δ_u =	0.0378	in
Δ_m =	0.0288	in
f(p) =	1.1898	, (AISC 360-10, J2-9)
F_w =	68.28	ksi, (AISC 360-10, J2-8)

CHECK WEB PLATE FLEXURE CAPACITY WITH VON-MISES REDUCTION (AISC Manual, page 10-103)

D = 20.5 in

$f_v = [(P/A)^2 + (T/A + 6Pe / tD^2)^2]^{0.5}$ = 15.2 ksi

$F_{cr} = (F_y^2 - 3 f_v^2)^{0.5}$ = 42.5 ksi

M = Pe = 37.1 ft-k < $F_{cr}\ Z / \Omega$ = 139.2 ft-k [Satisfactory]

CHECK WEB PLATE FOR SHEAR CAPACITY (AISC 360-05 G2)

P / A = 3.4 ksi < $0.6\ F_y\ C_v / \Omega_v = 0.6\ F_y\ 1.0 / 1.5$ = 20 ksi [Satisfactory]

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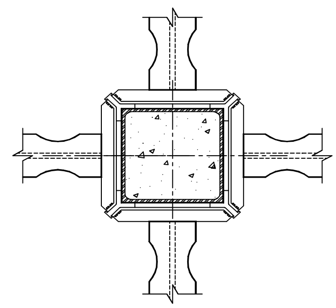
Seismic Bi-axial Moment Frame Design Based on AISC 358-10 & ACI 318-11

DESIGN CRITERIA

Concrete-filed 16 in. square HSS or Built-up box column (AISC 358-10 Chapter 10) is ***the first*** bi-axial bending column which can be used in seismic moment frames based on all versions of UBC/IBC/CBC (2010 CBC 2205A.5 & 2013 CBC 2206A.2). The beam to column connection (Collar/Bolts) can directly be purchased from the ConXtech ConXL (AISC Fabricator Certified Plant). But because bi-axial bending capacity is lower than about each principal axis, the bi-axial strong column-weak beam conditions have also to be checked based on ACI 318-11 filled composite capacity.

INPUT DATA & DESIGN SUMMARY

COLUMN SIZE	$b = h =$	16	in, *out-of width*
	$t =$	2.25	in, *web thickness*
COLUMN STEEL YIELD STRESS	$F_y =$	50	ksi
CONCRETE STRENGTH	$f_c' =$	5	ksi
BEAM	W24X103		
BEAM STEEL YIELD STRESS	$F_y =$	50	ksi
BEAMS AT ONE JOINT (2, 3, or 4)	$n =$	4	*, 4 - beams connected*
BEAM LENGTH	$L =$	24	ft
SEISMIC CHECK DIRECTION TO PRINCIPAL FRAME	$\theta =$	45	deg.



THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK BEAM LIMITATIONS (AISC 358-10, I0.3.1 & AISC 341-10 Table D1.1)

W24X103	**>**	W18		
	<	W30		
$t_f =$ 0.98	**<**	1.0	in	
$b_f =$ 9.00	**<**	12.0	in	
$L / d =$ 11.76	**>**	7		
$b_f / (2t_f) =$ 4.59	**<**	$0.3 (E_s / F_y)^{0.5} =$	7.22	**[Satisfactory]**

CHECK COLUMN LIMITATIONS (AISC 358-10, I0.3.2 & AISC 341-10 Table D1.1)

$t =$ 2.25	**>**	0.375	in	
$b =$ 16.00	**>**	16	in	
$f_c' =$ 5.00	**>**	3	ksi	
$(b - 2t) / t =$ 5.11	**<**	$2.26 (E / F_y)^{0.5} =$	54.43	**[Satisfactory]**

CHECK BI-AXIAL STRONG COLUMN-WEAK BEAM CONDITION AT $\boldsymbol{\theta}$ DIRECTION (AISC 358-10, I0.3.8, ACI 318-11 Chapter 10 & 21)

$\Sigma M_{nc} =$ 3003.7 ft-kips **>** $(6/5)\ \Sigma M_{nb} =$ 2941.1 ft-kips

(ACI 318-11 21-1) **[Satisfactory]**

where $\Sigma M_{nc} =$ 2 x M_{nc} *, top & bottom columns*

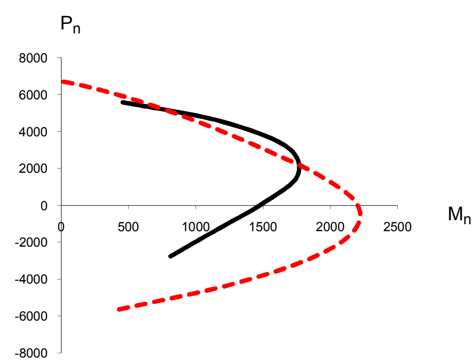
M_{nc} @ $P_u / \phi =$ 1501.9 ft-kips

$P_u =$ 128 kips, *($A_g f_c'$/10 suggested, ACI 318-11 21.6.1)*

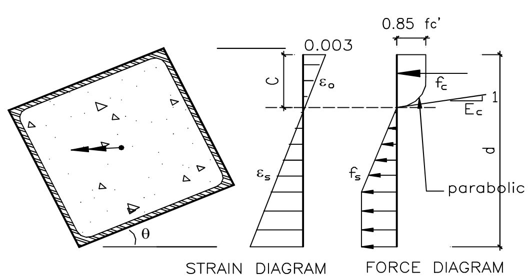
$$\varepsilon_O = \frac{2\left(0.85 f_C'\right)}{E_C} \quad , \quad E_C = 57\sqrt{f_C'} \quad , \quad E_S = 29000 ksi$$

$$f_C = \begin{cases} 0.85 f_C' \left[2\left(\frac{\varepsilon_C}{\varepsilon_O}\right) - \left(\frac{\varepsilon_C}{\varepsilon_O}\right)^2 \right] & , \quad for \quad 0 < \varepsilon_C < \varepsilon_O \\ 0.85 f_C' \quad , \quad for \quad \varepsilon_C \geq \varepsilon_O \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S \quad , \quad for \quad \varepsilon_S \leq \varepsilon_t \\ f_y \quad , \quad for \quad \varepsilon_S > \varepsilon_t \end{cases}$$



Solid Line - Tension Controlled
Dash Line - Compression Controlled



$\Sigma M_{nb} = [$ 2 $\cos\theta +$ 2 $\cos(90 - \theta)\ ]\ M_{nb} =$ 2450.9 ft-kips, *at* $\boldsymbol{\theta}$ *direction*

$M_{nb} = F_y Z_{RBS} =$ 866.5 ft-kips, *about principal axis. (AISC 358-10 10.8, C_{pr} & R_y not apply to ACI 318)*

$Z_{RBS} =$ 280 - 72.03 = 207.97 in^3, (AISC 318-10 10.3.1.6)

Purchaser's LOGO

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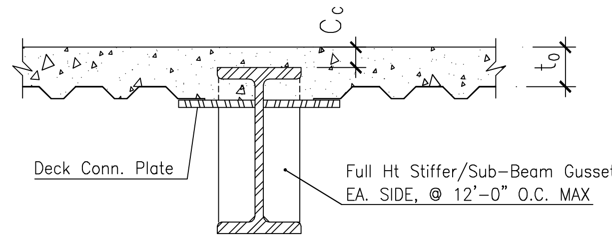
Thin Composite Beam/Collector Design Based on AISC 360-10 & ACI 318-11

DESIGN CRITERIA

Top flange within concrete can drag heavy diaphragm force to lateral frames, reduce floor system depth of composite steel-concrete, and not need shear studs. But if axial in tension the flexure capacity reduced.



INPUT DATA & DESIGN SUMMARY

BEAM/COLLECTOR SECTION **W21X44**

THE DESIGN IS ADEQUATE.

STEEL YIELD STRESS	F_y =	**50**	ksi
CONCRETE STRENGTH	$f_c{}'$ =	**5**	ksi
CONCRETE COVER	C_c =	**1**	in, 0.75" min., (ACI 318-11 7.7)
	t_0 =	**4.25**	in, 2.20 Min.
BEAM/COLLECTOR SPAN	L =	**30**	ft
SPACING (Tributary Width)	B =	**28**	ft, o.c.
COLLOCTOR AXIAL LOAD, *LRFD*	P_u =	**80**	kips, SD level, at center of W21X44
STRONG AXIS POSITIVE MOMENT, *LRFD*	M_u =	**380**	ft-kips, SD level
SHEAR LOAD, *LRFD*	V_u =	**75**	kips, SD level

ANALYSIS

DETERMINE CAMBER/SHORING ON NON-COMPOSITE

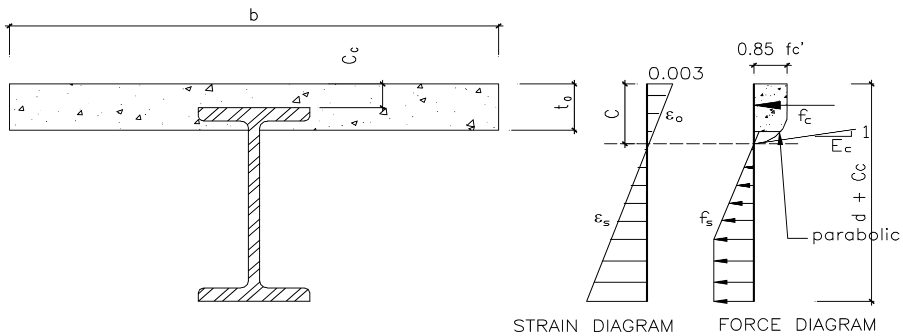
w = **2.497** *kips / ft, floor system self weight, to W21X44, on non-composite*

$\Delta = 5wL^4 / 384\ EI$ = 1.86 *in, deflection of W21X44*

< *L /* **180** = 2.00 *in* **[Satisfactory]**

Camber = **0.75** Δ = ***1.40*** *in*

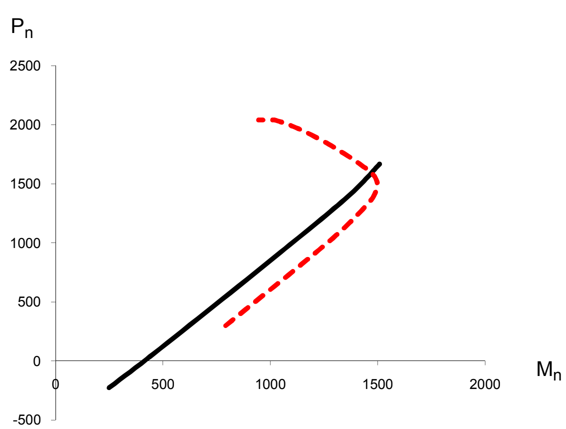
CHECK FLEXURAL & AXIAL CAPACITY (AISC 360-10, I3, ACI 318-11 Chapter 10 & 21)



$$\varepsilon_o = \frac{2(0.85 f_c')}{E_c}\ ,\quad E_c = 57\sqrt{f_c'}\ ,\quad E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f_c'\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] , & for \quad 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f_c' , & for \quad \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s , & for \quad \varepsilon_s \le \varepsilon_t \\ f_y , & for \quad \varepsilon_s > \varepsilon_t \end{cases}$$



Solid Line - Tension Controlled
Dash Line - Compression Controlled

b = *MIN (L / 4 , B)* = 90 *in, (AISC 360-10 I3.1a)*

ϕ = **0.9** *, (AISC 360-10 I3)*

M_n @ P_u / ϕ = *475.8 ft-kips*

ϕM_n = *428.2 ft-kips*

> $M_u + 0.5 C_c P_u$ = *383.3 ft-kips*

[Satisfactory]

CHECK SHEAR CAPACITY (AISC 360-10, G2)

ϕV_n = *195.6 kips* **>** V_u = *75.0 kips* **[Satisfactory]**

(cont'd)

CHECK WEB PLATE FOR TENSION CAPACITY (AISC 365-05 D)

$T / A =$ 4.7 ksi $<$ $F_y / \Omega_t = F_y / 1.67 =$ 29.94 ksi [Satisfactory]

CHECK WEB NET SHEAR FRACTURE (AISC 360-10 J4.2)

F_u = 70 ksi (AISC Manual, Pg. 2-39)

$P_{allow} = 0.6\, F_u / \Omega\, [\, D - n\, (d_s + 1/8\,)\,]\, t$ = 166 kips > 43 kips [Satisfactory]

CHECK WEB NET TENSION FRACTURE (AISC 360-10 J4.1)

F_u = 70 ksi

$T_{allow} = F_u / \Omega\, [\, D - n\, (d_s + 1/8\,)\,]\, t$ = 276 kips > 60 kips [Satisfactory]

CHECK CAPACITY OF EACH FLANGE BOLTS (AISC 360-10 J3)

One flange bolt group capacity = 69 kips > $(M / d + 0.5\, T) =$ 67 kips [Satisfactory]

CHECK EACH FLANGE PLATE FOR TENSION CAPACITY (AISC 365-05 D)

$(M / d + 0.5\, T) / A =$ 11.83 ksi $<$ $F_y / \Omega_t = F_y / 1.67 =$ 29.94 ksi [Satisfactory]

CHECK FLANGE NET SHEAR FRACTURE (AISC 360-10 J4.2)

$V_{allow} = 0.6\, F_u / \Omega\, [\, 1.5 + 3\, (n - 1) - n\, (d_s + 1/8\,)\,]\, 2\, t_{min}$ = 89 kips

> 67 kips [Satisfactory]

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Bolt Connection Design Based on AISC Manual 14th Edition (AISC 360-10)

INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS F_y = **50** ksi

SIDE PLATE THICKNESS t_s = **0.375** in by **1** One Side

A_b = **5** in^2, (Total Section Area)

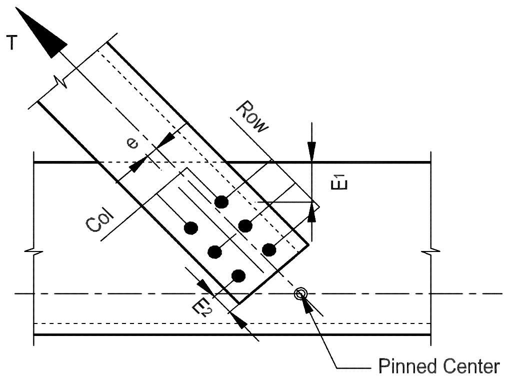
E_2 = **3** in

BASE PLATE THICKNESS t_b = **0.5** in

E_1 = **3** in

TOTAL AXIAL LOAD, ASD T = **59** kips, (Single Shear)

e = **1** in, (to bolts center)



BOLT DIAMETER ϕ = **1** in (1 in)

BOLT MATERIAL (A307, A325, A490) ASTM = **A325**

HOLE TYPE (STD, NSL, OVS, SSL, LSL) => **STD**

STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

THE DESIGN IS ADEQUATE.

CONNECTION TYPE (SC, N, X) => **N**

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

BOLTS NUMBER **3** Rows **2** Columns, (Total ***6*** Bolts)

MINIMUM BOLT SPACING s = **3** in

ANALYSIS

CHECK DIMENSION LIMITATIONS (AISC 360-10 J3)

Bolt spacing required = 3.00 in, (Tab J3.3) < ***3*** in [Satisfactory]

Edge spacing required = 1.75 in, (Tab J3.4) < $Min(E_1, E_2)$ [Satisfactory]

CHECK BOLT CAPACITY (AISC 360-10 J3)

Maximum single bolt shear $v = (T / 6) \times 1.67$ = 16.4 kips / bolt

< [Satisfactory]

Allowable shear capacity R_n / Ω_v = 18.8 kips / bolt, (AISC 14th Table 7)

CHECK NET TENSION FRACTURE OF SIDE PLATE (AISC 360-10 J4.1)

F_{up} = 58 ksi (for plate, AISC Manual 14th Edition, page. 2-39)

$(F_{up} / \Omega) [A_b - Col (\phi + 1/8) t_s]$ = 126 kips > T [Satisfactory]

CHECK BASE PLATE BLOCK SHEAR (AISC 360-10 J4)

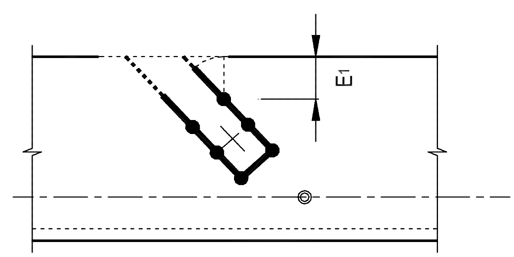
$0.6 A_v F_{up} / \Omega + A_t F_{up} / \Omega$ = 135 kips

> $6 v = T \times 1.67$ = 98 kips

[Satisfactory]

Where A_v = 6.1875 in^2, (Total Shear Area)

A_t = 0.94 in^2, (End Tension Area)



CHECK SIDE PLATE BLOCK SHEAR (AISC 360-10 J4)

$0.6 A_v F_{up} / \Omega + A_t F_{up} / \Omega$ = 101 kips

> $6 v = T \times 1.67$ = 98 kips

[Satisfactory]

Where A_v = 4.64 in^2, (Total Shear Area)

A_t = 0.70 in^2, (End Tension Area)

Purchaser's LOGO	PROJECT :		PAGE :	
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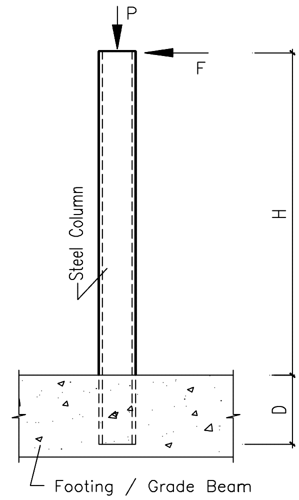
Cantilever Column System (SCCS/OCCS) Design Based on AISC 341-10/360-10 & ACI 318-14



INPUT DATA & DESIGN SUMMARY

COLUMN SHAPE (Square Tube or Pipe) & SIZE **HSS9X9X5/8** < == Tube

H = **8** ft

D = **22** in, *Embedment Depth*

GRAVITY LOAD P = **3** kips, *ASD level*

LATERAL LOAD F = **20** kips, *SD level*

(1=SCCS, 2=OCCS, 3=Wind) ==> **1** Ω_0 = 1.25 , *ASCE 7 Tab. 12.2-1*

STEEL YIELD STRESS F_y = **50** ksi

CONCRETE STRENGTH f_c' = **3** ksi

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK LIMITING WIDTH THICKNESS RATIO (AISC 341-10 E5.5a, E6.5a, & Tab.D1.1)

$0.038\ E_s / F_y$ = 22.040 <== *Not Apply*

b / t = 12.4 < $0.55\ (E_s / F_y)^{0.5}$ = 13.246

where t = 0.625 in **[Satisfactory]**

b = 7.75 in

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 341-10 E5.4a, E6.4a & AISC 360-10 H1)

$$\begin{cases} \dfrac{P_r}{P_{rc}} + \dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}\right) \ , \ for \ \dfrac{P_r}{P_{rc}} \ge 0.2 \\ \dfrac{P_r}{2P_{rc}} + \left(\dfrac{M_{rx}}{M_{cx}}\right) \ , \ for \ \dfrac{P_r}{P_{rc}} < 0.2 \end{cases} = \quad 0.99 \quad < \quad 1.0$$

[Satisfactory]

Where $P_r = (1 + 0.14\ S_{DS})\ P$ = 3.42 kips, *ASD level, (ASCE 7-10 12.4.3.2)*

$M_{rx} = 0.7\ \Omega_0\ F\ H$ = 140.00 ft-kips, *ASD level*

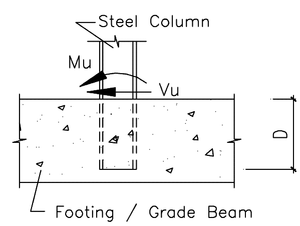
$P_{rc} = 15\%\ (P_n / \Omega_c)$ = 66.50 kips > P_r **[Satisfactory]**

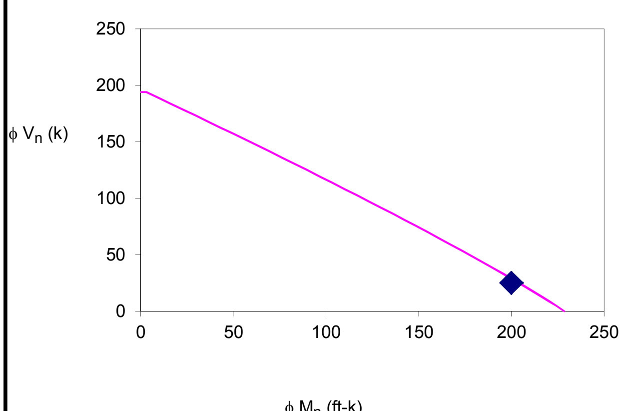
$M_{cx} = M_n / \Omega_b$ = 144.96 ft-kips > M_{rx} **[Satisfactory]**

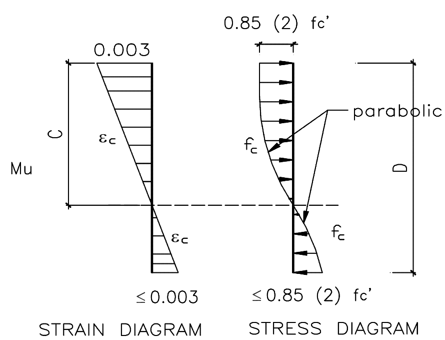
CHECK BASE FIXED MOMENT CONDITION (ACI 318 21 & 22)

$V_u = \Omega_0\ F$ = 25.00 kips, *SD level, (ASCE 7-10 12.4.3.2)*

$M_u = \Omega_0\ F\ H$ = 200.00 ft-kips







ϕM_n = 204 ft-kips @ V_u = 25 kips

> M_u = 200 ft-kips

[Satisfactory]

$\phi V_{n,max}$ = 193.90 kips, when C = 15.1 in

> V_u = 25 ft-kips **[Satisfactory]**

$$\varepsilon_O = \frac{2 f_c'}{E_C} 0.85 Min\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 2\right) \ , \quad E_C = 57\sqrt{f_c'}$$

$$f_c = \begin{cases} 0.85 Min\left(\sqrt{\dfrac{A_2}{A_1}}\ ,\ 2\right) f_c' \left[2\left(\dfrac{\varepsilon_C}{\varepsilon_O}\right) - \left(\dfrac{\varepsilon_C}{\varepsilon_O}\right)^2\right] \ , \ for \ \ 0 < \varepsilon_C < \varepsilon_O \\ 0.85 Min\left(\sqrt{\dfrac{A_2}{A_1}}\ ,\ 2\right) f_c' \ , \ for \ \ \varepsilon_C \ge \varepsilon_O \end{cases}$$

where ϕ = 0.65 , (ACI 318 21.2)

Bearing factor = 2, (ACI 318 14.5.1.1)

b = effective bearing width = 95% b_f = 8.55 in

Purchaser's LOGO

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Non-Prismatic Composite Girder Design Based on AISC 360-10 / 2013 CBC / 2015 IBC

INPUT DATA & DESIGN SUMMARY

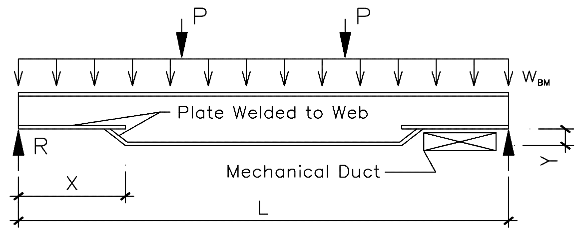
				A	d	I_x	S_x	Z_x
GIRDER SECTION	= >	**W24X55**	= >	16.3	23.6	1360	115	135

FLOOR DECK TYPE = > **W3-6 1/4" LW**

REDUCED DIMENSION (one end or both ends)

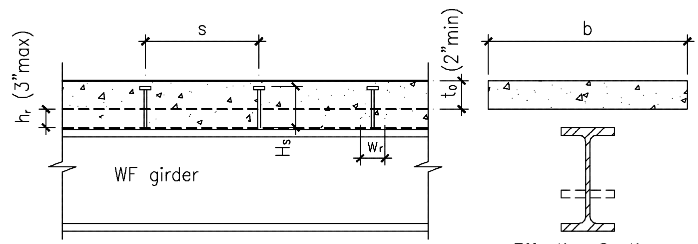
X = **60** in

Y = **10** in



GIRDER SPAN	L =	**28**	ft
GIRDER SPACING (TRIB. WIDTH)	B =	**21**	ft, o.c.
GIRDER SELF WEIGHT, ASD	w_{BM} =	**55.47**	lbs / ft
NUMBER OF EQUAL POINT LOAD	N =	**2**	
EQUAL POINT LOAD, ASD	P =	**30.417**	kips @ 9.33" o.c.
RIBS PERPENDICULAR TO GIRDER ?		**No**	(parallel)
GIRDER YIELD STRESS	F_y =	**50**	ksi
CONCRETE STRENGTH	f_c' =	**3**	ksi
SHEAR STUD DIA. (1/2, 5/8, 3/4)	ϕ =	**3/4**	in
STUDS SPACING **1**	row @	**18**	in o.c.

(Total Studs 20)



Effective Section

THE GIRDER DESIGN IS ADEQUATE.

USE C = 0.92" AT MID GIRDER.

ANALYSIS

DETERMINE REACTIONS, MOMENTS & SHEARS

$R = 0.5\,(w_{BM}\,L + N\,P) =$ 31.19 kips

$M_{max} = 0.5\,R\,L - 0.125\,w_{BM}\,L^2 - \Sigma(P\,D_i) =$ 289.3 ft-kips, at middle of girder

$M_{red} = R\,X - 0.5\,w_{BM}\,X^2 =$ 155.3 ft-kips, at reduced section

$V_{max} = R =$ 31.19 kips

CHECK DIMENSION REQUIREMENTS

t_o =	3.25	in	>	2	in	[Satisfactory]	(AISC 360-10 I3.2c)
h_r =	3	in	<	3	in	[Satisfactory]	(AISC 360-10 I3.2c)
ϕ =	3/4	in	<	3/4	in	[Satisfactory]	(AISC 360-10 I3.2c)
$H_s = h_r + 1.5$ =	4.5	in	<	$h_r + t_o - 0.5$ = 5.75	in	[Satisfactory]	(AISC I3.2c)
s =	18	in o.c.	<	$MAX[\,8(h_r + t_o)\,,\,36\,]$ = 50	in o.c.	[Satisfactory]	
			>	$4\,\phi$ = 3	in o.c.	[Satisfactory]	(AISC 360-10 I3.2d)
w_r =	6	in	>	2	in	[Satisfactory]	(AISC 360-10 I3.2c)

DETERMINE COMPOSITE PROPERTIES FOR PLASTIC DESIGN

$b = MIN\,(L/4\,,\,B) =$ 84 in, (AISC 360-10 I3.1a)

$h = t_0 + h_r + d =$ 29.9 in, (total height)

$A_{ctr} = 0.85\,f_c'\,b\,t_0\,/\,F_y =$ 13.9 in^2

$A_{total} = A_{ctr} + A =$ 30.2 in^2

$A_{fill} = A - 2A_f - A_w =$ 0.30 in^2

$A_f =$ 3.54 in^2

$t_w =$ 0.40 in

$t_f =$ 0.51 in

$$y_b = \begin{cases} h - \dfrac{A F_y}{0.85 f_c' b} & for\ \ A_{total} \le 2 A_{ctr} \\ d - \dfrac{0.5 A_{total} - A_{ctr}}{b_f} & for\ \ A_{total} \le 2\left(A_{ctr} + A_f\right) \\ t_f + \dfrac{A_{total} - A_f - 0.5 A_{fill}}{t_w} & for\ \ A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases} =$$

23.4 in, (plastic neutral axis to bottom)

$$y = \begin{cases} 0.5\left(t_0 + h_r + y_b\right) \ \ for\ \ A_{total} \le 2 A_{ctr} \\ h - \dfrac{0.5 t_0 A_{ctr} + 0.5 d A + \left(0.5 A_{total} - A_{ctr}\right)\left(h - d - y_b\right)}{0.5 A_{total}} \ \ for\ \ A_{total} \le 2\left(A_{ctr} + A_f\right) \\ h - \dfrac{0.5 t_0 A_{total} + A_f\left(t_0 + h_r + t_f\right) + 0.5 A_{fill}\left(t_0 + h_r + 2 t_f\right) + t_w\left(d - y_b - t_f\right)\left(h - 0.5 d - 0.5 y_b + 0.5 t_f\right) + t_w\left(y_b - t_f\right)\left(0.5 y_b + 0.5 t_f\right)}{0.5 A_{total}} \ \ for\ \ A_{total} > 2\left(A_{ctr} + A_f\right) \end{cases}$$

thus, $y =$ 17.0 in, (moment arm between centroid of tensile force and the resultant compressive force.)

$Z_{tr} = 0.5\,y\,A_{total} =$ 257 in^3

(cont'd)

DETERMINE COMPOSITE PROPERTIES FOR ELASTIC DESIGN

$n = \frac{E}{E_c} =$ 13.01 , (ACI 318-14 19.2.2.1)

$A_{ctr} = b\, t_0 / n =$ 21.0 in^2

$y_b = \frac{A_{ctr}(d+h_r+0.5t_0)+0.5Ad}{A_{ctr}+A} =$ 21.0 in, (elastic neutral axis to bottom)

$I_{tr} = I_x + A(y_b - 0.5d)^2 + \frac{A_{ctr}t_0^2}{12} + A_{ctr}(0.5t_0+h_r+d-y_b)^2 =$ 3853 in^4

$S_{tr} = \frac{I_{tr}}{y_b} =$ 183 in^3, referred to steel bottom.

$S_t = \frac{I_{tr}}{(d+h_r+t_0-y_b)} =$ 438 in^3, referred to concrete top.

CHECK BENDING & SHEAR CAPACITIES

Middle Bottom : $M_{max} = (Z_{tr} / Z_x)\, M_{DL} + M_{LL} =$ 366.8 ft-kips

$< M_n / \Omega_b = Z_{tr}\, F_y / \Omega_b =$ 640.1 ft-kips, (AISC 360 I3.2a) [Satisfactory]

where $\Omega_b =$ 1.67 (AISC 360-10 I3.2a)

$3.76(E / F_y)^{0.5} =$ 90.55 $>$ $h / t_w =$ 59.75

$w_{deck} =$ 43.50 lbs / ft^2

Reduced Section :

Steel ==>	A	d	I_x	S_x	Z_x	A_w
	12.4	13.6	369.4	54.3	62.0	5.4

$M_{red} = (Z_{tr} / Z_x)\, M_{DL} + M_{LL} =$ 222.6 ft-kips

$< M_n / \Omega_b = Z_{tr}\, F_y / \Omega_b =$ 380.5 ft-kips, (AISC 360 I3.2a) [Satisfactory]

where $Z_{tr} =$ 152 in^3

Shear : $V_{max} =$ 31.19 kips $<$ $V_n / \Omega_v = 0.6F_y A_w C_v / \Omega_v =$ 95.679 kips, (AISC 360-10 I3.1b)

[Satisfactory]

where $2.24\,(E / F_y)^{0.5} =$ 53.946

$k_v =$ 5 (AISC 360-10 G2.1b) $C_v =$ 0.9915 (AISC 360-10 G2.1b)

$(k_v\, E / F_y)^{0.5} =$ 53.852 $\Omega_v =$ 1.67 (AISC 360-10 G1)

CHECK SHEAR CONNECTOR CAPACITY

$M_{max} =$ 289.3 ft-kips $<$ $M_n / \Omega_b = Z_x\, F_y / \Omega_b =$ 336.8 ft-kips **<== No Shear Stud Required**

where $\Omega_b =$ 1.67 (AISC 360-10 F1 & F2-1)

$C_f = MIN\,(0.85\, f_c' A_c\, ,\, F_y A_s) =$ 696.15 kips, (AISC 360-10 C-I3.1)

$S_{eff} = Min[\, M_{max} / (0.66\, F_y)\, ,\, S_{tr}\,] =$ 105 in^3, referred to steel bottom.

$V' = MAX\left[\left(\frac{S_{eff}-S_s}{S_{tr}-S_s}\right)^2, 0.25\right] C_f =$ 174.04 kips, (AISC 360-10 C-I3-5)

$Q_n = MIN\,[\, 0.5\, A_{sc}\, (f_c' E_c)^{0.5}\, ,\, R_g R_p A_{sc} F_u\,] =$ 18.06 kips, (AISC 360-10 I3.2d)

where $w_c =$ 115 pcf

$E_c = w_c^{\,1.5}\; 33\, (f_c')^{0.5} =$ 2229.1 ksi

$A_{sc} =$ 0.44 in^2

$F_u =$ 58 ksi

$R_g =$ 1.00 (AISC 360-10 Table I3.2b)

$R_p =$ 0.75 (AISC 360-10 Table I3.2b)

$\Sigma Q_n = Q_n\, N_r\, X_1 / s =$ 168.59 kips $<$ V' [Satisfactory]

CHECK INITIAL DEFLECTION / CAMBER AND STRESS ON NON-COMPOSITE

$DL =$ 75% *Self Weight* $I =$ 369.4 in^4, reduced section conservatively.

$w_{DL} =$ 41.60 lbs / ft $Z_x =$ 62.0 in^3, reduced section conservatively.

$P_{DL} =$ 6.78 kips @ 9.33" o.c.

$e =$ 0.036 $L =$ 28.00 ft

$\Delta_{Mid} = \frac{eP_{DL}L^3}{EI} + \frac{5w_{DL}L^4}{384EI} =$ 0.92 in, downward at middle of girder.

USE C = 0.92" AT MID GIRDER.

(cont'd)

M_{max} = 68.7 ft-kips < $M_n / \Omega_b = Z_x F_y / \Omega_b$ = 154.7 ft-kips [Satisfactory]

CHECK LIVE LOAD DEFLECTION ON COMPOSITE

P = 23.63 kips I_{tr} = 1402.6 in^4, reduced section conservatively.

$\Delta_{Mid} = \frac{e P_{LL} L^3}{E I_{tr}} =$ 0.79 in, downward at middle of girder.

< $L / 360$ = 0.93 in [Satisfactory]

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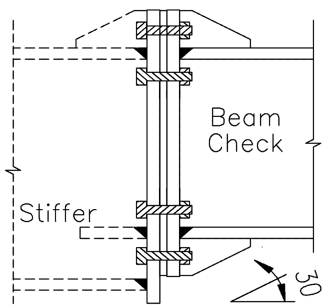
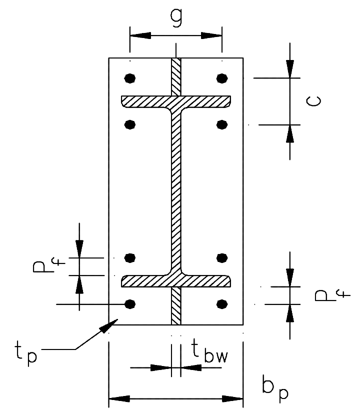
Endplate Splice Moment Connection Based on AISC 341-10, 358-10, 360-10 & FEMA-350

DESIGN CRITERIA

THE NON-SEISMIC MOMENT CONNECTION IS BASED ON BSEP-SMF BEAM TO COLUMN DESIGN METHOD, BUT WHICH HAS RELEASED BEAM & COLUMN SECTION LIMITS, BEAM-COLUMN RATIO REQUIREMENT, AND BENDING MOMENT AT THE COLUMN FACE FROM MEMBER CAPACITY TO ACTUAL BEAM END FORCE.





INPUT DATA & DESIGN SUMMARY

BOLTS	ϕ =	1 3/4	in
GRADES (A325 or A490)		A325	
PLATE & SHIM	t_p =	3/4	in

BEAM SECTION (Check End) => W18X50

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.7	18.0	0.36	7.50	0.57	88.9	800	7.38	1.65	101	0.97

BENDING MOMENT AT THE CONNECTION FACE	M_f =	550	ft-kips, SD level
SHEAR FORCE AT THE CONNECTION FACE	V_g =	60	kips, SD level
STRUCTURAL STEEL YIELD STRESS	F_y =	50	ksi

THE DESIGN IS ADEQUATE.

ANALYSIS

$g = \mathrm{Max}(b_{bf} - \phi\ ,\ t_w + 3\ \phi) =$ 6.00 in Where t_w = 1.00 in, *max web thinness for both sides*

$P_f = 1.5\ \phi =$ 2.00 in (AISC 358 Tab 6.1)

$c = 2\ P_f + t_{bf} =$ 4.57 in

$b_p = g + 3\ \phi =$ 11.25 in

CHECK BEAM LOCAL BUCKLING LIMITATIONS (AISC 341-10 Tab. D1.1)

$b_f / (2t_f) =$ 6.58 < $0.3\ (E_s / F_y)^{0.5} =$ 7.22 **[Satisfactory]**

Where E_s = 29000 ksi

$h / t_w =$ 45.23 < $2.45\ (E_s / F_y)^{0.5} =$ 59.00 **[Satisfactory]**

CHECK BENDING MOMENT AT THE CONNECTION FACE (FEMA Sec. 3.6.1.1.2)

M_f = input value for non-seismic

= 550 ft-kips < $2\ T_{ub}\ (d_0 + d_i) =$ 719 ft-kips **[Satisfactory]**

Where $d_0 = d_b + P_f - 0.5\ t_{bf} =$ 19.72 in T_b = 103 kips, (AISC 360-10, Tab. J3.1)

$d_i = d_0 - c =$ 15.15 in A_{bt} = 1.37 in^2 / bolt

$F_{fu} = M_f / (d_b - t_{bf}) =$ 378.66 kips

T_{ub} = 123.7 kips, (FEMA Sec. 3.6.1.1 & 3.6.2.1.2)

> $(0.00002305\ P_f^{0.591}\ F_{fu}^{2.583} / (t_p^{0.895}\ d_{bt}^{1.909}\ t_{bw}^{0.327}\ b_p^{0.965}) + T_b =$ 112.6 kips

[Satisfactory]

CHECK SHEAR CAPACITY AT THE CONNECTION FACE (FEMA Sec. 3.6.1.1.3)

A_b = 1.37 in^2 > $[2\ M_f / L + V_g] / 3F_v =$ 1.12 **[Satisfactory]**

Where V_g = input = 60.0 kips

L = 18 ft, *beam length between two endplates*

$F_v = \phi\ F_{nv} =$ 36 ksi, (AISC 360-10, Tab. J3.2)

CHECK END PLATE THICKNESS (AISC 358-10 Eq 6.10-13)

t_p = 0.75 in > $[1.11\ M_f / \phi_d\ F_{yp}\ Y_p)]^{0.5} =$ 0.40 in **[Satisfactory]**

Where Y_p = 1267 in, (AISC 358-10 Tab. 6.3 Case 1)

F_{yp} = 36 ksi ϕ_d = 1.0

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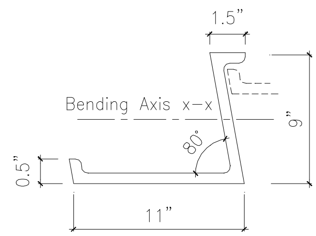
Flexure Capacity for Z-Profile Tread and Riser Based on AISC 360-10

INPUT DATA & DESIGN SUMMARY



SECTION	8 Gage (0.1644 in Thickness)		
STEEL YIELD STRESS	F_y =	50	ksi
BENDING UNBRACED LENGTH	L_{bx} =	18	ft, (AISC 360-10 F2.2.c)
MAXIMUM BENDING MOMENT	M_{rx} =	1.35	ft-kips, ASD

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK YIELDING CAPACITY

$M_x = M_n / \Omega_b =$ 22.77 / 1.67 = 13.64 ft-kips

> M_{bx} **[Satisfactory]**

Where $M_n = F_y\ S_{min} =$ 22.77 ft-kips, (AISC 360-10 F12)

$S_{min} =$ 5.46 in^3

CHECK LATERAL-TORSIONAL BUCKLING

$M_x = M_n / \Omega_b =$ 2.39 / 1.67 = 1.43 ft-kips

> M_{bx} **[Satisfactory]**

Where $M_n =$ 2.39 ft-kips, (AISC 360-10 F10.2)

$b =$ 9 in

$t =$ 0.1644 in

$E =$ 29000 ksi

$C_b =$ 1.00

$M_y = 0.8\ F_y\ S_{min} =$ 18.22 ft-kips, (AISC 360-10 F10.2)

$$M_e = \frac{0.66 E b^4 t C_b}{L_b^2}\left(\sqrt{1+0.78\left(\frac{tL_b}{b^2}\right)^2}-1\right) =$$ 2.67 ft-kips, (AISC 360-10 F10-6a)

CHECK LEG LOCAL BUCKLING

$M_x = M_n / \Omega_b =$ 2.50 / 1.67 = 1.50 ft-kips

> M_{bx} **[Satisfactory]**

Where $M_n = 0.8\ F_{cr}\ S_{min} =$ 2.50 ft-kips, (AISC 360-10 F10-8)

$F_{cr} = 0.71\ E / (b/t)^2 =$ 6.87 ksi, (AISC 360-10 F10.9)

Purchaser's LOGO

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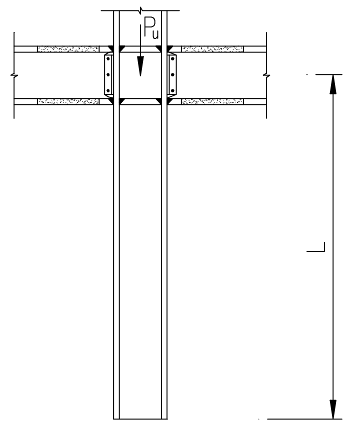
Strong-Column Weak-Beam Design Based on AISC 341-10 and AISC 360-10

DESIGN CRITERIA

1. The strong-column weak-beam (SC/WB) is basic concept in seismic design. But in steel design, the codes only provide the equation at plastic level (AISC 341-10 E3-1). The seismic steel column cannot be buckled for any forces from zero to plastic, before connected beams all plastic failed/hinged.
2. There are two buckling that have to be checked in SC/WB design: one is Lateral-Torsional Buckling (AISC 360-10 F2.2), another Flexural Buckling (AISC 360-10 E3).
3. If the column pinned at bottom, the L_b (AISC 360-10 F2.2) and kL (AISC 360-10 E3) are larger than the frame height (AISC 360-10 C).

INPUT DATA & DESIGN SUMMARY



COLUMN (Tube, Pipe, or WF) & SIZE **W14X120** < == **W Shape**

TOP COLUMN? (1 or 2) N_c = **1** , (bottom only)

COLUMN LENGTH kL = **21** ft L_b = **21** ft

(for fixed bottom kL = L_b = L, and for pinned kL = L_b =2L, suggested)

BEAM PLASTIC MODULUS $Z_{b,left}$ = **51** in^3 $Z_{b,right}$ = **51** in^3

(Z_b may be input zero for one side beam only)

STEEL YIELD STRESS F_y = **50** ksi

AXIAL LOAD ON THE COLUMN P_u = **200** kips, SD level

THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE COLUMN REQUIRED NOMINAL FLEXURAL STRENGTH (AISC 341-10 E3.4a)

$$M^*_{pc} \geq 1.1 R_y F_y \left(Z_{b,left} + Z_{b,right}\right) + \left(M_{uv,left} + M_{mv,right}\right) - F_y Z_{c,top} + \frac{P_u}{A_g} = \quad 514.72 \text{ ft-kips}$$

Where
R_y = 1.1 (AISC 341-10 Tab. A3.1)
$M_{uv,left}$ = 0 ft-kips
$M_{uv,right}$ = 0 ft-kips
$Z_{c,top}$ = 0 in^3
A_g = 35.3 in^2

CHECK BOTTOM COLUMN CAPACITY WITH COMBINED COMPRESSION AND BENDING (AISC 360-10, H1)

$$\frac{P_u}{\phi_c P_n}\left(1.5 - 0.5\frac{P_u}{\phi_c P_n}\right) + \left(\frac{M^*_{pc}}{C_b \phi_b M_n}\right)^2 = \quad 0.7373 \quad < \quad 1.0 \quad \textbf{[Satisfactory]}$$

Where
ϕ_c = 0.9 , (AISC 360-10 E1)
P_n = 1267.48 kips, (AISC 360-10 E3)
C_b = 1
ϕ_b = 0.9 , (AISC 360-10 F1)
M_n = 817.32 ft-kips, (AISC 360-10 F2)

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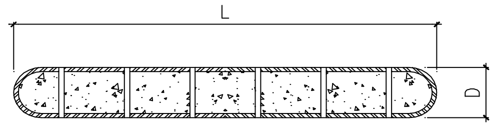
Composite Plate Shear Wall Design Based on AISC 341-16 & ACI 318-14 - Concrete Filled (C-PSW/CF)

DESIGN CRITERIA

1. The ACI 318-14 22.4 limited the maximum concrete compressive stress to ***0.85 f_c '***. But the AISC 341-16 H7.5 change it to ***1.0 fc'*** for boundary elements, and ***0.7 f_c '*** for non-boundary elements. This limitation governs the compression controlled flexural strength, so the software user may input the limitation value.
2. The C-PSW/CF benefits include that steel can provide confinement of the concrete, reinforcing bars are typically not necessary, concrete core delays local buckling of steel plate, and steel can act as formwork for accelerated construction.

INPUT DATA & DESIGN SUMMARY



HALF CIRCULAR WALL SECTION

WALL LENGTH	*L* =	**12**	*ft, (3.66 m)*
WALL THICKNESS	*D* =	**8.625**	*in, (219 mm).*
STEEL PLATE THICKNESS	t_s =	**0.375**	*in, (10 mm).*
STEEL YIELD STRESS	F_y =	**50**	*ksi, (345 MPa)*
CONCRETE STRENGTH	f_c ' =	**5**	*ksi, (34 MPa)*
THE MAX CONCRETE STRESS	β =	**0.85**	*f_c ' (0.85 on ACI, 1.0 on AISC)*

THE DESIGN IS ADEQUATE.

SECTION FORCES (LRFD)	M_u =	**5000**	*ft-kips, (6779 kN-m)*
	P_u =	**3500**	*kips, (15569 kN)*
	V_u =	**3000**	*kips, (13345 kN)*

TIE BAR	*dia* =	**0.875**	*in, (22 mm).*
	s =	**16**	*in. O.C., Horizontal & Vertical, (406 mm, spacing).*

ANALYSIS

CHECK STEEL PLATE LIMITATION (AISC 341-16 H7.4c)

D / t_s = 23.00 **<** $0.044 (E_s / F_y)$ = 25.52 **[Satisfactory]**

CHECK TIE BAR LIMITATION (AISC 341-16 H7.4)

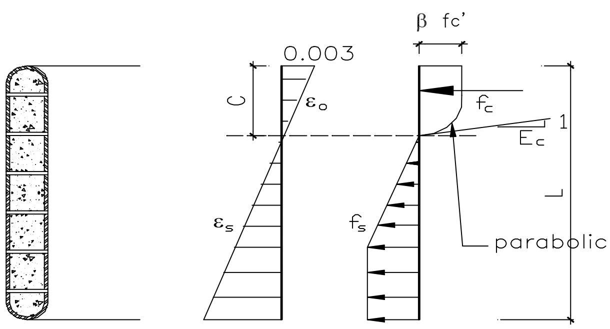
s = 16.00 **<** $1.8\, t_s (E_s / F_y)^{0.5}$ = 16.26 *in, (413 mm).* **[Satisfactory]**

$T_{req} = 2\, t_s^2\, F_y + 0.25\, t_s^2\, F_y\, [6 / (18\, (t_s / s)^2 + 1)]$ = 24.51 *kips, (109 kN)*

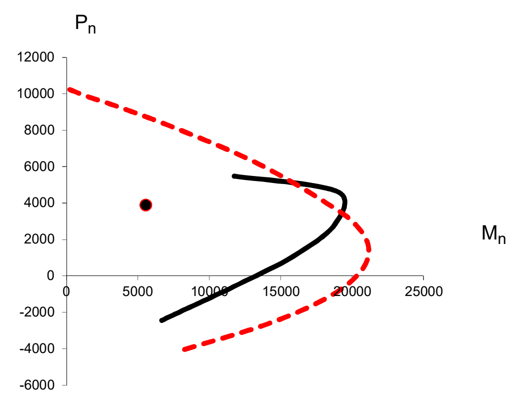
< $0.25\, \phi\, \pi\, dia^2\, F_y$ = 27.06 *kips, (120 kN)* **[Satisfactory]**

where ϕ = 0.9

CHECK FLEXURAL & AXIAL CAPACITY



STRAIN DIAGRAM FORCE DIAGRAM



P_u / ϕ = 3888.9 *kips, (17299 kN)*

M_{nc} @ P_u / ϕ = 17560.6 *ft-kips, (23809 kN-m)*

> M_u / ϕ = 5555.6 *ft-kips, (7532 kN-m)*

[Satisfactory]

Solid Line - Tension Controlled

Dash Line - Compression Controlled

CHECK SHEAR CAPACITY (AISC 341-16 H7.5b)

V_u / ϕ = 3333.3 *kips, (14827 kN)* **<** $\kappa\, A_{sw}\, F_y$ = 3870.9 *kips, (17219 kN)* **[Satisfactory]**

where A_{sw} = 101.5 in^2 (65504 mm^2)

κ = 0.763 , (AISC 341-16 H7-8)

Purchaser's LOGO

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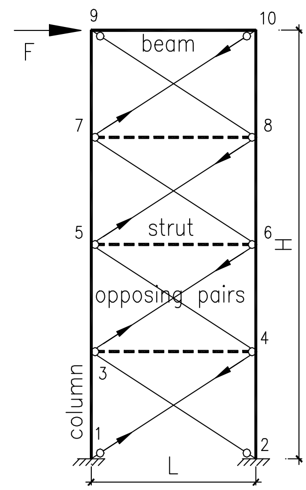
Multi-Tiered Braced Frame Design Based on AISC 341-16

DESIGN CRITERIA

1. Many Multi-Tiered Braced Frame (***MT-OCBF*** & ***MT-SCBF***) design are governed by allowable roof drift (ASCE 7 12.8.6), not by member capacities. At the allowable roof drift, the maximum roof seismic load, ***F*** (1.5 E for MT-OCBF and 1.0 E for MT-SCBF), which the frame can support, is determined/limited.
2. Assume conservatively tension-only bracing used, so no matter if brace buckling or not, how much second-order and geometric imperfection effects in the allowable roof drift, if the maximum roof seismic load, ***F***, less than actual, the frame member sizes have to be increased.

INPUT DATA



DIMENSION L = 15 *ft, (4.57 m)*
H = 45 *ft, (13.72 m)*

ROOF SEISNIC LOAD
F = 98 *kips, (436 kN), LRFD/SD Level*

Steel	*A*		*I_x (in plane)*	
Member	*(in^2)*	*(mm^2)*	*(in^4)*	*(mm^4)*
Column	19.1	12323	533	221851350
Top Beam	10.3	6645	510	212278027
Strut	11.7	7548		
Brace	5.85	3774		

MODULUS OF ELASTICITY
E = 29000 *ksi, (199948 MPa)*

ALLOWABLE ROOF DRIFT Δ_a = 0.02 *H, (ASCE 7 Table 12.12-1)*
C_d = 4 *, (ASCE 7 Tab 12.2-1)*
I = 1 *, (2015 IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)*

ANALYSIS & DESIGN SUMMARY

CHECK ROOF DRIFT $\Delta = \delta_x C_d / I =$ 6.31 *in, (160 mm)* < Δ_a **[Satisfactory]**

DESIGN INFORMATION & MAX SECTION FORCES

Steel	*Length*		*Axial P*		*M_x (in plane)*		*M_y (out-of plane)*	
Member	*(ft)*	*(m)*	*(kips)*	*(kN)*	*(ft-kips)*	*(kN-m)*	*(ft-kips)*	*(kN-m)*
Column	45	13.72	289.3	1287.0	47.5	64.4	39.1	53.0
Top Beam	15	4.57	94.4	420.1	31.8	43.1		
Strut	15	4.57	99.2	441.2				
Brace	18.75	5.72	-123.5	-549.3				

where

1. *Finite Element Method used.*
2. *Brace tension-only.*
3. *Column M_y based on AISC 341-16 F1.4c.g or F2.4e.3 .*
4. *The yellow cells may be input roof gravity loads (negative for downward).*

JOINT LOADS, REACTIONS, AND DISPLACEMENTS

Joint	*F_x / R_x (Horizontal)*		*F_y / R_y (Vertical)*		*R_M (Anticlockwise)*		*δ_x (Horizontal to Right)*		*δ_y (Vertical Up)*	
	(kips)	*(kN)*	*(kips)*	*(kN)*	*(ft-kips)*	*(kN-m)*	*(in)*	*(mm)*	*(in)*	*(mm)*
1	-94.10	-418.6	-284.12	-1263.8	61.74	83.7	0	0	0	0
2	-3.90	-17.3	289.32	1287.0	47.50	64.4	0	0	0	0
3							0.284	7	0.05	1
4							0.236	6	-0.07	-2
5							0.675	17	0.09	2
6							0.623	16	-0.12	-3
7							1.117	28	0.11	3
8							1.067	27	-0.16	-4
9	98.00	435.9	***-4.45658***	-19.8			***1.58***	40	0.11	3
10			***-4.45658***	-19.8			1.52	39	-0.18	-5

Purchaser's LOGO (Please email us your LOGO at purchase.)	PROJECT :		PAGE :
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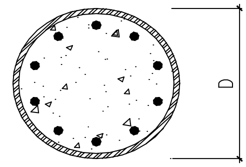
Filled Composite Column (FCC) Design for C-SMF/C-IMF/C-OCF Based on ASCE 7-16, AISC 341-16 & ACI 318-14

DESIGN CRITERIA

1. The Filled Composite Column (FCC), as seismic column, is good for strong column weak beam, bi-axial bending, and there are no confinement requirement of transverse reinforcement on ACI 318-14 18.7.
2. The FCC benefits also include that steel can provide confinement of the concrete, reinforcing bars are typically not necessary, concrete core delays local buckling of steel plate, and steel can act as formwork for accelerated construction.

INPUT DATA & DESIGN SUMMARY

COLUMN HEIGHT	H =	18	*ft, (5.49 m)*
COLUMN OUTSIDE DIAMETER	D =	40	*in, (1016 mm).*
STEEL PLATE THICKNESS	t_s =	1.125	*in, (29 mm).*
STEEL YIELD STRESS	F_y =	50	*ksi, (345 MPa)*
CONCRETE STRENGTH	f_c' =	5	*ksi, (34 MPa)*
THE MAX CONCRETE STRESS	β =	0.85	*f_c' (0.85 on ACI, 1.0 on AISC)*



FCC SECTION

THE DESIGN IS ADEQUATE.

SECTION FORCES	V_u =	5000	*kips, (22241 kN)*	
(Strength Level, LRFD)	P_u =	2000	*kips, (8896 kN)*	*<== For this P_u, the max ϕM_n =* 6439.9 *ft-kips, (8731 kN-m).*
	M_u =	2800	*ft-kips, (3796 kN-m)*	*<== For this M_u, the max $P_c = \phi P_n$ =* 7946.3 *kips, (35347 kN),*
				P_{re} / P_c = 0.252 *, (AISC 341-16 G3.4a)*

COLUMN VERTICAL REINFORCEMENT 8 # 8 Cover 0.75 *in, (19 mm) to steel plate.*

(Input zero if no vertical bars)

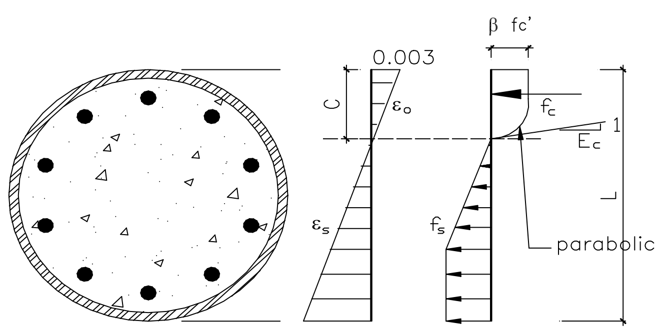
ANALYSIS

CHECK STEEL PLATE LIMITATION (AISC 341-16 TABLE D1.1)

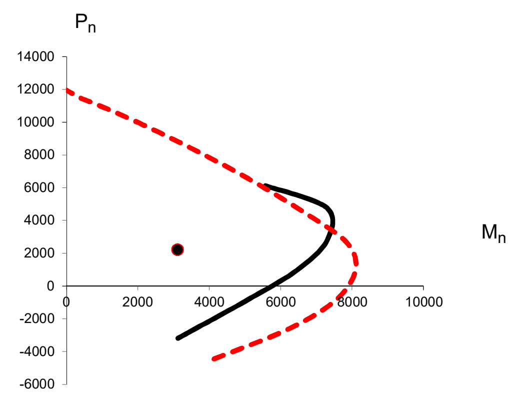
D / t_s = 35.56 **<** $0.085\ E_s / (R_y F_y)$ = 37.92 **[Satisfactory]**

where R_y = 1.3 , (AISC 341-16 TABLE A3.1)

CHECK FLEXURAL & AXIAL CAPACITY



STRAIN DIAGRAM FORCE DIAGRAM



P_u / ϕ = 2222.2 *kips, (9885 kN)*

M_{nc} @ P_u / ϕ = 7155.4 *ft-kips, (9701 kN-m)*

> M_u / ϕ = 3111.1 *ft-kips, (4218 kN-m)*

[Satisfactory]

where ϕ = 0.9 , (AISC 341-16 B3.2)

Solid Line - Tension Controlled

Dash Line - Compression Controlled

CHECK SHEAR CAPACITY (AISC 341-16 H7.5b)

V_u / ϕ = 5555.6 *kips, (24712 kN)* **<** $\kappa A_{stl} F_y$ = 6131.5 *kips, (27274 kN)*

where A_{stl} = 137.4 in^2 (88642 mm^2) **[Satisfactory]**

κ = 0.893 , (AISC 341-16 H7-8)

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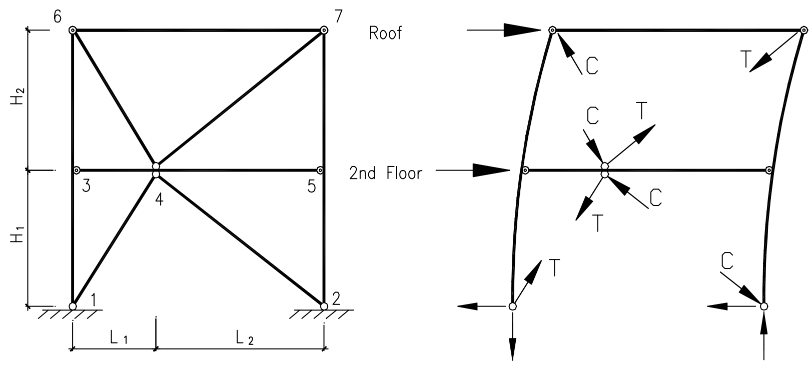
Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-16

DESIGN CRITERIA

1. Input story drifts, within ASCE 7-16 12.8.6, may change beam and column sizes of SCBF on Capacity-Limited Horizontal Seismic Load Effect, $\boldsymbol{E_{cl}}$. But input all zero story drifts, the Capacity-Limited Horizontal Seismic Loads can not directly apply to design of column and connection.
2. For non-symmetrical SCBF, both opposite of symmetrical cases have to be checked.



INPUT DATA & DESIGN SUMMARY

BRACE SECTION (Tube or Pipe)
HSS8X8X5/8

BEAM SECTION W14X109

COLUMN SECTION W14X159

DIMENSIONS

H_2 = 16 *ft, (4.9 m)*
H_1 = 18 *ft, (5.5 m)*
L_1 = 6.5 *ft, (2.0 m)*
L_2 = 8 *ft, (2.4 m)*

	Story Drift Input		$\Delta_a\ I / C_d$ *(Allowable Story Drift)*	
Roof	0.77	*in, (20 mm)*	0.77	*in, (20 mm)*
2nd Floor	0.86	*in, (22 mm)*	0.86	*in, (22 mm)*

POST-BUCKLING BRACE? (check both ***Yes*** and ***No*** required.) ***No*** , *(AISC 341-16 F2.3b)*

PARTIAL FIXED CONNECTION OF BEAM TO COLUMN ? ***30*** % $\boldsymbol{I_{BEAM}}$
(0 = Pinned , 100% = Fixed)

	Capacity-Limited Horizontal Seismic Loads		*Story Shear Force*	
Roof	**175.6**	*kips, (781.2 kN)*	**175.6**	*kips, (781.2 kN)*
2nd Floor	**-99.7**	*kips, (-443.6 kN)*	**75.9**	*kips, (337.6 kN)*

Reaction

$R_{1,X}$ =	**330.9**	*kips, (1471.9 kN)*
$R_{1,Y}$ =	**1548.5**	*kips, (6887.8 kN)*
$R_{2,X}$ =	**-255.0**	*kips, (-1134.3 kN)*
$R_{2,Y}$ =	**1548.5**	*kips, (6887.8 kN)*

THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE EXPECTED BRACE STRENGTH IN COMPRESSION AND IN TENSION (AISC 341-16 F2.3)

				A	$\boldsymbol{r_{min}}$	***t***	***b***	***h***
Brace	=>	HSS8X8X5/8	**Tube**	16.40	2.98	0.63	8.00	8.00

$D / t = 0.053\ E_s / (R_y\ F_y)$ = 25.70 , for Pipe
$h / t = 0.65\ (E_s / (R_y\ F_y))^{0.5}$ = 14.31 , for Tube
> Actual **[Satisfactory]**

$\boldsymbol{T} = R_yF_yA_g$ = ***980.72*** kips (Tension) Where R_y = **1.3** , (AISC 341-16 Table A3.1)
F_y = **46** ksi

$\boldsymbol{C} =$ Min($R_yF_yA_g$, $1/0.877\ F_{cre}A_g$) or $0.3\ F_{cre}$ E_s = 29000 ksi, $\lambda_c = (L_c / r)\ (R_y\ F_y / E)^{0.5}$

Brace	***Joint No.***	***Joint No.***	***L (ft)***	$\boldsymbol{\lambda_c}$	$\boldsymbol{F_e}$	$\boldsymbol{F_{cre}}$	***C*** *(kips)*
1	1	4	19.14	3.50	48.31	35.62	666.12
2	2	4	19.70	3.60	45.61	34.54	645.94
3	4	6	17.27	3.15	59.33	39.22	733.38
4	4	7	17.89	3.27	55.30	38.03	711.16

(cont'd)

DETERMINE JOINT DEFLECTIONS AND MEMBER FORCES BY FEM

Joint	X (ft)	Y (ft)	Δ_X (in)	Δ_Y (in)	F_X (k)	F_Y (k)
1	0	0	0	0	-331	-1549
2	14.5	0	0	0	255	1549
3	0	18	0.86	0.10	-100	0
4	6.5	18	0.87	-0.22	0	0
5	14.5	18	0.87	-0.15	0	0
6	0	34	1.63	0.20	176	0
7	14.5	34	1.55	-0.28	0	0

Member	L (ft)	P (k)	V (k)	M (ft-k)
1 - 4	19.14	-980.7	0	0
2 - 4	19.70	645.9	0	0
4 - 6	17.27	733.4	0	0
4 - 7	17.89	-980.7	0	0
3 - 4	***6.50***	***-96.4***	***60.0***	***326.7***
4 - 5	***8.00***	***-4.6***	***74.4***	***326.7***
6 - 7	***14.50***	***450.5***	***6.6***	***54.2***
1 - 3	18.00	-626.1	2.2	39.2
2 - 5	18.00	958.2	7.3	131.8
3 - 6	16.00	-686.1	1.1	41.7
5 - 7	16.00	883.8	12.0	137.0

CHECK BEAM CAPACITY (AISC 360-16 & AISC 341-16)

		A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
Beam	**W14X109**	32	14.3	0.53	14.60	0.86	173	1240	6.22	3.74	192	1.46

$b_f / (2t_f)$ =	8.49	<	8.79	**[Satisfactory]**	Where	R_y =	**1.2**	, (AISC 341-16 Table A3.1)	
h / t_w =	21.68	<	51.25	**[Satisfactory]**		F_y =	**50**	ksi	
P_u =	**450.5** kips	<	***1360*** kips	**[Satisfactory]**		L_x =	**14.5**	ft	
M_u =	**326.7** ft-kips	<	***720*** ft-kips	**[Satisfactory]**		L_y =	**8**	ft	
(P_u , M_u) =	0.73	<	1.00	**[Satisfactory]**		L_b =	**8**	ft	

CHECK COLUMN CAPACITY (AISC 360-16 & AISC 341-16)

		A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
COLUMN	**W14X159**	46.7	15.0	0.75	15.60	1.19	254	1900	6.38	4.00	287	1.79

$b_f / (2t_f)$ =	6.55	<	7.04	**[Satisfactory]**	Where	L_x =	**32**	ft
h / t_w =	15.33	<	43.03	**[Satisfactory]**		L_y =	**18**	ft
P_u =	**958.2** kips	<	***1612.3*** kips	**[Satisfactory]**		L_b =	**18**	ft
M_u =	**137.0** ft-kips	<	***1076.3*** ft-kips	**[Satisfactory]**				
(P_u , M_u) =	0.71	<	1.00	**[Satisfactory]**				

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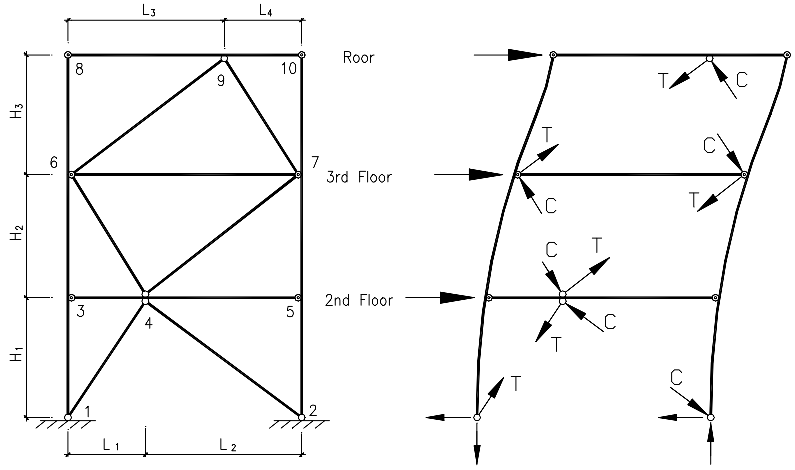
Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-16

DESIGN CRITERIA

1. Input story drifts, within ASCE 7-16 12.8.6, may change beam and column sizes of SCBF on Capacity-Limited Horizontal Seismic Load Effect, E_{cl} . But input all zero story drifts, the Capacity-Limited Horizontal Seismic Loads can not directly apply to design of column and connection.
2. For non-symmetrical SCBF, both opposite of symmetrical cases have to be checked.



INPUT DATA & DESIGN SUMMARY

BRACE SECTION (Tube or Pipe)
HSS8X8X5/8

BEAM SECTION **W14X211**

COLUMN SECTION **W14X283**

DIMENSIONS

				Story Drift Input		Δ_a I / C_d (Allowable Story Drift)	
H_3 =	14	ft, (4.3 m)					
H_2 =	16	ft, (4.9 m)					
H_1 =	18	ft, (5.5 m)					
L_1 =	6.5	ft, (2.0 m)					
L_2 =	8	ft, (2.4 m)	Roof	0.67	in, (17 mm)	0.67	in, (17 mm)
L_3 =	10	ft, (3.0 m)	3rd Floor	0.77	in, (20 mm)	0.77	in, (20 mm)
L_4 =	4.5	ft, (1.4 m)	2nd Floor	0.86	in, (22 mm)	0.86	in, (22 mm)

POST-BUCKLING BRACE? (check both ***Yes*** and ***No*** required.) ***Yes*** , (AISC 341-16 F2.3b)

PARTIAL FIXED CONNECTION OF BEAM TO COLUMN ? ***30*** % I_{BEAM}
(0 = Pinned , 100% = Fixed)

Capacity-Limited Horizontal Seismic Loads			*Story Shear Force*		*Reaction*		
					$R_{1,X}$ =	**335.4**	kips, (1491.8 kN)
Roof	**563.2**	kips, (2505.1 kN)	**563.2**	kips, (2505.1 kN)	$R_{1,Y}$ =	**1777.4**	kips, (7905.7 kN)
3rd Floor	**-172.7**	kips, (-768.0 kN)	**390.5**	kips, (1737.0 kN)	$R_{2,X}$ =	**-59.1**	kips, (-262.8 kN)
2nd Floor	**-114.2**	kips, (-508.0 kN)	**276.3**	kips, (1229.0 kN)	$R_{2,Y}$ =	**1777.4**	kips, (7905.7 kN)

THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE EXPECTED BRACE STRENGTH IN COMPRESSION AND IN TENSION (AISC 341-16 F2.3)

				A	***r_{min}***	***t***	***b***	***h***
Brace	= >	HSS8X8X5/8	**Tube**	16.40	2.98	0.63	8.00	8.00

$D / t = 0.053\ E_s / (R_y F_y)$ = 25.70 , for Pipe
$h / t = 0.65\ (E_s / (R_y F_y))^{0.5}$ = 14.31 , for Tube
> Actual **[Satisfactory]**

T = $R_y F_y A_g$ = ***980.72*** kips (Tension) Where R_y = 1.3 , (AISC 341-16 Table A3.1)
F_y = 46 ksi

C = $\text{Min}(R_y F_y A_g\ ,\ 1/0.877\ F_{cre} A_g)$ or $0.3\ F_{cre}$ E_s = 29000 ksi, $\lambda_c = (L_c / r) (R_y F_y / E)^{0.5}$

Brace	*Joint No.*	*Joint No.*	*L (ft)*	λ_c	F_e	F_{cre}	*C (kips)*
1	1	4	19.14	3.50	48.31	35.62	175.26
2	2	4	19.70	3.60	45.61	34.54	169.95
3	4	6	17.27	3.15	59.33	39.22	192.95
4	4	7	17.89	3.27	55.30	38.03	187.11
5	6	9	17.20	3.14	59.78	39.34	193.57
6	7	9	14.71	2.69	81.83	44.04	216.68

(cont'd)

DETERMINE JOINT DEFLECTIONS AND MEMBER FORCES BY FEM

Joint	X (ft)	Y (ft)	Δ_X (in)	Δ_Y (in)	F_X (k)	F_Y (k)
1	0	0	0	0	-335	-1777
2	14.5	0	0	0	59	1777
3	0	18	0.86	0.08	-114	0
4	6.5	18	0.87	-0.08	0	0
5	14.5	18	0.87	-0.15	0	0
6	0	34	1.63	0.14	-173	0
7	14.5	34	1.59	-0.27	0	0
8	0	48	2.30	0.13	563	0
9	10	48	2.26	-0.48	0	0
10	14.5	48	2.26	-0.30	0	0

Member	L (ft)	P (k)	V (k)	M (ft-k)
1 - 4	19.14	-980.7	0	0
2 - 4	19.70	169.9	0	0
4 - 6	17.27	193.0	0	0
4 - 7	17.89	-980.7	0	0
6 - 9	17.20	-980.7	0	0
7 - 9	14.71	216.7	0	0
3 - 4	***6.50***	***-97.0***	***4.1***	***200.9***
4 - 5	***8.00***	***4.9***	***64.6***	***315.9***
6 - 7	***14.50***	***427.1***	***28.4***	***249.0***
8 - 9	***10.00***	***586.6***	***146.2***	***1151.3***
9 - 10	***4.50***	***82.8***	***445.6***	***1151.3***
1 - 3	18.00	-854.9	2.3	41.3
2 - 5	18.00	1622.1	9.9	178.9
3 - 6	16.00	-859.0	19.5	179.6
5 - 7	16.00	1557.5	5.0	137.0
6 - 8	14.00	146.2	23.4	310.5
7 - 10	14.00	445.6	82.8	853.8

CHECK BEAM CAPACITY (AISC 360-16 & AISC 341-16)

		A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
Beam	**W14X211**	62	15.7	0.98	15.80	1.56	338	2660	6.55	4.08	390	2.16

b_f / $(2t_f)$ =	5.06	<	8.79	**[Satisfactory]**	Where	R_y =	**1.2**	, (AISC 341-16 Table A3.1)
h / t_w =	11.61	<	54.16	**[Satisfactory]**		F_y =	**50**	ksi
P_u =	**586.6** kips	<	***2618.7*** kips	**[Satisfactory]**		L_x =	**14.5**	ft
M_u =	**1151.3** ft-kips	<	***1462.5*** ft-kips	**[Satisfactory]**		L_y =	**10**	ft
(P_u , M_u) =	0.92	<	1.00	**[Satisfactory]**		L_b =	**10**	ft

CHECK COLUMN CAPACITY (AISC 360-16 & AISC 341-16)

		A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
COLUMN	**W14X283**	83.3	16.7	1.29	16.10	2.07	459	3840	6.79	4.16	542	2.67

b_f / $(2t_f)$ =	3.89	<	7.04	**[Satisfactory]**	Where	L_x =	**36**	ft
h / t_w =	8.81	<	43.48	**[Satisfactory]**		L_y =	**18**	ft
P_u =	**1622.1** kips	<	***2788.1*** kips	**[Satisfactory]**		L_b =	**18**	ft
M_u =	**853.8** ft-kips	<	***2032.5*** ft-kips	**[Satisfactory]**				
(P_u , M_u) =	0.96	<	1.00	**[Satisfactory]**				

Purchaser's LOGO	PROJECT :		PAGE :
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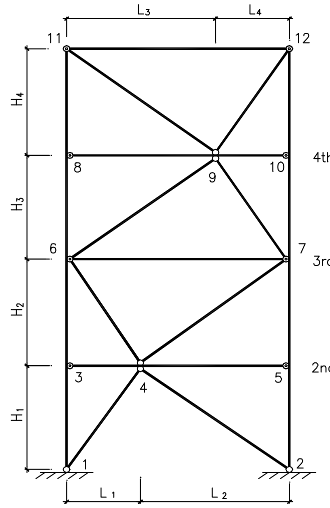
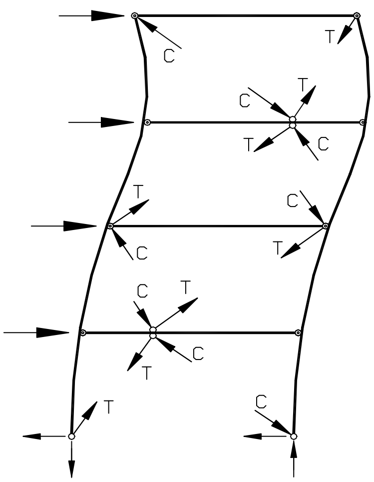
Plastic Mechanism Analysis, for Capacity-Limited Horizontal Seismic Load Effect, Based on AISC 341-16

DESIGN CRITERIA

1. Input story drifts, within ASCE 7-16 12.8.6, may change beam and column sizes of SCBF on Capacity-Limited Horizontal Seismic Load Effect, E_{cl}. But input all zero story drifts, the Capacity-Limited Horizontal Seismic Loads can not directly apply to design of column and connection.
2. For non-symmetrical SCBF, both opposite of symmetrical cases have to be checked.





INPUT DATA & DESIGN SUMMARY

BRACE SECTION (Tube or Pipe)
HSS8X8X5/8

BEAM SECTION **W14X145**

COLUMN SECTION **W14X283**

DIMENSIONS			
	H_4 =	16	ft, (4.9 m)
	H_3 =	14	ft, (4.3 m)
	H_2 =	16	ft, (4.9 m)
	H_1 =	18	ft, (5.5 m)
	L_1 =	6.5	ft, (2.0 m)
	L_2 =	8	ft, (2.4 m)
	L_3 =	10	ft, (3.0 m)
	L_4 =	4.5	ft, (1.4 m)

	Story Drift Input		Δ_a / C_d (Allowable Story Drift)	
Roof	0.77	in, (20 mm)	0.77	in, (20 mm)
4th Floor	0.67	in, (17 mm)	0.67	in, (17 mm)
3rd Floor	0.77	in, (20 mm)	0.77	in, (20 mm)
2nd Floor	0.86	in, (22 mm)	0.86	in, (22 mm)

POST-BUCKLING BRACE? (check both ***Yes*** and ***No*** required.) ***No*** , *(AISC 341-16 F2.3b)*

PARTIAL FIXED CONNECTION OF BEAM TO COLUMN ? ***30*** % I_{BEAM}
(0 = Pinned , 100% = Fixed)

	Capacity-Limited Horizontal Seismic Loads		*Story Shear Force*		*Reaction*		
Roof	**-120.4**	*kips, (-535.5 kN)*	**-120.4**	*kips, (-535.5 kN)*	$R_{1,X}$ =	**331.6**	*kips, (1475.0 kN)*
4th Floor	**431.6**	*kips, (1919.7 kN)*	**311.2**	*kips, (1384.2 kN)*	$R_{1,Y}$ =	**2980.3**	*kips, (13256.5 kN)*
3rd Floor	**-150.0**	*kips, (-667.1 kN)*	**161.2**	*kips, (717.1 kN)*	$R_{2,X}$ =	**-253.3**	*kips, (-1126.8 kN)*
2nd Floor	**-82.9**	*kips, (-368.9 kN)*	**78.3**	*kips, (348.2 kN)*	$R_{2,Y}$ =	**2980.3**	*kips, (13256.5 kN)*

THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE EXPECTED BRACE STRENGTH IN COMPRESSION AND IN TENSION (AISC 341-16 F2.3)

				A	r_{min}	*t*	*b*	*h*
Brace	= >	HSS8X8X5/8	**Tube**	16.40	2.98	0.63	8.00	8.00

$D / t = 0.053\ E_s / (R_y F_y)$ = 25.70 , for Pipe
$h / t = 0.65\ (E_s / (R_y F_y))^{0.5}$ = 14.31 , for Tube
> Actual **[Satisfactory]**

T = $R_y F_y A_g$ = ***980.72*** kips (Tension) Where R_y = **1.3** , (AISC 341-16 Table A3.1)
F_y = **46** ksi
C = Min($R_y F_y A_g$, 1/0.877 $F_{cre} A_g$) or 0.3 F_{cre} E_s = 29000 ksi, $\lambda_c = (L_c / r)(R_y F_y / E)^{0.5}$

Brace	*Joint No.*	*Joint No.*	*L (ft)*	λ_c	F_e	F_{cre}	*C (kips)*
1	1	4	19.14	3.50	48.31	35.62	666.12
2	2	4	19.70	3.60	45.61	34.54	645.94
3	4	6	17.27	3.15	59.33	39.22	733.38
4	4	7	17.89	3.27	55.30	38.03	711.16
5	6	9	17.20	3.14	59.78	39.34	735.71
6	7	9	14.71	2.69	81.83	44.04	823.57
7	9	11	18.87	3.45	49.70	36.14	675.85
8	9	12	16.62	3.04	64.05	40.46	756.56

(cont'd)

DETERMINE JOINT DEFLECTIONS AND MEMBER FORCES BY FEM

Joint	*X (ft)*	*Y (ft)*	*Δ_X (in)*	*Δ_Y (in)*	*F_X (k)*	*F_Y (k)*
1	0	0	0	0	-332	-2980
2	14.5	0	0	0	253	2980
3	0	18	0.86	0.18	-83	0
4	6.5	18	0.87	-0.14	0	0
5	14.5	18	0.87	-0.21	0	0
6	0	34	1.63	0.35	-150	0
7	14.5	34	1.53	-0.40	0	0
8	0	48	2.30	0.40	432	0
9	10	48	2.26	0.06	0	0
10	14.5	48	2.26	-0.44	0	0
11	0	64	3.07	0.44	-120	0
12	14.5	64	3.04	-0.52	0	0

Member	*L (ft)*	*P (k)*	*V (k)*	*M (ft-k)*
1 - 4	19.14	-980.7	0	0
2 - 4	19.70	645.9	0	0
4 - 6	17.27	733.4	0	0
4 - 7	17.89	-980.7	0	0
6 - 9	17.20	-980.7	0	0
7 - 9	14.71	823.6	0	0
9 - 11	18.87	675.9	0	0
9 - 12	16.62	-980.7	0	0
3 - 4	***6.50***	***-88.8***	***62.1***	***318.1***
4 - 5	***8.00***	***3.0***	***72.3***	***318.1***
6 - 7	***14.50***	***717.6***	***3.7***	***34.9***
8 - 9	***10.00***	***417.4***	***87.2***	***631.8***
9 - 10	***4.50***	***6.7***	***269.8***	***631.8***
11 - 12	***14.50***	***237.8***	***14.1***	***122.6***
1 - 3	18.00	-2057.9	1.5	26.8
2 - 5	18.00	2390.1	9.0	162.3
3 - 6	16.00	-2120.0	7.4	59.0
5 - 7	16.00	2317.7	6.0	98.2
6 - 8	14.00	-646.2	14.2	158.1
7 - 10	14.00	660.2	21.0	261.4
8 - 11	16.00	-559.0	0.0	82.3
10 - 12	16.00	930.0	27.7	320.8

CHECK BEAM CAPACITY (AISC 360-16 & AISC 341-16)

		A	***d***	***t_w***	***b_f***	***t_f***	***S_x***	***I_x***	***r_x***	***r_y***	***Z_x***	***k***
Beam	**W14X145**	42.7	14.8	0.68	15.50	1.09	232	1710	6.33	3.98	260	1.69

$b_f / (2t_f)$ =	7.11		<	8.79	**[Satisfactory]**	Where	R_y =	**1.2**	, (AISC 341-16 Table A3.1)
h / t_w =	16.79		<	49.53	**[Satisfactory]**		F_y =	**50**	ksi
P_u =	**717.6**	kips	<	***1798*** kips	**[Satisfactory]**		∠$_x$ =	**14.5**	ft
M_u =	**631.8**	ft-kips	<	***975*** ft-kips	**[Satisfactory]**		∠$_y$ =	**10**	ft
(P_u , M_u) =	0.98		<	1.00	**[Satisfactory]**		∠$_b$ =	**10**	ft

CHECK COLUMN CAPACITY (AISC 360-16 & AISC 341-16)

		A	***d***	***t_w***	***b_f***	***t_f***	***S_x***	***I_x***	***r_x***	***r_y***	***Z_x***	***k***
COLUMN	**W14X283**	83.3	16.7	1.29	16.10	2.07	459	3840	6.79	4.16	542	2.67

$b_f / (2t_f)$ =	3.89		<	7.04	**[Satisfactory]**	Where	∠$_x$ =	**36**	ft
h / t_w =	8.81		<	39.51	**[Satisfactory]**		∠$_y$ =	**18**	ft
P_u =	**2390.1**	kips	<	***2788.1*** kips	**[Satisfactory]**		∠$_b$ =	**18**	ft
M_u =	**320.8**	ft-kips	<	***2032.5*** ft-kips	**[Satisfactory]**				
(P_u , M_u) =	1.00		<	1.00	**[Satisfactory]**				

Purchaser's LOGO

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Double-Tee Connection Design for SMF Based on AISC 341-10/16, 358-16, 360-10/16

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION => **W12X120**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
35.3	13.1	0.71	12.30	1.11	163	1070	5.51	3.13	186	1.70

BEAM SECTION => **W18X50**

A	d	t_w	b_f	t_f	S_x	I_x	r_x	r_y	Z_x	k
14.7	18.0	0.36	7.50	0.57	88.9	800	7.38	1.65	101	0.97

T-STUBS

d_{max}	t_{st}	b_{ft}	t_{ft}	$t_{st, eff}$	k
17.36	1.5	11.2	2.5	1.39	1.34

STRUCTURAL STEEL YIELD STRESS F_y = **50** ksi

THE FACTOR GRAVITY LOAD ON THE BEAM w_u = **1.2** klf

THE FACTOR AXIAL LOAD ON THE COLUMN P_u = **600** kips

BEAM LENGTH BETWEEN COL. CENTERS L_c = **20** ft

AVERAGE STORY HEIGHT OF ABOVE & BELOW h = **12** ft

THE SMF DESIGN IS ADEQUATE.

(Continuity column stiffeners 5/8 x 6 with 7/16" fillet weld to web & CP to flanges. A doubler plate is required with thickness of 1/8 in.)

BOLTS ϕ_{tb} = **1** in, (T-Stub to Beam)

ϕ_{tc} = **1 5/8** in, (T-Stub to Column)

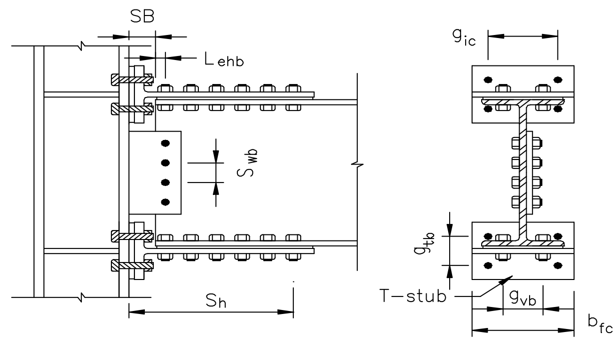
GRADES (A325 or A490) **A490**

END PLATE SHIM t_{shim} = **1/4** in

`

NUMBER COLUMNS N_c = **2** (Top & Bot)

NUMBER BEAM N_b = **1** (One Side Only)



ANALYSIS

DETERMINE BOLT NUMBERS AND DIMENSIONS

g_{ic} = 6.51 in

4 Tension Bolts. (AISC 358-16 Fig 13.3)

L_{sp} = 8.00 in t_{sp} = 0.36 in, Min.

S_{wb} = 3.00 in W_T = 7.50 in

Max 1 Row Web Bolt Numbers = 2

g_{vb} = 3.93 in g_{tb} = 6.35 in

Max 2 Row Flange Bolt Numbers = 10 S_h = 15.76 in

CHECK BEAM & T-STUB LOCAL BUCKLING LIMITATIONS (AISC 358-16 13.3.1 & AISC 341-10/16 Tab. D1.1)

$b_f / (2t_f)$ = 6.58 < $0.32\,[E_s / (R_y F_y)]^{0.5}$ = 7.04 **[Satisfactory]**

Where E_s = 29000 ksi, R_y = **1.2** , (AISC 341-16 Table A3.1)

h / t_w = 45.23 < $2.57\,[E_s / (R_y F_y)]^{0.5}$ = 56.50 **[Satisfactory]**

W18 < W24 **[Satisfactory]**

W_t = 50.02 lbs/ft < 55 lbs/ft **[Satisfactory]**

t_f < 5/8 in **[Satisfactory]**

L_f / d = 12.61 > 9 **[Satisfactory]**

g_{tb} / t_{ft} = < 7.0 **[Satisfactory]**

CHECK COLUMN LOCAL BUCKING LIMITATIONS (AISC 358-16 13.3.1 & AISC 341-10/16 Tab. D1.1)

W12 < **W14** **[Satisfactory]**

$b_f / (2t_f)$ = 5.54 < $0.32\,[E_s / (R_y F_y)]^{0.5}$ = 7.04 **[Satisfactory]**

h / t_w = 13.66 <

$3.96[E_s/(R_yF_y)]^{0.5}(1\text{-}3.04P_u/\phi_bP_y)$ = N/A , for $P_u/\phi_bP_y \le 0.114$

$1.29[E_s/(R_yF_y)]^{0.5}(2.12\text{-}P_u/\phi_bP_y)$ = 49.41 , for $P_u/\phi_bP_y > 0.114$

[Satisfactory] Where ϕ_c = 0.9 , $P_y = F_yA$ = 1765 kips

CHECK BEAM - COLUMN RATIO REQUIREMENT (AISC 341-10/16 E3.4a)

$\Sigma M_{pc}^* / (\Sigma M_{pb}^*)$ = 1.46 > 1.00 **[Satisfactory]**

Where $\Sigma M_{pc}^* = N_c Z_c (F_{yc} - P_u / A_g)$ = 1023 ft-kips

$\Sigma M_{pb}^* = N_b (M_{hinge} + M_v)$ = 701 ft-kips, at center of column

$M_v = V_{hinge} S_{hc} = [2M_{hinge} /(L\text{-}2S_{hc}) + w_u(L\text{-}2S_h)/2]\, S_{hc}$ = 145 ft-kips

$M_{pr} = M_{hinge} = C_{pr}R_yF_{yb}Z_b$ = 556 ft-kips

R_y = 1.2 (AISC 341 Tab. A3.1) $S_{hc} = S_h + d_c$ = 22.31 in

C_{pr} = 1.1 (FEMA Sec. 3.5.5.1)

CHECK SHEAR BOLT REQUIREMENT (AISC 358-16 13.6)

$1.25\, M_{pr} /(d_b\, \phi\, r_{nv})$ = 5.8 bolts req'D < 10 bolts **[Satisfactory]**

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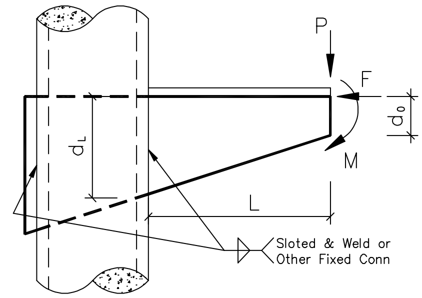
Steel Corbel Design Based on AISC-ASD 9th, Appendix F

DESIGN CRITERIA

1. IN ORDER TO QUALIFY UNDER THIS DESIGN, THE TOP & BOT FLANGES SHALL INPUT ZERO (APP. F7.1.b, page 5-102), ALTHOUGH TOP FLANGE ALWAYS EXISTS.
2. FOR RECTANGLE GUSSET PLATE, INPUT d_0 EQUAL TO d_L .
3. THE END FORCES, P, F & M, SHOULD INCLUDE IMPACT FACTOR, IF THEY ARE FROM DYNAMIC LOADS. (A4.2, page 5-29)



INPUT DATA & DESIGN SUMMARY

CORBEL DIMENSIONS

b_f =	1.25	in (32 mm)
t_f =	1.25	in (32 mm)
t_w =	**1.25**	in (32 mm)
d_L =	**22**	in (559 mm)
d_0 =	**12**	in (305 mm)
L =	**3**	ft (0.9 m)

STEEL YIELD STRESS F_y = **50** ksi (345 N / mm^2)

THE CORBEL DESIGN IS ADEQUATE.

LOADS (ASD)

F =	**16.0**	kips (71.2 kN)
P =	**80.0**	kips (355.8 kN)
M =	**30.0**	ft-kips (133.4 kN-m)

ANALYSIS

DETERMINE ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma}=\begin{cases}\frac{2}{3}\left[1.0-\frac{F_y}{6B\sqrt{F_{s\gamma}^2+F_{w\gamma}^2}}\right]F_y\le 0.60F_y \ , \ for \ \ F_{b\gamma}>F_y/3\\ B\sqrt{F_{s\gamma}^2+F_{w\gamma}^2} \ , \ for \ \ F_{b\gamma}\le F_y/3\end{cases}$$

where $A_f = t_f\, b_f$

$A_{To} = t_f\, b_f + d_0\, t_w / 6$

γ = MIN[(d_L - d_0) / d_0 , 0.268 L/d_0, 6.0]

I_{To} = ($t_f\, b_f^3$ + $d_0\, t_w^3$ / 6) / 12

$$r_{To}=\sqrt{\frac{I_{To}}{A_{To}}}$$

$$F_{s\gamma}=\frac{12000}{h_s L d_o/A_f}$$

$$h_s=1.0+0.0230\gamma\sqrt{\frac{Ld_0}{A_f}}$$

$$F_{w\gamma}=\frac{170000}{\left(h_w L/r_{To}\right)^2}$$

$$h_w=1.0+0.00385\gamma\sqrt{\frac{L}{r_{To}}}$$

$$B=\frac{1.75}{1.0+0.25\sqrt{\gamma}}$$

	Length	γ	A_{To}	I_{To}	r_{To}	H_s	H_w	$F_{s\gamma}$	$F_{w\gamma}$	B	$F_{b\gamma}$
Corbel	3.029	0.80	4.06	1	0.36	1.31	1.03	33.20	16.0707	1.43	**28.06**

DETERMINE ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v=\begin{cases}0.4F_y \ , \ for \ \ h/t_w\le 380\sqrt{F_y}\\ \frac{C_v F_y}{2.89}\le 0.4F_y \ , \ for \ \ h/t_w>380\sqrt{F_y}\end{cases}$$

where $h = d_L$ - 2 t_f

$$k_v=\begin{cases}4.0+\frac{5.34}{(a/h)^2} \ , \ for \ \ a/h\le 1.0\\ 5.34+\frac{4.0}{(a/h)^2} \ , \ for \ \ a/h>1.0\end{cases}$$

$$C_v=\begin{cases}\frac{45000k_v}{F_y(h/t_w)^2} \ , \ for \ \ C_v\le 0.8\\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \ , \ for \ \ C_v>0.8\end{cases}$$

	a	h	h/t_w	$380/F_y^{0.5}$	K_v	C_v	$F_{b\gamma}$
Corbel	3.0	19.50	16	54	6.51	4.40	**20.00**

DETERMINE ALLOWABLE COMPRESSIVE STRESS (APP. F7.3, pg 5-102)

$$F_{a\gamma}=\begin{cases}\frac{\left(1.0-\frac{S^2}{2C_c^2}\right)F_y}{\frac{5}{3}+\frac{3S}{8C_c}-\frac{S^3}{8C_c^3}} \ , \ for \ \ S\le C_c\\ \frac{12\pi^2E}{23S^2} \ , \ for \ \ S>C_c\end{cases}$$

where K_γ = (effective length factor by an analysis)

$S = K_\gamma\, l / r_{ox}$

$$C_c=\sqrt{\frac{2\pi^2E}{F_y}}$$

E = 29000 ksi

	l	K_γ	C_c	I_x	A	r_{ox}	S	$F_{a\gamma}$
Corbel	3.0	2	107	236.25	18.13	3.61	19.94	**28.31**

CHECK EACH SECTION CAPACITIES

	d_0	d_L	t_f	b_f	t_w	A_f
Corbel	12	22	1.25	1.25	1.25	1.5625

(cont'd)

Section	Fixed	1/3 L	2/3 L	End
d (in)	22	19	15	12
I (in^4)	1487.29	949.75	559.21	292.50
A_w (in^2)	27.50	23.33	19.17	15.00
N (kips)	16.0	16.0	16.0	16.0
V (kips)	80.0	80.0	80.0	80.0
M (ft-k)	270	190	110	30
f_a (ksi)	0.52	0.60	0.72	0.88
F_a (ksi)	28.31	28.31	28.31	28.31
f_v (ksi)	2.91	3.43	4.17	5.33
F_v (ksi)	20.00	20.00	20.00	20.00
f_b (ksi)	24.0	22.4	18.1	7.4
F_b (ksi)	28.06	28.06	28.06	28.06

$f_a < F_a$ **[Satisfactory]** $f_v < F_v$ **[Satisfactory]** $f_b < F_b$ **[Satisfactory]**

CHECK COMBINED FLEXURE AND AXIAL FORCE (APP. F7.4, pg 5-104)

$$\left(f_a\ ,\ f_b\right)=\begin{cases}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{f_{bl}}{F_{b\gamma}}\ ,\ for\ \dfrac{f_{a0}}{F_{a\gamma}}\le 0.15\\ Larger\ of\ \begin{pmatrix}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{C_m' f_{bl}}{\left(1-\dfrac{f_{a0}}{F_{e\gamma}'}\right)F_{b\gamma}}\\ \dfrac{f_{a0}}{0.6F_y}+\dfrac{f_{bl}}{F_{b\gamma}}\end{pmatrix}\ ,\ for\ \dfrac{f_{a0}}{F_{a\gamma}}>0.15\end{cases}\le 1.3$$

	f_{a0}	f_{bl}	$F_{a\gamma}$	$F_{b\gamma}$	$F_{e\gamma}'$	C_m'	(f_a, f_b)
Column	0.52	23.96	28.31	28.06	375.47	1.00	**0.87**

[Satisfactory]

Technical Reference:
1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. AISC: "Design Guide 25: Frame Design Using Web-Tapered Members, 2010.

(Cont'd)

$d_{vb} = Z_x / [2\ t_{fb}\ (d_b - t_{fb})]\ [1 - R_y\ F_{yb} / (R_t\ F_{ub})] - 0.125 =$ 1.05 in, (AISC 358-16 13.6-3)

Where R_t = **1.2** , (AISC 341-16 Table A3.1) > ϕ_{tb} **[Satisfactory]**

F_{ub} = **65** ksi

$\phi\ r_{nv}$ = 80.03 kips / bolt, (AISC 358-16 13.6-4)

F_{nv} = **75** ksi

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.2)

$M_f = M_{hinge} + [2M_{hinge} / (L - 2S_{hc}) + w_u(L - 2S_{hc})/2]\ (S_{hc} - d_c /2)$

= 658 ft-kips < $2\ T_{ub}\ (d_0 + d_i) =$ 1256 ft-kips **[Satisfactory]**

Where $d_0 = d_b + P_f - 0.5\ t_{st} =$ 19.25 in T_b = 148 kips, (AISC 360 Tab. J3.1)

$d_i = d_0 - g_{tb} =$ 12.90 in A_{bt} = 2.07 in^2 / bolt

$F_{fu} = M_f / (d_b - t_{bf}) =$ 452.97 kips $P_f = 1.5\ \phi_{tc} =$ 2.00 in (AISC 358 Tab 6.1)

T_{ub} = 234.4 kips, (FEMA Sec. 3.6.1.1 & 3.6.2.1.2)

> $(0.00002305\ P_f^{0.591}\ F_{fu}^{2.583} / (t_p^{0.895}\ d_{bt}^{1.909}\ t_{bw}^{0.327}\ b_p^{0.965}) + T_b =$ 153.0 kips

[Satisfactory]

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.3)

A_b = 2.07 in^2 > $[2\ M_f / (L - d_c) + V_g\ ] / 3F_v =$ 0.48 **[Satisfactory]**

Where $V_g = w_u\ (L - d_c) / 2 =$ 11.3 kips

$F_v = \phi\ F_{nv} =$ 56.25 ksi, (AISC 360, Tab. J3.2)

CHECK SHEAR CAPACITY AT THE BEAM WEB

$1.25\ M_{pr} / (0.5\ L_h\ \phi\ r_{nv}) =$ 1.5 bolts req'D < 2 bolts **[Satisfactory]**

Where $\phi\ r_{nv}$ = 49.84 kips / bolt

CHECK T-STEM LIMITATIONS (AISC 358-16 13.6)

Max[(13.6-13) , (13.6-14) , (13.6-15)] = 1.4 in < t_{st} **[Satisfactory]**

Where F_{pr} = 417.7 kips, (AISC 358-16 13.6-11)

L_{vb} = 2.9 in

W_{Whit} = 7.3 in, (AISC 358-16 13.6-12)

S_1 = 12.8 in

CHECK T-STEM LIMITATIONS (AISC 358-16 13.6)

$[4\ \phi\ r_{nt}\ b' / (\phi_d\ F_{yt}\ p)]^{0.5} =$ 2.2 in < t_{ft} **[Satisfactory]**

Where p = 3.8 in, (AISC 358-16 13.6-22)

b' = 2.5 in, (AISC 358-16 13.6-24)

$\phi\ r_{nt}$ = 121.33 kips / bolt

CHECK END PLATE THICKNESS (AISC 358-10/16 Eq 6.10-13/Eq 6.8-13)

t_p = 2.750 in > $[1.11\ M_f / \phi_d\ F_{yp}\ Y_p)]^{0.5} =$ 0.42 in **[Satisfactory]**

Where Y_p = 1355 in, (AISC 358 Tab. 6.4 Case 1)

F_{yp} = 36 ksi ϕ_d = 1.0

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-10/16 Eq 6.10-13/Eq 6.8-13, FEMA Sec 3.3.3.1)

$t_{cf,\ reqD} = [1.11\ M_f / \phi_d\ F_{yc}\ Y_c)]^{0.5} =$ 0.33 in < $t_{cf,\ actual}$

Where Y_c = 1575 in, (AISC 358 Tab. 6.5 Stiffened)

$t_{cw,\ reqD} = M_f / [(d_b - t_{bf})(6\ k_c + 2\ t_p + t_{bf})\ F_{yc}] =$ 0.56 in < $t_{cw,\ actual}$

(The continuity plates may not be required.)

$t_{st} = t_{bf}$ for interior connection, or $(t_{bf} /2)$ for exterior connection = 0.57 in, USE 0.63 in, (5/8 in)

b_{st} = 6 in < $0.56\ (E / F_{yst})^{0.5}\ t_{st} =$ 9.93 in, (AISC 358-10/16 Eq 6.10-10/Eq 6.8-10)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr}\ A =$ 651.0 kips

Where ϕ_c = 0.9 ,(AISC 360 E1) $h_{st} = d_c - 2k_c =$ 9.7 in

$K = L_c/L =$ 0.75 $K\ h_{st} / r_{st} < 200$ (AISC 360 E2) **[Satisfactory]**

$I = t_{st}\ (2b_{st} + t_{wc})^3 / 12 =$ 107 in^4 F_e = 28768 ksi (AISC 360 E3)

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 20 in^2 F_{cr} = 35.98 ksi (AISC 360 E3)

$r_{st} = (I / A)^{0.5} =$ 2.31 in F_{yst} = 36 kips, plate yield stress

$P_{u,st} = R_{yb}\ F_{yb}\ b_{fb}\ t_{fb} =$ 256.5 kips < $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 J2.2b)

w = 7/16 in > w_{MIN} = 0.25 in

< w_{MAX} = 0.5625 in **[Satisfactory]**

The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6 t_{st} L_{nst} F_y / [(2)\ \phi\ F_w\ (0.707\ w)]$ = (0.625 x 6.7) x 36 / [(2) 0.75 (0.6x70)(0.707x7/16)] = 3.53 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 6.7 < $2(L_{net} - 0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)

(Cont'd)

$d_{vb} = Z_x / [2\ t_{fb}\ (d_b - t_{fb})]\ [1 - R_y\ F_{yb} / (R_t\ F_{ub})] - 0.125 =$ 1.05 in, (AISC 358-16 13.6-3) **>** ϕ_{tb} **[Satisfactory]**

Where R_t = 1.2 , (AISC 341-16 Table A3.1)

F_{ub} = 65 ksi

$\phi\, r_{nv}$ = 73.71 kips / bolt, (AISC 358-16 13.6-4)

F_{nv} = 75 ksi

CHECK BENDING MOMENT AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.2)

$M_f = M_{hinge} + [2M_{hinge} / (L - 2S_{hc}) + w_u(L - 2S_{hc})/2]\ (S_{hc} - d_c /2)$

= 687 ft-kips **<** $2\ T_{ub}\ (d_0 + d_i) =$ 1426 ft-kips **[Satisfactory]**

Where $d_0 = d_b + P_f - 0.5\ t_{st} =$ 19.84 in T_b = 148 kips, (AISC 360 Tab. J3.1)

$d_i = d_0 - g_{tb} =$ 16.68 in A_{bt} = 2.07 in^2 / bolt

$F_{fu} = M_f / (d_b - t_{bf}) =$ 472.64 kips $P_f = 1.5\ \phi_{tc} =$ 2.00 in (AISC 358 Tab 6.1)

T_{ub} = 234.4 kips, (FEMA Sec. 3.6.1.1 & 3.6.2.1.2)

> $(0.00002305\ P_f^{0.591}\ F_{fu}^{2.583} / (t_p^{0.895}\ d_{bt}^{1.909}\ t_{bw}^{0.327}\ b_p^{0.965}) + T_b =$ 165.4 kips

[Satisfactory]

CHECK SHEAR CAPACITY AT THE COLUMN FACE (FEMA Sec. 3.6.1.1.3)

A_b = 2.07 in^2 **>** $[2\ M_f / (L - d_c) + V_g\ ] / 3F_v =$ 0.67 **[Satisfactory]**

Where $V_g = w_u\ (L - d_c) / 2 =$ 39.7 kips

$F_v = \phi\ F_{nv} =$ 56.25 ksi, (AISC 360, Tab. J3.2)

CHECK SHEAR CAPACITY AT THE BEAM WEB

$1.25\ M_{pr} / (0.5\ L_h\ \phi\ r_{nv}) =$ 1.5 bolts req'D **<** 4 bolts **[Satisfactory]**

$\phi\, r_{nv}$ = 49.84 kips / bolt

CHECK END PLATE THICKNESS (AISC 358-10/16 Eq 6.10-13/Eq 6.8-13)

t_p = 0.775 in **>** $[1.11\ M_f / \phi_d\ F_{yp}\ Y_p)]^{0.5} =$ 0.43 in **[Satisfactory]**

Where Y_p = 1355 in, (AISC 358 Tab. 6.4 Case 1)

F_{yp} = 36 ksi ϕ_d = 1.0

CHECK CONTINUITY PLATE REQUIREMENT (AISC 358-10/16 Eq 6.10-13/Eq 6.8-13, FEMA Sec 3.3.3.1)

$t_{cf,\ reqD} = [1.11\ M_f / \phi_d\ F_{yc}\ Y_c)]^{0.5} =$ 0.34 in **<** $t_{cf,\ actual}$

Where Y_c = 1575 in, (AISC 358 Tab. 6.5 Stiffened)

$t_{cw,\ reqD} = M_f\ / [(d_b - t_{bf})(6\ k_c + 2\ t_p + t_{bf})\ F_{yc}] =$ 0.77 in **>** $t_{cw,\ actual}$

(The continuity plates required.)

$t_{st} = t_{bf}$ for interior connection, or $(t_{bf}/2)$ for exterior connection = 0.57 in, USE 0.63 in, (5/8 in)

b_{st} = 6 in **<** $0.56\ (E / F_{yst})^{0.5}\ t_{st} =$ 9.93 in, (AISC 358-10/16 Eq 6.10-10/Eq 6.8-10)

[Satisfactory]

$\phi_c P_{n,st} = \phi_c F_{cr}\ A =$ 651.0 kips

Where ϕ_c = 0.9 ,(AISC 360 E1) $h_{st} = d_c - 2k_c =$ 9.7 in

$K = L_c/L =$ 0.75 $K\ h_{st} / r_{st}$ < 200 (AISC 360 E2) **[Satisfactory]**

$I = t_{st}\ (2b_{st} + t_{wc})^3 / 12 =$ 107 in^4 F_e = 28768 ksi (AISC 360 E3)

$A = 2b_{st}t_{st} + 25(t_{wc})^2 =$ 20 in^2 F_{cr} = 35.98 ksi (AISC 360 E3)

$r_{st} = (I / A)^{0.5} =$ 2.31 in F_{yst} = 36 kips, plate yield stress

$P_{u,st} = R_{yb}\ F_{yb}\ b_{fb}\ t_{fb} =$ 256.5 kips **<** $\phi_c P_{n,st}$ **[Satisfactory]**

The best fillet weld size (AISC 360 J2.2b)

w = 7/16 in > w_{MIN} = 0.25 in

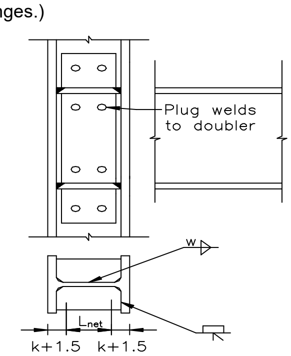
< w_{MAX} = 0.5625 in **[Satisfactory]**

The required weld length between A36 continuity plates and column web (FEMA Fig 3-6)

$L_w = 0.6 t_{st} L_{nst} F_y / [(2)\ \phi\ F_w\ (0.707\ w)]$ = (0.625 x 6.7) x 36 / [(2) 0.75 (0.6x70)(0.707x7/16)] = 3.53 in

Where $L_{net} = d_c - 2(k_c + 1.5) =$ 6.7 **<** $2(L_{net} - 0.5)$ **[Satisfactory]**

(Use complete joint penetration groove welds between continuity plates & column flanges.)



CHECK PANEL ZONE THICKNESS REQUIREMENT (AISC 341 E3.6e & FEMA Sec. 3.3.3.2)

$t_{ReqD} = MAX\ (t_1, t_2) =$ 0.84 in

$t_1 = C_y\ M_c\ (h - d_b) / [0.9\ (0.6)\ F_{yc}\ R_{yc}\ d_c\ (d_b - t_{fb})\ h] =$ 0.84 in

Where $C_y = S_b / (C_{pr}\ Z_{hing}) =$ 0.80

$S_b = 2I_b / d_b =$ 89 in^2

$I_b = I_x =$ 800 in^4

$M_c = \Sigma M_{pb}^* =$ 742 ft-kips

$t_2 = (d_z + w_z) / 90 = (d_b - 2t_{st} + d_c - 2k_c) / 90 =$ 0.29 in

Since t_{wc} = 0.71 in **<** t_{ReqD} , a doubler plate is required with thickness of 3/16 in.

Technical References:

FEMA 350: "Recommended Seismic Design Criteria for New Steel Moment-frame Buildings.", SAC Joint Venture, 2000.

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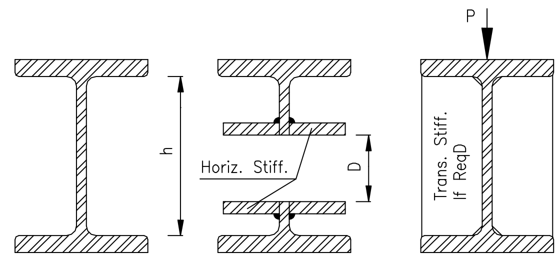
Proportions and Stiffeners Design for I-Shaped Member Based on AISC 360-16

DESIGN CRITERIA

1. Stiffeners, including full depth transverse stiffeners, one-half depth transverse stiffeners, and horizontal stiffeners around opening, should be provided by pair. Stiffeners may have the same thickness with web and not out width of flanges, when required.
2. Based on AISC 360-16, stiffeners may be required without concentrated load, and the most web opening needs horizontal & transverse stiffeners around opening.



INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	**50**	ksi, (345 MPa)
TOP FLANGE WIDTH	$b_{f,top}$ =	**16**	in (406 mm)
TOP FLANGE THICKNESS	$t_{f,top}$ =	**1.5**	in (38 mm)
BOTTOM FLANGE WIDTH	$b_{f,bot}$ =	**16**	in (406 mm)
BOTTOM FLANGE THICKNESS	$t_{f,bot}$ =	**1.5**	in (38 mm)
WEB THICKNESS	t_w =	**0.5**	in (13 mm)
WEB DEPTH	h =	**50**	in (1270 mm)
BEAM DEPTH	d =	53	in (1346 mm)

[Transverse Stiffeners Suggested]

[Bearing Stiffeners Must Be Provided]

CONCENTRATED LOAD (input zero if no load)	P =	**300**	kips, (1334 kN), ASD
THE MAX SHEAR FORCE	V =	**270**	kips, (1201 kN), ASD
OPENING DEPTH (input zero if no opening)	D =	**20**	in (508 mm)

ANALYSIS

CHECK PROPORTIONING LIMITS (AISC 360-16 F13.2)

I_{yc} / I_y = 0.50 **>** 0.10 h / t_w = 100 **<** 260

< 0.90 A_w / A_{fc} = 1.10 **<** 10

where I_{yc} = 512.00 in^4 where A_w = 26.50 in^4

I_y = 1024.31 in^4 A_{fc} = 24.00 in^4

[Satisfactory]

DETERMINE ALLOWABLE SHEAR STRENGTH , V_n / Ω_v (AISC 360-16 G2.1)

$h = d - t_{f,top} - t_{f,bot}$ = 50 in , h / t_w = 100 , A_w = 16.5 in^2 , at opening

k_v = 5.34 , (AISC 360 G2.3 (a)) Ω_v = 1.67 26.5 in^2 , at non-opening

$V_n = 0.6 F_y A_w C_v$

$V_{allowable} = V_n / \Omega_v$ = **181.45** kips, at opening $C_v = Min\left(1.0\ ,\ \frac{1.10}{h/t_w}\sqrt{\frac{k_v E}{F_y}}\right) =$ 0.612

291.43 kips, at non-opening

CHECK TRANSVERSE STIFFENERS FOR SHEAR (AISC 360-16 G2.3)

h / t_w = 100 **>** $2.46\ (E / F_y)^{0.5}$ = 59

$V_{allowable}$ = 291.43 kips, at non-opening **>** V = 270.00 kips, at the max shear section

[Unsatisfactory]

[Transverse Stiffeners Suggested]

CHECK LOCAL WEB YIELDING FOR THE CONCENTRATED LOAD. (AISC 360-16 J10.2)

P = 300.00 kips $k = t_{f,top} + w$ = 1.81 in

l_b = 0 in, bearing length, point. Ω = 1.5

$$\begin{cases} \frac{P}{t_w(l_b+5k)}\ ,\ for\ \ c>d & 66.21 \\ \frac{P}{t_w(l_b+2.5k)}\ ,\ for\ \ c\le d & 132.41 \end{cases} > F_y / \Omega$$

[Unsatisfactory]

[Bearing Stiffeners Must Be Provided]

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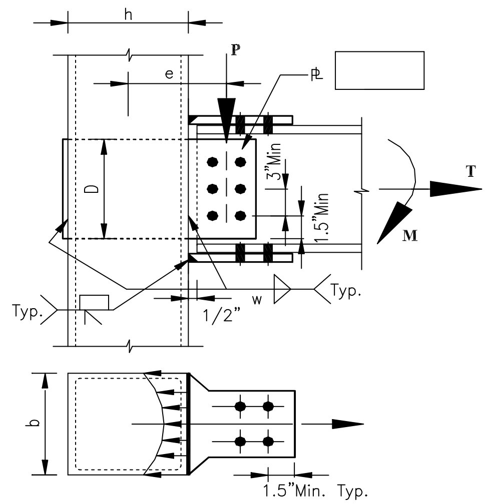
Moment Connection Design for Beam to Tube Column Based on AISC 360-16

DESIGN CRITERIA

1. The moment load, M, is supported by top and bottom flanges, and the vertical load, P, by beam web. But the axial load, T, may be supported by flanges/web, or by both, so the design conservatively double accounts that the axial load fully by both flanges and web.
2. The top/bottom flange plate supported tension/compression force to single side of column, so the stress is curved by tube web stiffness.
3. The additional thickness of connection plates and/or shims may be used to accommodate tolerances for fabrication.



INPUT DATA & DESIGN SUMMARY

TUBE COLUMN SIZE	h =	10	in	b =	10	in
	t_{col} =	0.75	in			
WF BEAM SECTION				=>	W24X94	
MOMENT LOAD, ASD				M =	75	ft-kips
VERTICAL SERVICE LOAD, ASD				P =	43	kips
HORIZONTAL TENSION LOAD, ASD				T =	53	kips
PLATE THICKNESS				t =	0.625	in
STEEL YIELD STRESS				F_y =	50	ksi
WELD SIZE				w =	0.5	in (1/2 in)
BOLT DIAMETER				ϕ =	1	in (1 in)
BOLT MATERIAL (A307, A325, A490)				ASTM =	A325	
HOLE TYPE (STD, NSL, OVS, SSL, LSL)				=>	STD	

STD = Standard round holes (d + 1/16 ")
NSL = Long or short-slotted hole normal to load direction
OVS = Oversize round holes
SSL = Short-slotted holes
LSL = Long-slotted holes

CONNECTION TYPE (SC, N, X) => **SC**

SC = Slip critical connection
N = Bearing-type connection with threads included in the shear plane
X = Bearing-type connection with threads excluded from the shear plane

THE DESIGN IS ADEQUATE.

WEB BOLT NO.	2	*rows &*	7	*bolts per row, (total 14 bolts.)*
EACH FLANGE BOLT NO.	2	*rows &*	3	*bolts per row, (total 6 bolts.)*

ANALYSIS

BEAM SECTION PROPERTIES (AISC Manual Table 1)

d	t_w	t_f	k	b_f
24.3	0.515	0.875	1.38	9.07

CHECK TUBE CAPACITY ON CURVED STRESS AT FLANGE PLATE (AISC 360 D)

(M / d + 0.5 T) = 63.54 kips < T_{allow} [Satisfactory]

The edge max stress F_y = 50.0 ksi

The middle stress $f_y = F_y\, 48\, (t_{col}^2/12)\, (0.5\, h) / b^3$ = 0.56 ksi, *(per pure math method.)*

Total capacity $T_{allow} = t\, b\, [F_y - 2\, (F_y - f_y)\, /3] / \Omega_t$ = 63.78 kips

CHECK CAPACITY OF WEB BOLTS (AISC 360 J3)

Allow shear per bolt = 11.5 kips / bolt, (R_n / Ω_v, AISC Manual Table 7)

$(P^2 + T^2)^{0.5}$ = 68 kips

No. of bolts required	=	5.9		Number of bolts used	=	14	bolts	[Satisfactory]
Bolt spacing required	=	3.00	in	Bolt spacing used	=	3.00	in	[Satisfactory]
Edge spacing required	=	1.25	in, (Tab J3.4)	Edge spacing used	=	1.25	in	[Satisfactory]
Number of rows required	=	2	rows	Number of rows used	=	2	rows	[Satisfactory]

Bolt group capacity = 161 kips > $(P^2 + T^2)^{0.5}$ = 68 kips
> P = 43 kips [Satisfactory]

CHECK CAPACITY OF WEB PLATE WELDING (AISC 360 J2)

e	=	8.25	in, (AISC 360 Table J3.4)	
Plate thickness	=	0.63	in	
Weld size,w	=	0.50	in	
Min allowable weld	=	0.25	in	[Satisfactory]
Max allowable weld	=	0.56	in	[Satisfactory]
t_e	=	0.35	in	
$I = 2\ (t_e\ d^3\ / 12)$	=	845.4	in^4	
Vertical shear = $P / A_w = P / 2\, d\, t_e$	=	2.5	ksi	
Bending stress = $0.5\, P\, e\, d / I$	=	5.1	ksi	
Tension stress = $T / A_w = T / 2\, d\, t_e$	=	3.1	ksi	

θ = 50.947 deg, (AISC 360 J2-5)
Δ_u = 0.0393 in
Δ_m = 0.0293 in
f(p) = 1.2003 , (AISC 360 J2-9)
F_w = 67.663 ksi, (AISC 360 J2-8)

Resultant Stress = $[\ (P/A_w)^2 + (0.5\, P\, e\, d / I + T/A_w)^2\ ]^{0.5}$ = 8.6 ksi

Allow shear $F_w / \Omega = F_w / 2.0$ = 33.8 ksi
> 8.6 ksi [Satisfactory]

(cont'd)

CHECK WEB PLATE FLEXURE CAPACITY WITH VON-MISES REDUCTION (AISC Manual page 10-103)

D = 20.5 in

$f_v = [(P/A)^2 + (T/A + 6Pe / tD^2)^2]^{0.5} =$ 12.7 ksi

$F_{cr} = (F_y^2 - 3 f_v^2)^{0.5} =$ 44.9 ksi

M = Pe = 29.6 ft-k < $F_{cr} Z / \Omega =$ 147.1 ft-k [Satisfactory]

CHECK WEB PLATE FOR SHEAR CAPACITY (AISC 360 G2)

P / A = 3.4 ksi < $0.6 F_y C_v / \Omega_v = 0.6 F_y 1.0 / 1.5 =$ 20 ksi [Satisfactory]

CHECK WEB PLATE FOR TENSION CAPACITY (AISC 360 D)

T / A = 4.1 ksi < $F_y / \Omega_t = F_y / 1.67 =$ 29.94 ksi [Satisfactory]

CHECK WEB NET SHEAR FRACTURE (AISC 360 J4.2)

F_u = 70 ksi (AISC Manual Pg. 2-39)

$P_{allow} = 0.6 F_u / \Omega [D - n (d_s + 1/8)] t$ = 166 kips > 43 kips [Satisfactory]

CHECK WEB NET TENSION FRACTURE (AISC 360 J4.1)

F_u = 70 ksi

$T_{allow} = F_u / \Omega [D - n (d_s + 1/8)] t$ = 276 kips > 53 kips [Satisfactory]

CHECK CAPACITY OF EACH FLANGE BOLTS (AISC 360 J3)

One flange bolt group capacity = 69 kips > (M / d + 0.5 T) = 64 kips [Satisfactory]

CHECK EACH FLANGE PLATE FOR TENSION CAPACITY (AISC 360 D)

(M / d + 0.5 T) / A = 11.21 ksi < $F_y / \Omega_t = F_y / 1.67 =$ 29.94 ksi [Satisfactory]

CHECK FLANGE NET SHEAR FRACTURE (AISC 360 J4.2)

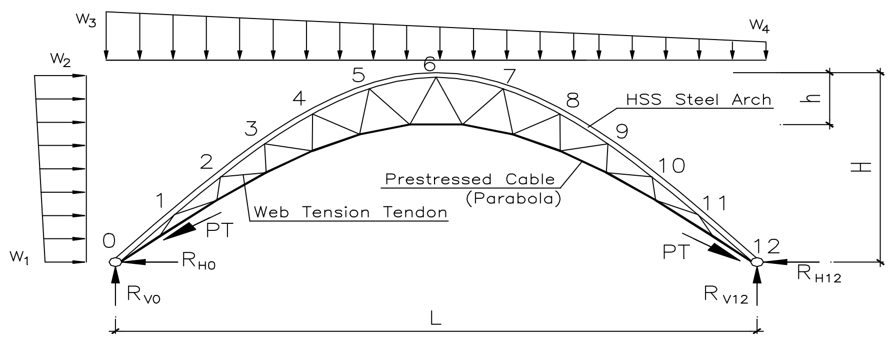
$V_{allow} = 0.6 F_u / \Omega [1.5 + 3 (n - 1) - n (d_s + 1/8)] 2 t_{min}$ = 89 kips > 64 kips [Satisfactory]

Purchaser's LOGO (Please email us your LOGO at purchase.)	PROJECT :		PAGE :	
	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

Prestressed Steel Arch Design Based on 2021 IBC/2019 CBC & AISC 360-16

DESIGN CRITERIA

1. Statically indeterminate prestressed steel arch contains secondary section forces, including secondary moments. The arch design forces have to include both the primary section forces and the secondary section forces. The secondary section forces are very important, which cannot directly be gotten from 3D/2D FEM analysis by input internal prestressed tension forces as external loads.
2. The loads w_1, w_2, w_3, and w_4 may be negative (outward) based on IBC/CBC 1605 wind/seismic load combination results. If the loads cause all final arch design section moments negative (top side still tension by prestress & loads), the in-plane, kL_x, reduced (steel cost reduced). But if positive (bottom side tension), the prestressed entire cable loosed, then $kL_x/r_x > 200$ design inadequate (AISC 360 E2).



INPUT DATA & DESIGN SUMMARY

HSS STEEL ARCH (Tube or Pipe) & SIZE

HSS20X12X5/8 <== Tube

A	r_x	r_y	S_x
35.0	7.3	4.9	188.0
S_y	I_x	I_y	Z_x
142.0	1880.0	851.0	230.0
Z_y	b	h	t
162.0	20.0	12.0	0.6

F_y = **46** ksi, (317 MPa)

SERVICE (ASD level) LOADS

w_1 = **0.3** kips / ft, (4.4 kN / m)

w_2 = **0.5** kips / ft, (7.3 kN / m)

w_3 = **0.2** kips / ft, (2.9 kN / m)

w_4 = **-0.5** kips / ft, (-7.3 kN / m)

THE DESIGN IS ADEQUATE.

PRESTRESSED CABLE FORCE

PT = **8.2** kips, (36 kN), ASD

DIMENSIONS

H = **80** ft, (24.38 m) h = **18** ft, (5.49 m) L = **150** ft, (45.72 m)

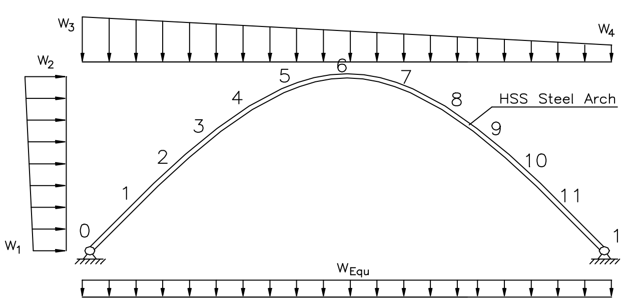
WEAK AXIS EFFECTIVE ARCH LENGTH kL_y = **24** ft, (7.32 m)

ANALYSIS

DETERMINE ARCH DESIGN FORCES

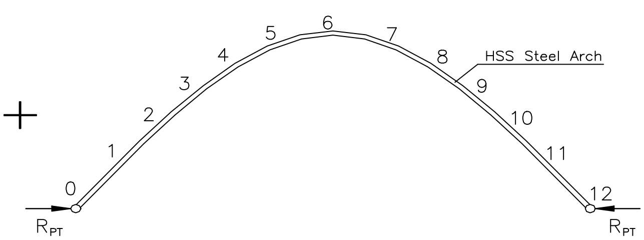
w_{Equ} = 8 PT (H - h) / L^2 = 0.181 kips / ft

R_{PT} = 6.43 kips - PT Sin[0.5 Atan(8L (H - h) / (16 (H - h)^2 - L^2))] = 2.19 kips



1/2 Analysis (Equivalent Load Method))

+



2/2 Analysis (Secondary Forces from Isolated)

No.	Location		1/2 Analysis (kips, ft-kips)			2/2 Analysis (kips, ft-kips)			1/2 & 2/2 (kips, ft-kips)		
Section	X (ft)	Y (ft)	Axial	Shear	Moment	Axial	Shear	Moment	Axial	Shear	Moment
0	-75.00	-80.00	25.62	9.74	-17	0.98	1.96	0	26.60	11.70	0
1	-66.00	-61.95	20.62	4.76	181.75	1.09	1.90	39.50	21.70	6.66	221.25
2	-57.00	-46.21	15.00	-0.37	264.72	1.24	1.81	73.96	16.23	1.44	338.68
3	-45.75	-29.77	8.34	-5.21	259.85	1.46	1.63	109.94	9.80	-3.58	369.78
4	-33.00	-15.49	0.93	-8.23	164.73	1.77	1.28	141.19	2.70	-6.94	305.92
5	-18.00	-4.61	-6.21	-7.19	19.58	2.12	0.54	165.00	-4.09	-6.65	184.58
6	**0**	**0**	-9.87	-2.50	-109.30	2.19	0	175.08	-7.68	-2.50	65.78
7	18.00	-4.61	-10.54	0.69	-154.00	2.12	-0.54	165.00	-8.42	0.15	11.00
8	33.00	-15.49	-11.38	1.83	-140.75	1.77	-1.28	141.19	-9.61	0.54	0.44
9	45.75	-29.77	-12.82	2.06	-105.66	1.46	-1.63	109.94	-11.37	0.42	4.27
10	57.00	-46.21	-14.64	1.93	-65.47	1.24	-1.81	73.96	-13.40	0.12	8.49
11	66.00	-61.95	-16.74	1.64	-30.05	1.09	-1.90	39.50	-15.65	-0.26	9.45
12	75.00	-80.00	-16.74	1.64	3	0.98	-1.96	0	-15.76	-0.32	0

(cont'd)

AXIAL COMPRESSION FORCE P_r = 26.60 *kips, (118 kN), ASD*

STRONG AXIS EFFECTIVE CURVE LENGTH kL_x = 19.12 *ft, (5.83 m)*

STRONG AXIS BENDING MOMENT M_{rx} = 369.78 *ft-kips, (501 kN-m), ASD*

STRONG AXIS BENDING UNBRACED CURVE LENGTH L_b = 24.0 *ft, (7.32 m), (AISC 360 F2.2.c)*

STRONG DIRECTION SHEAR LOAD, ASD V_{strong} = 11.70 *kips, (52 kN)*

WEAK AXIS BENDING MOMENT M_{ry} = 0.0 *ft-kips, (0 kN-m), ASD*

WEAK DIRECTION SHEAR LOAD, ASD V_{weak} = 0.0 *kips, (0 kN)*

TORSIONAL FORCE T_r = 0.0 *ft-kips, (0 kN-m), ASD*

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360 H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) \ , \ for \ \dfrac{P_r}{P_c}\ge 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) \ , \ for \ \dfrac{P_r}{P_c}< 0.2 \end{cases} = 0.72 < 1.33$$

[Satisfactory]
(2018 IBC, 1605.3.2)

Where $P_c = P_n / \Omega_c$ = 1280 / 1.67 = 766.39 kips, (AISC 360 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 881.67 / 1.67 = 527.94 ft-kips, (AISC 360 Chapter F)
> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 621.00 / 1.67 = 371.86 ft-kips, (AISC 360 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK TORSIONAL CAPACITY (AISC 360 H3.1)

$$T_c=\frac{1}{\Omega_T}T_n=\frac{1}{\Omega_T}\begin{cases} \left[2(B-t)(H-t)t-4.5(4-\pi)t^3\right]\begin{bmatrix} 0.6F_y \ , \ for \ \dfrac{h}{t}\le 2.45\sqrt{\dfrac{E}{F_y}} \\ 0.6F_y 2.45\sqrt{\dfrac{E}{F_y}}\dfrac{t}{h} \ , \ for \ \dfrac{h}{t}\le 3.07\sqrt{\dfrac{E}{F_y}} \\ 0.458\dfrac{E\pi^2}{(h/t)^2}, \ for \ \dfrac{h}{t}\le 260 \end{bmatrix} \ , \ for \ HSS \ Tube \\ \dfrac{\pi(D-t)^2 t}{2}Max\left[\dfrac{1.23E}{\sqrt{\dfrac{L}{D}}\left(\dfrac{D}{t}\right)^{(5/4)}} \ , \ \dfrac{0.60E}{\left(\dfrac{D}{t}\right)^{(3/2)}}\right] \ , \ for \ HSS \ Pipe \end{cases} = 378.1 \text{ ft-kips}$$

> T_r
[Satisfactory]

Where	B	H	h	t	D	E	Ω_T =	1.67	,ASD
	12.00	20.00	18.13	0.63		29000			

CHECK SHEAR CAPACITY (AISC 360 G2)

$V_{n,strong} / \Omega_v$ = 460.0 / 1.67 = 275.4 kips > V_{strong} = 11.7 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 276.0 / 1.67 = 165.3 kips > V_{weak} = 0.0 kips **[Satisfactory]**

CHECK COMBINED TORSION, SHEAR, COMPRESSION, AND BENDING CAPACITY (AISC 360 H3.2)

$$\begin{cases} \dfrac{P_r}{P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right)+\left[Max\left(\dfrac{V_{strong}}{V_{c,strong}} \ , \ \dfrac{V_{weak}}{V_{c,weak}}\right)+\dfrac{T_r}{T_c}\right]^2 \ , \ for \ \dfrac{T_r}{T_c}>0.2 \\ Torsion \ \ Neglected \ , \ for \ \dfrac{T_r}{T_c}\le 0.2 \end{cases} = 0.0 < 1.3$$

[Satisfactory]
(2018 IBC, 1605.3.2)

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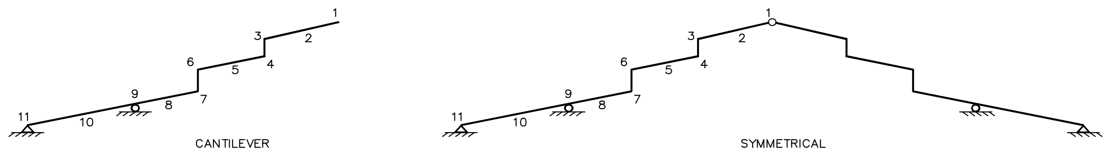
(Please email us your LOGO at purchase.)	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

Web-Tapered Roof Girder Design Based on AISC-ASD 9th Appendix F and 2018 IBC/2019 CBC 1605

DESIGN CRITERIA

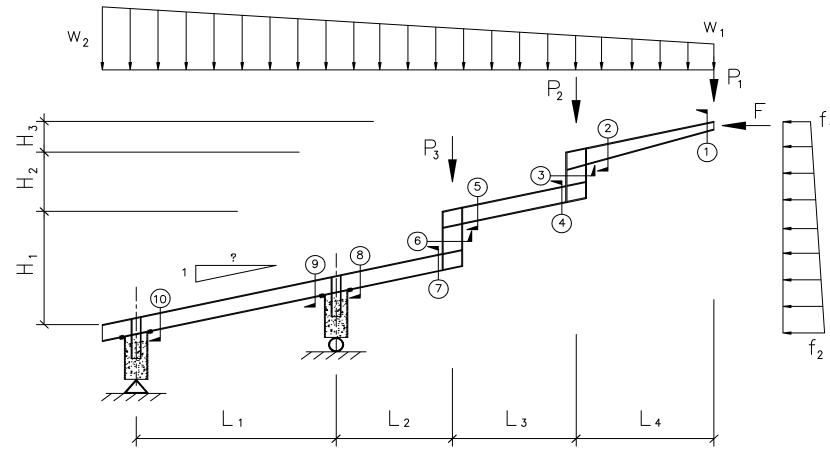


1. THIS IS CANTILEVER OR SYMMETRICAL DESIGN. FOR SYMMETRICAL, BOTH CASES OF CANTILEVER AND SYMMETRICAL HAVE TO BE CHECKED, SINCE THE LATERAL LOADS MAY NOT BE SYMMETRICAL.
2. IN ORDER TO QUALIFY UNDER THIS DESIGN, THE FLANGES SHALL BE OF EQUAL AND CONSTANT AREA.
3. DIAPHRAGM IS NOT BRACING MEMBER, SINCE ***L*** IS DIFFERENT WITH ℓ on AISC-ASD 9th F1.3, page 5-47.

INPUT DATA & DESIGN SUMMARY

DIMENSIONS

L_1 = 34.1 ft (10.4 m)
L_2 = 19.83 ft (6.0 m)
L_3 = 21 ft (6.4 m)
L_4 = 24.1 ft (7.3 m)
1 : ? = 1 : 4.5625 *(Slope Ratio)*
H_1 = 20.3 ft (6.2 m)
H_2 = 10.8 ft (3.3 m)
H_3 = 8.84 ft (2.7 m)



LOADS *(ASD level)*

w_1 = 53 *lbs / ft, (773 N / m)*
w_2 = 4208.2 *lbs / ft, (61371 N / m)*
f_1 = 40 *lbs / ft, (583 N / m)*
f_2 = 1588 *lbs / ft, (23159 N / m)*
P_1 = 0.1 *kips, (0.4 kN), Half only*
P_2 = 0.45 *kips, (2.0 kN)*
P_3 = 0.53 *kips, (2.4 kN)*
F = 0 *kips, (0.0 kN), Opposite*

THE BENT GIRDER DESIGN IS ADEQUATE.

STRUCTURAL SYMMETRICAL? **Yes** <== $\Delta_{1,X}$ = **0.00** *in (0 mm)* and $\Delta_{1,Y}$ = **-12.59** *in (-320 mm)*

SECTIONS

No.	b_f (in)	(mm)	t_f (in)	(mm)	t_w (in)	(mm)	d (in)	(mm)
1	20	508	1	25	1	25	18	457
2	20	508	1	25	1	25	36	914
3	20	508	1	25	1	25	42	1067
4	20	508	1	25	1	25	36	914
5	20	508	1	25	1	25	36	914
6	20	508	1	25	1	25	42	1067
7	20	508	1	25	1	25	36	914
8	20	508	1	25	1	25	36	914
9	20	508	1	25	1	25	36	914
10	20	508	1	25	1	25	36	914

STEEL YIELD STRESS F_y = 50 ksi (345 N / mm^2)
FULL HEIGHT STIFFENER SPACING a = 12 ft (3.7 m), O.C.
UNBRACED LENGTH / PURLIN SPACING $L_{b,top}$ = 12 ft (3.7 m)
UNBRACED LENGTH AT BOTTOM FLANGE $L_{b,bot}$ = 12 ft (3.7 m)

ANALYSIS

JOINT COORDINATES, DEFLECTIONS & MEMBER PROPERTIES

Joint No.	X (ft)	Y (ft)	Δ_X (in)	Δ_Y (in)
1	99.03	40.67	0.00	-12.59
2	86.98	36.62	-0.87	-10.00
3	74.93	32.57	-1.76	-7.34
4	74.93	26.37	-0.36	-7.33
5	64.43	24.07	-0.88	-5.00
6	53.93	21.77	-1.36	-2.76
7	53.93	11.82	0.59	-2.76
8	44.02	9.65	0.23	-1.11
9	34.10	7.47	-0.01	0
10	17.05	3.74	-0.09	0.38
11	0	0	0	0

Section No.	A (in^2)	I_x (in^4)
1	56.00	3234.7
2	74.00	15528.7
3	80.00	22146.7
4	74.00	15528.7
5	74.00	15528.7
6	80.00	22146.7
7	74.00	15528.7
8	74.00	15528.7
9	74.00	15528.7
10	74.00	15528.7

Member No.	L (ft)	A (in^2)	I_x (in^4)
1-2	12.71	60.50	6308.2
2-3	12.71	69.50	12455.2
3-4	6.20	80.00	22146.7
4-5	10.75	74.00	15528.7
5-6	10.75	74.00	15528.7
6-7	9.94	80.00	22146.7
7-8	10.15	74.00	15528.7
8-9	10.15	74.00	15528.7
9-10	17.45	74.00	15528.7
10-11	17.45	74.00	15528.7

(cont'd)

DETERMINE REACTIONS

$R_{11,X}$ = **59.8** *kips* $R_{9,Y}$ = **195.9** *kips* $R_{1,X}$ = **-26.7** *kips*

$R_{11,Y}$ = **16.1** *kips*

DETERMINE ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \text{ , } for \ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \text{ , } for \ F_{b\gamma} \le F_y/3 \end{cases}$$

where $A_f = t_f\, b_f$

$A_{To} = t_f\, b_f + d_0\, t_w / 6$

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}}$$

$$h_S = 1.0 + 0.0230\gamma\sqrt{\frac{Ld_0}{A_f}}$$

$$h_W = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}}$$

γ = MIN[(d_L - d_0) / d_0 , 0.268 L/d_0, 6.0]

I_{To} = ($t_f\, b_f^3$ + $d_0\, t_w^3$ / 6) / 12

$$F_{s\gamma} = \frac{12000}{h_S L d_O / A_f}$$

$$F_{w\gamma} = \frac{170000}{\left(h_W L / r_{To}\right)^2}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}}$$

MEMBER No.	***L (ft)***	d_0	d_L	t_f	b_f	t_w	A_f
1-2	12.71	18	27	1	20	1	20
2-3	12.71	27	36	1	20	1	20
3-4	6.20	42	42	1	20	1	20
4-5	10.75	36	36	1	20	1	20
5-6	10.75	36	36	1	20	1	20
6-7	9.94	42	42	1	20	1	20
7-8	10.15	36	36	1	20	1	20
8-9	10.15	36	36	1	20	1	20
9-10	17.45	36	36	1	20	1	20
10-11	17.45	36	36	1	20	1	20

MEMBER No.	γ	A_{To}	I_{To}	r_{To}	h_s	h_w	$F_{s\gamma}$	$F_{w\gamma}$	B	$F_{b\gamma}$
1-2	0.50	23.00	667	5.38	1.13	1.01	77.02	207.51	1.49	**30.00**
2-3	0.33	24.50	667	5.22	1.11	1.01	52.49	196.12	1.53	**30.00**
3-4	0.00	27.00	667	4.97	1.00	1.00	76.84	759.65	1.75	**30.00**
4-5	0.00	26.00	667	5.07	1.00	1.00	51.68	262.18	1.75	**30.00**
5-6	0.00	26.00	667	5.07	1.00	1.00	51.68	262.18	1.75	**30.00**
6-7	0.00	27.00	667	4.97	1.00	1.00	47.88	294.99	1.75	**30.00**
7-8	0.00	26.00	667	5.07	1.00	1.00	54.73	294.03	1.75	**30.00**
8-9	0.00	26.00	667	5.07	1.00	1.00	54.73	294.03	1.75	**30.00**
9-10	0.00	26.00	667	5.07	1.00	1.00	31.83	99.43	1.75	**30.00**
10-11	0.00	26.00	667	5.07	1.00	1.00	31.83	99.43	1.75	**30.00**

DETERMINE ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \text{ , } for \ h/t_w \le 380\sqrt{F_y} \\ \frac{C_v F_y}{2.89} \le 0.4F_y \text{ , } for \ h/t_w > 380\sqrt{F_y} \end{cases}$$

where $h = d_L - 2\, t_f$

$$k_v = \begin{cases} 4.0 + \frac{5.34}{(a/h)^2} \text{ , } for \ a/h \le 1.0 \\ 5.34 + \frac{4.0}{(a/h)^2} \text{ , } for \ a/h > 1.0 \end{cases}$$

$$C_v = \begin{cases} \frac{45000 k_v}{F_y (h/t_w)^2} \text{ , } for \ C_v \le 0.8 \\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \text{ , } for \ C_v > 0.8 \end{cases}$$

MEMBER No.	***a (ft)***	h	h/t_w	$380/F_y^{0.5}$	K_v	C_v	F_v
1-2	12.00	25	25	54	5.46	2.51	**20.00**
2-3	12.00	34	34	54	5.56	1.86	**20.00**
3-4	12.00	40	40	54	5.65	1.60	**20.00**
4-5	12.00	34	34	54	5.56	1.86	**20.00**
5-6	12.00	34	34	54	5.56	1.86	**20.00**
6-7	12.00	40	40	54	5.65	1.60	**20.00**
7-8	12.00	34	34	54	5.56	1.86	**20.00**
8-9	12.00	34	34	54	5.56	1.86	**20.00**
9-10	12.00	34	34	54	5.56	1.86	**20.00**
10-11	12.00	34	34	54	5.56	1.86	**20.00**

(cont'd)

DETERMINE ALLOWABLE COMPRESSIVE STRESS (APP. F7.3, pg 5-102)

$$F_{a\gamma}=\begin{cases}\dfrac{\left(1.0-\dfrac{S^2}{2C_c^2}\right)F_y}{\dfrac{5}{3}+\dfrac{3S}{8C_c}-\dfrac{S^3}{8C_c^3}} & ,\ for\ \ S\le C_c\\[2ex] \dfrac{12\pi^2E}{23S^2} & ,\ for\ \ S>C_c\end{cases}$$

where K_γ = (effective length factor by an analysis)

$S = K_\gamma\ l / r_{ox}$

$$C_c=\sqrt{\frac{2\pi^2E}{F_y}}$$

E = 29000 ksi

MEMBER No.	l	K_γ	C_c	r_{ox}	S	$F_{a\gamma}$
1-2	16.33	2.5	107	10.21	47.97	**24.66**
2-3	16.33	2.5	107	13.39	36.59	**26.30**
3-4	16.33	2.5	107	16.64	29.44	**27.22**
4-5	16.33	2.5	107	14.49	33.82	**26.67**
5-6	16.33	2.5	107	14.49	33.82	**26.67**
6-7	16.33	2.5	107	16.64	29.44	**27.22**
7-8	16.33	2.5	107	14.49	33.82	**26.67**
8-9	16.33	2.5	107	14.49	33.82	**26.67**
9-10	16.33	2.5	107	14.49	33.82	**26.67**
10-11	16.33	2.5	107	14.49	33.82	**26.67**

CHECK MEMBER CAPACITY AT EACH JOINT

Joint	1	2	3	4	5	6	7	8	9	10	11
d (in)	18	27	42	36	36	42	36	36	36	36	36
I (in^4)	3234.667	15528.66667	22146.7	15528.7	15528.7	22146.7	15528.7	15528.7	15528.7	15528.7	15528.7
A (in^2)	56.00	74.00	80.00	74.00	74.00	80.00	74.00	74.00	74.00	74.00	74.00
|N (kips)|	25.8	28.7	14.0	35.4	40.3	46.1	55.9	63.5	33.6	51.6	51.6
|V (kips)|	7.4	7.4	29.5	29.5	28.0	38.5	45.4	67.7	87.0	87.0	30.0
|M (ft-k)|	0.0	94.5	111.0	71.8	210.4	511.4	893.9	1354.5	2041.5	523.2	0.0
f_a (ksi)	0.46	0.39	0.18	0.48	0.54	0.58	0.76	0.86	0.45	0.70	0.70
F_a (ksi)	24.66	26.30	27.22	26.67	26.67	27.22	26.67	26.67	26.67	26.67	26.67
f_v (ksi)	0.41	0.28	0.70	0.82	0.78	0.92	1.26	1.88	2.42	2.42	0.83
F_v (ksi)	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00
f_b (ksi)	0.0	1.0	1.3	1.0	2.9	5.8	12.4	18.8	28.4	7.3	0.0
F_b (ksi)	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00

$f_a < F_a$ **[Satisfactory]** $f_v < F_v$ **[Satisfactory]** $f_b < F_b$ **[Satisfactory]**

CHECK COMBINED FLEXURE AND AXIAL FORCE (APP. F7.4, pg 5-104)

$$(f_a\ ,\ f_b)=\begin{cases}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{f_{bl}}{F_{b\gamma}} & ,\ for\ \ \dfrac{f_{a0}}{F_{a\gamma}}\le 0.15\\[3ex] Larger\ \ of\ \left(\begin{matrix}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{C_m' f_{bl}}{\left(1-\dfrac{f_{a0}}{F_{e\gamma}'}\right)F_{b\gamma}}\\[3ex] \dfrac{f_{a0}}{0.6F_y}+\dfrac{f_{bl}}{F_{b\gamma}}\end{matrix}\right) & ,\ for\ \ \dfrac{f_{a0}}{F_{a\gamma}}>0.15\end{cases}\quad\le\quad 1.0$$

Joint	f_{a0}	f_{bl}	$F_{a\gamma}$	$F_{b\gamma}$	$F_{e\gamma}'$	C_m'	(f_a, f_b)
1	0.46	0.00	24.66	30.00	64.89	0.99	0.02
2	0.39	0.99	26.30	30.00	111.52	1.00	0.05
3	0.18	1.26	27.22	30.00	172.27	1.00	0.05
4	0.48	1.00	26.67	30.00	130.59	1.00	0.05
5	0.54	2.93	26.67	30.00	130.59	1.00	0.12
6	0.58	5.82	27.22	30.00	172.27	1.00	0.22
7	0.76	12.43	26.67	30.00	130.59	0.99	0.44
8	0.86	18.84	26.67	30.00	130.59	0.99	0.66
9	0.45	28.40	26.67	30.00	130.59	1.00	0.96
10	0.70	7.28	26.67	30.00	130.59	1.00	0.27
11	0.70	0.00	26.67	30.00	130.59	1.00	0.03

[Satisfactory]

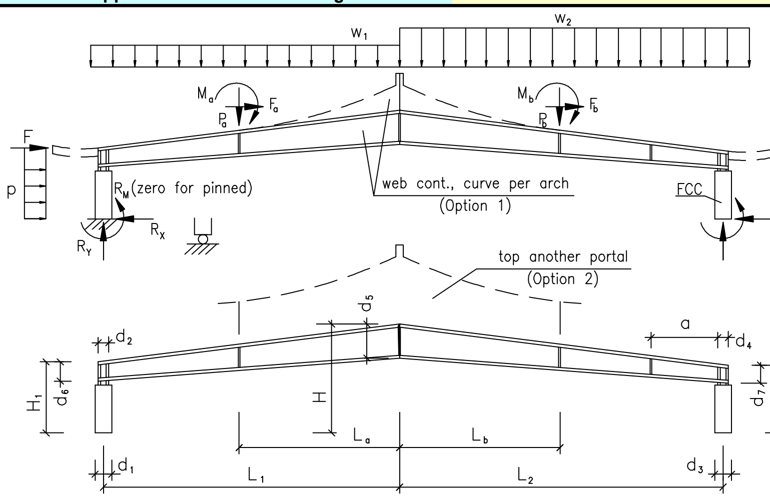
Technical Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. AISC: "Design Guide 25: Frame Design Using Web-Tapered Members, 2010.

Purchaser's LOGO (Please email us your LOGO at purchase.)	PROJECT :		PAGE :	
	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

Typical Frame Design of Web Curved Portal Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25

INPUT DATA & DESIGN SUMMARY



STEEL YIELD STRESS F_y = **50** ksi

CONCRETE STRENGTH f_c' = **5** ksi

LEFT FCC H_1 = **21.5** ft

(filled composite column) d_1 = **48** in

$t_{s,\,fcc}$ = **0.75** in

d_2 = **30** in, *vertical WF*

b_f = **20** in

t_f = **1** in

t_w = **0.75** in

RIGHT FCC H_2 = **22** ft

d_3 = **48** in

$t_{s,\,fcc}$ = **0.75** in

d_4 = **30** in, *vertical WF*

b_f = **20** in

t_f = **1** in

t_w = **0.75** in

LEFT BEAM DIMENSIONS b_f = **28** in, t_f = **1** in, t_w = **1** in, d_5 = **66** in, d_6 = **44** in, L_1 = **80** ft

RIGHT BEAM DIMENSIONS b_f = **28** in, t_f = **1** in, t_w = **1** in, d_5 = **66** in, d_7 = **44** in, L_2 = **82** ft

RIDGE HEIGHT H = **30** ft

GRAVITY LOAD w_1 = **1.02** kips / ft (" - " for wind uplift)

w_2 = **1.022** kips / ft (" - " for wind uplift)

LATERAL LOAD F = **114.8** kips (" - " to left direction)

p = **0.4** kips / ft (" - " to left direction)

BEAM STIFFENER SPACING a_{bm} = **24** ft

POINT LOADS

L_a =	**40**	ft	L_b =	**40**	ft
F_a =	**56**	kips	F_b =	**56**	kips
M_a =	**112**	ft-kips	M_b =	**112**	ft-kips
P_a =	**28**	kips	P_b =	**28**	kips

UNBRACED LENGTH / PURLIN SPACING $L_{b,top}$ = **10** ft

UNBRACED LENGTH AT BOTTOM FLANGE $L_{b,bot}$ = **40** ft

(Diaphragm is not bracing member. L is different with " l " in F1.3, pg 5-47)

BASE (d_1 & d_3) PINNED ? **No** , (fixed)

THE DESIGN IS INADEQUATE, SEE ANALYSIS BELOW

HORIZONTAL DRIFT Δ_H = **0.42** in

BEAM DEFLECTION Δ_{max} / L = **L / 720**

ANALYSIS

DETERMINE REACTIONS

R_X =	345.5	kips (Left)	R_X =	-110.1	kips (Right)
R_Y =	117.6	kips (Left)	R_Y =	103.8	kips (Right)
R_M =	4441.1	ft-kips (Left)	R_M =	-538.0	ft-kips (Right)

DETERMINE ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \ for \ F_{b\gamma} > F_y / 3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \ for \ F_{b\gamma} \le F_y / 3 \end{cases}$$

where $A_f = t_f\, b_f$

$A_{To} = t_f\, b_f + d_0\, t_w / 6$

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}}$$

$$h_S = 1.0 + 0.0230\gamma\sqrt{\frac{L d_0}{A_f}}$$

$$h_W = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}}$$

γ = MIN[$(d_L - d_0) / d_0$, $0.268\, L/d_0$, 6.0]

$I_{To} = (t_f\, b_f^3 + d_0\, t_w^3 / 6) / 12$

$$F_{s\gamma} = \frac{12000}{h_S L d_O / A_f}$$

$$F_{w\gamma} = \frac{170000}{(h_W L / r_{To})^2}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}}$$

(cont'd)

	Length	γ	A_{To}	I_{To}	r_{To}	H_s	H_w	$F_{s\gamma}$	$F_{w\gamma}$	B	$F_{b\gamma}$
Left Col	1.8	0.00	23.75	667	5.30	1.00	1.00	363.64	9862	1.75	**30.00**
Right Col	1.8	0.00	23.75	667	5.30	1.00	1.00	363.64	9862	1.75	**30.00**
L. Bm (+)	79.1	0.50	35.33	1830	7.20	1.16	1.01	54.96	601.92	1.49	**30.00**
(-)	79.1	0.50	35.33	1830	7.20	1.32	1.02	12.09	37.04	1.49	**28.54**
R. Bm (+)	81.1	0.50	35.33	1830	7.20	1.16	1.01	54.96	601.92	1.49	**30.00**
(-)	81.1	0.50	35.33	1830	7.20	1.32	1.02	12.09	37.04	1.49	**28.54**

DETERMINE ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ for \ h/t_w \le 380\sqrt{F_y} \\ \dfrac{C_v F_y}{2.89} \le 0.4F_y \ , \ for \ h/t_w > 380\sqrt{F_y} \end{cases}$$

where $h = d_L - 2\,t_f$

$$k_v = \begin{cases} 4.0 + \dfrac{5.34}{(a/h)^2} \ , \ for \ a/h \le 1.0 \\ 5.34 + \dfrac{4.0}{(a/h)^2} \ , \ for \ a/h > 1.0 \end{cases} \qquad C_v = \begin{cases} \dfrac{45000k_v}{F_y(h/t_w)^2} \ , \ for \ C_v \le 0.8 \\ \dfrac{190}{h/t_w}\sqrt{\dfrac{k_v}{F_y}} \ , \ for \ C_v > 0.8 \end{cases}$$

	a	*h*	h/t_w	$380/F_y^{0.5}$	K_v	C_v	F_v
Left Col	1.8	28.00	37	54	12.65	2.56	**20.00**
Right Col	1.8	28.00	37	54	12.65	2.56	**20.00**
Left Bm	24.0	64.00	64	54	5.54	0.99	**17.09**
Right Bm	24.0	64.00	64	54	5.54	0.99	**17.09**

DETERMINE ALLOWABLE COMPRESSIVE STRESS (APP. F7.3, pg 5-102)

$$F_{a\gamma} = \begin{cases} \dfrac{\left(1.0 - \dfrac{S^2}{2C_c^2}\right)F_y}{\dfrac{5}{3} + \dfrac{3S}{8C_c} - \dfrac{S^3}{8C_c^3}} \ , \ for \ S \le C_c \\ \dfrac{12\pi^2 E}{23S^2} \ , \ for \ S > C_c \end{cases}$$

where $K_\gamma =$ (effective length factor by an analysis)

$S = K_\gamma\, l / r_{ox}$

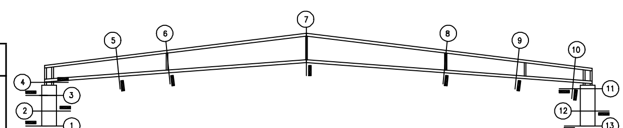
$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

$E =$ 29000 ksi

	l	K_γ	C_c	I_x	*A*	r_{ox}	*S*	$F_{a\gamma}$
Left Col	1.8	2.0	107	6188	62.50	9.95	4.42	**29.70**
Right Col	1.8	2.0	107	6188	62.50	9.95	4.42	**29.70**
Left Bm	79.1	2.5	107	20651	100.00	14.37	165.16	**5.47**
Right Bm	81.1	2.5	107	20651	100.00	14.37	169.22	**5.21**

CHECK EACH SECTION CAPACITIES

	d_0	d_L	t_f	b_f	t_w	A_f
Left Col	30	30	1	20	0.75	20
Right Col	30	30	1	20	0.75	20
Left Bm	44	66	1	28	1	28
Right Bm	44	66	1	28	1	28



Section	1	2	3	4	5	6	7	8	9	10	11	12	13
d (in)	30	30	30	30	51	59	66	59	51	44	30	30	30
I (in^4)	10688	10688	10688	10688	48164	65011	84942	65011	48164	34203	10688	10688	10688
A_w (in^2)	22.50	22.50	22.50	22.50	51.33	58.67	66.00	58.67	51.33	44.00	22.50	22.50	22.50
N (kips)	-117.6	-117.6	-117.6	-117.6	230.3	230.3	168.0	166.6	116.8	117.6	-103.8	-103.8	-103.8
V (kips)	341.21	338.34	338.34	341.21	75.80	75.80	12.59	72.57	72.57	341.21	110.11	110.11	110.11
M (ft-k)	4443.0	1987.4	-488.7	-1641.9	-729.6	793.2	1308.4	1384.5	-134.1	-1641.9	1480.7	471.3	-538.0
f_a (ksi)				1.88	2.15	2.01	1.38	1.45	1.09	1.18	1.66		
F_a (ksi)				29.70	5.47	5.47	5.47	5.21	5.21	5.21	29.70		
f_v (ksi)		FCC Design		15.16	1.48	1.29	0.19	1.24	1.41	7.75	4.89		FCC Design
F_v (ksi)				20.00	17.09	17.09	17.09	17.09	17.09	17.09	20.00		
f_b (ksi)				27.65	4.67	4.29	6.10	7.50	0.86	12.67	24.94		
F_b (ksi)				30.00	28.54	30.00	30.00	30.00	28.54	28.54	30.00		

$f_a < F_a$ **[Satisfactory]** $f_v < F_v$ **[Satisfactory]** $f_b < F_b$ **[Satisfactory]**

(cont'd)

CHECK COMBINED FLEXURE AND AXIAL FORCE (APP. F7.4, pg 5-104)

$$(f_a\ ,\ f_b)=\begin{cases}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{f_{bl}}{F_{b\gamma}}\ , & for\ \dfrac{f_{a0}}{F_{a\gamma}}\le 0.15\\ Larger\ of\ \begin{pmatrix}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{C_m' f_{bl}}{\left(1-\dfrac{f_{a0}}{F'_{e\gamma}}\right)F_{b\gamma}}\\ \dfrac{f_{a0}}{0.6F_y}+\dfrac{f_{bl}}{F_{b\gamma}}\end{pmatrix}\ , & for\ \dfrac{f_{a0}}{F_{a\gamma}}>0.15\end{cases}\le 1.3$$

	f_{a0}	f_{bl}	$F_{a\gamma}$	$F_{b\gamma}$	$F_{e\gamma}'$	C_m'	(f_a, f_b)
Left Col	0.00	27.65	29.70	30.00	7636.3	1.00	**0.92**
Right Col	0.00	24.94	29.70	30.00	7636.3	1.00	**0.83**
Left Bm	1.38	27.65	5.47	28.54	5.5	0.81	**1.30**
Right Bm	1.38	24.94	5.21	28.54	5.2	0.80	**1.22**

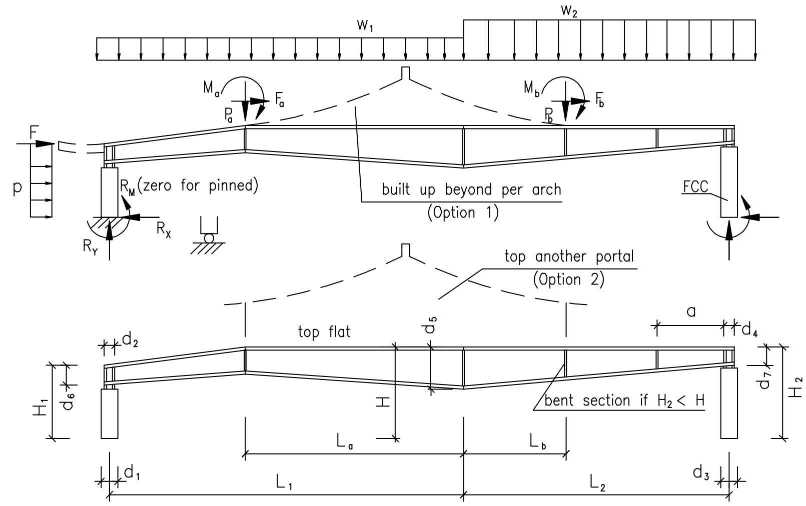
[Unsatisfactory]

Technical Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. AISC: "Design Guide 25: Frame Design Using Web-Tapered Members, 2010.

Mono Hip Frame Design of Web Curved Portal Based on AISC-ASD 9th Appendix F and/or AISC Design Guide 25

INPUT DATA & DESIGN SUMMARY



STEEL YIELD STRESS	F_y =	**50**	ksi
CONCRETE STRENGTH	f_c' =	**5**	ksi
LEFT FCC	H_1 =	**21.5**	ft
(filled composite column)	d_1 =	**48**	in
	$t_{s,\ fcc}$ =	**0.75**	in
	d_2 =	**30**	in, *vertical WF*
	b_f =	**20**	in
	t_f =	**1**	in
	t_w =	**0.75**	in
RIGHT FCC	H_2 =	**26**	ft
	d_3 =	**48**	in
	$t_{s,\ fcc}$ =	**0.75**	in
	d_4 =	**30**	in, *vertical WF*
	b_f =	**20**	in
	t_f =	**1**	in
	t_w =	**0.75**	in

LEFT BEAM DIMENSIONS	b_f =	**20**	in
	t_f =	**1**	in
	t_w =	**0.75**	in
	d_5 =	**40**	in
	d_6 =	**20**	in
	L_1 =	**60**	ft
RIGHT BEAM DIMENSIONS	b_f =	**20**	in
	t_f =	**1**	in
	t_w =	**0.75**	in
	d_5 =	**40**	in
	d_7 =	**20**	in
	L_2 =	**42**	ft

RIDGE HEIGHT	H =	**26**	ft
GRAVITY LOAD	w_1 =	**1.02**	kips / ft (" - " for wind uplift)
	w_2 =	**1.022**	kips / ft (" - " for wind uplift)
LATERAL LOAD	F =	**30.8**	kips (" - " to left direction)
	p =	**0.4**	kips / ft (" - " to left direction)
BEAM STIFFENER SPACING	a_{bm} =	**24**	ft

POINT LOADS

L_a =	**40**	ft	L_b =	**40**	ft
F_a =	**56**	kips	F_b =	**56**	kips
M_a =	**112**	ft-kips	M_b =	**112**	ft-kips
P_a =	**28**	kips	P_b =	**28**	kips

UNBRACED LENGTH / PURLIN SPACING $L_{b,top}$ = **10** ft

UNBRACED LENGTH AT BOTTOM FLANGE $L_{b,bot}$ = **40** ft

(Diaphragm is not bracing member. *L* is different with " ℓ " in F1.3, pg 5-47)

BASE (d_1 & d_3) PINNED ? **No** , (fixed)

THE FRAME DESIGN IS ADEQUATE.

HORIZONTAL DRIFT Δ_H = **0.24** in

BEAM DEFLECTION Δ_{max} / L = **L / 568**

ANALYSIS

DETERMINE REACTIONS

R_X =	155.2	kips (Left)	R_X =	-3.8	kips (Right)
R_Y =	80.2	kips (Left)	R_Y =	80.0	kips (Right)
R_M =	2185.0	ft-kips (Left)	R_M =	610.1	ft-kips (Right)

DETERMINE ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \ for \ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \ for \ F_{b\gamma} \le F_y/3 \end{cases}$$

where

$A_f = t_f\, b_f$

$A_{To} = t_f\, b_f + d_0\, t_w / 6$

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}}$$

$$h_S = 1.0 + 0.0230\gamma\sqrt{\frac{Ld_o}{A_f}}$$

$$h_W = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}}$$

γ = MIN[(d_L - d_0) / d_0 , 0.268 L/d_0, 6.0]

I_{To} = ($t_f\, b_f^3$ + $d_0\, t_w^3$ / 6) / 12

$$F_{s\gamma} = \frac{12000}{h_S L d_o / A_f}$$

$$F_{w\gamma} = \frac{170000}{(h_W L / r_{To})^2}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}}$$

(cont'd)

	Length	γ	A_{To}	I_{To}	r_{To}	H_s	H_w	$F_{s\gamma}$	$F_{w\gamma}$	B	$F_{b\gamma}$
Left Col	0.8	0.00	23.75	667	5.30	1.00	1.00	800.00	47732	1.75	**30.00**
Right Col	0.8	0.00	23.75	667	5.30	1.00	1.00	800.00	47732	1.75	**30.00**
L. Bm (+)	58.9	1.00	22.50	667	5.44	1.25	1.02	79.88	337.54	1.40	**30.00**
(-)	58.9	1.00	22.50	667	5.44	1.50	1.04	16.62	20.37	1.40	**25.79**
R. Bm (+)	40.8	1.00	22.50	667	5.44	1.25	1.02	79.88	337.54	1.40	**30.00**
(-)	40.8	1.00	22.50	667	5.44	1.50	1.04	16.62	20.37	1.40	**25.79**

DETERMINE ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ for \ h/t_w \le 380\sqrt{F_y} \\ \dfrac{C_v F_y}{2.89} \le 0.4F_y \ , \ for \ h/t_w > 380\sqrt{F_y} \end{cases}$$

where $h = d_L - 2\,t_f$

$$k_v = \begin{cases} 4.0 + \dfrac{5.34}{(a/h)^2} \ , \ for \ a/h \le 1.0 \\ 5.34 + \dfrac{4.0}{(a/h)^2} \ , \ for \ a/h > 1.0 \end{cases}$$

$$C_v = \begin{cases} \dfrac{45000k_v}{F_y(h/t_w)^2} \ , \ for \ C_v \le 0.8 \\ \dfrac{190}{h/t_w}\sqrt{\dfrac{k_v}{F_y}} \ , \ for \ C_v > 0.8 \end{cases}$$

	a	h	h/t_w	$380/F_y^{0.5}$	K_v	C_v	F_v
Left Col	0.8	28.00	37	54	45.87	4.87	**20.00**
Right Col	0.8	28.00	37	54	45.87	4.87	**20.00**
Left Bm	24.0	38.00	51	54	5.41	1.23	**20.00**
Right Bm	24.0	38.00	51	54	5.41	1.23	**20.00**

DETERMINE ALLOWABLE COMPRESSIVE STRESS (APP. F7.3, pg 5-102)

$$F_{a\gamma} = \begin{cases} \dfrac{\left(1.0 - \dfrac{S^2}{2C_c^2}\right)F_y}{\dfrac{5}{3} + \dfrac{3S}{8C_c} - \dfrac{S^3}{8C_c^3}} \ , \ for \ S \le C_c \\ \dfrac{12\pi^2 E}{23S^2} \ , \ for \ S > C_c \end{cases}$$

where $K_\gamma =$ (effective length factor by an analysis)

$S = K_\gamma\, l / r_{ox}$

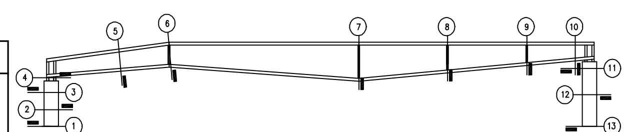
$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

$E =$ 29000 ksi

	l	K_γ	C_c	I_x	A	r_{ox}	S	$F_{a\gamma}$
Left Col	0.8	2.0	107	6188	62.50	9.95	2.01	**29.87**
Right Col	0.8	2.0	107	6188	62.50	9.95	2.01	**29.87**
Left Bm	58.9	2.5	107	2500	55.00	6.74	261.93	**2.18**
Right Bm	40.8	2.5	107	2500	55.00	6.74	181.36	**4.54**

CHECK EACH SECTION CAPACITIES

	d_0	d_L	t_f	b_f	t_w	A_f
Left Col	30	30	1	20	0.75	20
Right Col	30	30	1	20	0.75	20
Left Bm	20	40	1	20	0.75	20
Right Bm	20	40	1	20	0.75	20



Section	1	2	3	4	5	6	7	8	9	10	11	12	13
d (in)	30	30	30	30	27	33	40	33	27	20	30	30	30
I (in^4)	10688	10688	10688	10688	8296	13426	20000	13426	8296	4500	10688	10688	10688
A_w (in^2)	22.50	22.50	22.50	22.50	20.00	25.00	30.00	25.00	20.00	15.00	22.50	22.50	22.50
N (kips)	-80.2	-80.2	-80.2	-80.2	119.9	119.9	60.4	59.6	-17.4	80.2	-80.0	-80.0	-80.0
V (kips)	150.92	148.06	148.06	150.92	62.76	62.76	29.90	77.09	77.09	150.92	3.82	3.82	3.82
M (ft-k)	2186.9	1095.0	-17.4	-806.4	-301.1	327.6	657.6	-546.3	-626.3	-806.4	703.1	656.6	610.1
f_a (ksi)				1.28	2.00	1.84	0.86	0.92	0.29	1.46	1.28		
F_a (ksi)				29.87	2.18	2.18	2.18	4.54	4.54	4.54	29.87		
f_v (ksi)		FCC Design		6.71	3.14	2.51	1.00	3.08	3.85	10.06	0.17	FCC Design	
F_v (ksi)				20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00		
f_b (ksi)				13.58	5.81	4.88	7.89	8.14	12.08	21.50	11.84		
F_b (ksi)				30.00	25.79	30.00	30.00	25.79	25.79	25.79	30.00		

$f_a < F_a$ **[Satisfactory]** $f_v < F_v$ **[Satisfactory]** $f_b < F_b$ **[Satisfactory]**

(cont'd)

CHECK COMBINED FLEXURE AND AXIAL FORCE (APP. F7.4, pg 5-104)

$$\left(f_a\ ,\ f_b\right)=\begin{cases}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{f_{bl}}{F_{b\gamma}}\ ,\ for\ \dfrac{f_{a0}}{F_{a\gamma}}\le 0.15\\ Larger\ of\ \begin{pmatrix}\dfrac{f_{a0}}{F_{a\gamma}}+\dfrac{C_m' f_{bl}}{\left(1-\dfrac{f_{a0}}{F'_{e\gamma}}\right)F_{b\gamma}}\\ \dfrac{f_{a0}}{0.6F_y}+\dfrac{f_{bl}}{F_{b\gamma}}\end{pmatrix}\ ,\ for\ \dfrac{f_{a0}}{F_{a\gamma}}>0.15\end{cases}\le 1.3$$

	f_{a0}	f_{bl}	$F_{a\gamma}$	$F_{b\gamma}$	$F_{e\gamma}'$	C_m'	(f_a, f_b)
Left Col	0.00	13.58	29.87	30.00	36959.5	1.00	**0.45**
Right Col	0.00	11.84	29.87	30.00	36959.5	1.00	**0.39**
Left Bm	0.86	13.58	2.18	25.79	2.2	0.74	**1.04**
Right Bm	0.86	21.50	4.54	25.79	4.5	0.85	**1.07**

[Satisfactory]

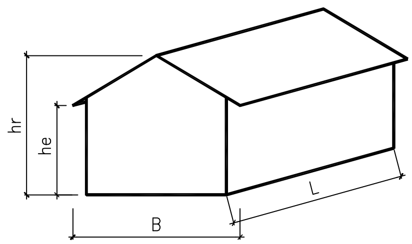
Technical Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.
2. AISC: "Design Guide 25: Frame Design Using Web-Tapered Members, 2010.

Wind Analysis for Low-rise Building, Based on ASCE 7-22

INPUT DATA

Exposure category (B, C or D, ASCE 7-22 26.7.3)		C	
Importance factor (ASCE 7-22 Table 1.5-2)	I_w =	1.00	for all Category
Basic wind speed (ASCE 7-22 26.5.1)	V =	120	*mph, (193.12 kph)*
Topographic factor (ASCE 7-22 26.8 & Figure 26.8-1)	K_{zt} =	1	Flat
Building height to eave	h_e =	25.33	*ft, (7.72 m)*
Building height to ridge	h_r =	29.38	*ft, (8.96 m)*
Building length	L =	50	*ft, (15.24 m)*
Building width, including overhangs	B =	40	*ft, (12.19 m)*
Overhang sloped width	O_h =	1.5	*ft, (0.46 m)*
Effective area of components (or Solar Panel area)	A =	12	ft^2, <== Overhang? (Yes or No) No
		(1.12	*m^2)*



DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	30.98	*kips, (138 kN), SD level (LRFD level), Typ.*
Max horizontal force normal to building length, B, face	=	24.85	*kips, (111 kN)*
Max total horizontal torsional load	=	203.858	*ft-kips, (276 kN-m)*
Max total upward force	=	48.33	*kips, (215 kN)*

ANALYSIS

Velocity pressure

$q_h K_d = (0.00256 K_z K_{zt} K_e V^2) K_d$ = **35.35** x **0.85** = **30.04** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 26.10-1 page 277)

K_z = velocity pressure exposure coefficient evaluated at height, h, (Tab. 26.10-1, pg 277) = **0.96**

K_d = wind directionality factor. (Tab. 26.6-1, for building, page 274) = **0.85**

h = mean roof height = **27.36** ft

K_e = ground elevation factor. (***1.0*** per Sec. 26.9, page 275)

< 60 ft, [Satisfactory] (ASCE 7-22 26.2.1)

< Min (L, B), [Satisfactory] (ASCE 7-22 26.2.2)

Design pressures for MWFRS

$p = q_h K_d [(G C_{pf}) - (G C_{pi})]$

where: p = pressure in appropriate zone. (Eq. 28.3-1, page 311). p_{min} = **16** psf (ASCE 7-22 28.3.6)

$G C_{pf}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 28.3-1, page 295)

$G C_{pi}$ = product of gust effect factor and internal pressure coefficient.(Tab. 26.13-1, Enclosed Building, page 280)

= **0.18** or **-0.18**

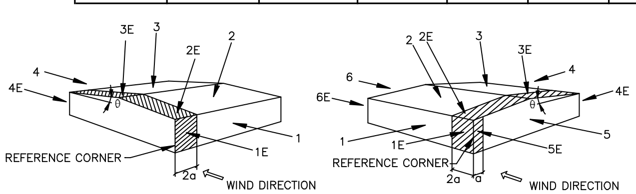
a = width of edge strips, Fig 28.3-1, page 295, MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3] = **4.00** **ft**

Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 11.45			Roof angle θ = 0.00		
	$G C_{pf}$	Net Pressure with		$G C_{pf}$	Net Pressure with	
		$(+GC_{pi})$	$(-GC_{pi})$		$(+GC_{pi})$	$(-GC_{pi})$
1	0.46	8.29	19.10	-0.45	-18.93	-8.11
2	-0.69	-26.14	-15.32	-0.69	-26.14	-15.32
3	-0.42	-17.95	-7.13	-0.37	-16.52	-5.71
4	-0.35	-15.93	-5.11	-0.45	-18.93	-8.11
5				0.40	6.61	17.43
6				-0.29	-14.12	-3.30
1E	0.69	15.37	26.19	-0.48	-19.83	-9.01
2E	-1.07	-37.56	-26.74	-1.07	-37.56	-26.74
3E	-0.60	-23.40	-12.58	-0.53	-21.33	-10.52
4E	-0.52	-21.04	-10.22	-0.48	-19.83	-9.01
5E				0.61	12.92	23.74
6E				-0.43	-18.33	-7.51

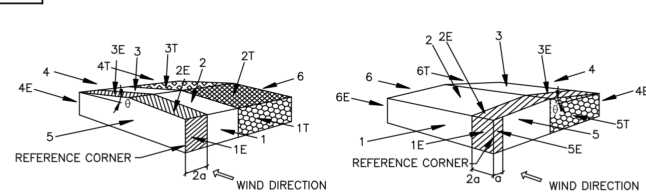
Net Pressures (psf), Torsional Load Cases

Surface	Roof angle θ = 11.45		
	$G C_{pf}$	Net Pressure with	
		$(+GC_{pi})$	$(-GC_{pi})$
1T	0.11	-2.02	8.80
2T	-0.17	-10.52	0.30
3T	-0.10	-8.50	2.32
4T	-0.09	-8.03	2.79
	Roof angle θ = 0.00		
Surface	$G C_{pf}$	Net Pressure with	
		$(+GC_{pi})$	$(-GC_{pi})$
5T	0.10	-2.40	8.41
6T	-0.07	-7.51	3.30



Load Case 1 (Transverse) Load Case 2 (Longitudinal)

Basic Load Cases



Load Case 3 (Transverse) Load Case 4 (Longitudinal)

Torsional Load Cases

Basic Load Case 1 (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)
1	1064	8.82	20.32
2	857	-22.40	-13.13
3	857	-15.38	-6.11
4	1064	-16.95	-5.44
1E	203	3.12	5.31
2E	163	-6.13	-4.37
3E	163	-3.82	-2.05
4E	203	-4.26	-2.07
Σ	Horiz.	31.29	31.29
	Vert.	-46.78	-25.15
Min. wind 28.4.4	Horiz.	21.88	21.88
	Vert.	-32.00	-32.00

$p_{overhang}$ = **-21.03** psf
(ASCE 7-22 28.3.5)

Basic Load Case 2 (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)
2	857	-22.40	-13.13
3	857	-14.16	-4.89
5	888	5.87	15.48
6	888	-12.54	-2.94
2E	163	-6.13	-4.37
3E	163	-3.48	-1.72
5E	206	2.66	4.89
6E	206	-3.77	-1.55
Σ	Horiz.	24.85	24.85
	Vert.	-36.93	-14.39
Min. wind 28.4.4	Horiz.	17.51	17.51
	Vert.	-32.00	-32.00

Torsional Load Case 3 (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)	Torsion (ft-k) ($+GC_{pi}$)	Torsion (ft-k) ($-GC_{pi}$)
1	431	3.57	8.23	37	86
2	347	-9.07	-5.32	-19	-11
3	347	-6.23	-2.47	13	5
4	431	-6.86	-2.20	72	23
1E	203	3.12	5.31	65	111
2E	163	-6.13	-4.37	-26	-18
3E	163	-3.82	-2.05	16	9
4E	203	-4.26	-2.07	90	44
1T	633	-1.28	5.57	16	-70
2T	510	-5.36	0.15	13	0
3T	510	-4.34	1.18	-11	3
4T	633	-5.08	1.77	-64	22
Total Horiz. Torsional Load, M_T				204	204

Torsional Load Case 4 (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)	Torsion (ft-k) ($+GC_{pi}$)	Torsion (ft-k) ($-GC_{pi}$)
2	857	-22.40	-13.13	-9	-5
3	857	-14.16	-4.89	6	2
5	341	2.26	5.95	18	47
6	341	-4.82	-1.13	38	9
2E	163	-6.13	-4.37	28	20
3E	163	-3.48	-1.72	-16	-8
5E	206	2.66	4.89	48	88
6E	206	-3.77	-1.55	68	28
5T	547	-1.31	4.60	13	-45
6T	547	-4.11	1.81	-40	18
Total Horiz. Torsional Load, M_T				152.8	152.8

Design pressures for components and cladding

$p = q_h K_d [(G C_p) - (G C_{pi})]$

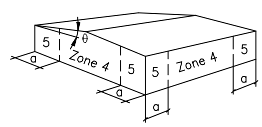
where: p = pressure on component. (Eq. 30.3-1, pg 316)

p_{min} = 16.00 psf (ASCE 7-22 30.2.2)

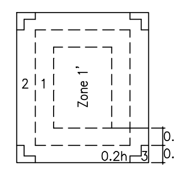
$G C_p$ = external pressure coefficient.

see table below. (ASCE 7-22 30.3.2)

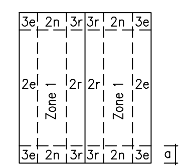
θ = 11.45 °



Walls



Roof θ≤7°



Roof θ>7°

Comp.	Effective Area (ft^2)	Zone 1 GC_p	Zone 1 $-GC_p$	Zone 1' GC_p	Zone 1' $-GC_p$	Zone 2 GC_p	Zone 2 $-GC_p$	Zone 2e GC_p	Zone 2e $-GC_p$	Zone 2n GC_p	Zone 2n $-GC_p$	Zone 2r GC_p	Zone 2r $-GC_p$
	12	0.52	-2.00	-	-	-	-	0.52	-2.00	0.52	-2.89	0.52	-2.89

Comp.	Effective Area (ft^2)	Zone 3 GC_p	Zone 3 $-GC_p$	Zone 3e GC_p	Zone 3e $-GC_p$	Zone 3r GC_p	Zone 3r $-GC_p$	Zone 4 GC_p	Zone 4 $-GC_p$	Zone 5 GC_p	Zone 5 $-GC_p$
	12	-	-	0.52	-2.89	0.52	-3.46	0.99	-1.09	0.99	-1.37

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 1' Positive	Zone 1' Negative	Zone 2 Positive	Zone 2 Negative	Zone 2e Positive	Zone 2e Negative	Zone 2n Positive	Zone 2n Negative	Zone 2r Positive	Zone 2r Negative
	20.93	-65.50					20.93	-65.50	20.93	-92.14	20.93	-92.14

Comp. & Cladding Pressure (psf)	Zone 3 Positive	Zone 3 Negative	Zone 3e Positive	Zone 3e Negative	Zone 3r Positive	Zone 3r Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
			20.93	-92.14	20.93	-109.29	35.03	-38.04	35.03	-46.63

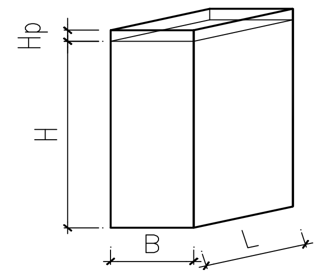
*(The Max Pressure **109.29** psf)*

Purchaser's LOGO	PROJECT :		PAGE :	
(Please email us your LOGO at purchase.)	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

Wind Analysis for Building with h > 60 ft, Based on ASCE 7-22

INPUT DATA

Exposure category (B, C or D, ASCE 7-22 26.7.3)			C	
Importance factor (ASCE 7-22 Table 1.5-2)	I_w	=	1.00	for all Category
Basic wind speed (ASCE 7-22 26.5.1 or 2021 IBC)	V	=	113.842	mph, (183.21 kph)
Topographic factor (ASCE 7-22 26.8 & Figure 26.8-1)	K_{zt}	=	1	Flat
Building height to roof	H	=	157	ft, (47.85 m)
Parapet height	H_P	=	4	ft, (1.22 m)
Building length	L	=	300	ft, (91.44 m)
Building width	B	=	180	ft, (54.86 m)
Natural frequency (ASCE 7-22 26.11)	n_1	=	0.95541	Hz, (1 / T)
Effective area of mullion	A_M	=	55	ft^2 (5.12 m^2)
Effective area of panel	A_P	=	27	ft^2 (2.51 m^2)



DESIGN SUMMARY

Max building horizontal force normal to building length, L, face	=	1999.9	kips, (8896 kN), SD/LRFD level, Typ.
Max overturning moment at wind normal to building length, L, face	=	301994.6	ft-kips, (409450 kN-m)
Max building horizontal force normal to building length, B, face	=	1070.1	kips, (4760 kN)
Max overturning moment at wind normal to building length, B, face	=	264065.0	ft-kips, (358024 kN-m)
Max building upward force	=	2142.0	kips, (9528 kN)
Max building torsion force	=	67497.8	ft-kips, (91515 kN-m)

ANALYSIS

Velocity pressures

$q_z = 0.00256\, K_z K_{zt} K_e V^2$

where: q_z = velocity pressure at height, z. (Eq. 26.10-1, page 277) p_{min} = **16** psf (ASCE 7-22 27.1.5)

K_z = velocity pressure exposure coefficient evaluated at height, z. (Tab. 26.10-1, page 277)

K_d = wind directionality factor. (Tab. 26.6-1, for building, page 274) = **0.85**

z = height above ground

z (ft)	0 - 15	20	25	30	40	50	60	70	80	90	100	120
K_z	0.85	0.90	0.94	0.98	1.04	1.09	1.13	1.17	1.21	1.24	1.26	1.31
K_d q_z (psf)	23.97	25.38	26.51	27.64	29.33	30.74	31.87	33.00	34.12	34.97	35.53	36.94
z (ft)	140	160	161	161	161	161	161	161	161	161		
K_z	1.36	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39		
K_d q_z (psf)	38.35	39.20	39.26	39.26	39.26	39.26	39.26	39.26	39.26	39.26		

Design pressures for MWFRS

$p = q K_d G C_p - q_h K_d (G C_{pi})$

where: p = pressure on surface for rigid building with all h. (Eq. 27.3-1, page 281).

$q K_d$ = K_d q_z for windward wall at height z above the ground, see table above.

$G C_{p\,i}$ = internal pressure coefficient. (Tab. 26.13-1, Enclosed Building, page 280) = **0.18** or **-0.18**

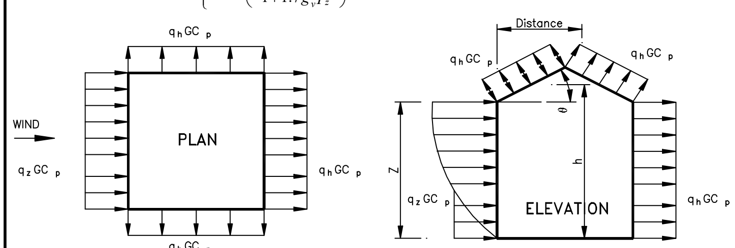
$q_h K_d$ = K_d q_z value at mean roof height, h, for leeward wall, side walls, and roof.

C_p = external pressure coefficient, see right down tables. (Fig. 27.3-1, page 284)

G = gust effect factor (ASCE 7-22 26.11, Page 278)

$$G = \begin{cases} 0.925\left(\dfrac{1+1.7 I_z\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_z}\right), & for \quad n_1 < 1.0 \\ 0.925\left(\dfrac{1+1.7 g_Q I_z Q}{1+1.7 g_v I_z}\right), & for \quad n_1 \geq 1.0 \end{cases} = \mathbf{0.856}$$

I_z =	0.17	z =	94.2	Q =	0.84
z_{min} =	15	g_Q =	3.4	L_z =	617
c =	0.2	g_R =	4.18	β =	0.05
R_h =	0.168	R_B =	0.148	R_L =	0.028
N_1 =	4.62	R_n =	0.053	R =	0.120
h =	157	g_v =	3.4	V_z =	127.5



q G C_p Figure for Gable, Hip Roof, page 275

Fig. 27.3-1 fo θ < 10°, page 275

Roof	h / B	Distance	Cp
To L Face	0.89	80.5	-1.01
To L Face	0.89	161	-0.74
To L Face	0.89	180	-0.66
To L Face	0.89	180	
Roof	**h / L**	**Distance**	**Cp**
To B Face	0.54	80.5	-0.91
To B Face	0.54	161	-0.89
To B Face	0.54	300	-0.51
To B Face	0.54	300	

Fig. 27.3-1, page 275

Wall	Direction	L / B	Cp
Windward Wall	All	All	0.80
Leeward Wall	To L Dir	0.60	-0.50
Leeward Wall	To B Dir	1.67	-0.37
Side Wall	All	All	-0.70

(cont'd)

Design pressures for MWFRS parapets

$p_p = q_p K_d G C_{pn}$

where: p_p = combined net pressure on parapet. (Eq. 27.3-3, page 286).

$q_p K_d$ = velocity pressure evaluated at the top of the parapet = **46.18** **x** **0.85** **=** **39.26** psf

$G C_{pn}$ = internal pressure coefficient. (page 286) **=** **1.50** or **-1.00**

Hence, MWFRS Net Pressures are given by following tables (ASCE 7-22 27.3.1 & 27.3.4, Page 281 & 286)

Surface	**z (ft)**	**P (psf) with GC_{Pi}**	**P (psf) with $-GC_{Pi}$**
Windward Wall	0 - 15	9.35	23.49
	20	10.32	24.45
	25	11.09	25.22
	30	11.86	26.00
	40	13.02	27.16
	50	13.99	28.12
	60	14.76	28.89
	70	15.53	29.67
	80	16.31	30.44
	90	16.89	31.02
	100	17.27	31.41
	120	18.24	32.37
	140	19.20	33.34
	160	19.78	33.92
	161	19.82	33.95

Surface	**z (ft)**	**P (psf) with GC_{Pi}**	**P (psf) with $-GC_{Pi}$**
Side Wall	All	-30.59	-16.46

Parapet	**P (psf) with GC_{Pn}**	**P (psf) with $-GC_{Pn}$**
Normal to Face	58.88	-39.26

Normal to L Face		**P (psf) with**		**Normal to B Face**		**P (psf) with**	
Surface	**z (ft)**	**GC_{Pi}**	**$-GC_{Pi}$**	**Surface**	**z (ft)**	**GC_{Pi}**	**$-GC_{Pi}$**
Leeward	All	-23.87	-9.74	Leeward	All	-19.39	-5.26

Normal to L Face		**P (psf) with**		**Normal to B Face**		**P (psf) with**	
Surface	**Dist. (ft)**	**GC_{Pi}**	**$-GC_{Pi}$**	**Surface**	**Dist. (ft)**	**GC_{Pi}**	**$-GC_{Pi}$**
Roof	0 - 80.5	-41.03	-26.90	Roof	0 - 80.5	-37.66	-23.53
	161	-32.01	-17.88		161	-36.82	-22.69
	180	-29.17	-15.04		300	-24.36	-10.23

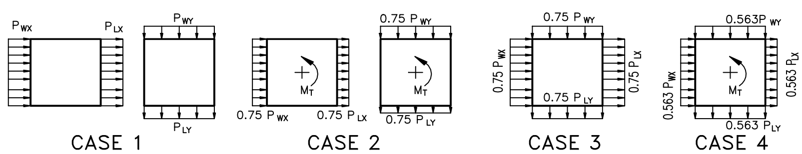


Figure 27.3-8, page 291

Base Forces		Normal to L Face Case 1	Normal to L Face Case 2	Normal to B Face Case 1	Normal to B Face Case 2	Wind with Angle Case 3	Wind with Angle Case 4	ASCE-7
V_{Base}	(kips)	2000	1500	1070	803	2303	1277	
M_{Base}	(ft - kips)	301995	226496	264065	198049	424545	225854	Fig. 27.4-8
M_T	(ft - kips)	0	67498	0	21669	0	66935	Page 271
F_{Upward}	(kips)	1549	1162	1307	980	2142	1141	
V_{min}	(kips)	773	773	464	464	927	901	Min. wind
$F_{Up,min}$	(kips)	864	864	864	864	864	864	27.4.7

Design pressures for components and cladding

$p = q (G C_p) - q_i (G C_{pi})$

where: p = pressure on component for building with h > 60 ft. (Eq. 30.5-1, page 350)

p_{min} = 16.00 psf (ASCE 7-22 30.2.2)

q = q_z for windward wall at height z above the ground, see table above.

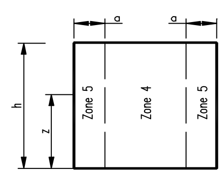
q_h = q_z value at mean roof height, h, for leeward wall, side walls, and roof.

$G C_{p\,i}$ = internal pressure coefficient. (Tab. 26.13-1, pg 271) = **0.18** or **-0.18**

a = Zone width = MAX[MIN(0.1B, 0.1L), 3] = **18.0** ft, (Fig 30.5-1 note 8, pg 363)

$G C_p$ = external pressure coefficient. (Fig 30.5-1 note 8, pg 363)

Wall Comp.	**Actual Effective Area (ft^2)**	**Zone 4 GC_P**	**Zone 4 $-GC_P$**	**Zone 5 GC_P**	**Zone 5 $-GC_P$**
Mullion	55	0.81	-0.84	0.81	-1.55
Panel	27	0.87	-0.88	0.87	-1.73

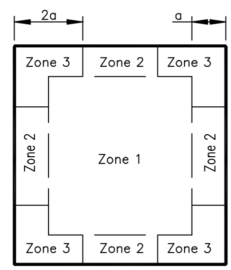


WALL ELEVATION

cont'd

z (ft)	Mullion Pressure (psf)				Panel Pressure (psf)			
	Zone 4		Zone 5		Zone 4		Zone 5	
	Positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
0 - 15	23.63	-39.93	23.63	-67.86	25.22	-41.66	25.22	-74.80
20	25.02	-39.93	26.70	-67.86	26.70	-41.66	26.70	-74.80
25	26.13	-39.93	27.89	-67.86	27.89	-41.66	27.89	-74.80
30	27.24	-39.93	29.07	-67.86	29.07	-41.66	29.07	-74.80
40	28.91	-39.93	30.86	-67.86	30.86	-41.66	30.86	-74.80
50	30.30	-39.93	32.34	-67.86	32.34	-41.66	32.34	-74.80
60	31.41	-39.93	33.53	-67.86	33.53	-41.66	33.53	-74.80
70	32.52	-39.93	34.71	-67.86	34.71	-41.66	34.71	-74.80
80	33.64	-39.93	35.90	-67.86	35.90	-41.66	35.90	-74.80
90	34.47	-39.93	36.79	-67.86	36.79	-41.66	36.79	-74.80
100	35.03	-39.93	37.38	-67.86	37.38	-41.66	37.38	-74.80
120	36.42	-39.93	38.87	-67.86	38.87	-41.66	38.87	-74.80
140	37.81	-39.93	40.35	-67.86	40.35	-41.66	40.35	-74.80
160	38.64	-39.93	41.24	-67.86	41.24	-41.66	41.24	-74.80
161	38.70	-39.93	41.30	-67.86	41.30	-41.66	41.30	-74.80

Roof	Effective Area (ft^2)	Zone 1 - GC_P	Zone 2 - GC_P	Zone 3 - GC_P
Components and Cladding	0	-1.40	-2.30	-3.20
	10	-1.40	-2.30	-3.20
	59	-1.17	-1.98	-2.79
	108	-1.10	-1.87	-2.65
	157	-1.05	-1.81	-2.57
	206	-1.01	-1.76	-2.50
	255	-0.99	-1.72	-2.45
	304	-0.96	-1.69	-2.41
	353	-0.94	-1.66	-2.38
	402	-0.93	-1.64	-2.35
	451	-0.91	-1.62	-2.32
	500	-0.90	-1.60	-2.30
	38016	-0.90	-1.60	-2.30



ROOF PLAN

Roof	Effective Area (ft^2)	Net Pressure (psf)		
		Zone 1	Zone 2	Zone 3
Components and Cladding	0	-62.02	-97.35	-132.68
	10	-62.02	-97.35	-132.68
	59	-53.12	-84.89	-116.65
	108	-50.09	-80.64	-111.19
	157	-48.21	-78.01	-107.82
	206	-46.85	-76.10	-105.36
	255	-45.77	-74.60	-103.44
	304	-44.89	-73.37	-101.85
	353	-44.14	-72.32	-100.50
	402	-43.49	-71.41	-99.32
	451	-42.91	-70.60	-98.29
	500	-42.40	-69.88	-97.35
	38016	-42.40	-69.88	-97.35

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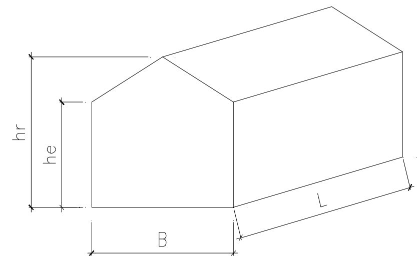
Wind Analysis for Enclosed Building, Based on DSA IR 16-7 (SEAOC)

INPUT DATA

Exposure category (B, C or D)		C	
Importance factor, IR 16-7 3.1, (0.87, 1.0 or 1.15)	I =	1.00	Category II
Basic wind speed (IBC Tab 1609.3.1V_{3S})	V =	85	mph
Topographic factor (IR 16-7 3.3.2)	K_{zt} =	1	Flat
Building height to eave	h_e =	11	ft
Building height to ridge	h_r =	18	ft
Building length	L =	100	ft
Building width	B =	50	ft
Effective area of components	A =	28	ft^2



DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	18.00 kips
Max overturning moment at wind normal to building length, L, face	=	267.64 ft - kips
Max horizontal force normal to building length, B, face	=	10.71 kips
Max overturning moment at wind normal to building length, B, face	=	79.19 ft - kips
Max total horizontal torsional load	=	250.01 ft-kips
Max total upward force	=	58.80 kips

ANALYSIS

Velocity pressure (IR 16-7 Table 1)

q_s = 18.50 psf

Design pressures for MWFRS (IR 16-7 Equation 1)

$$P_{net} = q_s K_z C_{net} [I K_{zt}]$$

where: $P_{net, min}$ = **10** psf on projected area (IR 16-7 3.2)

K_z = velocity pressure exposure coefficient evaluated at height, h, (IR 16-7 3.3.2) = **0.85**

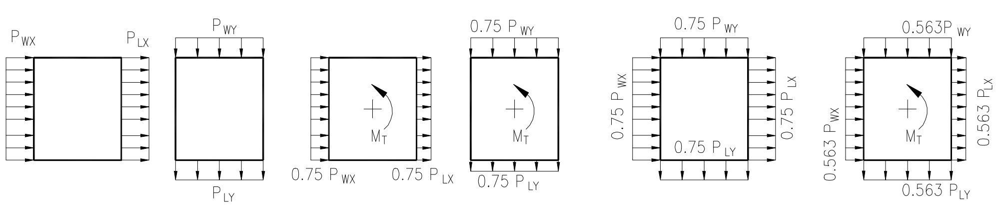
h = mean roof height = **14.50** ft **< 75 ft, [Satisfactory] (IR 16-7 2.3)**

(SEAOC updated limitation to 75 ft)

C_{net} (IR 16-7 Table 2) (Roof angle θ = 15.64 °)

C_{net}, Area, & P_{net}		Normal to L Face		Normal to B Face		Normal to L Face		Normal to B Face	
		Case 1	Case 2	Case 1	Case 2	Case 3	Case 4	Case 3	Case 4
Windward Wall	C_{net}	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43
	Area (ft^2)	1100	1100	725	725	1100	1100	725	725
	P_{net} (psf)	7	7	7	7	7	7	7	7
Leeward Wall	C_{net}	-0.51	-0.51	-0.51	-0.51	-0.51	-0.51	-0.51	-0.51
	Area (ft^2)	1100	1100	725	725	1100	1100	725	725
	P_{net} (psf)	-8	-8	-8	-8	-8	-8	-8	-8
Side Wall	C_{net}	-0.66	-0.66	-0.66	-0.66	-0.66	-0.66	-0.66	-0.66
	Area (ft^2)	725	725	1100	1100	725	725	1100	1100
	P_{net} (psf)	-10	-10	-10	-10	-10	-10	-10	-10
Windward Roof	C_{net}	-0.84	-0.12						
	Area (ft^2)	2596	2596						
	P_{net} (psf)	-13	-2						
Leeward Roof	C_{net}	-0.66	-0.66						
	Area (ft^2)	2596	2596						
	P_{net} (psf)	-10	-10						

(SEAOC updated more 45° by wall)



CASE 1 CASE 2 CASE 3 CASE 4

cont'd

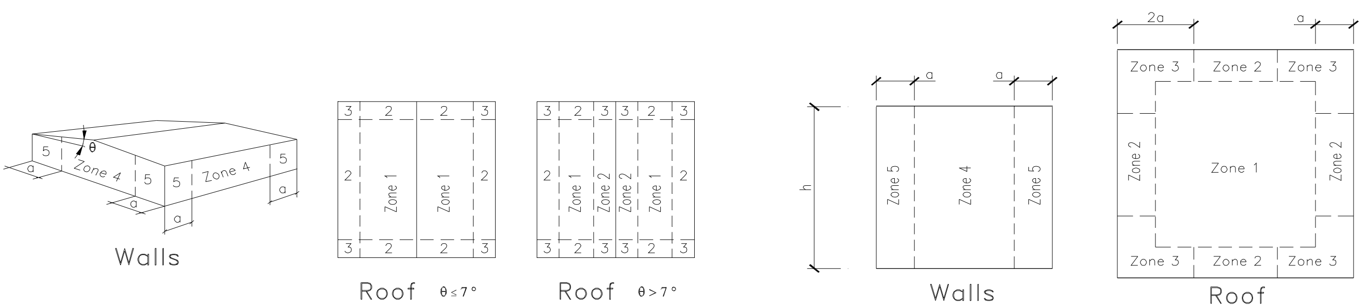
Base Forces		Normal to L Face		Normal to B Face		Normal to L Face		Normal to B Face		Minimum	
		Case 1	Case 2	Case 1	Case 2	Case 3	Case 4	Case 3	Case 4	to L	to B
V_{Base}	(kips)	14.32	16.67	10.71	8.04	12.19	9.15	8.04	6.03	18.00	7.25
M_{Base}	(ft - kips)	234.34	-267.64	79.19	59.39	67.06	50.34	59.39	44.58	162.00	53.58
M_T	(ft - kips)	0.00	250.01	0.00	60.27	0.00	182.52	0.00	182.52		
F_{Upward}	(kips)	-58.80	-22.93								

Design pressures for components and cladding

$$P_{net} = q_s\, K_z\, C_{net}\, [\, I\, K_{zt}]$$

where: $P_{net,\,min}$ = **10** psf (IR 16-7 3.2)

a = width of edge strips, ASCE 7 Fig 6-10, note 9, page 54, MAX[MIN(0.1B, 0.4h), 0.04B,3] = **5.00 ft**



Zone Designation for h ≤ 60'-0"　　　　Zone Designation for h > 60'-0"

C_{net} (IR 16-7 Table 2)

C_{net}	Effective Area (ft^2)	Zone 1		Zone 2		Zone 3		Zone 4		Zone 5	
		C_{net}	$-C_{net}$	C_{net}	$-C_{net}$	C_{net}	$-C_{net}$	C_{net}	$-C_{net}$	C_{net}	$-C_{net}$
Roof	28	0.55	-0.98	0.55	-1.58	0.55	-2.39	0.99	-1.08	0.99	-1.32
Overhang			-1.43		-1.87		-2.95				

Pressure (psf)	Zone 1		Zone 2		Zone 3		Zone 4		Zone 5	
	Positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
Roof	10.00	-15.47	10.00	-24.81	10.00	-37.64	15.58	-16.99	15.58	-20.77
Overhang		-22.51		-29.40		-46.32				

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Seismic Analysis Based on IBC 09 (Equivalent Lateral-Force Procedure, ASCE 7-05 12.8 & Supplement 2)

INPUT DATA

Typical floor height	h =	9	ft
Typical floor weight	w_x =	800	k
Number of floors	n =	40	
Importance factor (ASCE 11.5.1)	I =	1.25	(IBC Tab.1604.5)
Building location	Zip Code	92868	
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02	
The coefficient(ASCE Tab. 12.2.1)	R =	5.5	

DESIGN SUMMARY

Total base shear

$V = 0.07\ W$, (SD) = **2,189** k, (SD)

$= 0.05\ W$, (ASD) = **1,564** k, (ASD)

Seismic design category = **D**

Latitude: 33.784

Longitude: -117.874

S_S = 137.545 %g , S_{ms} = 1.375 g , F_a = 1.000

S_1 = 49.629 %g , S_{m1} = 0.746 g , F_v = 1.504

S_{DS} = **0.917** g , S_{D1} = **0.498** g

h_n = 360.0 ft k = 1.58 , (ASCE 12.8.3, pg 130) x = 0.75 , (ASCE Tab 12.8-2)

W = 32,000 k $\Sigma w_x h^k$ = 137,376,298 $T_a = C_t (h_n)^x$ = 1.65 Sec, (ASCE 12.8.2.1)

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Level Name	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft	Diaphragm force ΣF_i k	ΣW_i k	F_{px} k
40	Roof		360.0	800	8,570,835	0.062	136.6			136.6	800	183
		9.00						136.6				
39	40th		351.0	800	8,235,486	0.060	131.2		1,229	267.8	1,600	183
		9.00						267.8				
38	39th		342.0	800	7,905,058	0.058	126.0		3,639	393.8	2,400	183
		9.00						393.8				
37	38th		333.0	800	7,579,605	0.055	120.8		7,183	514.5	3,200	183
		9.00						514.5				
36	37th		324.0	800	7,259,185	0.053	115.7		11,814	630.2	4,000	183
		9.00						630.2				
35	36th		315.0	800	6,943,854	0.051	110.6		17,486	740.9	4,800	183
		9.00						740.9				
34	35th		306.0	800	6,633,675	0.048	105.7		24,154	846.6	5,600	183
		9.00						846.6				
33	34th		297.0	800	6,328,712	0.046	100.8		31,773	947.4	6,400	183
		9.00						947.4				
32	33th		288.0	800	6,029,030	0.044	96.1		40,299	1,043.5	7,200	183
		9.00						1,043.5				
31	32th		279.0	800	5,734,699	0.042	91.4		49,691	1,134.9	8,000	183
		9.00						1,134.9				
30	31th		270.0	800	5,445,792	0.040	86.8		59,904	1,221.6	8,800	183
		9.00						1,221.6				
29	30th		261.0	800	5,162,383	0.038	82.3		70,899	1,303.9	9,600	183
		9.00						1,303.9				
28	29th		252.0	800	4,884,554	0.036	77.8		82,634	1,381.7	10,400	183
		9.00						1,381.7				
27	28th		243.0	800	4,612,387	0.034	73.5		95,069	1,455.2	11,200	183
		9.00						1,455.2				
26	27th		234.0	800	4,345,970	0.032	69.3		108,166	1,524.5	12,000	183
		9.00						1,524.5				
25	26th		225.0	800	4,085,396	0.030	65.1		121,887	1,589.6	12,800	183
		9.00						1,589.6				
24	25th		216.0	800	3,830,763	0.028	61.0		136,193	1,650.6	13,600	183
		9.00						1,650.6				
23	24th		207.0	800	3,582,175	0.026	57.1		151,048	1,707.7	14,400	183
		9.00						1,707.7				
22	23th		198.0	800	3,339,741	0.024	53.2		166,417	1,760.9	15,200	183
		9.00						1,760.9				
21	22th		189.0	800	3,103,579	0.023	49.5		182,266	1,810.4	16,000	183
		9.00						1,810.4				
20	21th		180.0	800	2,873,813	0.021	45.8		198,559	1,856.2	16,800	183
		9.00						1,856.2				
19	20th		171.0	800	2,650,577	0.019	42.2		215,264	1,898.4	17,600	183
		9.00						1,898.4				

Level No.	Level Name	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	F_x k	V_x k	O. M. k-ft	ΣF_i k	ΣW_i k	F_{px} k
												(cont'd)
18	19th		162.0	800	2,434,015	0.018	38.8		232,350	1,937.2	18,400	183
		9.00						1,937.2				
17	18th		153.0	800	2,224,281	0.016	35.4		249,784	1,972.6	19,200	183
		9.00						1,972.6				
16	17th		144.0	800	2,021,542	0.015	32.2		267,538	2,004.8	20,000	183
		9.00						2,004.8				
15	16th		135.0	800	1,825,982	0.013	29.1		285,581	2,033.9	20,800	183
		9.00						2,033.9				
14	15th		126.0	800	1,637,798	0.012	26.1		303,886	2,060.0	21,600	183
		9.00						2,060.0				
13	14th		117.0	800	1,457,210	0.011	23.2		322,427	2,083.2	22,400	183
		9.00						2,083.2				
12	13th		108.0	800	1,284,460	0.009	20.5		341,176	2,103.7	23,200	183
		9.00						2,103.7				
11	12th		99.0	800	1,119,820	0.008	17.8		360,109	2,121.6	24,000	183
		9.00						2,121.6				
10	11th		90.0	800	963,594	0.007	15.4		379,203	2,136.9	24,800	183
		9.00						2,136.9				
9	10th		81.0	800	816,129	0.006	13.0		398,435	2,149.9	25,600	183
		9.00						2,149.9				
8	9th		72.0	800	677,826	0.005	10.8		417,784	2,160.7	26,400	183
		9.00						2,160.7				
7	8th		63.0	800	549,156	0.004	8.8		437,231	2,169.5	27,200	183
		9.00						2,169.5				
6	7th		54.0	800	430,681	0.003	6.9		456,756	2,176.3	28,000	183
		9.00						2,176.3				
5	6th		45.0	800	323,094	0.002	5.1		476,343	2,181.5	28,800	183
		9.00						2,181.5				
4	5th		36.0	800	227,276	0.002	3.6		495,976	2,185.1	29,600	183
		9.00						2,185.1				
3	4th		27.0	800	144,408	0.001	2.3		515,642	2,187.4	30,400	183
		9.00						2,187.4				
2	3rd		18.0	800	76,206	0.001	1.2		535,328	2,188.6	31,200	183
		9.00						2,188.6				
1	2nd		9.0	800	25,552	0.000	0.4		555,026	2,189.0	32,000	183
		9.00						2,189.0				
	Ground		0.0						574,727			

(Spanning headers: "Lateral force @ each level" over F_x, V_x, O. M.; "Diaphragm force" over ΣF_i, ΣW_i, F_{px}.)

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Four Story Seismic Analysis Based on CBC 2010 Chapter A

Determine Base Shear (Derived from ASCE 7-05 Sec. 12.8, Supplement 2, & CBC 2010 Sec. 1614A.1.7)

$V = \text{MAX}\{ \text{MIN} [S_{D1}I / (RT) \;,\; S_{DS}\, I / R] \;,\; \text{MAX}(0.044S_{DS}I \;,\; 0.01) \;,\; 0.5S_1\, I / R \}\, W$

$= \text{MAX}\{ \text{MIN}[0.15W \;,\; 0.18W] ,\; 0.06W \;,\; 0.00W \}$ ^ (for $S_1 \geq 0.6$ g only)

= 0.15 W, (SD)

= 0.11 W, (ASD) = **1454.89** kips

Where			
	S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
	S_{D1} =	0.54	(ASCE 7-05 Sec 11.4.4)
	S_1 =	0.55	(ASCE 7-05 Sec 11.4.1)
	R =	8	(ASCE 7-05 Tab 12.2-1)
	I =	1.5	(IBC 09 Tab 1604A.5 & ASCE 7-05 Tab 11.5-1)
	C_t =	0.028	(0.028 for steel MRF, 0.016 for concrete MRF, & 0.03 steel EBF)
	h_n =	52.0	ft
	x =	0.8	(ASCE 7-05 Tab 12.8-2)

$T = C_t\,(h_n)^x =$ 0.661 sec, (ASCE 7-05 Sec 12.8.2.1)

Calculate Vertical Distribution of Forces & Allowable Elastic Drift (ASCE 7-05, Sec 12.8.3 & 12.8.6)

Level	W_x	h_x	h_x^k	$W_x h_x^k$	F_x , ASD (12.8-11)		$\delta_{xe,allowable,\,ASD}$
Roof	120	52	71.4	8571	27.3	(0.23 Wx)	0.3
4TH	3500	40	53.8	188278	598.9	(0.17 Wx)	0.3
3RD	4050	28	36.6	148199	471.4	(0.12 Wx)	0.3
2ND	5620	16	20.0	112349	357.4	(0.06 Wx)	0.4
	13290.0			457396	1454.9		

Where
k = 1 for T <= 0.5
k = 0.5 T + 0.75 for T @ (0.5 , 2.5)
k = 2 for T >= 2.5

$\delta_{xe,allowable,\,ASD} = \Delta_a\, I / (1.4\, C_d)$, (ASCE 7-05 Sec 12.8.6)

C_d = 5.5 ,(ASCE 7-05 Tab 12.2-1)

Δ_a = 0.01 h_{sx}, (ASCE 7-05 Tab 12.12-1, & Sec 12.12)

Calculate Diaphragm Forces (ASCE 7-05, Sec 12.10.1.1)

Level	W_x	ΣW_x	F_x	ΣF_x	F_{px} , ASD, (12.10-1)	
Roof	120.0	120.0	27.3	27.3	27.3	(0.23 Wx)
4TH	3500.0	3620.0	598.9	626.1	672.0	(0.19 Wx)
3RD	4050.0	7670.0	471.4	1097.5	777.6	(0.19 Wx)
2ND	5620.0	13290.0	357.4	1454.9	1079.0	(0.19 Wx)
	13290.0		1454.9			

Where $F_{min} = 0.2\, S_{DS}\, I\, W_x / 1.5$, ASD

$F_{max} = 0.4\, S_{DS}\, I\, W_x / 1.5$, ASD

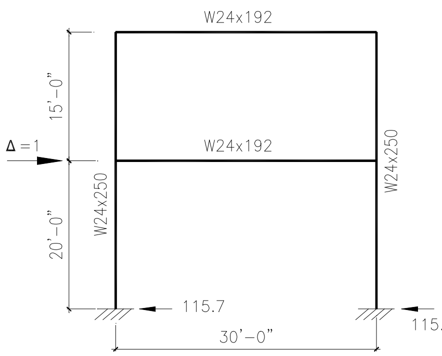
Method to Calculate Redundancy Factor, ρ, Based on IBC 2009

There are big updated to calculate redundancy factor from new IBC 2009. (ASCE 7-05 Sec 12.3.4)

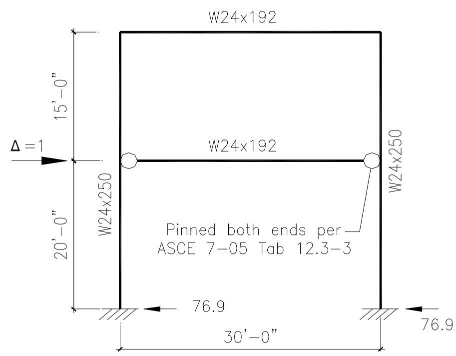
ρ = 1.3 , only for Seismic Design Categories D through F, and an individual element removed will be causing the remaining structural to suffer a reduction of **Story Strength** of more than 35% or creating an extreme torsional irregularity.

ρ = 1.0 , all other cases.

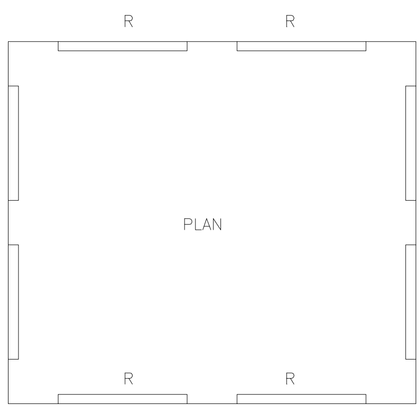
Note: To verify Redundancy Factor 1.0, the relative **Rigidity** of each frame at calculation level have to be known, since the Code Redundancy Provision Committee envisioned Story Strength as Rigidity, and SEAOC Seismology Committee endorsed this interpretation (SEAOC Seismic Design Manual -1, page 72).



R = 115.7 + 115.7 = 231.4



R = 76.9 + 76.9 = 153.8



$$\frac{4\times 231.4-(3\times 231.4+153.8)}{4\times 231.4} = 8.4\% \le 33\%$$

$$\rho = 1.0$$

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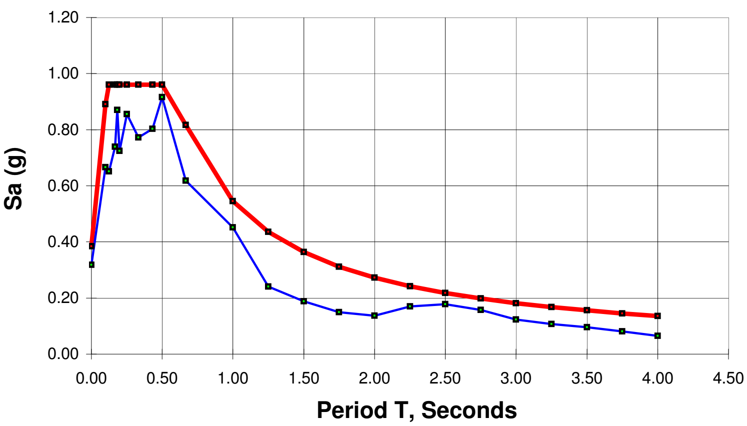
Response Spectra Analysis Based on IBC 09

INPUT DATA

SEISMIC DESIGN PARAMETER	S_{DS} =	0.96	(ASCE 7-05 11.4.4)
SEISMIC DESIGN PARAMETER	S_{D1} =	0.545	(ASCE 7-05 11.4.4)
LONG-PERIOD TRANSITION PERIOD	T_L =	8	(ASCE 7-05 11.4.5)

RESPONSE SPECTRA



ANALYSIS

Station	T (s)	S_a(g) (IBC/CBC)	A/g (El Centro)
0	0.00	0.38	0.32
1	0.10	0.89	0.67
2	0.13	0.96	0.65
3	0.17	0.96	0.74
4	0.18	0.96	0.87
5	0.20	0.96	0.72
6	0.25	0.96	0.86
7	0.33	0.96	0.77
8	0.43	0.96	0.80
9	0.50	0.96	0.92
10	0.67	0.82	0.62
11	1.00	0.55	0.45
12	1.25	0.44	0.24
13	1.50	0.36	0.19
14	1.75	0.31	0.15
15	2.00	0.27	0.14
16	2.25	0.24	0.17
17	2.50	0.22	0.18
18	2.75	0.20	0.16
19	3.00	0.18	0.12
20	3.25	0.17	0.11
21	3.50	0.16	0.10
22	3.75	0.15	0.08
23	4.00	0.14	0.06

STATIC ANALYSIS PERIOD (ASCE 7-05 12.8.2)

APPROXIMATE FOUNDATIONAL PERIOD	$T_a = C_t (h_n)^x$ =	0.799	sec, (ASCE 7-05 Sec 12.8.2.1)
SUBSTANTIATED ANALYSIS PERIOD	T_b =	1.095	sec, (may from RISA, ETABS, RAM)

T = Max[T_a , Min(T_b , C_uT_a)] = 1.095 sec.

where C_u = 1.400 , (ASCE 7-05 Tab. 12.8-1)

SCALE FACTOR

BASE SHEAR BY ELASTIC DYNAMIC ANALYSIS (ASCE 7-05 12.9)	V_{DX} =	3000	kips	V_{DY} = 3500	kips
BASE SHEAR BY STATIC ANALYSIS (ASCE 7-05 12.8)	V_S =	735.09	kips		
RESPONSE MODIFICATION COEFFICIENT (ASCE 7 Tab 12.2-1)	R =	8			
IMPORTANCE FACTOR	I =	1.5		**SF**$_X = V_X / V_D$ = 0.245	
DSA/OSHPD PROJECT (CBC 2010 Chapter A) ?	== >	Yes		**SF**$_Y = V_Y / V_D$ = 0.210	

V_X = Max(0.85 V_S , V_{DX} I / R) = 624.83 kips, (ASCE 7-05 12.9.2 & 12.9.4)
V_X = Max(1.0 V_S , V_{DX} I / R) = 735.09 kips, (ASCE 7-05 12.9.2 & CBC 10 1614A.1.9)

V_Y = Max(0.85 V_S , V_{DY} I / R) = 656.25 kips, (ASCE 7-05 12.9.2 & 12.9.4)
V_Y = Max(1.0 V_S , V_{DY} I / R) = 735.09 kips, (ASCE 7-05 12.9.2 & CBC 10 1614A.1.9)

Notes:

See following link for more information.

http://www.engineering-international.com/NonlinearGuide.pdf

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Three Story Seismic Analysis Based on IBC 09 / CBC 10

Determine Base Shear (Derived from ASCE 7-05 Sec. 12.8 & Supplement 2)

$V = \text{MAX}\{\ \text{MIN}\ [\ S_{D1} I / (RT)\ ,\ S_{DS}\ I / R\ ]\ ,\ \text{MAX}(0.044 S_{DS} I\ ,\ 0.01)\ ,\ 0.5 S_1\ I / R\ \}\ W$

$= \text{MAX}\{\ \text{MIN}[\ 0.29W\ ,\ 0.15W\ ]\ ,\ 0.04W\ ,\ 0.00W\ \}$ ^ (for $S_1 \geq 0.6$ g only)

= 0.15 W, (SD)

= 0.11 W, (ASD) = **1067.60** kips

Where

S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
S_{D1} =	0.55	(ASCE 7-05 Sec 11.4.4)
S_1 =	0.54	(ASCE 7-05 Sec 11.4.1)
R =	6.5	(ASCE 7-05 Tab 12.2-1)
I =	1	(IBC 09 Tab 1604.5 & ASCE 7-05 Tab 11.5-1)
C_t =	0.02	(ASCE 7-05 Tab 12.8-2)
h_n =	36.0	ft
x =	0.75	(ASCE 7-05 Tab 12.8-2)

$T = C_t (h_n)^x =$ 0.294 sec, (ASCE 7-05 Sec 12.8.2.1)

Calculate Vertical Distribution of Forces & Allowable Elastic Drift (ASCE 7-05, Sec 12.8.3 & 12.8.6)

Level	**W_x**	**h_x**	**h_x^k**	**$W_x h_x^k$**	**F_x** , ASD (12.8-11)		$\delta_{xe,allowable,\ ASD}$
Roof	120	36	36.0	4320	26.0	(0.22 Wx)	0.5
3RD	4400	24	24.0	105600	636.5	(0.14 Wx)	0.5
2ND	5600	12	12.0	67200	405.1	(0.07 Wx)	0.5
	10120.0			177120	1067.6		

Where

k = 1	for T <= 0.5	
k = 0.5 T + 0.75	for T @ (0.5 , 2.5)	
k = 2	for T >= 2.5	

$\delta_{xe,allowable,\ ASD} = \Delta_a\ I / (1.4\ C_d)$, (ASCE 7-05 Sec 12.8.6)

C_d = 4 ,(ASCE 7-05 Tab 12.2-1)

Δ_a = 0.02 h_{sx}, (ASCE 7-05 Tab 12.12-1)

Calculate Diaphragm Forces (ASCE 7-05, Sec 12.10.1.1)

Level	**W_x**	**ΣW_x**	**F_x**	**ΣF_x**	**F_{px}** , ASD, (12.10-1)	
Roof	120.0	120.0	26.0	26.0	26.0	(0.22 Wx)
3RD	4400.0	4520.0	636.5	662.6	645.0	(0.15 Wx)
2ND	5600.0	10120.0	405.1	1067.6	716.8	(0.13 Wx)
	10120.0		1067.6			

Where $F_{min} = 0.2\ S_{DS}\ I\ W_x / 1.5$, ASD

$F_{max} = 0.4\ S_{DS}\ I\ W_x / 1.5$, ASD

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Two Story Seismic Analysis Based on IBC 09 / CBC 10

Determine Base Shear (Derived from ASCE 7-05 Sec. 12.8 & Supplement 2)

V = MAX{ MIN [S_{D1}I / (RT) , S_{DS} I / R] , MAX(0.044S_{DS}I , 0.01) , 0.5S_1 I / R } W

= MAX{ MIN[0.39W , 0.15W] , 0.04W , 0.00W }

^ (for $S_1 \geq 0.6$ g only)

= 0.15 W, (SD)

= 0.11 W, (ASD) = **603.43** kips

Where

S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
S_{D1} =	0.55	(ASCE 7-05 Sec 11.4.4)
S_1 =	0.54	(ASCE 7-05 Sec 11.4.1)
R =	6.5	(ASCE 7-05 Tab 12.2-1)
I =	1	(IBC 09 Tab 1604.5 & ASCE 7-05 Tab 11.5-1)
C_t =	0.02	(ASCE 7-05 Tab 12.8-2)
h_n =	24.0	ft
x =	0.75	(ASCE 7-05 Tab 12.8-2)

T = $C_t (h_n)^x$ = 0.217 sec, (ASCE 7-05 Sec 12.8.2.1)

Calculate Vertical Distribution of Forces & Allowable Elastic Drift (ASCE 7-05, Sec 12.8.3 & 12.8.6)

Level	**W_x**	**h_x**	**h_x^k**	**$W_x h_x^k$**	**F_x**, ASD (12.8-11)		$\delta_{xe,allowable,\ ASD}$
Roof	120	24	24.0	2880	24.8	(0.21 Wx)	0.5
2ND	5600	12	12.0	67200	578.6	(0.10 Wx)	0.5
	5720.0			70080	603.4		

Where

k = 1	for T <= 0.5
k = 0.5 T + 0.75	for T @ (0.5 , 2.5)
k = 2	for T >= 2.5

$\delta_{xe,allowable,\ ASD} = \Delta_a$ I / (1.4 C_d), (ASCE 7-05 Sec 12.8.6)

C_d = 4 ,(ASCE 7-05 Tab 12.2-1)

Δ_a = 0.02 h_{sx}, (ASCE 7-05 Tab 12.12-1)

Calculate Diaphragm Forces (ASCE 7-05, Sec 12.10.1.1)

Level	**W_x**	**ΣW_x**	**F_x**	**ΣF_x**	**F_{px}**, ASD, (12.10-1)	
Roof	120.0	120.0	24.8	24.8	24.8	(0.21 Wx)
2ND	5600.0	5720.0	578.6	603.4	716.8	(0.13 Wx)
	5720.0		603.4			

Where $F_{min} = 0.2\ S_{DS}\ I\ W_x / 1.5$, ASD

$F_{max} = 0.4\ S_{DS}\ I\ W_x / 1.5$, ASD

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One Story Seismic Analysis Based on IBC 09 / CBC 10

Determine Base Shear (Derived from ASCE 7-05 Sec. 12.8 & Supplement 2)

V = MAX{ MIN [$S_{D1}I$ / (RT) , S_{DS} I / R] , MAX(0.044S_{DS}I , 0.01) , 0.5S_1 I / R } W

= MAX{ MIN[0.58W , 0.15W] , 0.04W , 0.00W }

^ (for $S_1 \geq 0.6$ g only)

= 0.15 W, (SD)

= 0.11 W, (ASD) = **12.66** kips

Where

S_{DS} =	0.96	(ASCE 7-05 Sec 11.4.4)
S_{D1} =	0.55	(ASCE 7-05 Sec 11.4.4)
S_1 =	0.54	(ASCE 7-05 Sec 11.4.1)
R =	6.5	(ASCE 7-05 Tab 12.2-1)
I =	1	(IBC 09 Tab 1604.5 & ASCE 7-05 Tab 11.5-1)
C_t =	0.02	(ASCE 7-05 Tab 12.8-2)
h_n =	14.0	ft
x =	0.75	(ASCE 7-05 Tab 12.8-2)

$T = C_t (h_n)^x$ = 0.145 sec, (ASCE 7-05 Sec 12.8.2.1)

Calculate Vertical Distribution of Forces & Allowable Elastic Drift (ASCE 7-05, Sec 12.8.3 & 12.8.6)

Level	**W_x**	**h_x**	**h_x^k**	**$W_xh_x^k$**	**F_x**, ASD (12.8-11)		**$\delta_{xe,allowable, ASD}$**
Roof	120	14	14.0	1680	12.7	(0.11 Wx)	0.6
	120.0			1680	12.7		

Where
k = 1 for T <= 0.5
k = 0.5 T + 0.75 for T @ (0.5 , 2.5)
k = 2 for T >= 2.5

$\delta_{xe,allowable, ASD} = \Delta_a$ I / (1.4 C_d), (ASCE 7-05 Sec 12.8.6)

C_d = 4 ,(ASCE 7-05 Tab 12.2-1)

Δ_a = 0.02 h_{sx}, (ASCE 7-05 Tab 12.12-1)

Calculate Diaphragm Forces (ASCE 7-05, Sec 12.10.1.1)

Level	**W_x**	**ΣW_x**	**F_x**	**ΣF_x**	**F_{px}**, ASD, (12.10-1)	
Roof	120.0	120.0	12.7	12.7	15.4	(0.13 Wx)
	120.0		12.7			

Where
F_{min} = 0.2 S_{DS} I W_x / 1.5 , ASD
F_{max} = 0.4 S_{DS} I W_x / 1.5 , ASD

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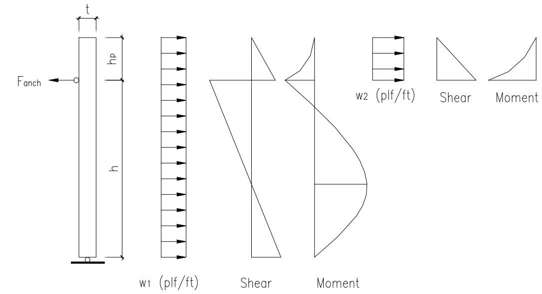
Lateral Force for One-Story Wall Based on 2009 IBC

INPUT DATA

WALL THICKNESS	t	=	8	in
PARAPET HEIGHT	h_p	=	4	ft
WALL HEIGHT	h	=	14	ft
TOTAL WALL DENSITY	ρ	=	150	pcf
SEISMIC PARAMETER	S_{DS}	=	0.54	(ASCE 7-05 Sec 11.4.4)
SEISMIC DESIGN CATEGOR`	SDC	=	C	
DIAPHRAGM FLEXIBLE ? (0=no, 1=yes)			1	Yes
SEISMIC IMPORTANCE FACTOR	I	=	1	(ASCE 7-05 Tab 11.5-1)
WIND IMPORTANCE FACTOR	I	=	1	(ASCE 7-05 Tab 6-1)
BASIC WIND SPEED	V	=	90	mph
EXPOSURE CATEGORY (B, C, D)		=	C	
TOPOGRAPHIC FACTOR	K_{Zt}	=	1	Flat, (ASCE 6.5.7.2)



DESIGN SUMMARY

Out-of-plane force for wall design w_1 = 22.5 psf (Wind governs)
Out-of-plane force for parapet design w_2 = 61.7 psf (Wind governs)
Out-of-plane force for anchorage design F_{anch} = 440 plf (Horizontal direction)
(The governing seismic forces have been reduced by 0.7 for ASD)

WIND ANALYSIS

Out-of-plane wind force for wall design (ASCE 7-05, Eq.6-22)

$$w_{1,wind} = q_h\left[(GC_P)-(GC_{Pi})\right] = \left(0.00256K_hK_{Zt}K_dV^2I\right)\left[(GC_P)-(GC_{Pi})\right] \quad = \quad 22.5 \text{ psf}$$

Where : K_h = 0.86 , K_d = 0.85 , GC_p = -1.30 , GC_{pi} = 0.18
(mean roof h = 16 ft, changeable) (ASCE Tab. 6-4) (corner ? Yes , TA = 18.67 ft^2) (ASCE Fig. 6-5)
(ASCE Tab. 6-3) (ASCE Fig. 6-11A)

Out-of-plane wind force for parapet design (ASCE 7-05, Eq.6-24)

$$w_{2,wind} = q_p\left[(GC_P)-(GC_{Pi})\right] = \left(0.00256K_hK_{Zt}K_dV^2I_w\right)\left[(GC_P)-(GC_{Pi})\right] \quad = \quad 61.7 \text{ psf, (ASCE 7-05,6.5.12.4.4)}$$

Where : K_p = 0.88 , K_d = 0.85 , GC_p = -1.40 , GC_p = -2.80 GC_{pi} = 0.18
(ASCE Tab. 6-3) (ASCE Tab. 6-4) = 1.00 (roof, ASCE Fig. 6-11B) (roof, ASCE Fig. 6-5)
(TA = 5.333 ft^2)
(wall, ASCE Fig. 6-11A)

Out-of-plane wind force for anchorage design

$$F_{anch,wind} = \frac{h}{2}w_{1,wind} + h_p\left(1+\frac{h_p}{2h}\right)w_{2,wind} \quad = \quad 440 \text{ plf (Horizontal)}$$

SEISMIC ANALYSIS

Out-of-plane seismic force for wall design (ASCE 7-05, Sec.12.11.1)

$$w_{1,seismic} = MAX\left(0.4IS_{DS}W_p \ ,0.1W_p\right) \quad = \quad 0.22\ W_p = \quad 21.6 \text{ psf}$$

Where : W_p = 100.0 psf , I = 1.0
(IBC Tab 1604.5 & ASCE Tab 11.5-1)

Out-of-plane seismic force for parapet design (ASCE 7-05, Sec. 13.3.1)

$$w_{2,seismic} = MAX\left[0.3S_{DS}I_pW_p \ , \ MIN\left(\frac{1.2a_pS_{DS}I_pW_p}{R_p} \ , \ 1.6S_{DS}I_pW_p\right)\right] \quad = \quad 0.65\ W_p = \quad 64.8 \quad \text{psf}$$

Where : a_p = 2.5 , I_p = 1.0 , R_p = 2.5
(ASCE Tab. 13.5-1) (ASCE Sec. 13.1.3) (ASCE Tab. 13.5-1)

Out-of-plane seismic force for anchorage design

For masonry or concrete under seismic design category A & B, both flexible & rigid diaphragm (ASCE 7-05 Sec. 12.11.2)

$$F_{anch,seismic} = MAX\left[0.4S_{DS}IW_p\frac{(h+h_p)^2}{2h} \ , \ 0.1W_p\frac{(h+h_p)^2}{2h} \ , \ 400S_{DS}I \ , \ F_{min}\right] = \quad 2.80\ W_p =$$

280 plf (Horizontal)
(Not applicable)

Where : F_{min} = 280 plf
(ASCE Sec. 12.11.2 & 11.7.3)

For seismic design category C and above, flexible diaphragm (ASCE 7-05 Sec. 12.11.2.1)

$$F_{anch,seismic} = MAX\left[0.8S_{DS}IW_p\frac{(h+h_p)^2}{2h} \ , \ 0.1W_p\frac{(h+h_p)^2}{2h} \ , \ 400S_{DS}I \ , \ F_{min}\right] = \quad 5.00\ W_p =$$

500 plf (Horizontal)
(Applicable)

For seismic design category C and above, rigid diaphragm (ASCE 7-05 Sec. 12.11.2 & Sec. 13.3.1)

$$F_{anch,seismic} = MAX\left\{MAX\left[0.3S_{DS}I_p \ , \ MIN\left(\frac{1.2a_pS_{DS}I_p}{R_p} \ , \ 1.6S_{DS}I_p\right)\right]W_p\frac{(h+h_p)^2}{2h} \ , \ 400S_{DS}I \ , \ F_{min}\right\}$$

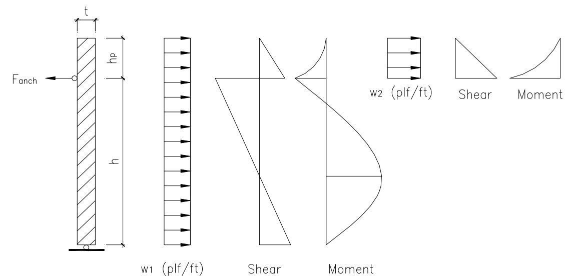
= 5.00 W_p = 500 plf (Horizontal) (Not applicable)

Where : a_p = 1.0 R_p = 1.5
(ASCE Tab. 13.5-1) (1.5, ASCE 13.4.2 or 2.5, ASCE Tab 13.5-1)

Lateral Force for One-Story Wall Based on IBC 2003

INPUT DATA

WALL THICKNESS	t	=	8	in
PARAPET HEIGHT	h_p	=	4	ft
WALL HEIGHT	h	=	14	ft
TOTAL WALL DENSITY	ρ	=	150	pcf
SEISMIC PARAMETER	S_{DS}	=	0.54	(IBC Sec.1615.1.3)
SEISMIC DESIGN CATEGOR`	SDC	=	C	
DIAPHRAGM FLEXIBLE ? (0=no, 1=yes)			1	Yes
IMPORTANCE FACTOR	I_w	=	1	(IBC Tab. 1604.5)
BASIC WIND SPEED	V	=	90	mph
EXPOSURE CATEGORY (B, C, D)		=	C	
TOPOGRAPHIC FACTOR	K_{Zt}	=	1	Flat, (ASCE Eq.6-3)



DESIGN SUMMARY

Out-of-plane force for wall design w_1 = 22.5 psf (Wind governs)
Out-of-plane force for parapet design w_2 = 61.7 psf (Wind governs)
Out-of-plane force for anchorage design F_{anch} = 440 plf (Horizontal direction)
(The governing seismic forces have been reduced by 0.7 for ASD)

WIND ANALYSIS

Out-of-plane wind force for wall design (ASCE 7-02, Eq.6-22)

$$w_{1,wind} = q_h\left[(GC_P)-(GC_{Pi})\right] = \left(0.00256K_hK_{Zt}K_dV^2I_w\right)\left[(GC_P)-(GC_{Pi})\right] \quad = \quad 22.5 \text{ psf}$$

Where : K_h = 0.86 , K_d = 0.85 , GC_p = -1.30 , GC_{pi} = 0.18
(mean roof h = 16 ft, changeable) (ASCE Tab. 6-4) (corner ? Yes , TA = 18.67 ft[2]) (ASCE Fig. 6-5)
(ASCE Tab. 6-3) (ASCE Fig. 6-11A)

Out-of-plane wind force for parapet design (ASCE 7-02, Eq.6-24)

$$w_{2,wind} = q_p\left[(GC_P)-(GC_{Pi})\right] = \left(0.00256K_hK_{Zt}K_dV^2I_w\right)\left[(GC_P)-(GC_{Pi})\right] \quad = \quad 61.7 \text{ psf, (ASCE7-02,6.5.12.4.4)}$$

Where : K_p = 0.88 , K_d = 0.85 , GC_p = -1.40 , GC_p = -2.80 GC_{pi} = 0.18
(ASCE Tab. 6-3) (ASCE Tab. 6-4) = 1.00 (roof, ASCE Fig. 6-11B) (roof, ASCE Fig. 6-5)
(TA = 5.333 ft[2])
(wall, ASCE Fig. 6-11A)

Out-of-plane wind force for anchorage design

$$F_{anch,wind} = \frac{h}{2}w_{1,wind} + h_p\left(1+\frac{h_p}{2h}\right)w_{2,wind} \quad = \quad 440 \text{ plf (Horizontal)}$$

SEISMIC ANALYSIS

Out-of-plane seismic force for wall design (IBC 2003, Sec.1620.2.7)

$$w_{1,seismic} = MAX\left(0.4I_ES_{DS}W_p \ ,0.1W_p\right) \quad = \quad 0.22\ W_p = \quad 21.6 \text{ psf}$$

Where : W_p = 100.0 psf , I_E = 1.0
(IBC Sec.1604.5)

Out-of-plane seismic force for parapet design (ASCE 7-02, Sec. 9.6.1.3)

$$w_{2,seismic} = MAX\left[0.3S_{DS}I_pW_p \ ,\ MIN\left(\frac{1.2a_pS_{DS}I_pW_p}{R_p}\ ,\ 1.6S_{DS}I_pW_p\right)\right] \quad = \quad 0.65\ W_p = \quad 64.8 \text{ psf}$$

Where : a_p = 2.5 , I_p = 1.0 , R_p = 2.5
(ASCE Tab.9.6.2.2) (ASCE Sec. 9.6.1.5) (ASCE Tab.9.6.2.2)

Out-of-plane seismic force for anchorage design

For seismic design category A & B, both flexible & rigid diaphragm (IBC 2003, Sec.1604.8.2 & 1620.2.7)

$$F_{anch,seismic} = MAX\left[0.4S_{DS}I_EW_p\frac{(h+h_p)^2}{2h}\ ,\ 0.1W_p\frac{(h+h_p)^2}{2h}\ ,\ 400S_{DS}I_E\ ,\ F_{min}\right] =$$

2.80 W_p =
280 plf (Horizontal)
(Not applicable)

Where : F_{min} = 280 plf
(IBC Sec.1604.8.2)

For seismic design category C and above, flexible diaphragm (IBC 2003, Sec.1604.8.2 ,1620.2.7, & 1620.3.1)

$$F_{anch,seismic} = MAX\left[0.8S_{DS}I_EW_p\frac{(h+h_p)^2}{2h}\ ,\ 0.1W_p\frac{(h+h_p)^2}{2h}\ ,\ 400S_{DS}I_E\ ,\ F_{min}\right] =$$

5.00 W_p =
500 plf (Horizontal)
(Applicable)

For seismic design category C and above, rigid diaphragm (IBC 2003, Sec.1604.8.2 ,1620.2.7, & ASCE Sec. 9.6.1.3)

$$F_{anch,seismic} = MAX\left\{MAX\left[0.3S_{DS}I_p\ ,\ MIN\left(\frac{1.2a_pS_{DS}I_p}{R_p}\ ,\ 1.6S_{DS}I_p\right)\right]W_p\frac{(h+h_p)^2}{2h}\ ,\ 400S_{DS}I_E\ ,\ F_{min}\right\}$$

= 3.00 W_p = 300 plf (Horizontal) (Not applicable)

Where : a_p = 1.0
(ASCE Tab.9.6.2.2)

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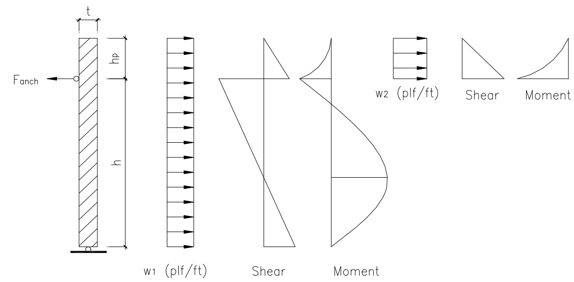
Lateral Force for One-Story Wall Based on IBC 2000



INPUT DATA

WALL THICKNESS	t	=	8	in
PARAPET HEIGHT	h_p	=	4	ft
WALL HEIGHT	h	=	14	ft
TOTAL WALL DENSITY	ρ	=	150	pcf
SEISMIC PARAMETER	S_{DS}	=	0.54	(IBC Sec.1615.1.3)
SEISMIC DESIGN CATEGORY	SDC	=	C	
DIAPHRAGM FLEXIBLE ? (0=no, 1=yes)			1	Yes
IMPORTANCE FACTOR	I_w	=	1	(IBC Tab. 1604.5)
BASIC WIND SPEED	V	=	90	mph
EXPOSURE CATEGORY (B, C, D)		=	C	
TOPOGRAPHIC FACTOR	K_{Zt}	=	1	Flat, (ASCE Eq.6-1)

DESIGN SUMMARY

Out-of-plane force for wall design w_1 = 23.9 psf (Wind governs)
Out-of-plane force for parapet design w_2 = 61.7 psf (Wind governs)
Out-of-plane force for anchorage design F_{anch} = 450 plf (Horizontal direction)
(The governing seismic forces have been reduced by 0.7 for ASD)

WIND ANALYSIS

Out-of-plane wind force for wall design (ASCE 7-98, Eq.6-18)

$$w_{1,wind} = q_h\left[\left(GC_P\right)-\left(GC_{Pi}\right)\right] = \left(0.00256K_hK_{Zt}K_dV^2I_w\right)\left[\left(GC_P\right)-\left(GC_{Pi}\right)\right] \quad = \quad 23.9 \text{ psf}$$

Where : K_h = 0.86 , K_d = 0.85 , GC_p = -1.40 , GC_{pi} = 0.18
(mean roof h = 16 ft, changeable) (ASCE Tab. 6-6) (corner ? Yes , TA = 10 ft^2) (ASCE Tab. 6-7)
(ASCE Tab. 6-5) (ASCE Fig. 6-5A)

Out-of-plane wind force for parapet design (ASCE 7-98, Eq.6-18)

$$w_{2,wind} = q_h\left[\left(GC_P\right)-\left(GC_{Pi}\right)\right] = \left(0.00256K_hK_{Zt}K_dV^2I_w\right)\left[\left(GC_P\right)-\left(GC_{Pi}\right)\right] \quad = \quad 61.7 \text{ psf}$$

Where : K_h = 0.88 , K_d = 0.85 , GC_p = -1.40 , GC_p = -2.80 GC_{pi} = 0.18
(ASCE Tab. 6-5) (ASCE Tab. 6-6) = 1.00 (roof, ASCE 7-02 Fig. 6-11B) (roof, ASCE Fig. 6-5)
(wall, ASCE 7-02 Fig. 6-11A)

Out-of-plane wind force for anchorage design

$$F_{anch,wind} = \frac{h}{2}w_{1,wind} + h_p\left(1+\frac{h_p}{2h}\right)w_{2,wind} \quad = \quad 450 \text{ plf (Horizontal)}$$

SEISMIC ANALYSIS

Out-of-plane seismic force for wall design (IBC 2000, Sec.1620.1.7)

$$w_{1,seismic} = MAX\left(0.4I_ES_{DS}W_p \ , 0.1W_p\right) \quad = \quad 0.22 \ W_p = \quad 21.6 \text{ psf}$$

Where : W_p = 100.0 psf , I_E = 1.0
(IBC Sec.1621.1.6)

Out-of-plane seismic force for parapet design (IBC 2000, Sec. 1621.1.4)

$$w_{2,seismic} = MAX\left[0.3S_{DS}I_pW_p \ , \ MIN\left(\frac{1.2a_pS_{DS}I_pW_p}{R_p} \ , \ 1.6S_{DS}I_pW_p\right)\right] \quad = \quad 0.65 \ W_p = \quad 64.8 \quad \text{psf}$$

Where : a_p = 2.5 , I_p = 1.0 , R_p = 2.5
(IBC Tab.1621.2) (IBC Sec.1621.1.6) (IBC Tab.1621.2)

Out-of-plane seismic force for anchorage design

For seismic design category A & B, both flexible & rigid diaphragm (IBC 2000, Sec.1604.8.2 & 1620.1.7)

$$F_{anch,seismic} = MAX\left[0.4S_{DS}I_EW_p\frac{\left(h+h_p\right)^2}{2h} \ , \ 0.1W_p\frac{\left(h+h_p\right)^2}{2h} \ , \ 400S_{DS}I_E \ , \ F_{min}\right] = \quad 2.50 \ W_p =$$

250 plf (Horizontal)
(Not applicable)

Where : F_{min} = 200 plf
(IBC Sec.1604.8.2)

For seismic design category C and above, flexible diaphragm (IBC 2000, Sec.1604.8.2 ,1620.1.7, & 1620.2.1)

$$F_{anch,seismic} = MAX\left[0.8S_{DS}I_EW_p\frac{\left(h+h_p\right)^2}{2h} \ , \ 0.1W_p\frac{\left(h+h_p\right)^2}{2h} \ , \ 400S_{DS}I_E \ , \ F_{min}\right] = \quad 5.00 \ W_p =$$

500 plf (Horizontal)
(Applicable)

For seismic design category C and above, rigid diaphragm (IBC 2000, Sec.1604.8.2 ,1620.1.7, & 1621.1.4)

$$F_{anch,seismic} = MAX\left\{MAX\left[0.3S_{DS}I_p \ , \ MIN\left(\frac{1.2a_pS_{DS}I_p}{R_p} \ , \ 1.6S_{DS}I_p\right)\right]W_p\frac{\left(h+h_p\right)^2}{2h} \ , \ 400S_{DS}I_E \ , \ F_{min}\right\}$$

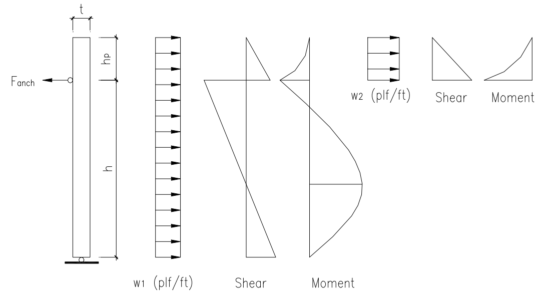
= 3.00 W_p = 300 plf (Horizontal) (Not applicable)

Where : a_p = 1.0
(IBC Tab.1621.2)

Lateral Force for One-Story Wall Based on 2010 CBC

INPUT DATA

WALL THICKNESS	t	=	8	in	
PARAPET HEIGHT	h_p	=	4	ft	
WALL HEIGHT	h	=	14	ft	
TOTAL WALL DENSITY	ρ	=	150	pcf	
SEISMIC PARAMETER	S_{DS}	=	0.54	(ASCE 7-05 Sec 11.4.4)	
SEISMIC DESIGN CATEGOR`	SDC	=	C		
DIAPHRAGM FLEXIBLE ? (0=no, 1=yes)			1	Yes	
SEISMIC IMPORTANCE FACTOR	I	=	1.5	(ASCE 7-05 Tab 11.5-1)	
WIND IMPORTANCE FACTOR	I	=	1	(ASCE 7-05 Tab 6-1)	
BASIC WIND SPEED	V	=	90	mph	
EXPOSURE CATEGORY (B, C, D)		=	C		
TOPOGRAPHIC FACTOR	K_{Zt}	=	1	Flat, (ASCE 6.5.7.2)	



DESIGN SUMMARY

Out-of-plane force for wall design w_1 = 22.7 psf (Seismic governs)
Out-of-plane force for parapet design w_2 = 68.0 psf (Seismic governs)
Out-of-plane force for anchorage design F_{anch} = 525 plf (Horizontal direction)
(The governing seismic forces have been reduced by 0.7 for ASD)

WIND ANALYSIS

Out-of-plane wind force for wall design (ASCE 7-05, Eq.6-22)

$$w_{1,wind}=q_h\left[(GC_P)-(GC_{Pi})\right]=\left(0.00256K_hK_{Zt}K_dV^2I\right)\left[(GC_P)-(GC_{Pi})\right] = 22.5 \text{ psf}$$

Where : K_h = 0.86 , K_d = 0.85 , GC_p = -1.30 , GC_{pi} = 0.18
(mean roof h = 16 ft, changeable) (ASCE Tab. 6-4) (corner ? Yes , TA = 18.67 ft^2) (ASCE Fig. 6-5)
(ASCE Tab. 6-3) (ASCE Fig. 6-11A)

Out-of-plane wind force for parapet design (ASCE 7-05, Eq.6-24)

$$w_{2,wind}=q_p\left[(GC_P)-(GC_{Pi})\right]=\left(0.00256K_hK_{Zt}K_dV^2I_w\right)\left[(GC_P)-(GC_{Pi})\right] = 61.7 \text{ psf, (ASCE 7-05,6.5.12.4.4)}$$

Where : K_p = 0.88 , K_d = 0.85 , GC_p = -1.40 , GC_p = -2.80 GC_{pi} = 0.18
(ASCE Tab. 6-3) (ASCE Tab. 6-4) = 1.00 (roof, ASCE Fig. 6-11B) (roof, ASCE Fig. 6-5)
(TA = 5.333 ft^2)
(wall, ASCE Fig. 6-11A)

Out-of-plane wind force for anchorage design

$$F_{anch,wind}=\frac{h}{2}w_{1,wind}+h_p\left(1+\frac{h_p}{2h}\right)w_{2,wind} = 440 \text{ plf (Horizontal)}$$

SEISMIC ANALYSIS

Out-of-plane seismic force for wall design (ASCE 7-05, Sec.12.11.1)

$$w_{1,seismic}=MAX\left(0.4IS_{DS}W_p\ ,0.1W_p\right) = 0.32\ W_p = 32.4 \text{ psf}$$

Where : W_p = 100.0 psf , I = 1.5
(IBC Tab 1604.5 & ASCE Tab 11.5-1)

Out-of-plane seismic force for parapet design (ASCE 7-05, Sec. 13.3.1)

$$w_{2,seismic}=MAX\left[0.3S_{DS}I_pW_p\ ,\ MIN\left(\frac{1.2a_pS_{DS}I_pW_p}{R_p}\ ,\ 1.6S_{DS}I_pW_p\right)\right] = 0.97\ W_p = 97.2 \text{ psf}$$

Where : a_p = 2.5 , I_p = 1.5 , R_p = 2.5
(ASCE Tab. 13.5-1) (ASCE Sec. 13.1.3) (ASCE Tab. 13.5-1)

Out-of-plane seismic force for anchorage design

For masonry or concrete under seismic design category A & B, both flexible & rigid diaphragm (ASCE 7-05 Sec. 12.11.2)

$$F_{anch,seismic}=MAX\left[0.4S_{DS}IW_p\frac{(h+h_p)^2}{2h}\ ,\ 0.1W_p\frac{(h+h_p)^2}{2h}\ ,\ 400S_{DS}I\ ,\ F_{min}\right] = 3.75\ W_p = 375 \text{ plf (Horizontal)}$$

(Not applicable)

Where : F_{min} = 280 plf
(2010 CBC 1604.8.2, ASCE Sec. 12.11.2 & 11.7.3)

For seismic design category C and above, flexible diaphragm (ASCE 7-05 Sec. 12.11.2.1)

$$F_{anch,seismic}=MAX\left[0.8S_{DS}IW_p\frac{(h+h_p)^2}{2h}\ ,\ 0.1W_p\frac{(h+h_p)^2}{2h}\ ,\ 400S_{DS}I\ ,\ F_{min}\right] = 7.50\ W_p = 750 \text{ plf (Horizontal)}$$

(Applicable)

For seismic design category C and above, rigid diaphragm (ASCE 7-05 Sec. 12.11.2 & Sec. 13.3.1)

$$F_{anch,seismic}=MAX\left\{MAX\left[0.3S_{DS}I_p\ ,\ MIN\left(\frac{1.2a_pS_{DS}I_p}{R_p}\ ,\ 1.6S_{DS}I_p\right)\right]W_p\frac{(h+h_p)^2}{2h}\ ,\ 400S_{DS}I\ ,\ F_{min}\right\}$$

= 7.50 W_p = 750 plf (Horizontal) (Not applicable)

Where : a_p = 1.0 R_p = 1.5
(ASCE Tab. 13.5-1) (1.5, ASCE 13.4.2 or 2.5, ASCE Tab 13.5-1)

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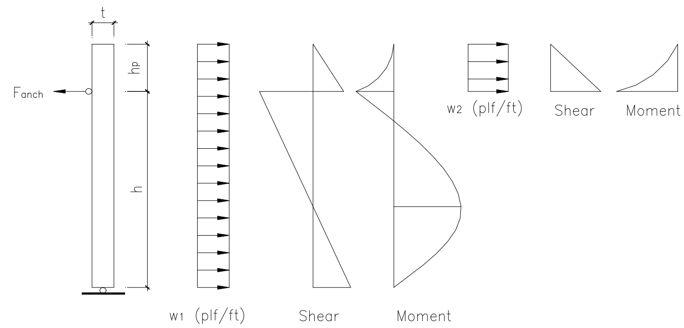
Lateral Force for One-Story Wall Based on UBC 97



INPUT DATA

WALL THICKNESS	t	=	12	in
PARAPET HEIGHT	h_p	=	2	ft
WALL HEIGHT	h	=	44	ft
TOTAL WALL DENSITY	ρ	=	58	pcf
SEISMIC ZONE (1, 2A, 2B, 3, or 4)			4	Zone 4
SEISMIC COEFFICIENT	C_a	=	0.44	(UBC Tab. 16-Q)
IMPORTANCE FACTOR	I_p	=	1	(UBC Tab. 16-K)
BASIC WIND VELOCITY	V	=	70	mph
EXPOSURE TYPE (B, C, D)			B	

DESIGN SUMMARY

Out-of-plane force for wall design w_1 = 18.5 psf (Seismic governs)

Out-of-plane force for parapet design w_2 = 60.8 psf (Seismic governs)

Out-of-plane force for anchorage design F_{anch} = 877 plf (Horizontal direction)

(The governing seismic forces have been divided by 1.4 for ASD)

WIND ANALYSIS

Out-of-plane wind force for wall design (UBC 97 Sec.1620)

$$w_{1,wind} = C_e C_q q_s I_w \quad = \quad 16.3 \text{ psf}$$

Where : C_e = 0.87 (UBC Tab.16-G), C_q = 1.49 (UBC Tab.16-H), q_s = 12.60 psf (UBC Tab.16-F), I_w = 1.00 (UBC Tab.16-K)

Mean roof h = 45 ft   Corner ? Yes (1.5 Yes, 1.2 No)

TA = 16 ft^2 (10 ft^2 default)

[C_q - 0.2 + (100-TA)/450] for TA @ [10,100]

Out-of-plane wind force for parapet design (UBC 97 Sec.1620)

$$w_{2,wind} = C_e C_q q_s I_w \quad = \quad 16.4 \text{ psf}$$

Where : C_e = 0.87 (UBC Tab.16-G), C_q = 1.50, TA = 8 ft^2, q_s = 12.60 psf (UBC Tab.16-F), I_w = 1.00 (UBC Tab.16-K)

Out-of-plane wind force for anchorage design

$$F_{anch,wind} = C_e C_q q_s I_w \frac{\left(h_p + h\right)^2}{2h} \quad = \quad 391 \text{ plf (Horizontal)}$$

Where : C_e = 0.87 (UBC Tab.16-G), C_q = 1.49 (UBC Tab.16-H), q_s = 12.60 (UBC Tab.16-F), I_w = 1.00 (UBC Tab.16-K)

SEISMIC ANALYSIS

Out-of-plane seismic force for wall design (UBC 97 Sec.1632.2)

$$w_{1,seismic} = \frac{1}{2}\left\{ MAX\left[0.7 C_a I_p W_p \ ,\ MIN\left(\frac{4 a_p C_a I_p W_p}{R_p} \ ,\ 4 C_a I_p W_p\right)\right] + MAX\left[0.7 C_a I_p W_p \ ,\ MIN\left(\frac{a_p C_a I_p W_p}{R_p} \ ,\ 4 C_a I_p W_p\right)\right]\right\} = 0.45\ W_p = 25.9 \text{ psf}$$

Where : a_p = 1.0 (UBC Tab.16-O), R_p = 3.0 (UBC Tab.16-O), W_p = 58.0 psf

Out-of-plane seismic force for parapet design (UBC 97 Sec.1632.2)

$$w_{2,seismic} = MAX\left[0.7 C_a I_p W_p \ ,\ MIN\left(\frac{4 a_p C_a I_p W_p}{R_p} \ ,\ 4 C_a I_p W_p\right)\right] = 1.47\ W_p = 85.1 \text{ psf}$$

Where : a_p = 2.5 (UBC Tab.16-O), R_p = 3.0 (UBC Tab.16-O), W_p = 58.0 psf

Out-of-plane seismic force for anchorage design (UBC 97 Sec.1632.2 & 1633.2.8.1)

$$F_{anch,seismic} = MAX\left[0.7 C_a I_p W_p \ ,\ MIN\left(\frac{4 a_p C_a I_p W_p}{R_p} \ ,\ 4 C_a I_p W_p\right) \ ,\ F_{min}\right] = 0.88\ W_p = 1227 \text{ plf (Horizontal)}$$

Where : a_p = 1.5 (UBC Tab.16-O), R_p = 3.0 (UBC Tab.16-O), W_p = 1395 plf (Horizontal), F_{min} = 420 plf (UBC 1633.2.8.1)

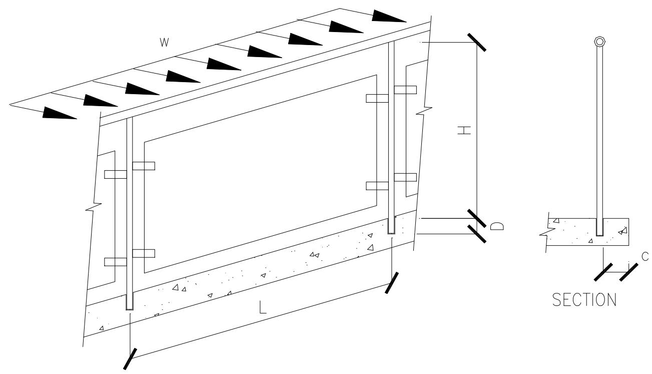
Handrail Design Based on AISC 360-05 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

HANDRAIL SECTION	= >	PIPE-1 1/2		= >	**A**	**Z**	**S**	**t**	**D**
HANDRAIL YIELD STRESS	F_y =	35	ksi		0.80	0.42	0.33	0.15	1.90
BALUSTER SECTION	= >	PIPE-1 1/4		= >	**A**	**Z**	**S**	**t**	**D**
BALUSTER YIELD STRESS	F_y =	35	ksi		0.67	0.31	0.24	0.14	1.66

HANDRAIL SPAN	L =	48	in
BALUSTER HEIGHT	H =	36	in
BALUSTER SLEEVE DEPTH	D =	4	in
EDGE DISTANCE TO SLEEVE	c =	1	in
CONCRETE STRENGTH	f_c' =	3	ksi
HORIZ. LOAD PERP. TO HANDRAIL	w =	50	plf

(UBC Tab.16-B, IBC 1607.7.1)



THE BRACE DESIGN IS ADEQUATE.

ANALYSIS

CHECK HANDRAIL CAPACITIES (AISC 360-05 F7, G5 or F8, G6)

$M = \frac{wL^2}{8} =$ 100 ft-lbs $\qquad V = \frac{wL}{2} =$ 100 lbs

$f_b = \frac{M}{S} =$ 3.68 ksi < ***(4/3) (M_n / Ω_b) / S = (4/3) (F_y Z / 1.67) / S =*** 36.09 ksi [Satisfactory]

(where 4/3 from IBC 1607.7.1.3, Typical.)

$f_v = \frac{V}{A} =$ 0.25 ksi < ***(4/3) (0.6 F_y C_v / Ω_v) = (4/3) (0.6 F_y 1.0 / 1.67) =*** 16.77 ksi [Satisfactory]

CHECK BALUSTER CAPACITIES (AISC 360-05 F7, G5 or F8, G6)

$M = wLH =$ 600 ft-lbs $\qquad V = wL =$ 200 lbs

$f_b = \frac{M}{S} =$ 30.64 ksi < ***(4/3) (M_n / Ω_b) / S = (4/3) (F_y Z / 1.67) / S =*** 36.27 ksi [Satisfactory]

$f_v = \frac{V}{A} =$ 0.60 ksi < ***(4/3) (0.6 F_y C_v / Ω_v) = (4/3) (0.6 F_y 1.0 / 1.67) =*** 16.77 ksi [Satisfactory]

CHECK CONCRETE BREAKOUT STRENGTH AT BALUSTER SLEEVE (ACI 318-08 Appendix D)

$$\phi V_{cb} = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} \left(7 \left(\frac{l}{d} \right)^{0.2} \sqrt{d} \sqrt{f'_c} c^{1.5} \right) = \quad 0.562 \text{ kips} \quad > \quad V_u \quad \text{[Satisfactory]}$$

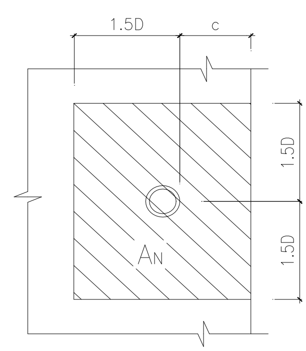
where : ϕ = 0.75

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

A_V / A_{Vo} and $\psi_{cd,V}$ terms are 1.0 for single shear sleeve not influenced by more than one free edge.

ℓ is load bearing length of the anchor for shear, not to exceed 8d.

V_u = 1.4 V = 0.280 kips



CHECK CONCRETE PRYOUT STRENGTH AT BALUSTER SLEEVE (ACI 318-08 Appendix D)

$$\phi V_{cp} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{(9D^2)} \left(0.7 + \frac{0.3c}{1.5D} \right) \psi_{c,N} \left(24 \sqrt{f'_c} D^{1.5} \right)$$

= 6.901 kips > V_u [Satisfactory]

where : $\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

k_{cp} = 2.0 for D > 2.5 in.

A_N = 3 D (1.5D + c) = 84.00 in^2

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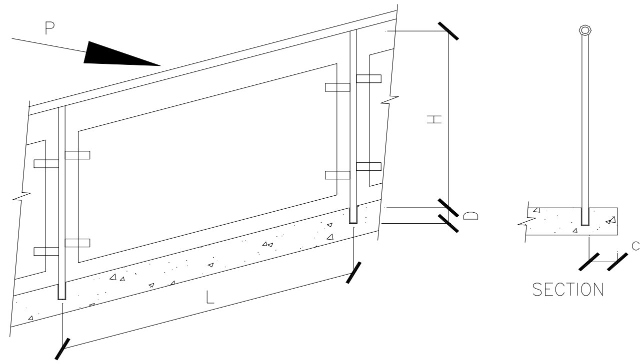
Handrail Design Based on AISC 360-05 & ACI 318-08

INPUT DATA & DESIGN SUMMARY

					A	Z	S	t	D
HANDRAIL SECTION	= >	PIPE-1 1/2		= >	A	Z	S	t	D
HANDRAIL YIELD STRESS	F_y =	35	ksi		0.80	0.42	0.33	0.15	1.90
BALUSTER SECTION	= >	PIPE-1 1/4		= >	A	Z	S	t	D
BALUSTER YIELD STRESS	F_y =	35	ksi		0.67	0.31	0.24	0.14	1.66

HANDRAIL SPAN L = 48 in
BALUSTER HEIGHT H = 36 in
BALUSTER SLEEVE DEPTH D = 4 in
EDGE DISTANCE TO SLEEVE c = 1 in

CONCRETE STRENGTH f_c' = 3 ksi
POINT LOAD PERP. TO HANDRAIL P = 200 lbs
(UBC Tab.16-B, IBC 1607.7.1)



THE BRACE DESIGN IS ADEQUATE.

ANALYSIS

CHECK HANDRAIL CAPACITIES (AISC 360-05 F7, G5 or F8, G6)

$M = \frac{PL}{4} =$ 200 ft-lbs, (P @ middle) $\quad V = P =$ 200 lbs, (P @ end)

$f_b = \frac{M}{S} =$ 7.36 ksi < ***(4/3) (M_n / Ω_b) / S = (4/3) (F_y Z / 1.67) / S =*** 36.09 ksi [Satisfactory]

(where 4/3 from IBC 1607.7.1.3, Typical.)

$f_v = \frac{V}{A} =$ 0.50 ksi < ***(4/3) (0.6 F_y C_v / Ω_v) = (4/3) (0.6 F_y 1.0 / 1.67) =*** 18.67 ksi [Satisfactory]

CHECK BALUSTER CAPACITIES (AISC 360-05 F7, G5 or F8, G6)

$M = PH =$ 600 ft-lbs $\quad V = P =$ 200 lbs

$f_b = \frac{M}{S} =$ 30.64 ksi < ***(4/3) (M_n / Ω_b) / S = (4/3) (F_y Z / 1.67) / S =*** 36.27 ksi [Satisfactory]

$f_v = \frac{V}{A} =$ 0.60 ksi < ***(4/3) (0.6 F_y C_v / Ω_v) = (4/3) (0.6 F_y 1.0 / 1.67) =*** 18.67 ksi [Satisfactory]

CHECK CONCRETE BREAKOUT STRENGTH AT BALUSTER SLEEVE (ACI 318-08 Appendix D)

$$\phi V_{cb} = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{A_V}{A_{Vo}} \psi_{cd,V} \psi_{c,V} \left(7 \left(\frac{l}{d} \right)^{0.2} \sqrt{d} \sqrt{f'_c} c^{1.5} \right) =$$ 0.562 kips > V_u [Satisfactory]

where : ϕ = 0.75

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

A_V / A_{Vo} and $\psi_{cd,V}$ terms are 1.0 for single shear sleeve not influenced by more than one free edge.

ℓ is load bearing length of the anchor for shear, not to exceed 8d.

V_u = ***1.4 V =*** 0.280 kips

CHECK CONCRETE PRYOUT STRENGTH AT BALUSTER SLEEVE (ACI 318-08 Appendix D)

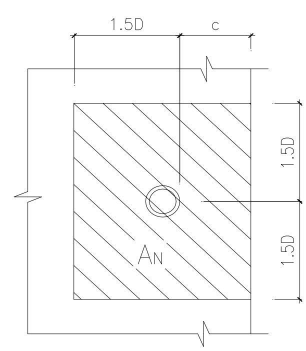
$$\phi V_{cp} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{(9D^2)} \left(0.7 + \frac{0.3c}{1.5D} \right) \psi_{c,N} \left(24 \sqrt{f'_c} D^{1.5} \right)$$

= 6.901 kips > V_u [Satisfactory]

where : $\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

k_{cp} = 2.0 for D > 2.5 in.

A_n = ***3 D (1.5D + c) =*** 84.00 in^2



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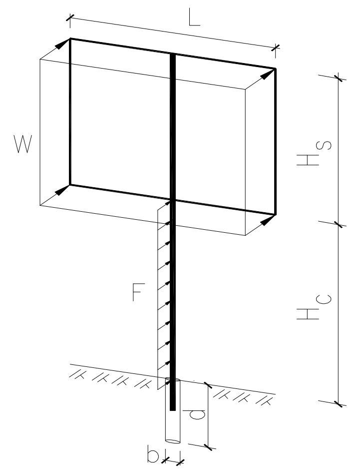
Sign Design Based on AISC 360-05, ACI 318-08, and IBC 09 1807.3

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION (Tube, Pipe, or WF)		HSS8X8X5/16	**Tube**
COLUMN YIELD STRESS	F_y =	46	ksi
DIMENSIONS	L =	20	ft
	H_S =	8	ft
	H_C =	10	ft
SIGN GRAVITY LOAD (lbs / ft^2)	D =	10	psf
SIGN LATERAL LOAD (lbs / ft^2)	W =	25	psf, ASD
COLUMN LATERAL LOAD (plf)	F =	8	plf, ASD
DIAMETER OF POLE FOOTING	b =	2	ft
ALLOW SOIL PRESSURE	Q_a =	1	ksf
LATERAL SOIL CAPACITY	P_P =	0.266	ksf / ft
RESTRAINED @ GRADE ?(1=yes,0=no)		0	No



Use 2 ft dia x 8.61 ft deep footing unrestrained @ ground level

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY OF COLUMN (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}< 0.2 \end{cases} = 1.28 < 4/3$$ **[Satisfactory]**

Where P_r = 1.68 kips

M_{rx} = 56.4 ft-kips, at bottom of column

M_{ry} = 16.92 ft-kips, 30% ***M_{rx} used***

$P_c = P_n / \Omega_c$ = 292 / 1.67 = 175.01 kips, (AISC 360-05 Chapter E)

> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 96.217 / 1.67 = 57.615 ft-kips, (AISC 360-05 Chapter F)

> $3/4\ M_{rx}$ **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 96.217 / 1.67 = 57.615 ft-kips, (AISC 360-05 Chapter F)

> $3/4\ M_{ry}$ **[Satisfactory]**

DESIGN POLE FOOTING (IBC 09 1807.3)

By trials, use pole depth, d = 8.606 ft

Lateral bearing @ bottom, $S_3 = 2\ P_P\ Min(\ d\ , 12'\ )$ = 4.58 ksf

Lateral bearing @ $d / 3$, $S_1 = 2\ P_P\ Min(\ d / 3\ , 12'\ )$ = 1.53 ksf

Require Depth is given by

$$d=\begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & for\ \ nonconstrained \\ \sqrt{\frac{4.25Ph}{bS_3}} & for\ \ constrained \end{cases} = 8.606\ \text{ft}$$ **[Satisfactory]**

Where $P = V_{max}$ = 4.08 kips

$A = 2.34\ P / (b\ S_1)$ = 3.13

$h = M_{max} / V_{max}$ = 13.82 ft

CHECK VERTICAL SOIL BEARING CAPACITY (ACI, Sec. 15.2.2)

$q_{soil} = (D\ H_S\ L + col\ wt\ ) / (\pi\ b^2 / 4)$ = 0.53 ksf, (net weight of pole footing included.)

< Q_a **[Satisfactory]**

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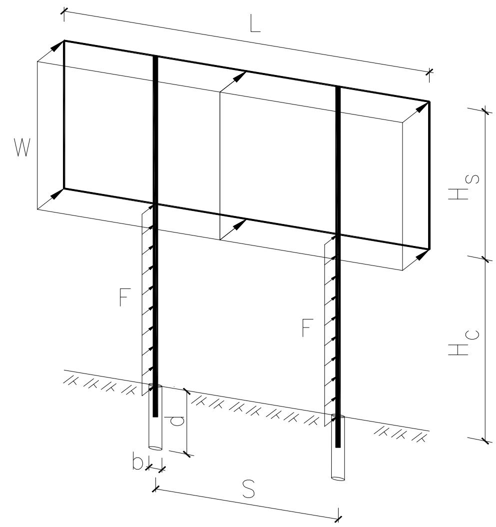
Sign Design Based on AISC 360-05, ACI 318-08, and IBC 09 1807.3

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION (WF, Tube, or Pipe)		W8X24	**W Shape**
COLUMN YIELD STRESS	F_y =	50	ksi
DIMENSIONS	L =	20	ft
	H_S =	8	ft
	H_C =	10	ft
SIGN GRAVITY LOAD (lbs / ft^2)	D =	10	psf
SIGN LATERAL LOAD (lbs / ft^2)	W =	25	psf, ASD
COLUMN LATERAL LOAD (plf)	F =	8	plf, ASD
DIAMETER OF POLE FOOTING	b =	3	ft
ALLOW SOIL PRESSURE	Q_a =	1	ksf
LATERAL SOIL CAPACITY	P_P =	0.266	ksf / ft
RESTRAINED @ GRADE ?(1=yes,0=no)		0	No



Use 3 ft dia x 5.73 ft deep footing unrestrained @ ground level

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY OF COLUMN (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}\ge 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}< 0.2 \end{cases} = 1.15 < 4/3$$ **[Satisfactory]**

Where P_r = 0.86 kips

M_{rx} = 28.4 ft-kips, at bottom of column

M_{ry} = 8.52 ft-kips, 30% M_{rx} *used*

$P_c = P_n / \Omega_c$ = 98 / 1.67 = 58.956 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 63.567 / 1.67 = 38.064 ft-kips, (AISC 360-05 Chapter F)
> $3/4\ M_{rx}$ **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 35.708 / 1.67 = 21.382 ft-kips, (AISC 360-05 Chapter F)
> $3/4\ M_{ry}$ **[Satisfactory]**

DESIGN POLE FOOTING (IBC 09 1807.3)

By trials, use pole depth, d = 5.73 ft

Lateral bearing @ bottom, $S_3 = 2\,P_P\ Min(\,d\,,12'\,)$ = 3.05 ksf

Lateral bearing @ $d/3$, $S_1 = 2\,P_P\ Min(\,d/3\,,12'\,)$ = 1.02 ksf

Require Depth is given by

$$d=\begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & for\quad nonconstrained \\ \sqrt{\frac{4.25Ph}{bS_3}} & for\quad constrained \end{cases} = 5.73\ \text{ft}$$ **[Satisfactory]**

Where $P = V_{max}$ = 2.08 kips

$A = 2.34\,P / (b\,S_1)$ = 1.59

$h = M_{max} / V_{max}$ = 13.65 ft

CHECK VERTICAL SOIL BEARING CAPACITY (ACI, Sec. 15.2.2)

$q_{soil} = (0.5\,D\,H_S\ L + col\ wt\,) / (\pi\ b^2 / 4)$ = 0.12 ksf, (net weight of pole footing included.)
< Q_a **[Satisfactory]**

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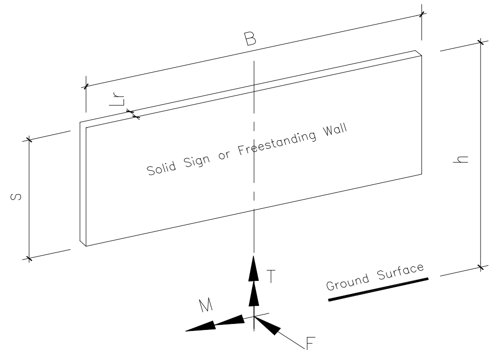
Wind Analysis for Freestanding Wall & Sign Based on ASCE 7-2010

INPUT DATA

Exposure category (B, C or D)		=	C	
Importance factor, 1.0 only, (Table 1.5-2)	I_w	=	1.00	
Basic wind speed (ASCE 7-10 26.5.1)	V	=	136	mph
Topographic factor (26.8 & Table 26.8-1)	K_{zt}	=	1	Flat
Height of top	h	=	35	ft
Vertical dimension (for wall, s = h)	s	=	8	ft
Horizontal dimension	B	=	100	ft
Dimension of return corner	L_r	=	2	ft



DESIGN SUMMARY

Max horizontal wind pressure	p	=	125 psf
Max total horizontal force at centroid of base	F	=	51.09 kips
Max bending moment at centroid of base	M	=	1583.73 ft-kips
Max torsion at centroid of base	T	=	1021.76 ft-kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h K_{zt} K_d V^2$ = **40.65** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 29.3-1 page 307 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 29.3-1, pg 310) = **1.01**

K_d = wind directionality factor. (Tab. 26.6-1, for building, page 250) = **0.85**

h = height of top = **35.00** ft

Wind Force Case A: resultant force though the geometric center (Sec. 29.4.1 & Fig. 29.1-1)

$p = q_h\, G\, C_f$ =	=	**64**	psf
$F = p\, A_s$	=	**51.09**	kips
M = F (h - 0.5s) for sign, F (0.55h) for wall	=	**1583.73**	ft-kips
T =	=	**0.00**	ft-kips

where: G = gust effect factor. (Sec. 26.9) = **0.85**

C_f = net force coefficient. (Fig. 29.4-1, page 311) **1.85**

A_s = B s = **800.0** ft^2

Wind Force Case B: resultant force at 0.2 B offset of the geometric center (Sec. 29.4.1 & Fig. 29.1-1)

p = Case A	=	**64**	psf
F = Case A	=	**51.09**	kips
M = Case A	=	**1583.73**	ft-kips
T = 0.2 F B	=	**1021.76**	ft-kips

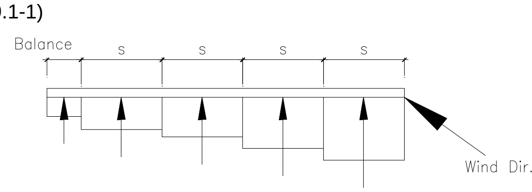
Wind Force Case C: resultant force different at each region (Sec. 29.4.1 & Fig. 29.1-1)

$p = q_h\, G\, C_f$

$F = \Sigma\, p\, A_s$

M = Σ [F (h - 0.5s) for sign, F (0.55h) for wall]

$T = \Sigma\, T_s$



Distance	C_f	P_i	A_{si}	F_i	M_i	T_i
(ft)	(Fig.6-20)	(psf)	(ft^2)	(kips)	(ft-kips)	(ft-kips)
8.0	3.628	125	64	8.02	248.74	369.10
16.0	2.575	89	64	5.69	176.52	216.38
24.0	1.975	68	64	4.37	135.39	131.02
32.0	1.408	49	64	3.11	96.54	68.51
40.0	1.283	44	64	2.84	87.97	39.73
80.0	0.908	31	64	2.01	62.27	-20.09
100.0	0.901	31	160	4.98	154.34	-199.14
Σ				**31.02**	**961.77**	**605.51**

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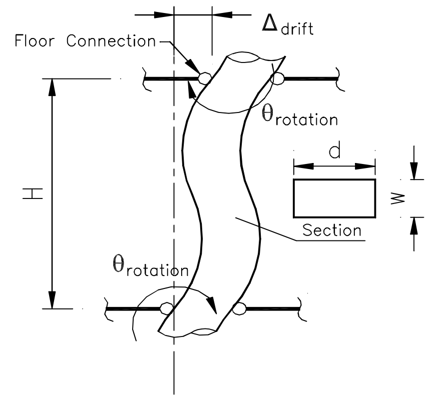
MCE Level Seismic Design for Metal Pipe/Riser Based on ASCE 7-10 & AISI S100

INPUT DATA & DESIGN SUMMARY

BUILDING STORY HEIGHT	$H =$	18	ft
METAL YIELD STRESS (33 or 55)	$F_y =$	55	ksi
SECTION SIZE	$w =$	48	in
	$d =$	84	in
	$t =$	43	mils
	(	18	Gauge)
MAX STORY DRIFT (MCE level)	$\Delta_{drift} =$	6	in, (< 0.04 H, CBC 2205A.4.1.1)
ALLOWABLE ROTATION	$\theta_{rotation} =$	0.028	rad, (detail/site verify, < 0.03)



THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE SECTION FORCES BY FULL SECTION CONSERVATIVELY

[k] =

EA/L	0	0	-EA/L	0	0
	$12EI/((1+\beta)L^3)$	$6EI/((1+\beta)L^2)$	0	$-12EI/((1+\beta)L^3)$	$6EI/((1+\beta)L^2)$
		$(4+\beta)EI/((1+\beta)L)$	0	$-6EI/((1+\beta)L^2)$	$(2-\beta)EI/((1+\beta)L)$
			EA/L	0	0
				$12EI/((1+\beta)L^3)$	$-6EI/((1+\beta)L^2)$
					$(4+\beta)EI/((1+\beta)L)$

=

1598.544	0.000	0.000	-1598.544	0.000	0.000
0.000	230.068	24847.334	0.000	-230.068	24847.334
0.000	24847.334	4307051.945	0.000	-24847.334	1059972.212
-1598.544	0.000	0.000	1598.544	0.000	0.000
0.000	-230.068	-24847.334	0.000	230.068	-24847.334
0.000	24847.334	1059972.212	0.000	-24847.334	4307051.945

Where								
	L	=	18	ft	thk	=	0.0451	in
	E	=	29000	ksi	A	=	11.91	in^2
	G	=	11153.85	ksi	I	=	12092.57	in^4
	β	=	$12 E I k / (G A L^2) =$ 0.815		k	=	1.2	

{F} = [k] {Δ} = [k]

0.000
0.000
0.028
0.000
6.000
0.028

=

0.000
11.043
1192.672
0.000
-11.043
1192.672

$V =$ 11.043 kips, SD = 7.730 kips, ASD

$M =$ 1193 in-k, SD = 69.6 ft-k, ASD

CHECK FLEXURAL CAPACITY BY EFFECTIVE SECTION ON FOUR LEGS (AISI C3.1)

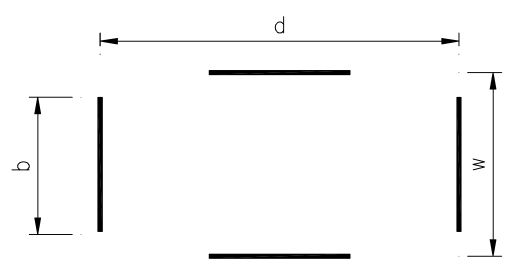
M_n/Ω_b = $F_y I_{eff} / (0.5\, d\, \Omega_b) =$ 94.1 ft-kips

> **M** **[Satisfactory]**

Where $b = 200$ (thk) = 9.02 in, (AISI B1.2a)

$I_{eff} = 2\,(thk)\, b^3 / 12 + 2\,(thk)\, b\, (0.5\, d)^2 =$ 1440.71 in^4

$\Omega_b =$ 1.67



CHECK SHEAR CAPACITY BY EFFECTIVE SECTION ON TWO LEGS (AISI C3.2)

V_n/Ω_V = $F_y A_{eff} / (\Omega_v) =$ 27.968 kips

> **V** **[Satisfactory]**

Where $A_{eff} = 2\,(thk)\, b =$ 0.81 in^2

$\Omega_v =$ 1.60

CHECK CAPACITY COMBINED BENDING & SHEAR AT THE SAME SECTION (AISI C3.3.1)

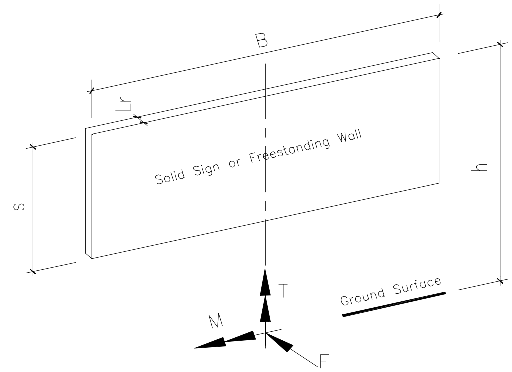
$$\sqrt{\left(\frac{\Omega_b M}{M_n}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} = 0.788998 < 1.0 \quad \textbf{[Satisfactory]}$$

$$\left(\frac{\Omega_b M}{M_n}\right) = 0.74 > 0.5 \qquad \left(\frac{\Omega_v V}{V_n}\right) = 0.28 < 0.7$$

$$0.6\left(\frac{\Omega_b M}{M_n}\right) + \left(\frac{\Omega_v V}{V_n}\right) = 1 < 1.3 \quad \textbf{[Satisfactory]}$$

Wind Analysis for Freestanding Wall & Sign Based on ASCE 7-05

INPUT DATA



Exposure category (B, C or D)		=	C	
Importance factor, pg 73, (0.87, 1.0 or 1.15)	I	=	1.00	Category II
Basic wind speed (3 sec. gust wind)	V	=	90	mph
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt}	=	1	Flat
Height of top	h	=	35	ft
Vertical dimension (for wall, s = h)	s	=	8	ft
Horizontal dimension	B	=	100	ft
Dimension of return corner	L_r	=	2	ft

DESIGN SUMMARY

Max horizontal wind pressure	p	=	55 psf
Max total horizontal force at centroid of base	F	=	22.37 kips
Max bending moment at centroid of base	M	=	693.57 ft-kips
Max torsion at centroid of base	T	=	447.46 ft-kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2\, I$ = **17.80** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 6-15, page 27)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) = **1.01**

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) = **0.85**

h = height of top = **35.00** ft

Wind Force Case A: resultant force though the geometric center (Sec. 6.5.14 & Fig. 6-20)

$p = q_h\, G\, C_f =$ = **28** psf

$F = p\, A_s$ = **22.37** kips

M = F (h - 0.5s) for sign, F (0.55h) for wall = **693.57** ft-kips

T = = **0.00** ft-kips

where: G = gust effect factor. (Sec. 6.5.8, page 26). = **0.85**

C_f = net force coefficient. (Fig. 6-20, page 73) **1.85**

A_s = B s = **800.0** ft^2

Wind Force Case B: resultant force at 0.2 B offset of the geometric center (Sec. 6.5.14 & Fig. 6-20)

p = Case A = **28** psf

F = Case A = **22.37** kips

M = Case A = **693.57** ft-kips

T = 0.2 F B = **447.46** ft-kips

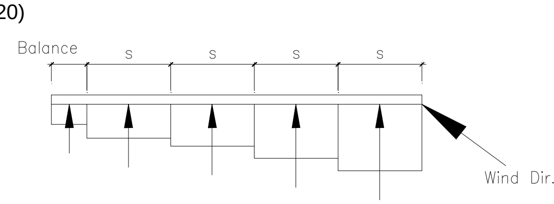
Wind Force Case C: resultant force different at each region (Sec. 6.5.14 & Fig. 6-20)

$p = q_h\, G\, C_f$

$F = \Sigma\, p\, A_s$

M = Σ [F (h - 0.5s) for sign, F (0.55h) for wall]

$T = \Sigma\, T_s$



Distance	C_f	P_i	A_{si}	F_i	M_i	T_i
(ft)	(Fig.6-20)	(psf)	(ft^2)	(kips)	(ft-kips)	(ft-kips)
8.0	3.628	55	64	3.51	108.93	161.64
16.0	2.575	39	64	2.49	77.30	94.76
24.0	1.975	30	64	1.91	59.29	57.38
32.0	1.408	21	64	1.36	42.28	30.00
40.0	1.283	19	64	1.24	38.53	17.40
80.0	0.908	14	64	0.88	27.27	-8.80
100.0	0.901	14	160	2.18	67.59	-87.21
Σ				**13.59**	**421.19**	**265.17**

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Snow Load Analysis Based on ASCE 7-2010

INPUT DATA & DESIGN SUMMARY

BASIC GROUND SNOW LOAD (ASCE Sec. 7.2)	P_g =	20	psf, (ASCE page 29)	$P_{f,\ roof}$ =	15.75	psf
SNOW EXPOSURE FACTOR	C_e =	1	(Tab. 7-2, pg 30)	$P_{f,\ overhang}$ =	33.60	psf
THERMAL FACTOR (0.85 , 1.0 , 1.1 , 1.2, or 1.3)	C_t =	1.2	(Tab. 7-3, pg 30)	$P_{f,\ valley}$ =	33.60	psf
IMPORTANCE FACTOR	I_s =	1	(Tab. 1.5-1 & 1.5-2, 7.3.3 pg 29)	$P_{f,\ parapet}$ =	38.06	psf
ROOF SLOPE		4 / 12		W_d =	5.38	ft
PARAPET HEIGHT (DRIFT CORNER HEIGHT)	h_r =	4	ft, see fig. below			
LENGTH OF THE ROOF UPWIND OF THE DRIFT	L_u =	35	ft, see fig. below	P_1 =	25.00	psf
OBSTRUCTED SLIPPERY SURFACE (Sec. 7.4, pg 31 & 36) **?** (1=Yes, 0=No)		0	Unobstructed	P_2 =	20.75	psf
				P_3 =	6.23	psf
				P_4 =	33.63	psf
				x =	6.21	ft

ANALYSIS

FLAT SNOW LOADS (Sec 7.3, pg 29)

$P_f = 0.7C_eC_t\, I_s\, P_g$ = 16.80 psf

Where α_{roof} = 18.4 ^{0}

W = 17.5 ft

$P_{f,\ min}$ = 0.00 psf, (Sec. 7.3.4, pg 29)

ROOF SNOW LOADS (Sec. 7.4, pg 31)

$P_s = C_s\, P_f$ = 15.75 psf, (Eq.7.4-1)

Where C_s = 0.938

, Derived from Fig 7-2, page 86, as following table

C_t	0.85 or 1.0	1.1	1.2 or 1.3
Unobstructed	$C_{s,1}$	$C_{s,3}$	$C_{s,5}$
Obstructed	$C_{s,2}$	$C_{s,4}$	$C_{s,6}$

$C_{s,1}$ = MIN [(70 - α) / 65, 1.0] , Fig. 7-2a dash line

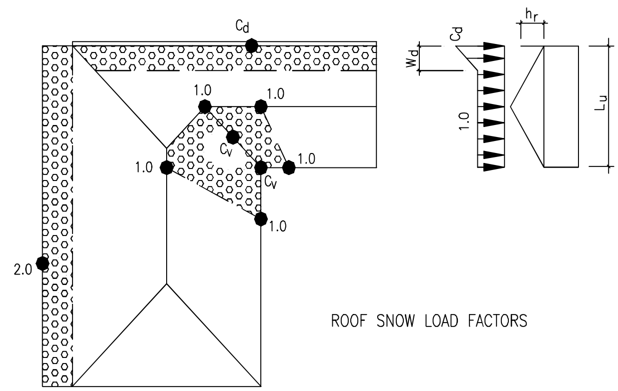
$C_{s,2}$ = MIN [(70 - α) / 40, 1.0] , Fig. 7-2a solid line

$C_{s,3}$ = MIN [(70 - α) / 60, 1.0] , Fig. 7-2b dash line

$C_{s,4}$ = MIN [(70 - α) / 32.5, 1.0] , Fig. 7-2b solid line

$C_{s,5}$ = MIN [(70 - α) / 55, 1.0] , Fig. 7-2c dash line

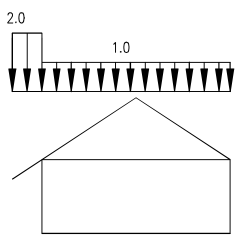
$C_{s,6}$ = MIN [(70 - α) / 25, 1.0] , Fig. 7-2c solid line



ROOF SNOW LOAD FACTORS

SNOW LOADS AT OVERHANG, VALLEY, AND PARAPET CORNER

$P_{f,\ overhang} = 2\ P_f$ = 33.60 psf, (Sec.7.4.5, pg 31)



$P_{f,\ valley} = C_v\, P_f$ = 33.60 psf, (Sec.7.6.3, pg 32)

Where $C_v = 2 / C_e$ = 2.00 , (Fig. 7-6, pg 40)

$P_{f,\ parapet} = C_d\, P_s = \mathrm{MIN}[\gamma(h_d + h_b),\ \gamma h_r]$

= 38.06 psf, (Sec.7.7.1, pg 32)

Where $\gamma = \mathrm{MIN}(0.13P_g + 14,\ 30)$ = 16.60 pcf, (7.7-1)

$h_b = P_s\ /\ \gamma$ = 0.95 ft, (Sec. 7.1)

$h_c = h_r - h_b$ = 3.05 ft, (Fig. 7.8)

$h_c\ /\ h_b$ = 3.22 **>** 0.2, (Sec.7.7.1)

(Drift load need be considered)

$h_d = 0.75\ [0.43(L_u)^{1/3}(P_g + 10)^{1/4} - 1.5]$

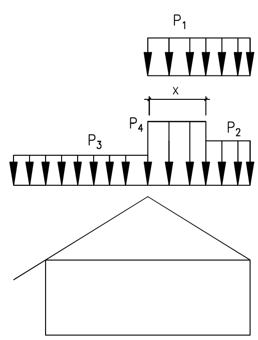
= 1.34 ft, (Sec.7.8 & Fig.7-9)

C_d = 2.42 , (see fig. right)

$W_d = \begin{cases} 4h_d = & 5.38 \text{ ft, for } h_d<h_c \\ 4h_d^2\ /\ h_c = & \text{N/A ft, for } h_d>h_c \end{cases}$

(Sec.7.7.1, pg 33)

$W_{d,max} = 8\ h_c$ = 24.41 ft



ROOF UNBLANCED SNOW LOADS

ROOF UNBALANCED SNOW LOADS

$P_{surcharge}$ = 5.00 psf, (Sec. 7.10, pg 33)

$P_1 = P_{surcharge} + I_s\ P_g$ = 25.00 psf, (unbalanced load, Fig 7-5)

$P_2 = (P_{surcharge} + P_{s)}$ = 20.75 psf, (unbalanced load, Fig 7-5)

$P_3 = 0.3\ P_2$ = 6.23 psf, (unbalanced load, Fig 7-5)

$P_4 = P_2 + h_d\ \gamma\ /\ (1\ /\ \mathrm{Tan}\ \alpha_{roof})^{0.5}$ = 33.63 psf

$x = (8\ /\ 3)\ h_d\ (1\ /\ \mathrm{Tan}\ \alpha_{roof})^{0.5}$ = 6.21 ft

Note : Where flat roof snow loads exceed 30 psf, the seismic dead load shall include 20% design snow load. (ASCE 7-10 Sec. 12.7.2.4)

Snow Load Analysis Based on UBC 97

INPUT DATA & DESIGN SUMMARY (UBC Sec. 1614)

TOTAL SNOW LOAD	S =	75	psf	$w_{snow} = S - R_s(\alpha - 20) =$	61.3	psf
PITCH DEGREE	α =	30	°	Where		

$R_s = \{$ S/40 - 1/2 = 1.375 psf, (for $\alpha > 20^o$)
N/A (for $\alpha < 20^o$)

INPUT DATA & DESIGN SUMMARY (UBC / CBC Appendix Chapter 16)

BASIC GROUND SNOW LOAD	P_g =	35	psf			
SNOW EXPOSURE FACTOR	C_e =	0.9	(Tab. A-16-A)	$P_{f,\ roof}$ =	31.50	psf
IMPORTANCE FACTOR	I =	1	(Tab. A-16-B)	$P_{f,\ overhang}$ =	63.00	psf
OBSTRUCTED SLIPPERY SURFACE ON ROOF **?** (1=Yes, 0=No)		0	Unobstructed	$P_{f,\ valley}$ =	59.72	psf
HEATED GREENHOUSES **?** (1=Yes, 0=No)	= = >	0	Unheated	$P_{f,\ parapet}$ =	51.38	psf
ROOF SLOPE	α =	18	°	W_d =	4.29	ft
PARAPET HEIGHT (DRIFT CORNER HEIGHT)	h_r =	4	ft, see fig. below			
LENGTH FROM PARAPET TO ROOF EDGE	W =	35	ft, see fig. below			

THE ROOF SNOW LOADS (CBC 40-1-1)

$P_f = C_e\ I\ P_g$ = 31.50 psf

THE APPLICABLE ROOF SNOW LOADS (CBC Sec. 1640)

$P_{f,\ roof} = C_s\ P_f$ = 31.50 psf

Where C_sP_f is derived from CBC Sec.1640 as following table

	P_g	[0, 20]	(20, 70]	(70, 100]	(100, greater]
Unheated	Unobstructed	Max($C_{s,1}P_f$, P_g)	Max($C_{s,1}P_f$, 20)	Max($C_{s,1}P_f$, 70C_eI)	Max($C_{s,5}P_f$, 70C_eI)
	Obstructed	Max($C_{s,2}P_f$, P_g)	Max($C_{s,2}P_f$, 20)	Max($C_{s,2}P_f$, 70C_eI)	Max($C_{s,5}P_f$, 70C_eI)
Heated	Unobstructed	Max($C_{s,3}P_f$, P_g)	Max($C_{s,3}P_f$, 20)	Max($C_{s,3}P_f$, 70C_eI)	Max($C_{s,5}P_f$, 70C_eI)
	Obstructed	Max($C_{s,4}P_f$, P_g)	Max($C_{s,4}P_f$, 20)	Max($C_{s,4}P_f$, 70C_eI)	Max($C_{s,5}P_f$, 70C_eI)

$C_{s,1}$ = MIN [1-(α - 30) / 40, 1.0] , CBC (40-2-1)
$C_{s,2}$ = MIN [1-(α - 45) / 25, 1.0] , CBC (40-2-2)
$C_{s,3}$ = MIN [1-(α - 15) / 55, 1.0] , CBC (40-2-3)
$C_{s,4}$ = MIN [1-(α - 30) / 40, 1.0] , CBC (40-2-4)
$C_{s,5}$ = MIN [1-(α - 20)(P_f - 20) / (40P_f), 1.0] , CBC (40-2-5)

THE SNOW LOADS AT OVERHANG, VALLEY, AND PARAPET CORNER

$P_{f,\ overhang} = 2\ P_{f,\ roof}$ = 63.00 psf, (CBC Fig. A-16-10)

$P_{f,\ valley} = C_v\ P_{f,\ roof}$ = 59.72 psf, (CBC Fig. A-16-12)

Where θ = 90 °, roof intersection angle

C_v = 1.90 (CBC Fig. A-16-11)

$P_{f,\ parapet} = C_d\ P_{f,\ roof} = P_m$ = MIN[D(h_d + h_b), Dh_r]

= 51.38 psf, (CBC Eq. 44-4)

Where D = MIN(0.13P_g+ 14, 35) = 18.55 pcf, (44-2)

$h_b = P_{f,\ roof}$ / D = 1.70 ft, (44-3)

(h_r - h_b) / h_b = 1.36 **>** 0.2, (44-3)

(Drift load need be considered)

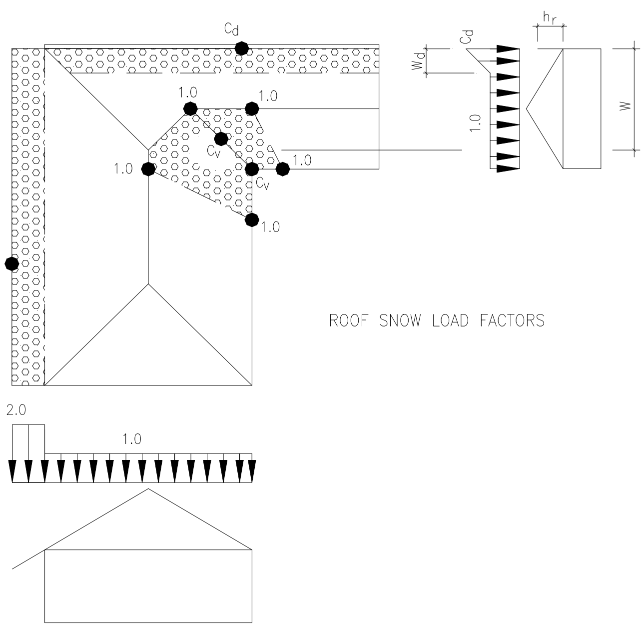
W_b = MIN[W , 50] = 35.00 ft, (Sec.1644.5)

$h_d = 0.5\ [0.43(W_b)^{1/3}(P_g + 10)^{1/4} - 1.5]$

= 1.07 ft,(1644.5 & 44-1)

$C_d = P_m / P_{f,\ roof}$ = 1.63

W_d = MIN[4(h_r - h_b), 4h_d] = 4.29 ft, (Sec.1644.2)



Note : Where design snow loads exceed 30 psf, the seismic dead load shall include 25% design snow load. (UBC 1630.1.1)

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PROJECT :
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Snow Load Analysis Based on ASCE 7-05 / IBC 2009

INPUT DATA & DESIGN SUMMARY

BASIC GROUND SNOW LOAD (ASCE Sec. 7.2)	P_g =	**20**	(ASCE page 81)	$P_{f,\ roof}$ =	15.75	psf
SNOW EXPOSURE FACTOR	C_e =	**1**	(Tab. 7-2, pg 92)	$P_{f,\ overhang}$ =	33.60	psf
THERMAL FACTOR (0.85 , 1.0 , 1.1 , 1.2)	C_t =	**1.2**	(Tab. 7-3, pg 93)	$P_{f,\ valley}$ =	33.60	psf
IMPORTANCE FACTOR	I =	**1**	(Tab. 7-4, pg 93)	$P_{f,\ parapet}$ =	38.06	psf
ROOF SLOPE		**4** / 12		W_d =	5.38	ft
PARAPET HEIGHT (DRIFT CORNER HEIGHT)	h_r =	**4**	ft, see fig. below			
LENGTH OF THE ROOF UPWIND OF THE DRIFT	L_u =	**35**	ft, see fig. below	P_1 =	25.00	psf
OBSTRUCTED SLIPPERY SURFACE (Sec. 7.4.0, pg 81) **?** (1=Yes, 0=No)		**0**	Unobstructed	P_2 =	20.75	psf
				P_3 =	6.23	psf
				P_4 =	33.63	psf
				x =	6.21	ft

ANALYSIS

FLAT SNOW LOADS (Sec 7.3, pg 81)

$P_f = 0.7C_eC_t\ I\ P_g$ = 16.80 psf

Where α_{roof} = 18.4 ^{0}

W = 17.5 ft

$P_{f,\ min}$ = 0.00 psf, (Sec. 7.3.4, pg 81)

ROOF SNOW LOADS (Sec. 7.4, pg 81)

$P_s = C_s\ P_f$ = 15.75 psf, (Eq.7-2)

Where C_s = 0.938

, Derived from Fig 7-2, page 86, as following table

C_t	0.85 or 1.0	1.1	1.2
Unobstructed	$C_{s,1}$	$C_{s,3}$	$C_{s,5}$
Obstructed	$C_{s,2}$	$C_{s,4}$	$C_{s,6}$

$C_{s,1}$ = MIN [(70 - α) / 65, 1.0] , Fig. 7-2a dash line

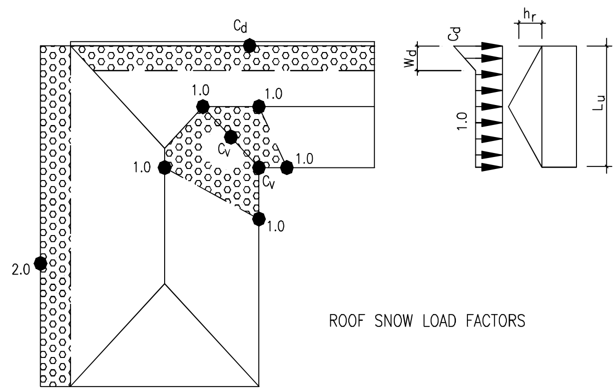
$C_{s,2}$ = MIN [(70 - α) / 40, 1.0] , Fig. 7-2a solid line

$C_{s,3}$ = MIN [(70 - α) / 60, 1.0] , Fig. 7-2b dash line

$C_{s,4}$ = MIN [(70 - α) / 32.5, 1.0] , Fig. 7-2b solid line

$C_{s,5}$ = MIN [(70 - α) / 55, 1.0] , Fig. 7-2c dash line

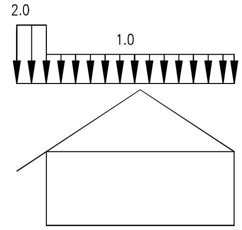
$C_{s,6}$ = MIN [(70 - α) / 25, 1.0] , Fig. 7-2c solid line



ROOF SNOW LOAD FACTORS

SNOW LOADS AT OVERHANG, VALLEY, AND PARAPET CORNER

$P_{f,\ overhang} = 2\ P_f$ = 33.60 psf, (Sec.7.4.5, pg 82)



$P_{f,\ valley} = C_v\ P_f$ = 33.60 psf, (Sec.7.6.3, pg 82)

Where $C_v = 2 / C_e$ = 2.00 , (Fig. 7-6, pg 90)

$P_{f,\ parapet} = C_d\ P_s = \mathrm{MIN}[\gamma(h_d + h_b),\ \gamma h_r]$

= 38.06 psf, (Sec.7.7.1, pg 83)

Where $\gamma = \mathrm{MIN}(0.13P_g + 14,\ 30)$ = 16.60 pcf, (7-3)

$h_b = P_s\ /\ \gamma$ = 0.95 ft, (Sec. 7.1)

$h_c = h_r - h_b$ = 3.05 ft, (Fig. 7.8)

h_c / h_b = 3.22 **>** 0.2, (Sec.7.7.1)

(Drift load need be considered)

$h_d = 0.75\ [0.43(L_u)^{1/3}(P_g + 10)^{1/4} - 1.5]$

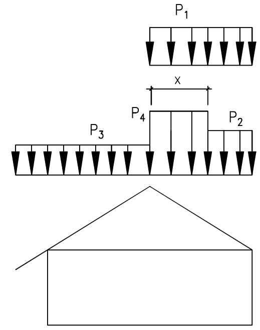
= 1.34 ft, (Sec.7.8 & Fig.7-9)

C_d = 2.42 , (see fig. right)

$$W_d = \begin{cases} 4h_d = 5.38 & \text{ft, for } h_d < h_c \\ 4h_d^2 / h_c = \text{N/A} & \text{ft, for } h_d > h_c \end{cases}$$

(Sec.7.7.1, pg 83)

$W_{d,max} = 8\ h_c$ = 24.41 ft



ROOF UNBLANCED SNOW LOADS

ROOF UNBALANCED SNOW LOADS

$P_{surcharge}$ = 5.00 psf, (Sec. 7.10, pg 83)

$P_1 = P_{surcharge} + I\ P_g$ = 25.00 psf, (unbalanced load, Fig 7-5)

$P_2 = (P_{surcharge} + P_s)$ = 20.75 psf, (unbalanced load, Fig 7-5)

$P_3 = 0.3\ P_2$ = 6.23 psf, (unbalanced load, Fig 7-5)

$P_4 = P_2 + h_d\ \gamma\ /\ (1 / \mathrm{Tan}\ \alpha_{roof})^{0.5}$ = 33.63 psf

$x = (8 / 3)\ h_d\ (1 / \mathrm{Tan}\ \alpha_{roof})^{0.5}$ = 6.21 ft

Note : Where flat roof snow loads exceed 30 psf, the seismic dead load shall include 20% design snow load. (ASCE 7-05 Sec. 12.7.2.4)

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Live Load Reduction Based on ASCE 7-2010

INPUT DATA & DESIGN SUMMARY

MEMBER TYPE (0=Beam, 1=Column)		1	Column
ROOF TRIBUTARY AREA SUPPORTED BY THE MEMBER	A_r =	975	ft^2, (if no roof, input 0.)
ROOF SLOPE		1 / 12	
NUMBER OF FLOORS	n =	4	
TOTAL FLOOR TRIBUTARY AREA SUPPORTED BY THE MEMBER	A_T =	3900	ft^2
FLOOR LIVE LOAD (ASCE 7-10 Table 4.1, page 17)	L_0 =	100	psf
FLOOR DEAD LOAD (for IBC/CBC Eq. 16-24)	D =	70	psf
GROUP A OCCUPANCIES ? (0=No, 1=Yes) (IBC/CBC 1607.9.2.1)		0	No

	UNIFORM	Σ
THE MINIMUM ROOF LIVE LOAD	12.00 psf	11.70 kips
THE MINIMUM FLOOR LIVE LOAD	49.02 psf	191.17 kips
TOTAL LOAD SUPPORTED BY THE COLUMN		202.87 kips

Note: Live loads are horizontal projected loads.

ANALYSIS

MINIMUM ROOF LIVE LOAD (ASCE 7-10 4.8)

$L_r = L_0 R_1 R_2 =$ 12.00 psf

Where $L_0 =$ 20 psf, (ASCE 7-10 Table 4-1, page 18)

$R_1 =$ 0.60

TRIBUTARY AREA	[0 ~ 200]	(200 ~ 600)	[600 ~ over)
R_1	1	1.2 -0.001 A_f	0.6

$R_2 =$ 1.00

ROOF SLOPE, F / 12	[0 ~ 4] / 12	(4 ~ 12) / 12	[12 ~ over) / 12
R_2	1	1.2 -0.05 F	0.6

$L_r =$ [12 , 20]

MINIMUM FLOOR LIVE LOAD BY INFLUENCE AREA METHOD (ASCE 7-10 4.7.2, page 15)

$L = L_0 [0.25 + 15 / (A_I)^{0.5}] =$ 49.02 psf <= =

Where $L_0 =$ 100 psf

$K_{LL} =$ 4 ,(ASCE 7-10 Table 4-2)

$A_I = K_{LL} A_T =$ 15600 ft^2

note:
1. Min reduced ,L, 50% for one level only, 40% for others.
2. Column live reaction can not be used to design footing, since K_{LL} different per ASCE 7-10 Table 4-2.

MINIMUM FLOOR LIVE LOAD BY TRIBUTARY AREA METHOD (IBC/CBC 1607.9.2)

L = (100 psf) x (100 - R) / 100 = 60.73 psf

Where r = 0.08

R = r (A_T - 150) = 39.3 <= =

note:
1. For floor live loads exceeding 100 psf, no reduction shall be made, except that design live loads on columns may be reduced 20 percent.
2. Max reduction ,R, per Eq 16-23, and 40% for one level only, 60% for others.
3. No reduction permitted in Group A occupancies.

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REVIEW BY :

Live Load Reduction Based on IBC 2009 / CBC 2010

INPUT DATA & DESIGN SUMMARY

MEMBER TYPE (0=Beam, 1=Column)		1	Column
ROOF TRIBUTARY AREA SUPPORTED BY THE MEMBER	A_r =	975	ft^2, (if no roof, input 0.)
ROOF SLOPE		1 / 12	
NUMBER OF FLOORS	n =	4	
TOTAL FLOOR TRIBUTARY AREA SUPPORTED BY THE MEMBER	A_T =	3900	ft^2
FLOOR LIVE LOAD (IBC Table 1607.1)	L_0 =	100	psf
FLOOR DEAD LOAD (for Eq. 16-24)	D =	70	psf
GROUP A OCCUPANCIES ? (0=No, 1=Yes) (IBC 1607.9.2.1)		0	No

	UNIFORM	Σ
THE MINIMUM ROOF LIVE LOAD	12.00 psf	11.70 kips
THE MINIMUM FLOOR LIVE LOAD	40.00 psf	156.00 kips
TOTAL LOAD SUPPORTED BY THE COLUMN		167.70 kips

Note: Live loads are horizontal projected loads.

ANALYSIS

MINIMUM ROOF LIVE LOAD (IBC 1607.11.2)

$L_r = L_0 R_1 R_2 =$ 12.00 psf

Where $L_0 =$ 20 psf, (IBC Tab 1607.1 Item 30)

$R_1 =$ 0.60

TRIBUTARY AREA	[0 ~ 200]	(200 ~ 600)	[600 ~ over)
R_1	1	1.2 -0.001 A_f	0.6

$R_2 =$ 1.00

ROOF SLOPE, F / 12	[0 ~ 4] / 12	(4 ~ 12) / 12	[12 ~ over) / 12
R_2	1	1.2 -0.05 F	0.6

L_r = [12 , 20]

MINIMUM FLOOR LIVE LOAD BY INFLUENCE AREA METHOD (IBC 1607.9.1)

$L = L_0 [0.25 + 15 / (A_I)^{0.5}] =$ 40.00 psf <= =

Where $L_0 =$ 100 psf

$K_{LL} =$ 4 ,(IBC Table 1607.9.1)

$A_I = K_{LL} A_T =$ 15600 ft^2

note: 1. Min reduced ,L, 50% for one level only, 40% for others.
2. Column live reaction can not be used to design footing, since K_{LL} different per Table 1607.9.1.

MINIMUM FLOOR LIVE LOAD BY TRIBUTARY AREA METHOD (IBC 1607.9.2)

L = (100 psf) x (100 - R) / 100 = 60.73 psf

Where r = 0.08

R = r (A_T - 150) = 39.3 <= =

note: 1. For floor live loads exceeding 100 psf, no reduction shall be made, except that design live loads on columns may be reduced 20 percent.
2. Max reduction ,R, per Eq 16-23, and 40% for one level only, 60% for others.
3. No reduction permitted in Group A occupancies.

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Live Load Reduction Based on CBC 2001

INPUT DATA & DESIGN SUMMARY

MEMBER TYPE (0=Beam, 1=Column)		**1**	Column
ROOF TRIBUTARY AREA SUPPORTED BY THE MEMBER	A_r =	**500**	ft^2, (if no roof, input 0.)
ROOF SLOPE		**4 / 12**	
NUMBER OF FLOORS	n =	**1**	
TOTAL FLOOR TRIBUTARY AREA SUPPORTED BY THE MEMBER	A_f =	**700**	ft^2
FLOOR LIVE LOAD (CBC Table 16-A, or Table 16A-A)	L =	**100**	psf
DSA or OSHPD PROJECT ? (0=No, 1=Yes)		**0**	No

	UNIFORM	Σ
THE MINIMUM ROOF LIVE LOAD	12.64 psf	6.32 kips
THE MINIMUM FLOOR LIVE LOAD	53.35 psf	37.34 kips
TOTAL LOAD SUPPORTED BY THE COLUMN		43.66 kips

Note: Live loads are horizontal projected loads.

ANALYSIS

MINIMUM ROOF LIVE LOAD (CBC Table 16-C)

METHOD	METHOD 1			METHOD 2		
	TRIBUTARY AREA			UNIFORM	REDUCTION	MAXIMUM
ROOF SLOPE	[0 ~ 200]	(200 ~ 600]	(600 ~ over)	L_r (psf)	r	R (%)
[0~4) / 12	20	16	12	20	0.08	40
[4~12) / 12	16	14	12	16	0.06	25
[12~over) / 12	12	12	12	12	0	0
				16	0.06	25
MINIMUM L_r		14 psf		12.64 psf		

MINIMUM FLOOR LIVE LOAD BY TRIBUTARY AREA METHOD (CBC 1607.5)

L = (100 psf) x (100 - R) / 100 = 60.00 psf

Where r = 0.08

R = r (A_f - 150) = 40.0 < = =

note: 1. For storage loads exceeding 100 psf, no reduction shall be made, except that design live loads on columns may be reduced 20 percent.
2. Max reduction ,R, 40% for one level only, 60% for others.

MINIMUM FLOOR LIVE LOAD BY INFLUENCE AREA METHOD (CBC 1607.6)

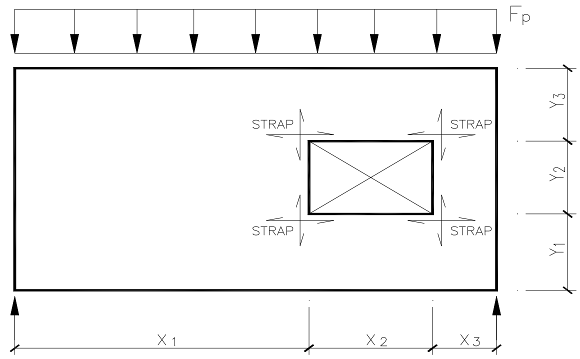
$L = L_0 [0.25 + 15 / (A_I)^{0.5}]$ = 53.35 psf < = =

Where L_0 = 100 psf

A_I = 2800 ft^2

note: 1. This method Not adopted by DSA or OSHPD.
2. Min reduced ,L, 50% for one level only, 40% for others.

Flexible Diaphragm with an Opening Analysis



INPUT DATA

LATERAL DIAPHRAGM FORCE :			F_p =	380	plf			
DIMENSIONS:	X_1 =	72	ft , X_2 =	24	ft , X_3 =	16	ft	
	Y_1 =	20	ft , Y_2 =	20	ft , Y_3 =	20	ft	

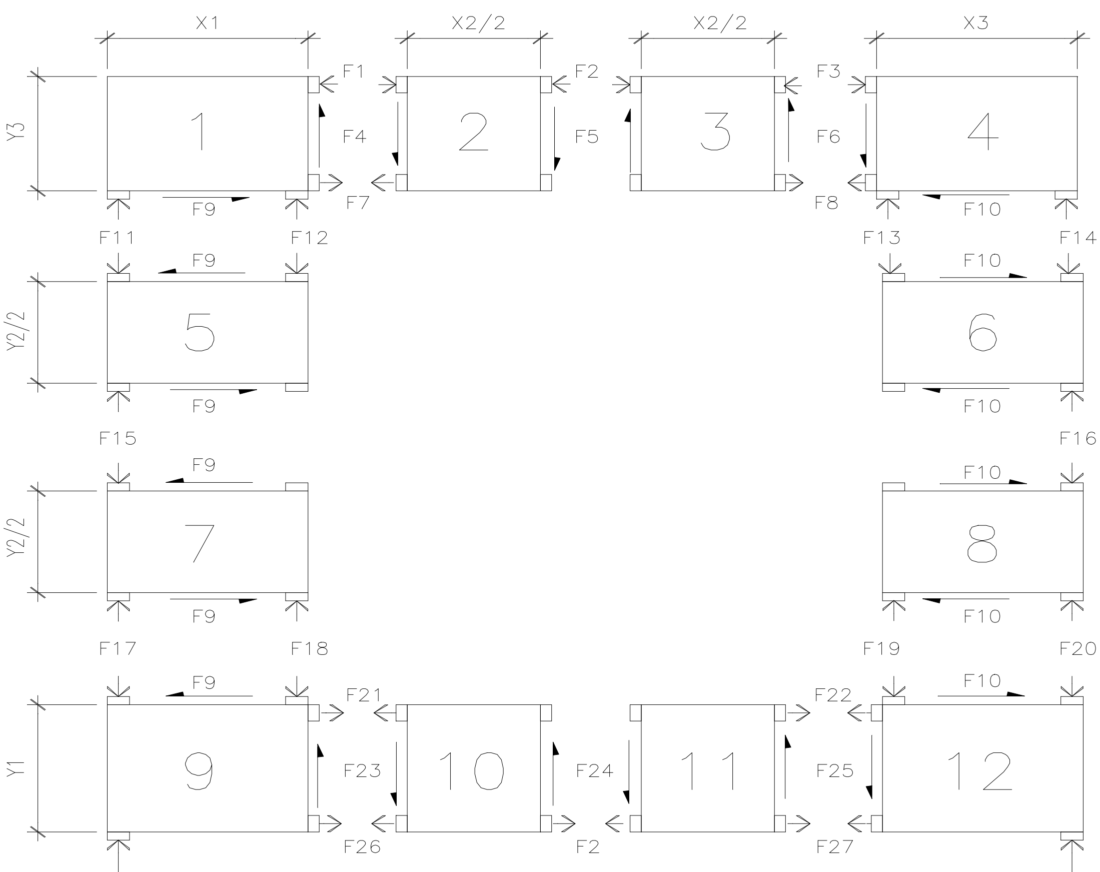
DESIGN SUMMARY

THE MAXIMUM STRAP FORCE, T = C = 4.66 kips

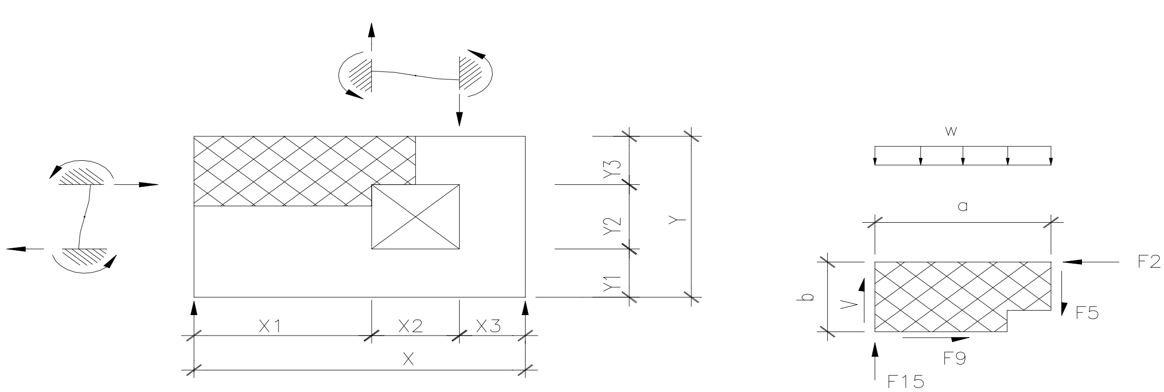
THE MAXIMUM SHEAR STRESS, v_x = 466 plf

v_y = 480 plf

ANALYSIS



VL FREE-BODY INDIVIDUAL PANELS OF DIAPHRAGM VR



ASSUME INFLECTION POINT AT MIDDLE OF OPENING

(cont'd)

REACTIONS : V_L = 21.280 kips , V_R = 21.280 kips

HATCHED AREA : $a = X_1 + 0.5\ X_2 =$ 84.00 ft , $b = 0.5\ Y_2 + Y_3 =$ 30.00 ft

$Y = Y_1 + Y_2 + Y_3 =$ 60.00 ft , $w = F_p\ b / Y =$ 190.0 plf

$M = a\ V_L - 0.5\ F_{px}\ a^2 =$ 446.9 ft-k, total moment at middle opening , $V = b\ V_L / Y =$ 10.640 kips

$F_2 = M / Y =$ 7448 lbf , $F_9 = F_2 =$ 7448 lbf

$F_5 = (F_2\ b - 0.5\ w\ a^2) / a =$ -5320 kips , $F_{15} = w\ a - V + F_5 =$ 0 kips

INDIVIDUAL PANEL	X (ft)	Y (ft)	v_x (plf)	v_y (plf)
1	72.00	20.00	138	480
2	12.00	20.00	209	266
3	12.00	20.00	323	380
4	16.00	20.00	223	299
5	72.00	10.00	103	103
6	16.00	10.00	466	466
7	72.00	10.00	103	103
8	16.00	10.00	466	466
9	72.00	20.00	138	480
10	12.00	20.00	209	266
11	12.00	20.00	323	380
12	16.00	20.00	223	299

NO.	FORCE (lbf)	NO.	FORCE (lbf)
F1	9956	F15	0
F2	7448	F16	0
F3	3572	F17	-2512
F4	3040	F18	-1034
F5	-5320	F19	-4655
F6	7600	F20	1108
F7	2508	F21	-2508
F8	-3876	F22	3876
F9	7448	F23	3040
F10	7448	F24	5320
F11	2512	F25	7600
F12	1034	F26	9956
F13	4655	F27	3572
F14	-1108		

Technical References:

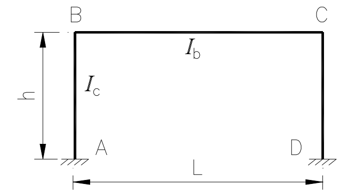
1. Kelly E. Cobeen, MS, SE: "Structural Engineering Review Workshop", BYA publications, 2005.
2. Doug Thompson, SE: "Guide to Design of Diaphragm", SEAOSC/NCSEA Seminar, 11-7-2009.

Formulas of Moment Resisting Frame at Bottom Fixed Condition

INPUT DATA & DESIGN SUMMARY

BEAM SECTION **W16X26** **W Shape** I_b = 301 in^4
ORIENTATION (x-x, y-y) **x-x** , (strong axis bending)

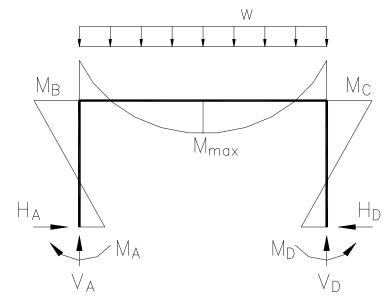
COLUMN SECTION **W10X30** **W Shape** I_c = 170 in^4
ORIENTATION (x-x, y-y) **x-x** , (strong axis bending)



BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft
STORY HEIGHT h = **14** ft

GRAVITY LOAD w = **1.8** klf

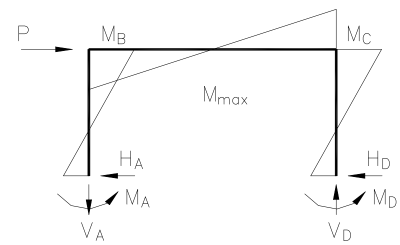
$H_A = H_D$ =	10.2	kips	$M_B = M_C$ =	95.5	ft-kips
$V_A = V_D$ =	27.0	kips	M_{max} =	107.0	ft-kips
$M_A = M_D$ =	47.8	ft-kips			



LATERAL LOAD P = **15** kips

$H_A = H_D$ =	7.5	kips	$M_B = M_C$ =	43.7	ft-kips
$V_A = V_D$ =	2.9	kips	Δ_H =	0.90	in
$M_A = M_D$ =	61.3	ft-kips	K_H =	16.6	kips / in

(K_H is frame stiffness.)



GRAVITY ANALYSIS

$$n = \frac{I_b}{I_c}\frac{h}{L} = 0.826 \qquad V_A = V_D = \frac{wL}{2} = 27.0 \text{ kips}$$

$$H_A = H_D = \frac{wL^2}{4h(2+n)} = 10.2 \text{ kips} \qquad M_A = M_D = \frac{wL^2}{12(2+n)} = 47.8 \text{ ft-kips}$$

$$M_{max} = \frac{wL^2(2+3n)}{24(2+n)} = 107.0 \text{ ft-kips} \qquad M_B = M_C = \frac{wL^2}{6(2+n)} = 95.5 \text{ ft-kips}$$

LATERAL ANALYSIS

$$a = h\left(\frac{3n}{6n+1}\right) = 5.83 \text{ ft} \qquad b = h\left(\frac{3n+1}{6n+1}\right) = 8.17 \text{ ft} \qquad H_A = H_D = \frac{P}{2} = 7.5 \text{ kips}$$

$$V_A = V_D = \frac{3nPh}{L(1+6n)} = 2.9 \text{ kips} \qquad M_A = M_D = \frac{Ph(1+3n)}{2(1+6n)} = 61.3 \text{ ft-kips}$$

$$E = 29000 \text{ ksi} \qquad M_B = M_C = \frac{3nPh}{2(1+6n)} = 43.7 \text{ ft-kips}$$

$$\Delta_H = \frac{P\left[\frac{b^3}{I_c} + a^2\left(\frac{a}{I_c} + \frac{L}{2I_b}\right)\right]}{6E} = 0.90 \text{ in} \qquad K_H = \frac{P}{\Delta_H} = 16.6 \text{ kips / in}$$

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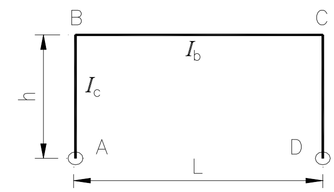
Formulas of Moment Resisting Frame at Bottom Pinned Condition

INPUT DATA & DESIGN SUMMARY



BEAM SECTION **W16X26** **W Shape** I_b = 301 in^4

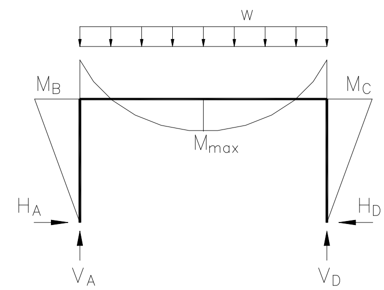
ORIENTATION (x-x, y-y) **x-x** , (strong axis bending)

COLUMN SECTION **W10X33** **W Shape** I_c = 171 in^4

ORIENTATION (x-x, y-y) **x-x** , (strong axis bending)

BEAM LENGTH BETWEEN COL. CENTERS L = **20** ft

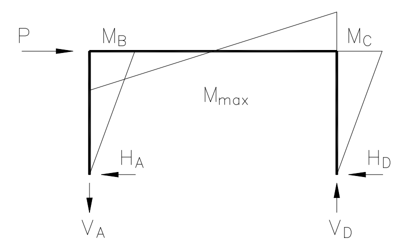
STORY HEIGHT h = **14** ft



GRAVITY LOAD w = **1.8** klf

$H_A = H_D =$ 2.4 kips $M_B = M_C =$ 32.9 ft-kips

$V_A = V_D =$ 18.0 kips $M_{max} =$ 57.1 ft-kips



LATERAL LOAD P = **15** kips

$H_A = H_D =$ 7.5 kips $M_B = M_C =$ 105.0 ft-kips

$V_A = V_D =$ 10.5 kips $\Delta_H =$ 3.36 in

$K_H =$ 4.5 kips / in

(K_H is frame stiffness.)

GRAVITY ANALYSIS

$$n = \frac{I_b}{I_c}\frac{h}{L} = 1.232 \qquad V_A = V_D = \frac{wL}{2} = 18.0 \text{ kips}$$

$$H_A = H_D = \frac{wL^2}{4h(3+2n)} = 2.4 \text{ kips} \qquad M_B = M_C = \frac{wL^2}{4(3+2n)} = 32.9 \text{ ft-kips}$$

$$M_{max} = \frac{wL^2(2n+1)}{8(2n+3)} = 57.1 \text{ ft-kips}$$

LATERAL ANALYSIS

$$H_A = H_D = \frac{P}{2} = 7.5 \text{ kips} \qquad E = 29000 \text{ ksi}$$

$$V_A = V_D = \frac{Ph}{L} = 10.5 \text{ kips} \qquad M_B = M_C = \frac{Ph}{2} = 105.0 \text{ ft-kips}$$

$$\Delta_H = \frac{Ph^3(1+2n)}{12nEI_c} = 3.36 \text{ in} \qquad K_H = \frac{P}{\Delta_H} = 4.5 \text{ kips / in}$$

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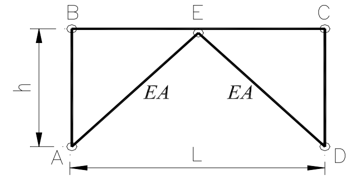
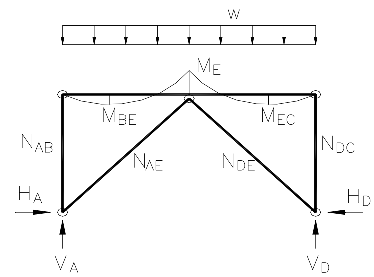
Formulas of Concentrically Braced Frame

INPUT DATA & DESIGN SUMMARY

BRACE SECTION **HSS6X6X1/2** **Tube** EA = 282460 kips

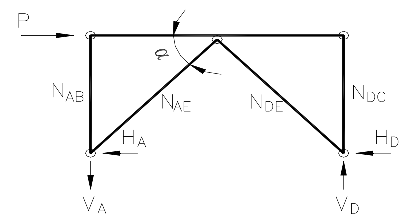


BEAM LENGTH BETWEEN COL. CENTERS L = **30** ft
STORY HEIGHT h = **14** ft



GRAVITY LOAD w = **1.8** klf

$H_A = H_D$ =	18.1	kips	$M_{BE} = M_{EC}$ =	28.5	ft-kips
$V_A = V_D$ =	27.0	kips	M_E =	50.6	ft-kips
$N_{AE} = N_{DE}$ =	24.7	kips	$N_{AB} = N_{DC}$ =	10.1	kips



LATERAL LOAD P = **15** kips

$H_A = H_D$ =	7.5	kips	$- N_{AE} = N_{DE}$ =	10.3	kips
$V_A = V_D$ =	7.0	kips	$- N_{AB} = N_{DC}$ =	0.0	kips
Δ_H =	0.01	in			
K_H =	1226.2	kips / in ,	(K_H is frame stiffness.)		

GRAVITY ANALYSIS (Neglected axial deformations.)

$$\alpha = \tan^{-1}\left(\frac{h}{0.5L}\right) = \quad 43.03\ ^0$$

$$V_A = V_D = \frac{wL}{2} = \quad 27.0 \text{ kips}$$

$$H_A = H_D = \frac{5wL}{16\tan\alpha} = \quad 18.1 \text{ kips}$$

$$M_{BE} = M_{EC} = \frac{9wL^2}{512} = \quad 28.5 \text{ ft-kips}$$

$$N_{AE} = N_{DE} = \frac{5wL}{16\sin\alpha} = \quad 24.7 \text{ kips}$$

$$M_E = \frac{wL^2}{32} = \quad 50.6 \text{ ft-kips}$$

LATERAL ANALYSIS

$$H_A = H_D = \frac{P}{2} = \quad 7.5 \text{ kips}$$

$$V_A = V_D = \frac{Ph}{L} = \quad 7.0 \text{ kips}$$

$$-N_{AE} = N_{DE} = \frac{P}{2\cos\alpha} = \quad 10.3 \text{ kips}$$

$$\Delta_H = \frac{PL}{4EA\cos^3\alpha} = \quad 0.01 \text{ in}$$

$$K_H = \frac{P}{\Delta_H} = \quad 1226.2 \text{ kips / in}$$

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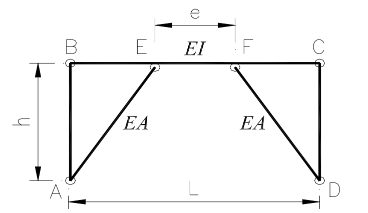
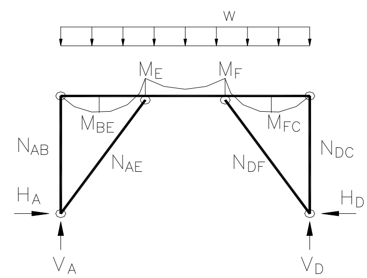
Formulas of Eccentrically Braced Frame

INPUT DATA & DESIGN SUMMARY



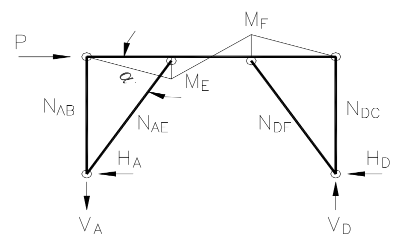
BRACE SECTION	**HSS6X6X1/2**	**Tube**	EA =	282460	kips
BEAM SECTION	**W16X26**	**W Shape**	EI =	8729000	kips-in^2
ORIENTATION (x-x, y-y)	**x-x**	, (strong axis bending)			

BEAM LENGTH BETWEEN COL. CENTERS	L =	**30**	ft
STORY HEIGHT	h =	**14**	ft
LINK LENGTH	e =	**4**	ft



GRAVITY LOAD w = **1.8** klf

$H_A = H_D$ =	16.0	kips	$M_{BE} = M_{FC}$ =	25.8	ft-kips
$V_A = V_D$ =	27.0	kips	$M_E = M_F$ =	24.5	ft-kips
$N_{AE} = N_{DF}$ =	23.5	kips	$N_{AB} = N_{DC}$ =	9.8	kips



LATERAL LOAD P = **15** kips

$H_A = H_D$ =	7.5	kips	$- N_{AE} = N_{DF}$ =	11.0	kips
$M_E = M_F$ =	14.0	ft-kips	$- N_{AB} = N_{DC}$ =	-1.1	kips
$Q_{EF} = V_A = V_D$ =	7.0	kips	Δ_H =	0.04	in
(Q_{EF} is link shear force.)			K_H =	384.5	kips / in
			(K_H is frame stiffness.)		

GRAVITY ANALYSIS (Neglected axial deformations.)

$$\alpha = \tan^{-1}\left(\frac{h}{0.5L - 0.5e}\right) = \quad 47.12\ ^{0}$$

$$M_E = M_F = \quad 24.5 \text{ ft-kips , (by moment distribution procedure.)}$$

$$V_A = V_D = \frac{wL}{2} = \quad 27.0 \text{ kips}$$

$$H_A = H_D = \frac{M_E}{h} + \frac{w\left(L^2 - e^2\right)}{8h} = \quad 16.0 \text{ kips}$$

$$N_{AE} = N_{DF} = \frac{H_A}{\cos\alpha} = \quad 23.5 \text{ kips}$$

$$M_{BE} = M_{FC} = \frac{w(L-e)^2}{32} - \frac{M_E}{2} = \quad 25.8 \text{ ft-kips}$$

LATERAL ANALYSIS

$$H_A = H_D = \frac{P}{2} = \quad 7.5 \text{ kips}$$

$$Q_{EF} = V_A = V_D = \frac{Ph}{L} = \quad 7.0 \text{ kips}$$

$$-N_{AE} = N_{DF} = \frac{P}{2\cos\alpha} = \quad 11.0 \text{ kips}$$

$$M_E = M_F = \frac{Peh}{2L} = \quad 14.0 \text{ ft-kips}$$

$$\Delta_H = \frac{Ph}{2EA\cos^2\alpha\sin\alpha} + \frac{Ph^2e^2}{12EIL} = \quad 0.04 \text{ in}$$

$$K_H = \frac{P}{\Delta_H} = \quad 384.5 \text{ kips / in}$$

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Rotation Analysis of Rigid Diaphragm Based on IBC 2009 / CBC 2010

INPUT DATA & DESIGN SUMMARY

DIAPHRAGM FORCES F_{px} = 1548.3 kips

F_{py} = 1548.3 kips

DIAPHRAGM EDGE COORDINATES

Start Point		End Point	
X (ft)	Y (ft)	X (ft)	Y (ft)
0	0	0	90
0	90	30	90
30	90	30	125
30	125	80	125
80	125	80	150
80	150	155	150
155	150	155	125
155	125	270	125
270	125	270	90
270	90	300	90
300	90	300	-10.67
300	-10.67	50	-30
50	-30		

LATERAL FRAME LOCATION, RELATIVE RIGIDITY, & FRAME LINE FORCES

Frame	Start Point		End Point		Relative	Frame F (kips)		Relative
Number	X (ft)	Y (ft)	X (ft)	Y (ft)	R	@ F_{px}	@ F_{py}	δ_{xe} (in)
1	30	0	30	90	1007.3	157.7	607.8	0.60
2	160	0	160	120	1087	8.6	543.9	0.50
3	270	0	270	90	1012.8	149.0	544.0	0.54
4	180	0	240	0	739.1	336.8	48.2	0.46
5	60	30.5	120	30.5	737.5	307.6	29.0	0.42
6	60	90	120	90	738.1	259.3	8.3	0.35
7	180	90	240	90	739.8	259.9	8.3	0.35
8	60	120	90	120	454.8	152.2	16.7	0.33
9	180	120	210	120	434.8	145.5	16.0	0.33
10	90	146	120	146	473.2	151.5	27.8	0.32

(cont'd)

DIAPHRAGM

A = 43108.8 ft^2

I_{cx} = 7.26E+08 ft^4

I_{cy} = 1.09E+09 ft^4

MASS MOMENT OF INERTIA

M = Wt / 32.174 , kips-sec^2 / ft ***(Wt / 384 , kips-sec^2 / in)***

I_{zc} = 8433 M , kips-ft-sec^2

(1214369 M , kips-in-sec^2)

CENTER OF MASS

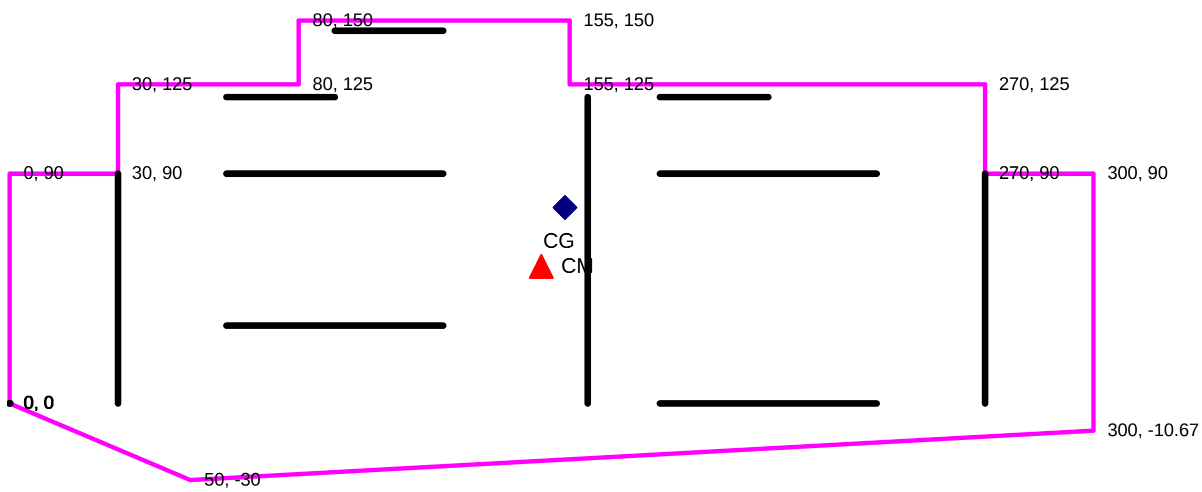
X_{cm} = 147.2 ft

Y_{cm} = 53.6 ft

CENTER OF RIGIDITY

X_{cg} = 153.7 ft

Y_{cg} = 76.7 ft



ANALYSIS

DETERMINE CENTER OF RIGIDITY

Frame Number	Relative R	Direction θ	R_x R $Cos^2\theta$	R_y R $Sin^2\theta$	X_g (ft)	Y_g (ft)	R_xY_g	R_yX_g
1	1007.3	90.0	0.0	1007.3	30.0	45.0	0.0	30219.0
2	1087	90.0	0.0	1087.0	160.0	60.0	0.0	173920.0
3	1012.8	90.0	0.0	1012.8	270.0	45.0	0.0	273456.0
4	739.1	0.0	739.1	0.0	210.0	0.0	0.0	0.0
5	737.5	0.0	737.5	0.0	90.0	30.5	22493.8	0.0
6	738.1	0.0	738.1	0.0	90.0	90.0	66429.0	0.0
7	739.8	0.0	739.8	0.0	210.0	90.0	66582.0	0.0
8	454.8	0.0	454.8	0.0	75.0	120.0	54576.0	0.0
9	434.8	0.0	434.8	0.0	195.0	120.0	52176.0	0.0
10	473.2	0.0	473.2	0.0	105.0	146.0	69087.2	0.0

$X_{cg} = \Sigma R_yX_g / \Sigma R_y$ = 153.7 ft

$Y_{cg} = \Sigma R_xY_g / \Sigma R_x$ = 76.7 ft

(cont'd)

Σ			4317.3	3107.1			331344.0	477595.0

DETERMINE ECCENTRICITY (ASCE 7-05 12.8.4.2)

	Actual	Additional	Total @ F_{px}		Total @ F_{py}	
	CG - CM	5% & A_x	Case 1	Case 2	Case 3	Case 4
e_x (ft)	6.5	15.0	21.5	-8.5		
e_y (ft)	23.1	9.0			32.1	14.1

A_x = 1 (ASCE 7-05 12.8.4.3)

(cont'd)

DETERMINE FRAME FORCES

Frame	r_x	r_y	T_y		T_x		$r_x^2 R_y$	$r_y^2 R_x$	F_x (kips)		F_y (kips)	
Number	(ft)	(ft)	Case 1	Case 2	Case 3	Case 4			Case 1	Case 2	Case 3	Case 4
1	-123.7	-31.7	33352.5	-13096.5	49699.2	21829.8	15416098	0	157.7	69.3	607.8	460.4
2	6.3	-16.7	33352.5	-13096.5	49699.2	21829.8	42994	0	-8.6	-3.8	535.9	543.9
3	116.3	-31.7	33352.5	-13096.5	49699.2	21829.8	13696262	0	-149.0	-65.5	404.7	544.0
4	56.3	-76.7	33352.5	-13096.5	49699.2	21829.8	0	4353483	336.8	296.6	48.2	-18.9
5	-63.7	-46.2	33352.5	-13096.5	49699.2	21829.8	0	1577419	307.6	283.4	29.0	-11.4
6	-63.7	13.3	33352.5	-13096.5	49699.2	21829.8	0	129623	252.3	259.3	-8.3	3.3
7	56.3	13.3	33352.5	-13096.5	49699.2	21829.8	0	129921	252.9	259.9	-8.3	3.3
8	-78.7	43.3	33352.5	-13096.5	49699.2	21829.8	0	850812	138.2	152.2	-16.7	6.6
9	41.3	43.3	33352.5	-13096.5	49699.2	21829.8	0	813397	132.1	145.5	-16.0	6.3
10	-48.7	69.3	33352.5	-13096.5	49699.2	21829.8	0	2269394	128.2	151.5	-27.8	10.9

(cont'd)

Σ							29155354	10124050				

$F_x = (F_{px}\, R_x) / (\Sigma\, R_x) - [T_x\, (R_x\, r_y + R_y\, r_x) / (\Sigma\, r_y^2\, R_x + \Sigma\, r_x^2\, R_y)]$

$F_y = (F_{py}\, R_y) / (\Sigma\, R_y) - [T_y\, (R_y\, r_x + R_x\, r_y) / (\Sigma\, r_x^2\, R_y + \Sigma\, r_y^2\, R_x)]$

Purchaser's LOGO

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Flexible Diaphragm Analysis

DIAPHRAGM DESIGN FORCE FACTOR F_{px} = 0.18 w_{px} , (ASCE 7-10 12.10.1.1)

DIAPHRAGM EDGE CORNER POINT

Corner Point	X (ft)	Y (ft)
1	0	66.66
2	60.02	66.66
3	122.64	3.03
4	60.02	0
5	112.31	0
6	86.53	0
7	122.64	18.6
8	112.31	8.26
9	122.64	28.9
10	91.67	28.9
11	122.64	38.21
12	115.69	38.21
13	115.69	52.92
14	86.53	52.92
15	0	93.18
16	0	118.12
17	77.98	127.12
18	115.69	138.11
19	67.43	118.12
20	77.98	107.57
21	115.69	93.18
22	86.53	93.18
23	88.52	118.12
24	115.69	118.12
25	67.43	127.12
26	88.52	127.12
27	186.14	138.11
28	186.14	48.08
29	186.13	54.89
30	151.59	11.07
31	134.14	11.09
32	134.14	3.03
33	130	3.03
34	130	0
35	168.9	11.09
36	184.15	3.03
37	168.9	3.03
38	173.16	3.03
39	173.16	0
40	184.15	38.03
41	186.14	38.03
42	249.25	48.08
43	214.72	54.89
44	214.72	48.08
45	186.14	122.28
46	199.28	122.28
47	199.28	115.18
48	151.59	138.11
49	130	138.11
50	130	151.32
51	173.16	138.11
52	173.16	151.32
53	249.25	76.99
54	214.72	115.18
55	214.72	76.99

DIAPHRAGM PANEL & SEISMIC DEAD LOAD

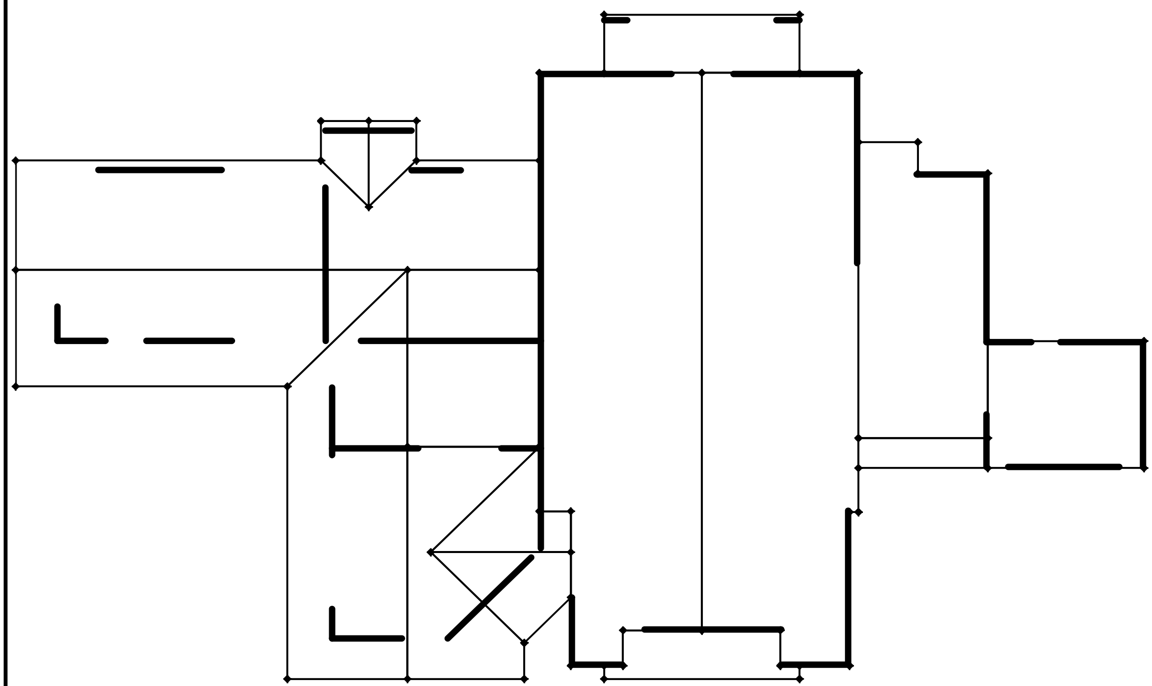
Panel No.	Corner 1	Corner 2	Corner 3	Corner 4	Corner 5	Corner 6	Corner 7	Corner 8	Corner 9	Seismic DL (psf)
1	15	16	19	20	23	24	21			18
2	25	17	20	19						23
3	20	17	26	23						23
4	1	15	22	2						18
5	4	2	22	6						18
6	14	22	21	13						50
7	6	14	13	10	8	5				18
8	10	13	12	11	9					18
9	10	9	7	8						18
10	3	11	12	18	48	30	31	32		17
11	49	50	52	51						23
12	34	33	32	31	35	37	38	39		23
13	30	48	27	41	40	36	37	35		17
14	45	46	47	54	43	29				18
15	29	43	44	28						23
16	55	53	42	44						17

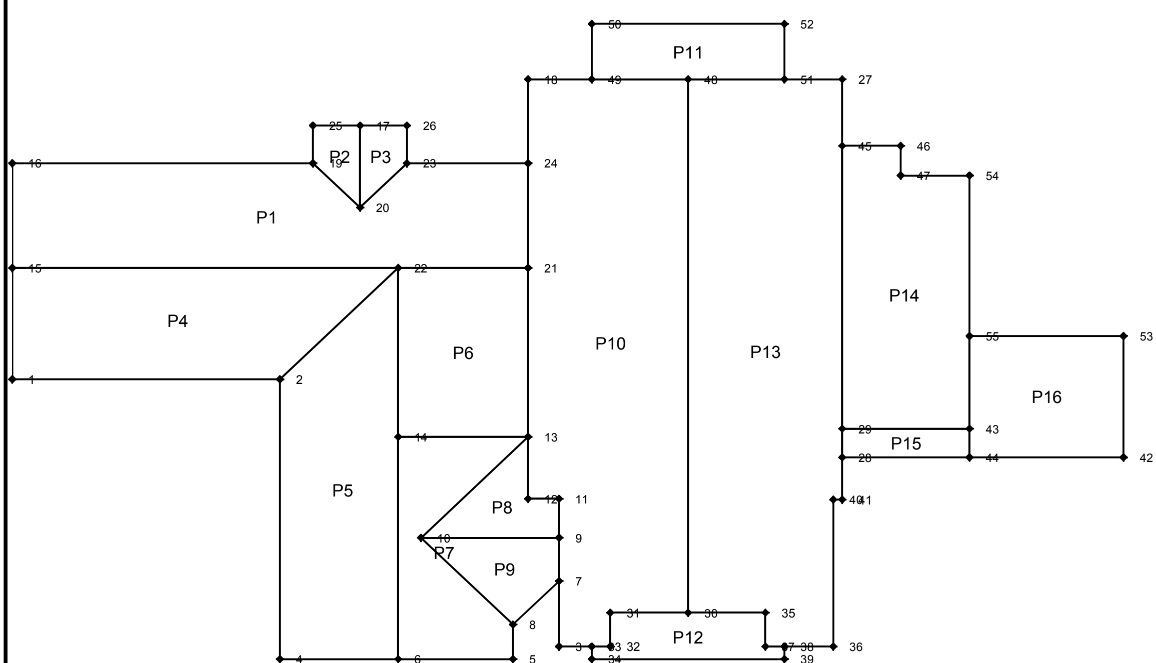
SHEAR WALL (LATERAL FRAME) LOCATION

SW NO.	Drag to Start (ft)	Start Point X (ft)	Start Point Y (ft)	End Point X (ft)	End Point Y (ft)	Drag fr End (ft)	Σ Drag Length	Wall Length
1	0	69.87	9.25	85.29	9.25	10.17	25.6	15.4
2	0	122.87	3.26	133.94	3.26	17	28.1	11.1
3	17	169.13	3.26	183.92	3.26	0	31.8	14.8
4	16.25	138.9	11.32	169.13	11.32	15	61.5	30.2
5	0	69.87	52.55	88.9	52.55	9	28.0	19.0
6	9	107.33	52.55	115.99	52.55	0	17.7	8.7
7	33	219.22	48.31	243.81	48.31	5.5	63.1	24.6
8	0	9.2	77.01	19.85	77.01	4.5	15.2	10.7
9	4.5	28.87	77.01	47.73	77.01	14.3	37.7	18.9
10	14.3	76.29	77.01	115.99	77.01	72	126.0	39.7
11	0	214.45	76.76	224.33	76.76	6.1	16.0	9.9
12	6.1	230.76	76.76	249.02	76.76	0	24.4	18.3
13	9	18.25	115.92	45.53	115.92	21	57.3	27.3
14	21	87.44	115.89	98.29	115.89	17.67	49.5	10.9
15	12.5	199.05	114.96	214.45	114.96	0	27.9	15.4
16	0	68.44	124.89	87.44	124.89	0	19.0	19.0
17	0	115.99	137.81	144.84	137.81	7	35.9	28.9
18	7	158.6	137.81	185.84	137.81	0	34.2	27.2
19	0	130	150.09	135.13	150.09	16.5	21.6	5.1
20	16.5	168.03	150.09	173.16	150.09	0	21.6	5.1
21	0	9.2	77.01	9.2	84.85	31	38.8	7.8
22	0	69.87	9.25	69.87	15.93	17.5	24.2	6.7
23	17.5	69.87	51.03	69.87	66.37	6	38.8	15.3
24	4	68.45	77.01	68.44	111.98	13	52.0	35.0
25	0	95.45	9.25	113.88	27.69	2	28.1	26.1
26	2	115.99	29.8	115.99	137.81	0	110.0	108.0
27	0	122.87	3.26	122.87	18.59	19.75	35.1	15.3
28	0	183.92	3.26	183.92	38.31	28	63.1	35.1
29	29	185.84	94.74	185.84	137.81	0	72.1	43.1
30	0	214.45	48.31	214.45	60.25	8.375	20.3	11.9
31	8.375	214.45	76.76	214.45	114.96	0	46.6	38.2
32	0	249.02	48.31	249.02	76.76	0	28.5	28.5

Tip: Input the START point of each wall at left/bottom & END at right/top, so easy to follow drag force locations.

(cont'd)





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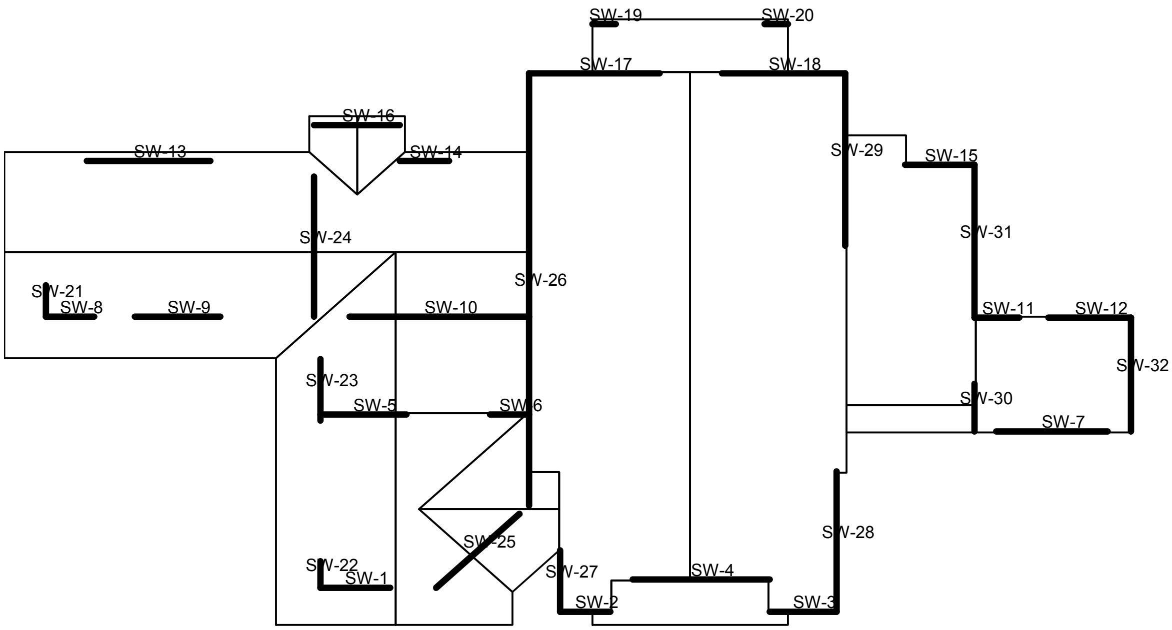

DIAPHRAGM PANEL LOAD AND CHORD FORCE

Panel No.	Area (ft^2)	Centroid X (ft)	Centroid Y (ft)	F_{px} (kips)	Chord Force L (ft)	Chord Force T / C (k)
1	2774	57.0	105.3	9.0	59.3	1.4
2	151	73.4	119.7	0.6	47.3	1.2
3	150	82.6	119.7	0.6	47.3	1.2
4	1943	37.0	80.7	6.3	59.3	1.4
5	2119	74.0	40.3	6.9	43.3	0.8
6	1174	101.1	73.1	10.6	45.9	2.4
7	944	96.8	25.5	3.1	34.0	0.5
8	353	109.8	36.3	1.1	34.0	0.5
9	373	110.3	21.5	1.2	34.0	0.5
10	4464	134.1	75.3	13.7	70.5	1.9
11	570	151.6	144.7	2.4	43.2	1.0
12	411	151.5	5.3	1.7	61.0	1.9
13	4458	168.9	73.3	13.6	70.5	1.9
14	1817	200.0	86.8	5.9	67.0	1.8
15	195	200.4	51.5	0.8	28.3	0.4
16	998	232.0	62.5	3.1	34.8	0.5
Σ	**22894**			**80.5**		

Note:
The Chord Force, T / C (kips), is from the equation,
T and/or C = Moment / Depth = $(F_{px} / Area)\ L^2 / 8$,
where L should be actual maximum supported span and not larger than $(3\ A)^{0.5}$ since diaphragm ratio 3 max.

(cont'd)

TRIBUTARY DIAPHRAGM AREA TO **HORIZONTAL (X DIRECTION)** SHEAR WALL

Panel	Shear Wall			Shear Wall			Shear Wall			Shear Wall			Σ
No.	Number	Top	Bottom	Number	Top	Bottom	Number	Top	Bottom	Number	Top	Bottom	%
1	13	3%	70%	14	3%	40%							116%
2	16	3%	100%										103%
3	16	3%	100%										103%
4	8	25%	20%	9	35%	30%							110%
5	10	3%	20%	5	20%	28%	1	28%	10%				109%
6	10	45%	35%	6	25%	0%							105%
7	5	0%	15%	6	0%	45%	25	45%	0%				105%
8	6	0%	50%	25	55%	0%							105%
9	25	55%	55%										110%
10	17	0%	30%	10	25%	25%	4	25%	0%	2	5%	0%	110%
11	19	0%	25%	20	0%	25%	17	25%	0%	18	25%	0%	100%
12	4	0%	100%										100%
13	18	0%	30%	10	25%	25%	4	25%	0%	3	5%	0%	110%
14	15	10%	50%	7	45%	0%							105%
15	7	100%											100%
16	11	0%	25%	12	0%	30%	7	50%	0%				105%

TRIBUTARY DIAPHRAGM AREA TO **VERTICAL (Y DIRECTION)** SHEAR WALL

Panel	Shear Wall			Shear Wall			Shear Wall			Shear Wall			Σ
No.	Number	Left	Right	Number	Left	Right	Number	Left	Right	Number	Left	Right	%
1	21	8%	30%	24	30%	20%	26	20%	0%				108%
2	24	0%	100%										100%
3	24	0%	50%	26	55%	0%							105%
4	21	5%	40%	24	40%	18%							103%
5	22	10%	40%	23	10%	40%	26	10%	0%				110%
6	23	0%	10%	24	0%	15%	26	80%	0%				105%
7	22	0%	25%	23	0%	25%	25	15%	15%	26	30%	0%	110%
8	23	0%	5%	25	20%	0%	26	80%	0%				105%
9	23	0%	8%	25	30%	0%	26	80%	0%				118%
10	26	0%	100%										100%
11	26	0%	55%	29	55%	0%							110%
12	26	0%	55%	28	55%	0%							110%
13	28	50%	0%	29	55%	0%							105%
14	29	0%	52%	31	52%	0%							104%
15	29	0%	50%	30	55%	0%							105%
16	30	0%	45%	31	0%	10%	32	50%	0%				105%

(cont'd)

DIAPHRAGM SHEAR STRESS AT EACH SIDE OF SHEAR WALL & DRAG FORCE AT TWO ENDS OF SHEAR WALL

Shear	**Diaphragm**								**Shear Wall**	
Wall	**Shear** (k)		**Length** (ft)		**Stress** (plf)		**Drag Force** (k)		**Force**	**Stress**
NO.	**L / T**	**R / B**	**L / T**	**R / B**	**L / T**	**R / B**	**Start**	**End**	(k)	(plf)
1	1.9	0.7	25.6	25.6	75	27	0.0	1.0	2.6	169
2	0.7	0.0	28.1	28.1	24	0	0.0	0.4	0.7	62
3	0.7	0.0	31.8	31.8	21	0	0.4	0.0	0.7	46
4	6.8	1.7	61.5	61.5	111	28	2.3	2.1	8.5	282
5	1.4	2.4	28.0	28.0	49	85	0.0	1.2	3.8	197
6	2.6	1.9	17.7	17.7	150	110	2.3	0.0	4.6	530
7	5.0	0.0	63.1	63.1	79	0	2.6	0.4	5.0	203
8	1.6	1.3	15.2	15.2	104	83	0.0	0.8	2.8	266
9	2.2	1.9	37.7	37.7	59	50	0.5	1.6	4.1	217
10	11.8	11.9	126.0	126.0	94	94	2.7	13.5	23.7	597
11	0.0	0.8	16.0	16.0	0	48	0.0	0.3	0.8	77
12	0.0	0.9	24.4	24.4	0	38	0.2	0.0	0.9	50
13	0.3	6.3	57.3	57.3	5	110	1.0	2.4	6.6	241
14	0.3	3.6	49.5	49.5	5	73	1.6	1.4	3.9	356
15	0.6	2.9	27.9	27.9	21	105	1.6	0.0	3.5	229
16	0.0	1.2	19.0	19.0	2	66	0.0	0.0	1.3	68
17	0.6	4.1	35.9	35.9	16	114	0.0	0.9	4.7	163
18	0.6	4.1	34.2	34.2	17	120	1.0	0.0	4.7	172
19	0.0	0.6	21.6	21.6	0	27	0.0	0.5	0.6	115
20	0.0	0.6	21.6	21.6	0	27	0.5	0.0	0.6	115
21	1.0	5.2	38.8	38.8	27	134	0.0	5.0	6.2	797
22	0.7	3.5	24.2	24.2	28	145	0.0	3.0	4.2	628
23	0.7	4.7	38.8	38.8	18	122	2.4	0.8	5.4	352
24	5.2	5.5	52.0	52.0	100	105	0.8	2.7	10.7	305
25	3.7	1.1	28.1	28.1	133	40	0.0	0.3	4.8	186
26	14.1	15.9	110.0	110.0	128	144	0.5	0.0	30.0	278
27	0.0	0.0	35.1	35.1	0	0	0.0	0.0	0.0	0
28	7.8	0.0	63.1	63.1	123	0	0.0	3.4	7.8	221
29	8.8	3.5	72.1	72.1	122	48	4.9	0.0	12.3	285
30	0.4	1.4	20.3	20.3	22	68	0.0	0.7	1.8	152
31	3.1	0.3	46.6	46.6	66	7	0.6	0.0	3.4	88
32	1.5	0.0	28.5	28.5	54	0	0.0	0.0	1.5	54
Max.					**150**	**145**	**4.9**	**13.5**	**30.0**	**797**

Daniel T. Li
Engineering International

PROJECT :
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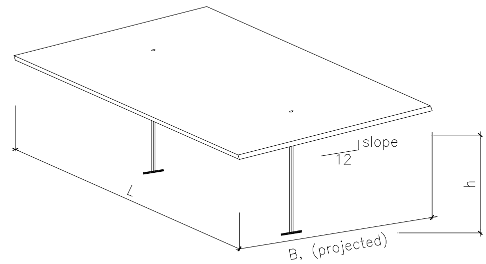
Wind Analysis for Open Structure (Solar Panels) Based on ASCE 7-2010

INPUT DATA



Exposure category (B, C or D)		=	C	
Importance factor, 1.0 only, (Table 1.5-2)	I_w	=	1.00	
Basic wind speed (ASCE 7-10 26.5.1)	V	=	136	mph
Topographic factor (26.8 & Table 26.8-1)	K_{zt}	=	1	Flat
Height of mean roof	h	=	10.5	ft
Roof slope			0	**: 12**
Roof length	L	=	16	ft
Roof horizontal projected width	B	=	11.33	ft
Effective area of component / cladding	A	=	10	ft^2

DESIGN SUMMARY

1. Main Wind-Force Resisting System

Max horizontal force / base shear from roof	=	0.00	kips
	+	0.00	kips (eave & columns increasing)
	Σ	0.00	kips
Max vertical download force	=	3.95	kips
Max vertical uplift force	=	4.48	kips
Max moment at centroid of base from roof	=	7.47	ft-kips
	+	0.75	ft-kips (eave & columns increasing)
	Σ	8.21	ft-kips

2. Component and Cladding Elements

Max inward pressure	=	69	psf
Max net outward pressure	=	103	psf

ANALYSIS

Velocity pressure

$q_h = 0.00256\ K_h\ K_{zt}\ K_d\ V^2$ = **34.21** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 28.3-1, pg 299) = **0.85**

K_d = wind directionality factor. (Tab. 26.6-1, for building, page 250) = **0.85**

h = height of mean roof = **10.50** ft

Main Wind-Force Resisting System (Sec. 27.4.3)

$p = q_h\ G\ C_N$

$F = p\ A_f$

where: G = gust effect factor. (Sec. 26.9) = **0.85**

θ = roof angle = **0.00** degree

< **45 deg.** **[Satisfactory]**

A_f = roof actual area. = **90.6** ft^2, windward

= **90.6** ft^2, leeward

C_N = net force coefficients. (Fig. 27.4-4, page 267)

Check Fig. 27.4-4 limitation h / B = 0.93 within [0.25 , 1.0] **[Satisfactory]**

θ	CASE	WIND TO BOTTOM, $\gamma = 0^o$				WIND TO TOP, $\gamma = 180^o$			
		CLEAR FLOW		OBSTRUCTED		CLEAR FLOW		OBSTRUCTED	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}
0.00	A	1.20	0.30	-0.50	-1.20	1.20	0.30	-0.50	-1.20
	p (psf)	34.89	8.72	-14.54	-34.89	34.89	8.72	-14.54	-34.89
	F (kips)	3.16	0.79	-1.32	-3.16	3.16	0.79	-1.32	-3.16
	B	-1.10	-0.10	-1.10	-0.60	-1.10	-0.10	-1.10	-0.60
	p (psf)	-31.99	-2.91	-31.99	-17.45	-31.99	-2.91	-31.99	-17.45
	F (kips)	-2.90	-0.26	-2.90	-1.58	-2.90	-0.26	-2.90	-1.58

(cont'd)

		WIND TO BOTTOM, $\gamma = 0^o$				WIND TO TOP, $\gamma = 180^o$			
		CLEAR FLOW		OBSTRUCTED		CLEAR FLOW		OBSTRUCTED	
		CASE A	CASE B	CASE A	CASE B	CASE A	CASE B	CASE A	CASE B
Horizontal force / base shear	H (kips)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Vertical download force	V (kips)	3.95	0.00	0.00	0.00	3.95	0.00	0.00	0.00
Vertical uplift force	V (kips)	0.00	3.16	4.48	4.48	0.00	3.16	4.48	4.48
Bending moment at centroid of base	M (ft-kips)	-6.719035	7.46559	-5.22592	3.732797	-6.719	7.465594	-5.2259	3.732797

Component and Cladding Elements (Sec. 30.8.2)

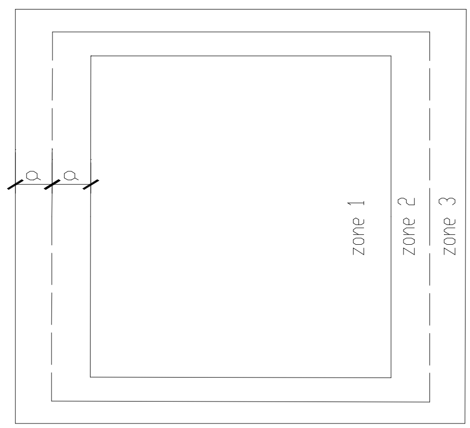
$p = q_h\ G\ C_N$

where: C_N = net force coefficients. (Fig. 30.8-1, page 351)

a = Max[Min(0.1L , 0.1B , 0.4h) , 0.04L , 0.04B , 3ft] , (Fig. 30.8-1, footnote 6) **=** **3.0** ft

For effective area, 10 sq.ft. as given

	CLEAR WIND FLOW						OBSTRUCTED WIND FLOW					
	ZONE 3		ZONE 2		ZONE 1		ZONE 3		ZONE 2		ZONE 1	
C_N	2.38	-3.24	1.80	-1.70	1.20	-1.10	0.99	-3.53	0.80	-1.80	0.50	-1.20
p (psf)	69.14	-94.24	52.34	-49.43	34.89	-31.99	28.86	-102.74	23.26	-52.34	14.54	-34.89



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Engineering International

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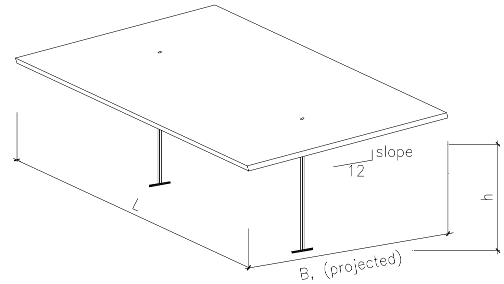
REVIEW BY :

Wind Analysis for Open Structure (Solar Panels) Based on ASCE 7-05 / IBC 2009 / CBC 2010

INPUT DATA

Exposure category (B, C or D)		=	C	
Importance factor, pg 73, (0.87, 1.0 or 1.15)	I	=	0.87	Category I
Basic wind speed (3 sec. gust wind)	V	=	90	mph
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt}	=	1	Flat
Height of mean roof	h	=	35	ft
Roof slope			3	**: 12**
Roof length	L	=	100	ft
Roof horizontal projected width	B	=	60	ft
Effective area of component / cladding	A	=	50	ft^2



DESIGN SUMMARY

1. Main Wind-Force Resisting System

Max horizontal force / base shear from roof	=	28.00	kips
	+	2.80	kips (eave & columns increasing)
	Σ	30.80	kips
Max vertical download force	=	111.99	kips
Max vertical uplift force	=	105.62	kips
Max moment at centroid of base from roof	=	1819.76	ft-kips
	+	181.98	ft-kips (eave & columns increasing)
	Σ	2001.74	ft-kips

2. Component and Cladding Elements

Max inward pressure	=	45	psf
Max net outward pressure	=	55	psf

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2\, I$ = **15.49** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 6-15, page 27)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) **= 1.01**

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) **= 0.85**

h = height of mean roof **= 35.00** ft

Main Wind-Force Resisting System (Sec. 6.5.13.2)

$p = q_h\, G\, C_N$

$F = p\, A_f$

where: G = gust effect factor. (Sec. 6.5.8, page 26). **= 0.85**

θ = roof angle **= 14.04** degree

< 45 deg. [Satisfactory]

A_f = roof actual area. **= 3092.3** ft^2, windward

= 3092.3 ft^2, leeward

C_N = net force coefficients. (Fig. 6-18A, page 66)

Check Fig. 6-18A limitation h / B = 0.58 within [0.25 , 1.0] **[Satisfactory]**

θ	CASE	WIND TO BOTTOM, $\gamma = 0^o$				WIND TO TOP, $\gamma = 180^o$			
		CLEAR FLOW		OBSTRUCTED		CLEAR FLOW		OBSTRUCTED	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}
14.04	A	-0.86	-1.26	-1.09	-1.50	1.25	1.59	0.32	-1.11
	p (psf)	-11.34	-16.61	-14.31	-19.75	16.44	20.89	4.25	-14.65
	F (kips)	-35.07	-51.35	-44.26	-61.06	50.83	64.61	13.14	-45.30
	B	-1.84	0.00	-2.05	-0.63	1.77	0.56	1.15	-0.30
	p (psf)	-24.17	0.00	-26.97	-8.24	23.36	7.39	15.12	-3.95
	F (kips)	-74.73	0.00	-83.40	-25.47	72.23	22.86	46.76	-12.21

(cont'd)

		WIND TO BOTTOM, $\gamma = 0^o$				WIND TO TOP, $\gamma = 180^o$			
		CLEAR FLOW		OBSTRUCTED		CLEAR FLOW		OBSTRUCTED	
		CASE A	CASE B	CASE A	CASE B	CASE A	CASE B	CASE A	CASE B
Horizontal force / base shear	H (kips)	-20.96	-18.13	-25.54	-26.40	28.00	23.06	-7.80	8.38
Vertical download force	V (kips)	0.00	0.00	0.00	0.00	111.99	92.25	0.00	33.51
Vertical uplift force	V (kips)	83.84	72.50	102.18	105.62	0.00	0.00	31.20	0.00
Bending moment at centroid of base	M (ft-kips)	481.8343	1789.84	634.176	1819.761	1193.04	43.75912	-1176.7	-618.538

Component and Cladding Elements (Sec. 6.5.13.3)

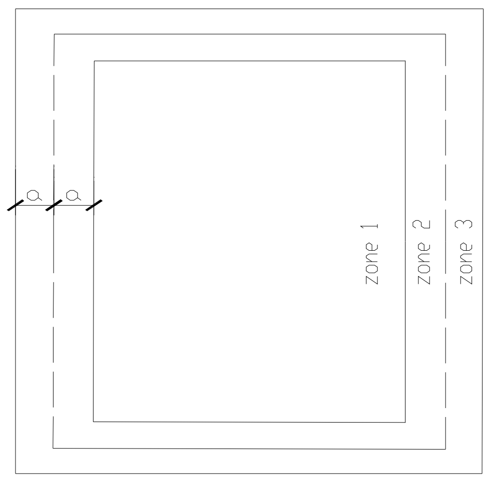
$\mathbf{p = q_h \, G \, C_N}$

where: C_N = net force coefficients. (Fig. 6-19A, page 70)

a = Max[Min(0.1L , 0.1B , 0.4h) , 0.04L , 0.04B , 3ft] , (Fig. 6-19B, footnote 6) = **6.0** ft

For effective area, 50 sq.ft. as given

	CLEAR WIND FLOW						OBSTRUCTED WIND FLOW					
	ZONE 3		ZONE 2		ZONE 1		ZONE 3		ZONE 2		ZONE 1	
C_N	3.43	-3.71	2.66	-2.80	1.77	-1.84	2.22	-4.16	1.72	-3.12	1.15	-2.05
p (psf)	45.20	-48.90	35.04	-36.82	23.36	-24.17	29.26	-54.78	22.68	-41.11	15.12	-26.97



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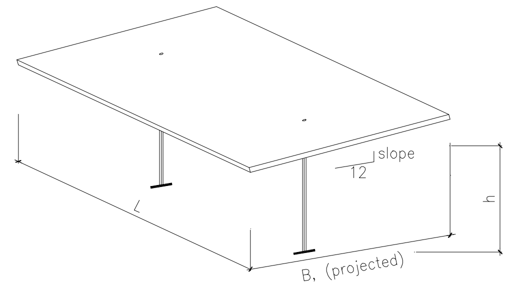
Wind Analysis for Open Structure Based on CBC 2001 / UBC 97

INPUT DATA

Exposure type (B, C or D)		=	C	
Importance factor, (CBC Tab.16-K)	I_w	=	1.00	
Basic wind speed (fastest mile wind)	V	=	75	mph
Height of centroid of roof area	h	=	35	ft
Roof slope			1	**: 12**
Roof length	L	=	100	ft
Roof horizontal projected width	B	=	60	ft



DESIGN SUMMARY

Max horizontal force / base shear from roof	=	39.14 kips
Max vertical download force	=	0.00 kips
Max vertical uplift force	=	78.56 kips
Max bending moment at centroid of base from roof	=	1370.00 ft-kips

ANALYSIS

Design pressures for wind force acting normal to the roof

$\mathbf{F = C_e\, C_q\, q_s\, I_w\, A_f}$ = (0.00 psf) A_f **= 0.00** kips, (inward)

= (13.09 psf) A_f **= 78.83** kips, (outward)

where: F = design wind load on open structural roof. (Sec. 1620).

θ = roof angle	**=**	**4.76**	degree
h = max top height for C_e calculation	**=**	**37.49**	ft
C_e = exposure and gust factor. (Tab. 16-G)	**=**	**1.29**	
C_q = pressure coefficient. (Tab. 16-H, Item 1)	**=**	**0.00**	inward
	=	**0.70**	outward
q_s = wind stagnation pressure. (Tab. 16-F)	**=**	**14.50**	psf
A_f = roof actual area.	**=**	**6020.8**	ft^2

Horizontal force / base shear	$H = F \sin\theta$	=	6.55 kips
Vertical download force	$V = F_{inward} \cos\theta$	=	0.00 kips
Vertical uplift force	$V = F_{outward} \cos\theta$	=	78.56 kips
Bending moment at centroid of base	$M = H\, h$	=	229.1 ft-kips

Design pressures for wind force acting to horizontal projected area

$\mathbf{F = C_e\, C_q\, q_s\, I_w\, A_f}$ = (74.82 psf) A_f **= 39.14** kips, (horizontal)

where:

C_q = pressure coefficient. (Tab. 16-H, Item 5)	**=**	**4.00**	horizontal
A_f = horizontal projected area normal to the wind, included columns area.	**=**	**523.2**	ft^2

Horizontal force / base shear	H = F	=	39.14 kips
Vertical uplift / download force	V	=	0.00 kips
Bending moment at centroid of base	M = F h	=	1370.0 ft-kips

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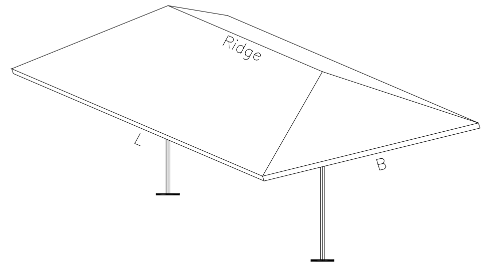
Wind Analysis for Shade Open Structure Based on ASCE 7-2010

INPUT DATA



Exposure category (B, C or D)		=	C	
Importance factor, 1.0 only, (Table 1.5-2)	I_w	=	1.00	
Basic wind speed (ASCE 7-10 26.5.1)	V	=	136	mph
Topographic factor (26.8 & Table 26.8-1)	K_{zt}	=	1	Flat
Height of ridge	H	=	35	ft
Roof slope			1	**: 12**
Roof length	L	=	100	ft
Roof horizontal projected width	B	=	60	ft
Effective area of component / cladding	A	=	50	ft^2

DESIGN SUMMARY

1. Main Wind-Force Resisting System

Max horizontal force / base shear from roof	=	8.40	kips
	+	9.28	kips (eave & columns increasing)
	Σ	17.68	kips
Max vertical download force	=	8.23	kips
Max vertical uplift force	=	261.33	kips
Max bending moment, at centroid of base, from roof	=	3371.27	ft-kips
	+	208.80	ft-kips (eave & columns increasing)
	Σ	3580.07	ft-kips

2. Component and Cladding Elements

Max inward pressure	=	76	psf
Max net outward pressure	=	146	psf

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h K_{zt} K_d V^2$ = **40.35** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 28.3-1, pg 299)	=	**1.00**	
K_d = wind directionality factor. (Tab. 26.6-1, for building, page 250)	=	**0.85**	
h = height of mean roof	=	**33.75**	ft

Main Wind-Force Resisting System (Sec. 27.4.3)

$p = q_h\, G\, C_N$

$F = p\, A_f$

where:

G = gust effect factor. (Sec. 26.9)	=	**0.85**	
θ = roof angle	=	**4.76**	degree
	<	**45 deg.**	**[Satisfactory]**
A_f = roof actual area.	=	**2107.3**	ft^2, front / back
	=	**903.1**	ft^2, each side

C_N = net force coefficients. (Fig. 27.4-5, page 268 & Fig. 27.4-7, page 270, and C_{NS} for two sides of sloped roof.)

Check Fig.27.4 limitation				
	h / L =	0.34	within [0.25 , 1.0]	**[Satisfactory]**
	h / B =	0.56	within [0.25 , 1.0]	**[Satisfactory]**

	CLEAR WIND FLOW						OBSTRUCTED WIND FLOW					
	CASE A			CASE B			CASE A			CASE B		
	C_{NW}	C_{NL}	C_{NS}	C_{NW}	C_{NL}	C_{NS}	C_{NW}	C_{NL}	C_{NS}	C_{NW}	C_{NL}	C_{NS}
	1.10	-0.30	-0.80	0.20	-1.20	0.80	-1.60	-1.00	-1.20	-0.90	-1.70	0.50
p (psf)	37.73	-10.29	-27.44	6.86	-41.15	27.44	-54.87	-34.30	-41.15	-30.87	-58.30	17.15
F (kips)	79.50	-21.68	-24.78	14.45	-86.72	24.78	-115.63	-72.27	-37.17	-65.04	-122.86	15.49

		CLEAR FLOW		OBSTRUCTED	
		CASE A	CASE B	CASE A	CASE B
Horizontal force / base shear	H (kips)	8.40	8.40	-3.60	4.80
Vertical download force	V (kips)	8.23	0.00	0.00	0.00
Vertical uplift force	V (kips)	0.00	22.64	261.33	156.39
Bending moment at centroid of base	M (ft-kips)	-705.6	1516.7	3089.2	3371.3

(cont'd)

DETERMINE HORIZONTAL WIND LOAD OF COLUMNS AND EAVE

$\mathbf{F_{col\ \&\ eave} = p_{col\ \&\ eave}\ A_{col\ \&\ eave}}$ = **9.28** kips

where: $p_{col\ \&\ eave}$ = horizontal wind pressure = **40.35** psf

$A_{col\ \&\ eave}$ = horizontal projected area of columns and eave **=** **230.0** ft^2

$\mathbf{M_{col\ \&\ eave}}$ = **208.8** ft-kips

Component and Cladding Elements of Roof (Sec. 30.8.2)

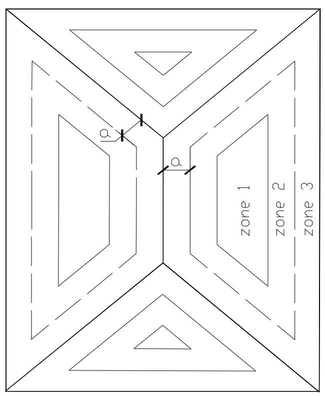
$\mathbf{p = q_h\ G\ C_N}$

where: C_N = net force coefficients. (Fig. 30.8-2, page 252)

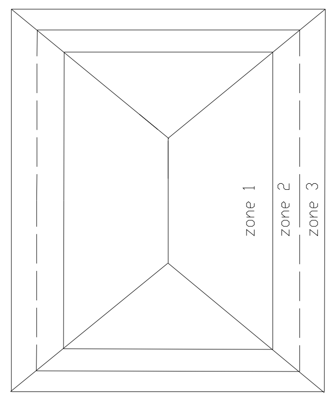
a = Max[Min(0.1L , 0.1B , 0.4h) , 0.04L , 0.04B , 3ft] , (Fig. 30.8-2, footnote 6) **=** **6.0** ft

For effective area, 50 sq.ft. as given

	CLEAR WIND FLOW						OBSTRUCTED WIND FLOW					
	ZONE 3		ZONE 2		ZONE 1		ZONE 3		ZONE 2		ZONE 1	
C_N	2.20	-3.27	1.74	-1.76	1.14	-1.16	0.97	-4.26	0.80	-2.31	0.50	-1.52
p (psf)	75.57	-112.03	59.55	-60.48	38.98	-39.90	33.41	-146.16	27.44	-79.16	17.15	-52.05



$\theta \geq 10°$



$\theta < 10°$

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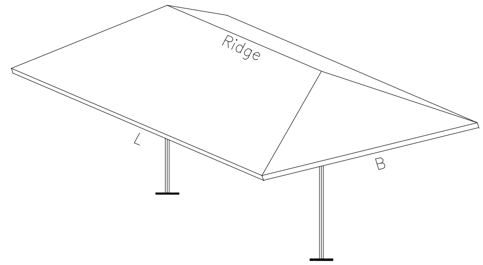
Wind Analysis for Shade Open Structure Based on ASCE 7-05 / IBC 2009

INPUT DATA

Exposure category (B, C or D)		=	C	
Importance factor, pg 73, (0.87, 1.0 or 1.15)	I	=	1.00	Category II
Basic wind speed (3 sec. gust wind)	V	=	90	mph
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt}	=	1	Flat
Height of ridge	H	=	35	ft
Roof slope			1	**: 12**
Roof length	L	=	100	ft
Roof horizontal projected width	B	=	60	ft
Effective area of component / cladding	A	=	50	ft^2



DESIGN SUMMARY

1. Main Wind-Force Resisting System

Max horizontal force / base shear from roof = 3.68 kips
\+ 4.06 kips (eave & columns increasing)
Σ 7.74 kips

Max vertical download force = 3.60 kips
Max vertical uplift force = 114.45 kips

Max bending moment, at centroid of base, from roof = 1476.39 ft-kips
\+ 91.44 ft-kips (eave & columns increasing)
Σ 1567.83 ft-kips

2. Component and Cladding Elements

Max inward pressure = 33 psf
Max net outward pressure = 64 psf

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2\, I$ = **17.67** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 6-15, page 27)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) = **1.00**

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) = **0.85**

h = height of mean roof = **33.75** ft

Main Wind-Force Resisting System (Sec. 6.5.13.2)

$p = q_h\, G\, C_N$

$F = p\, A_f$

where: G = gust effect factor. (Sec. 6.5.8, page 26). = **0.85**

θ = roof angle = **4.76** degree
< **45 deg. [Satisfactory]**

A_f = roof actual area. = **2107.3** ft^2, front / back
= **903.1** ft^2, each side

C_N = net force coefficients. (Fig. 6-18B, page 67 & Fig. 6-18D, page 69, and C_{NS} for two sides of sloped roof.)

Check Fig. 6-18 limitation h / L = 0.34 within [0.25 , 1.0] **[Satisfactory]**
h / B = 0.56 within [0.25 , 1.0] **[Satisfactory]**

	CLEAR WIND FLOW						OBSTRUCTED WIND FLOW					
	CASE A			CASE B			CASE A			CASE B		
	C_{NW}	C_{NL}	C_{NS}	C_{NW}	C_{NL}	C_{NS}	C_{NW}	C_{NL}	C_{NS}	C_{NW}	C_{NL}	C_{NS}
	1.10	-0.30	-0.80	0.20	-1.20	0.80	-1.60	-1.00	-1.20	-0.90	-1.70	0.50
p (psf)	16.52	-4.51	-12.02	3.00	-18.02	12.02	-24.03	-15.02	-18.02	-13.52	-25.53	7.51
F (kips)	34.81	-9.49	-10.85	6.33	-37.98	10.85	-50.64	-31.65	-16.28	-28.48	-53.80	6.78

		CLEAR FLOW		OBSTRUCTED	
		CASE A	CASE B	CASE A	CASE B
Horizontal force / base shear	H (kips)	3.68	3.68	-1.58	2.10
Vertical download force	V (kips)	3.60	0.00	0.00	0.00
Vertical uplift force	V (kips)	0.00	9.91	114.45	68.49
Bending moment at centroid of base	M (ft-kips)	-309.0	664.2	1352.9	1476.4

(cont'd)

DETERMINE HORIZONTAL WIND LOAD OF COLUMNS AND EAVE

$F_{col\ \&\ eave} = p_{col\ \&\ eave}\ A_{col\ \&\ eave}$ = **4.06** kips

where: $p_{col\ \&\ eave}$ = horizontal wind pressure = **17.67** psf

$A_{col\ \&\ eave}$ = horizontal projected area of columns and eave **=** **230.0** ft^2

$M_{col\ \&\ eave}$ = **91.4** ft-kips

Component and Cladding Elements of Roof (Sec. 6.5.13.3)

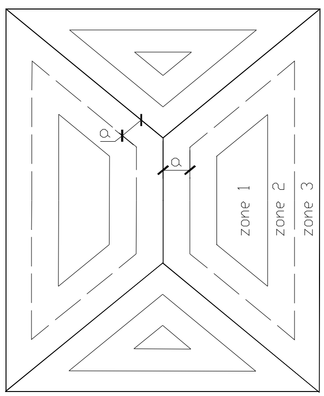
$p = q_h\ G\ C_N$

where: C_N = net force coefficients. (Fig. 6-19B, page 71)

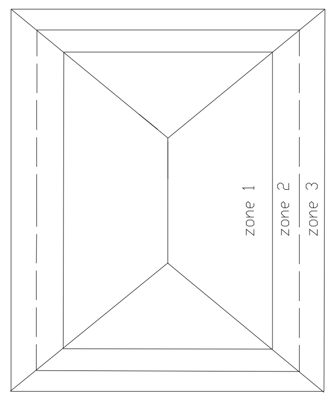
a = Max[Min(0.1L , 0.1B , 0.4h) , 0.04L , 0.04B , 3ft] , (Fig. 6-19B, footnote 6) **=** **6.0** ft

For effective area, 50 sq.ft. as given

	CLEAR WIND FLOW						OBSTRUCTED WIND FLOW					
	ZONE 3		ZONE 2		ZONE 1		ZONE 3		ZONE 2		ZONE 1	
C_N	2.20	-3.27	1.74	-1.76	1.14	-1.16	0.97	-4.26	0.80	-2.31	0.50	-1.52
p (psf)	33.09	-49.06	26.08	-26.49	17.07	-17.48	14.63	-64.01	12.02	-34.67	7.51	-22.79



$\theta \geq 10°$



$\theta < 10°$

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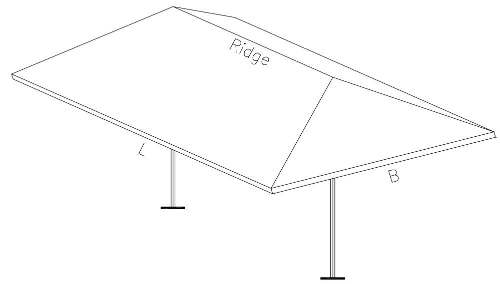
Wind Analysis for Shade Open Structure Based on CBC 2001 / UBC 97

INPUT DATA

Exposure type (B, C or D)		=	C	
Importance factor, (CBC Tab.16-K)	I_w	=	1.00	
Basic wind speed (fastest mile wind)	V	=	75	mph
Height of ridge	h	=	35	ft
Roof slope			4	: 12
Roof length	L	=	100	ft
Roof horizontal projected width	B	=	60	ft



DESIGN SUMMARY

Max horizontal force / base shear	=	54.14 kips
Max vertical download force	=	0.00 kips
Max vertical uplift force	=	85.08 kips
Max bending moment at centroid of base	=	1585.53 ft-kips

ANALYSIS

Design pressures for wind force acting normal to the roof

$F = C_e C_q q_s I_w A_f$ = (5.52 psf) A_f = **12.23** kips, (inward at front)

= (16.57 psf) A_f = **36.69** kips, (outward at front)

= (12.89 psf) A_f = **28.53** kips, (outward at back)

= (12.89 psf) A_f = **12.23** kips, (outward at each side)

where: F = design wind load on open structural roof. (Sec. 1620).

θ = roof angle	=	**18.43**	degree
C_e = exposure and gust factor. (Tab. 16-G)	=	**1.27**	
C_q = pressure coefficient. (Tab. 16-H, Item 1)	=	**0.30**	inward at front
	=	**0.90**	outward at front
	=	**0.70**	outward at back
	=	**0.70**	outward at sides
q_s = wind stagnation pressure. (Tab. 16-F)	=	**14.50**	psf
A_f = roof actual area.	=	**2213.6**	ft^2, front / back
	=	**948.7**	ft^2, each side

Horizontal force / base shear	H	=	12.89 kips
Vertical download force	V	=	0.00 kips
Vertical uplift force	V	=	85.08 kips
Bending moment at centroid of base	M	=	377.5 ft-kips

Design pressures for wind force acting to horizontal projected area

$F = C_e C_q q_s I_w A_f$ = (73.66 psf) A_f = **54.14** kips, (horizontal)

where: C_q = pressure coefficient. (Tab. 16-H, Item 5)	=	**4.00**	horizontal
A_f = horizontal projected area normal to the wind, included columns area.	=	**735.0**	ft^2

Horizontal force / base shear	H = F	=	54.14 kips
Vertical uplift / download force	V	=	0.00 kips
Bending moment at centroid of base	M = F h	=	1585.5 ft-kips

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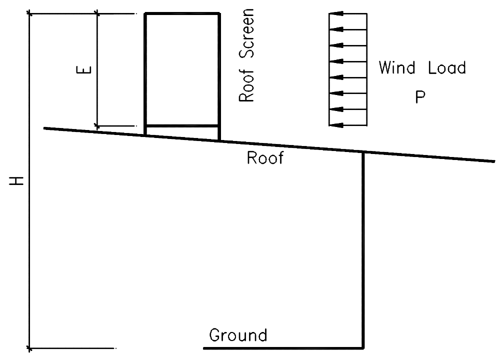
Wind Load, on Roof Screen / Roof Equipment, Based on ASCE 7-2010

INPUT DATA & DESIGN SUMMARY

BASIC WIND VELOCITY	V =	136	mph
EXPOSURE TYPE (B, C, D)	== >	C	
SCREEN TOP ELEVATION	H =	44	ft
HEIGHT OF STRUCTURE	h =	38	ft, (ASCE Fig. 29.5-1 note 3)
BUILDING WIDTH	B =	80	ft
BUILDING LENGTH	L =	150	ft
SCREEN VERTICAL SIZE	E =	5	ft
SCREEN HORIZONTAL SIZE	D =	20	ft, (ASCE Fig. 29.5-1)
VERTICAL PROJECTED AREA	A_f =	80	ft^2
HORIZONTAL PROJECTED AREA	A_r =	40	ft^2
TOPOGRAPHIC FACTOR	K_{Zt} =	1	Flat, (26.8 & Table 26.8-1)



Out-of-plane load for screen design **P = 80.1 psf, Horizontal**

F_h = 6.408 kips, Horizontal **F_v = 2.53 kips, Vertical**

WIND ANALYSIS

Out-of-plane wind force for screen design (ASCE 7-10, 29.5.1)

$$F_h = PA_f = q_h GC_r A_f = \left(0.00256 K_Z K_{Zt} K_d V^2\right) GC_r A_f =$$ (80.1 psf) A_f = 6.408 kips, Horizontal

Where : GC_r = Max{ Min[2 - A_f / (B h) , 1.9] , 1.0} = 1.90 , (ASCE 7-10 Eq. 29.5-2)

h_c = 41.5 ft, centroid height of the screen

B h = 3040 ft^2, building side area (ASCE 7-10 Eq. 29.5-2)

K_Z = 1.05 , (ASCE 7-10 Tab. 29.3-1)

K_d = 0.85 , (ASCE 7-10 26.6)

$$F_v = PA_r = q_h GC_r A_r = \left(0.00256 K_Z K_{Zt} K_d V^2\right) GC_r A_r =$$ (63.2 psf) A_r = 2.53 kips, Vertical

Where : GC_r = Max{ Min[2 - A_r / (B L) , 1.5] , 1.0} = 1.50 , (ASCE 7-10 Eq. 29.5-3)

B L = 12000 ft^2, building side area (ASCE 7-10 Eq. 29.5-3)

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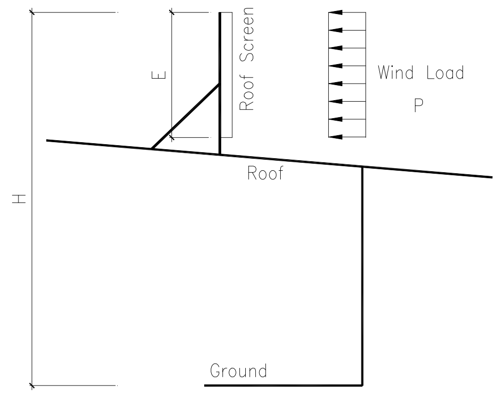
Wind Load on Roof Screen Based on IBC 2009 / ASCE 7-05

INPUT DATA & DESIGN SUMMARY

BASIC WIND VELOCITY	V =	85	mph
EXPOSURE TYPE (B, C, D)	== >	C	
SCREEN TOP ELEVATION	H =	44	ft
HEIGHT OF STRUCTURE	h =	38	ft, (ASCE Fig. 6-21 note 3)
BUILDING WIDTH	B =	80	ft
IMPORTANCE FACTOR	I =	1	(ASCE Tab. 6-1 & IBC Tab. 1604.5)
SCREEN VERTICAL SIZE	E =	5	ft
SCREEN HORIZONTAL SIZE	D =	20	ft, (ASCE Fig. 6-21)
DESIGN MEMBER TRIBUTARY AREA	A_f =	80	ft^2
TOPOGRAPHIC FACTOR	K_{Zt} =	1	Flat, (ASCE Eq.6-3)



Out-of-plane load for screen design **P = 35.0 psf**

F = 2.798 kips

WIND ANALYSIS

Out-of-plane wind force for screen design (ASCE 7-05, Eq.6-28)

$$F = PA_f = f q_z G C_f A_f = f\left(0.00256 K_z K_{Zt} K_d V^2 I\right) G C_f A_f =$$ (35.0 psf) A_f = 2.798 kips

Where : f	=	Max{ Min[2 - A_f / (B h) , 1.9] , 1.0} = 1.90	, (ASCE 6.5.15.1)
h_c	=	41.5	ft, centroid height of the screen
B h	=	3040	ft^2, building side area (ASCE 6.5.15.1)
K_z	=	1.05	, (ASCE Tab. 6-3)
K_d	=	0.85	, (ASCE Tab. 6-4)
G	=	0.85	, (ASCE 6.5.8.1)
C_f	=	1.32	, (ASCE Fig. 6-21)

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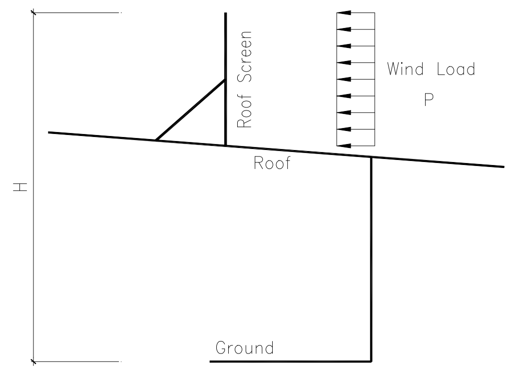
Wind Load on Roof Screen Based on CBC 2001 / UBC 97

INPUT DATA & DESIGN SUMMARY

SCREEN TOP HEIGHT	H	=	44	ft
IMPORTANCE FACTOR	I_w	=	1	(UBC Tab. 16-K)
BASIC WIND VELOCITY	V	=	75	mph
EXPOSURE TYPE (B, C, D)		=	C	
MEMBER TRIBUTARY AREA	TA	=	80	ft^2
MEMBER AT CORNER ?			Yes	

Out-of-plane load for screen design **P = 26.0 psf**



WIND ANALYSIS

Out-of-plane wind force for screen design (UBC 97 Sec.1620)

$P = C_e\ C_q\ q_s\ I_w =$ = 26.0 psf

Where : C_e = 1.33 (UBC Tab.16-G)

C_q = 1.34 (UBC Tab.16-H Item 2 parapet, Item 3 wall corner, & footnote 2)

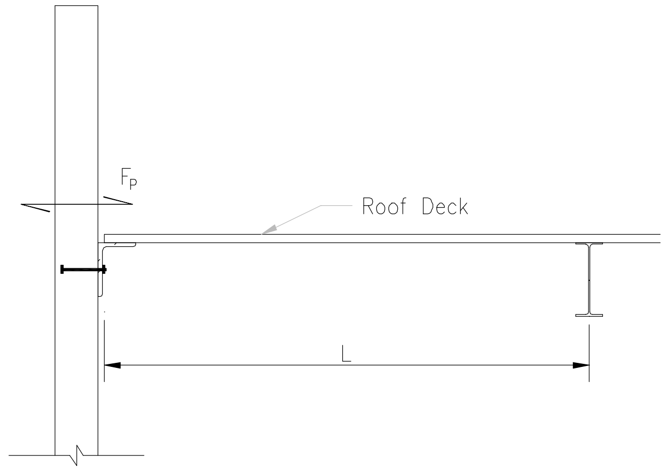
1.5 at corner, 1.3 at non-corner, and reduced by [C_q - 0.2 + (100-TA)/450] for TA @ [10,100]

q_s = 14.50 psf , (UBC Tab.16-F)

Axial Capacity of 1 1/2" Type "B" Roof Deck Based on ICBO ER-2078P

INPUT DATA & DESIGN SUMMARY

DECK VERT. SPAN LENGTH	L =	5	ft
GAGE (22,20,18,16) ? =>	== >	18	GA
DEAD LOAD	DL =	20	psf
LIVE LOAD	LL =	20	psf
WALL ANCHORAGE LOAD, ASD	F_P =	730	plf



THE DESIGN IS ADEQUATE.

ANALYSIS

PLB & HSB SECTION PROPERTIES (ER-2078P, Table 4, page 3)

GAGE	thk, in	I, in^4/ft	+S, in^3/ft	-S, in^3/ft	Wt, psf
16	0.0598	0.377	0.411	0.417	3.5
18	0.0478	0.302	0.322	0.335	2.9
20	0.0359	0.216	0.235	0.248	2.3
22	0.0299	0.175	0.187	0.198	1.9

A = 0.84 in^2

r = $(I / A)^{0.5}$ = 0.60 in

E = 29500 ksi

F_y = 38 ksi

$$F_e = \frac{\pi^2 E}{(kL/r)^2} = \quad 29.15 \text{ ksi, (AISI C4.1-1)}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \quad 1.14 \quad < \quad 1.5$$

$$F_n = \begin{cases} \left[\frac{0.877}{\lambda_c^2}\right] F_y \ , \ for \ \lambda_c > 1.5 \\ 0.658^{\lambda_c^2} F_y \ , \ for \ \lambda_c \le 1.5 \end{cases} = \quad 32.60 \text{ ksi}$$

M_n = S F_e = 9.39 ft-k / ft

P_n = A F_n = 27.31 klf

CHECK DECK CAPACITY (AISI C 5.2.1, Page 87)

M = (DL + LL) L^2 / 8 = 0.13 ft-k / ft

P = F_P = 0.73 klf

Ω_c = 1.8

Ω_b = 1.67

$$P_E = \frac{\pi^2 EI}{(KL)^2} = \quad 24.42 \text{ klf}$$

$$\alpha = 1 - \frac{\Omega_c P}{P_E} = \quad 0.95$$

$$\frac{0.75\Omega_c P}{P_n} = \quad 0.04 \quad < \quad 0.15$$,(0.75 from AISI A.4.1.2)

$$\frac{0.75\Omega_c P}{P_n} + \frac{\Omega_b M C_m}{\alpha M_n} = \quad 0.060 \quad < \quad 1.0$$ [Satisfactory]

Note: Roof live load & wall anchorage lateral load may be non-concurrent.

Technical References:

1. AISI STANDARD, 2001 Edition. American Iron and Steel Institute.

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Seismic Analysis Based on ASCE 7-22 - Equivalent Lateral Force (ELF) Procedure

INPUT DATA

Typical floor height	h =	**9**	*ft, (2.7 m)*
Typical floor weight	w_x =	**800**	*kips, (3558.4 kN)*
Number of floors	n =	**40**	
Importance factor (ASCE 11.5.1)	I_e =	**1.25**	
Design spectral response	S_{DS} =	**0.916**	g
	S_{D1} =	**0.498**	g
Mapped spectral response	S_1 =	**0.496**	g
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**	
The coefficient(ASCE Tab. 12.2.1)	R =	**5.5**	

DESIGN SUMMARY

Total base shear *(1 kips = 4.448 kN)*

V = 0.07 W, (SD) = **2,191** k, (SD)

= 0.05 W, (ASD) = **1,565** k, (ASD)

Seismic design category = **D**

h_n = 360.0 *ft, (109.7 m)*

W = 32,000 *kips, (142336.0 kN)*

k = 1.58 , (ASCE 12.8.3, page 125)

$\Sigma w_x h^k$ = 137,376,298

x = 0.75 , (ASCE Tab 12.8-2)

$T_a = C_t (h_n)^x$ = 1.65 Sec, (ASCE 12.8.2.1)

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Level Name	Floor to floor Height (ft)	Height h_x (ft)	Weight w_x (k)	$w_x h_x^k$	C_{vx}	Lateral force @ each level: F_x (k)	V_x (k)	O. M. (k-ft)	Diaphragm force: ΣF_i (k)	ΣW_i (k)	F_{px} (k)
40	Roof		360.0	800	8,570,835	0.062	136.7			136.7	800	183
		9.00						136.7				
39	40th		351.0	800	8,235,486	0.060	131.4		1,230	268.1	1,600	183
		9.00						268.1				
38	39th		342.0	800	7,905,058	0.058	126.1		3,643	394.1	2,400	183
		9.00						394.1				
37	38th		333.0	800	7,579,605	0.055	120.9		7,190	515.0	3,200	183
		9.00						515.0				
36	37th		324.0	800	7,259,185	0.053	115.8		11,826	630.8	4,000	183
		9.00						630.8				
35	36th		315.0	800	6,943,854	0.051	110.8		17,503	741.6	4,800	183
		9.00						741.6				
34	35th		306.0	800	6,633,675	0.048	105.8		24,177	847.4	5,600	183
		9.00						847.4				
33	34th		297.0	800	6,328,712	0.046	100.9		31,804	948.3	6,400	183
		9.00						948.3				
32	33th		288.0	800	6,029,030	0.044	96.2		40,338	1,044.5	7,200	183
		9.00						1,044.5				
31	32th		279.0	800	5,734,699	0.042	91.5		49,739	1,136.0	8,000	183
		9.00						1,136.0				
30	31th		270.0	800	5,445,792	0.040	86.9		59,962	1,222.8	8,800	183
		9.00						1,222.8				
29	30th		261.0	800	5,162,383	0.038	82.3		70,968	1,305.2	9,600	183
		9.00						1,305.2				
28	29th		252.0	800	4,884,554	0.036	77.9		82,714	1,383.1	10,400	183
		9.00						1,383.1				
27	28th		243.0	800	4,612,387	0.034	73.6		95,162	1,456.6	11,200	183
		9.00						1,456.6				
26	27th		234.0	800	4,345,970	0.032	69.3		108,271	1,525.9	12,000	183
		9.00						1,525.9				
25	26th		225.0	800	4,085,396	0.030	65.2		122,005	1,591.1	12,800	183
		9.00						1,591.1				
24	25th		216.0	800	3,830,763	0.028	61.1		136,325	1,652.2	13,600	183
		9.00						1,652.2				
23	24th		207.0	800	3,582,175	0.026	57.1		151,195	1,709.3	14,400	183
		9.00						1,709.3				
22	23th		198.0	800	3,339,741	0.024	53.3		166,579	1,762.6	15,200	183
		9.00						1,762.6				
21	22th		189.0	800	3,103,579	0.023	49.5		182,442	1,812.1	16,000	183
		9.00						1,812.1				
20	21th		180.0	800	2,873,813	0.021	45.8		198,751	1,858.0	16,800	183
		9.00						1,858.0				
19	20th		171.0	800	2,650,577	0.019	42.3		215,473	1,900.2	17,600	183
		9.00						1,900.2				

Level No.	Level Name	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft	Diaphragm force ΣF_i k	ΣW_i k	F_{px} k
												(cont'd)
18	19th		162.0	800	2,434,015	0.018	38.8		232,575	1,939.0	18,400	183
		9.00						1,939.0				
17	18th		153.0	800	2,224,281	0.016	35.5		250,026	1,974.5	19,200	183
		9.00						1,974.5				
16	17th		144.0	800	2,021,542	0.015	32.2		267,797	2,006.8	20,000	183
		9.00						2,006.8				
15	16th		135.0	800	1,825,982	0.013	29.1		285,858	2,035.9	20,800	183
		9.00						2,035.9				
14	15th		126.0	800	1,637,798	0.012	26.1		304,181	2,062.0	21,600	183
		9.00						2,062.0				
13	14th		117.0	800	1,457,210	0.011	23.2		322,739	2,085.3	22,400	183
		9.00						2,085.3				
12	13th		108.0	800	1,284,460	0.009	20.5		341,507	2,105.7	23,200	183
		9.00						2,105.7				
11	12th		99.0	800	1,119,820	0.008	17.9		360,458	2,123.6	24,000	183
		9.00						2,123.6				
10	11th		90.0	800	963,594	0.007	15.4		379,571	2,139.0	24,800	183
		9.00						2,139.0				
9	10th		81.0	800	816,129	0.006	13.0		398,822	2,152.0	25,600	183
		9.00						2,152.0				
8	9th		72.0	800	677,826	0.005	10.8		418,189	2,162.8	26,400	183
		9.00						2,162.8				
7	8th		63.0	800	549,156	0.004	8.8		437,655	2,171.6	27,200	183
		9.00						2,171.6				
6	7th		54.0	800	430,681	0.003	6.9		457,199	2,178.4	28,000	183
		9.00						2,178.4				
5	6th		45.0	800	323,094	0.002	5.2		476,805	2,183.6	28,800	183
		9.00						2,183.6				
4	5th		36.0	800	227,276	0.002	3.6		496,457	2,187.2	29,600	183
		9.00						2,187.2				
3	4th		27.0	800	144,408	0.001	2.3		516,142	2,189.5	30,400	183
		9.00						2,189.5				
2	3rd		18.0	800	76,206	0.001	1.2		535,848	2,190.7	31,200	183
		9.00						2,190.7				
1	2nd		9.0	800	25,552	0.000	0.4		555,564	2,191.1	32,000	183
		9.00						2,191.1				
	Ground		0.0						575,284			

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Four Story Seismic Analysis Based on ASCE 7-22

Determine Base Shear (Derived from ASCE 7-22 12.8)

$V = MAX\{ MIN [S_{D1}I_e / (RT) \;,\; S_{DS} I_e / R] \;,\; MAX(0.044S_{DS}I_e , 0.01) \;,\; 0.5S_1 I_e / R \} W$

^ (for $S_1 \geq 0.6$ g only)

$= MAX\{ MIN[0.15W \;,\; 0.18W] , 0.06W \;,\; 0.00W \}$

= 0.15 W, (SD)

= 0.11 W, (ASD) = **1454.89** kips

Where

S_{DS} =	0.96	(ASCE 7-22 Sec 11.4.4)
S_{D1} =	0.54	(ASCE 7-22 Sec 11.4.4)
S_1 =	0.55	(ASCE 7-22 Sec 11.4.3)
R =	8	(ASCE 7-22 Tab 12.2-1)
I_e =	1.5	(ASCE 7-22 Tab 1.5-2)
C_t =	0.028	(0.028 for steel MRF, 0.016 for concrete MRF, & 0.03 steel EBF)
h_n =	52.0	ft
x =	0.8	(ASCE 7-22 Tab 12.8-2)

$T = C_t (h_n)^x$ = 0.661 sec, (ASCE 7-22 Sec 12.8.2.1)

Calculate Vertical Distribution of Forces & Allowable Elastic Drift (ASCE 7-22, Sec 12.8.3 & 12.8.6)

Level	$\mathbf{W_x}$	$\mathbf{h_x}$	$\mathbf{h_x^k}$	$\mathbf{W_x h_x^k}$	$\mathbf{F_x}$, ASD (12.8-12)		$\delta_{xe,allowable,\ ASD}$
Roof	120	52	71.4	8571	27.3	(0.23 Wx)	0.3
4TH	3500	40	53.8	188278	598.9	(0.17 Wx)	0.3
3RD	4050	28	36.6	148199	471.4	(0.12 Wx)	0.3
2ND	5620	16	20.0	112349	357.4	(0.06 Wx)	0.4
	13290.0			457396	1454.9		

Where
k = 1 for T <= 0.5
k = 0.5 T + 0.75 for T @ (0.5 , 2.5)
k = 2 for T >= 2.5

$\delta_{xe,allowable,\ ASD} = \Delta_a I / (1.4\, C_d)$, (ASCE 7-22 12.8.6)

C_d = 5.5 ,(ASCE 7-22 Tab 12.2-1)

Δ_a = 0.01 hsx, (ASCE 7-22 Tab 12.12-1, & Sec 12.12)

Calculate Diaphragm Forces (ASCE 7-22, Sec 12.10.1.1)

Level	$\mathbf{W_x}$	$\mathbf{\Sigma W_x}$	$\mathbf{F_x}$	$\mathbf{\Sigma F_x}$	$\mathbf{F_{px}}$, ASD, (12.10-1)	
Roof	120.0	120.0	27.3	27.3	27.3	(0.23 Wx)
4TH	3500.0	3620.0	598.9	626.1	672.0	(0.19 Wx)
3RD	4050.0	7670.0	471.4	1097.5	777.6	(0.19 Wx)
2ND	5620.0	13290.0	357.4	1454.9	1079.0	(0.19 Wx)
	13290.0		1454.9			

Where
$F_{min} = 0.2\, S_{DS}\, I\, W_x / 1.5$, ASD
$F_{max} = 0.4\, S_{DS}\, I\, W_x / 1.5$, ASD

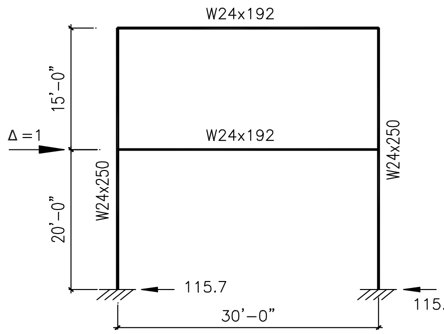
Method to Calculate Redundancy Factor, ρ, Based on ASCE 7-22

Redundancy Factor (ASCE 7-22 Sec 12.3.4)

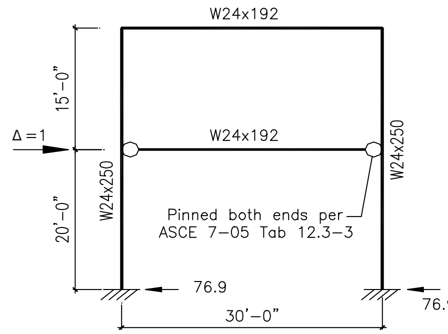
ρ = 1.3 , only for Seismic Design Categories D through F, and an individual element removed will be causing the remaining structural to suffer a reduction of **Story Strength** of more than 35% or creating an extreme torsional irregularity.

ρ = 1.0 , all other cases.

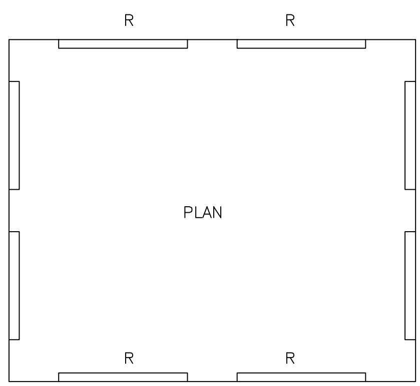
Note: To verify Redundancy Factor 1.0, the relative **Rigidity** of each frame at calculation level have to be known, since the Code Redundancy Provision Committee envisioned Story Strength as Rigidity, and SEAOC Seismology Committee endorsed this interpretation (SEAOC Seismic Design Manual -1, page 72).



R = 115.7 + 115.7 = 231.4



R = 76.9 + 76.9 = 153.8



$$\frac{4\times231.4-(3\times231.4+153.8)}{4\times231.4}=8.4\%\leq33\%$$

$$\rho=1.0$$

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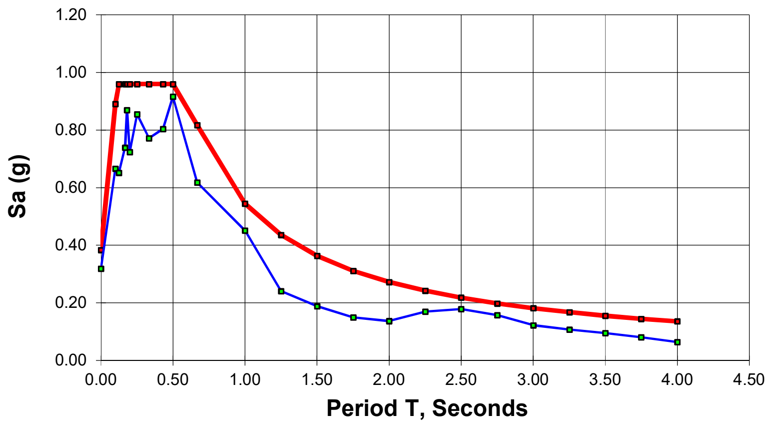
Modal Response Spectrum Analysis Based on ASCE 7-22

INPUT DATA

SEISMIC DESIGN PARAMETER	S_{DS} =	0.96	(ASCE 7-22 11.4.4)
SEISMIC DESIGN PARAMETER	S_{D1} =	0.545	(ASCE 7-22 11.4.4)
LONG-PERIOD TRANSITION PERIOD	T_L =	8	(ASCE 7-22 11.4.5.2)

RESPONSE SPECTRA



ANALYSIS

Station	T (s)	S_a(g) (IBC/CBC)	A/g (El Centro)
0	0.00	0.38	0.32
1	0.10	0.89	0.67
2	0.13	0.96	0.65
3	0.17	0.96	0.74
4	0.18	0.96	0.87
5	0.20	0.96	0.72
6	0.25	0.96	0.86
7	0.33	0.96	0.77
8	0.43	0.96	0.80
9	0.50	0.96	0.92
10	0.67	0.82	0.62
11	1.00	0.55	0.45
12	1.25	0.44	0.24
13	1.50	0.36	0.19
14	1.75	0.31	0.15
15	2.00	0.27	0.14
16	2.25	0.24	0.17
17	2.50	0.22	0.18
18	2.75	0.20	0.16
19	3.00	0.18	0.12
20	3.25	0.17	0.11
21	3.50	0.16	0.10
22	3.75	0.15	0.08
23	4.00	0.14	0.06

STATIC ANALYSIS PERIOD (ASCE 7-22 12.8.2)

APPROXIMATE FOUNDATIONAL PERIOD	$T_a = C_t (h_n)^x$ =	0.799	sec, (ASCE 7-22 Sec 12.8.2.1)
SUBSTANTIATED ANALYSIS PERIOD	T_b =	1.095	sec, (may from RISA, ETABS, RAM)

T = Max[T_a , Min(T_b , C_uT_a)] = 1.095 sec.

where C_u = 1.400 , (ASCE 7-22 Tab. 12.8-1)

SCALE FACTOR

BASE SHEAR BY ELASTIC DYNAMIC ANALYSIS (ASCE 7-22 12.9)	V_{DX} =	3000	kips	V_{DY} =	3500	kips
BASE SHEAR BY STATIC ANALYSIS (ASCE 7-22 12.8)	V_S =	735.09	kips			
RESPONSE MODIFICATION COEFFICIENT (ASCE 7 Tab 12.2-1)	R =	8				
IMPORTANCE FACTOR	I_e =	1.5		**SF**$_X = V_X / V_D$ =	0.245	
DSA/OSHPD PROJECT (CBC Chapter A) ?	== >	Yes		**SF**$_Y = V_Y / V_D$ =	0.210	

V_X = Max(100% V_S , V_{DX} I_e / R) = 735.09 kips, (ASCE 7-22 12.9.1.2 & 12.9.1.4)

V_X = Max(1.0 V_S , V_{DX} I_e / R) = 735.09 kips, (ASCE 7-22 12.9.1.2 & CBC 1616A.1.13)

V_Y = Max(100% V_S , V_{DY} I_e / R) = 735.09 kips, (ASCE 7-22 12.9.1.2 & 12.9.1.4)

V_Y = Max(1.0 V_S , V_{DY} I_e / R) = 735.09 kips, (ASCE 7-22 12.9.1.2 & CBC 1616A.1.13)

Notes:

See following link for more information.

http://www.engineering-international.com/NonlinearGuide.pdf

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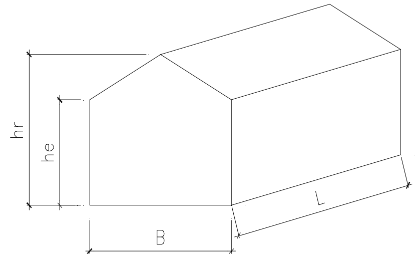
Wind Analysis for Low-rise Building, Based on ASCE 7-05 / IBC 2009 / CBC 2010

INPUT DATA

Exposure category (B, C or D)		C	
Importance factor, pg 77, (0.87, 1.0 or 1.15)	I =	1.00	Category II
Basic wind speed (IBC Tab 1609.3.1V_{3S})	V =	85	mph
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt} =	1	Flat
Building height to eave	h_e =	11	ft
Building height to ridge	h_r =	18	ft
Building length	L =	100	ft
Building width	B =	50	ft
Effective area of components (or Solar Panel area)	A =	28	ft^2



DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	18.00 kips
Max horizontal force normal to building length, B, face	=	7.26 kips
Max total horizontal torsional load	=	132.45 ft-kips
Max total upward force	=	51.97 kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\ K_h K_{zt} K_d V^2 I$ = **13.36** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 6-15, page 27)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) = **0.85**

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) = **0.85**

h = mean roof height = **14.50** ft

< 60 ft, [Satisfactory]
< Min (L, B), [Satisfactory]

Design pressures for MWFRS

$p = q_h [(G\ C_{pf}) - (G\ C_{pi})]$

where: p = pressure in appropriate zone. (Eq. 6-18, page 28). p_{min} = **10** psf (Sec. 6.1.4.1 & 6.1.4.2)

G $C_{p\,f}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 6-10, page 53 & 54)

G $C_{p\,i}$ = product of gust effect factor and internal pressure coefficient.(Fig. 6-5, Enclosed Building, page 47)

= **0.18** or **-0.18**

a = width of edge strips, Fig 6-10, note 9, page 54, MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3] = **5.00** **ft**

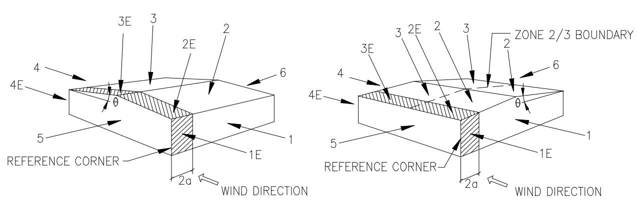
Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 15.64			Roof angle θ = 0.00		
	G $C_{p\,f}$	Net Pressure with		G $C_{p\,f}$	Net Pressure with	
		(+G$C_{p\,i}$)	(-G$C_{p\,i}$)		(+G$C_{p\,i}$)	(-G$C_{p\,i}$)
1	0.49	4.17	8.98	0.40	2.94	7.75
2	-0.69	-11.63	-6.82	-0.69	-11.63	-6.82
3	-0.45	-8.39	-3.58	-0.37	-7.35	-2.54
4	-0.39	-7.61	-2.80	-0.29	-6.28	-1.47
1E	0.74	7.55	12.36	0.61	5.75	10.56
2E	-1.07	-16.70	-11.89	-1.07	-16.70	-11.89
3E	-0.64	-11.00	-6.19	-0.53	-9.49	-4.68
4E	-0.58	-10.14	-5.33	-0.43	-8.15	-3.34
5	-0.45	-8.42	-3.61	-0.45	-8.42	-3.61
6	-0.45	-8.42	-3.61	-0.45	-8.42	-3.61

Net Pressures (psf), Torsional Load Cases

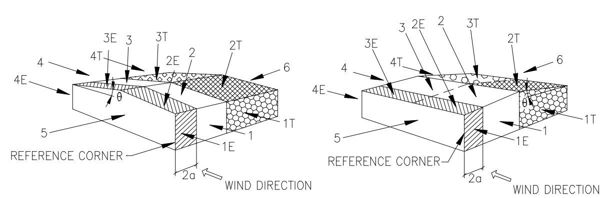
Surface	Roof angle θ = 15.64		
	G $C_{p\,f}$	Net Pressure with	
		(+G$C_{p\,i}$)	(-G$C_{p\,i}$)
1T	0.49	1.04	2.25
2T	-0.69	-2.91	-1.70
3T	-0.45	-2.10	-0.90
4T	-0.39	-1.90	-0.70

Surface	Roof angle θ = 0.00		
	G $C_{p\,f}$	Net Pressure with	
		(+G$C_{p\,i}$)	(-G$C_{p\,i}$)
1T	0.40	0.73	1.94
2T	-0.69	-2.91	-1.70
3T	-0.37	-1.84	-0.63
4T	-0.29	-1.57	-0.37



Transverse Direction Longitudinal Direction

Basic Load Cases



Transverse Direction Longitudinal Direction

Torsional Load Cases

Basic Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$
1	990	4.13	8.89
2	2337	-27.16	-15.92
3	2337	-19.61	-8.37
4	990	-7.53	-2.77
1E	110	0.83	1.36
2E	260	-4.34	-3.09
3E	260	-2.86	-1.61
4E	110	-1.12	-0.59
Σ	Horiz.	11.17	11.17
	Vert.	-51.97	-27.92
Min. wind	Horiz.	18.00	18.00
Sec. 6.1.4.1	Vert.	-50.00	-50.00

Basic Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$
1	601	1.77	4.66
2	2077	-24.15	-14.15
3	2077	-15.27	-5.27
4	601	-3.77	-0.88
1E	124	0.71	1.31
2E	519	-8.67	-6.18
3E	519	-4.93	-2.43
4E	124	-1.01	-0.41
Σ	Horiz.	7.26	7.26
	Vert.	-51.05	-26.99
Min. wind	Horiz.	7.25	7.25
Sec. 6.1.4.1	Vert.	-50.00	-50.00

Torsional Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$	Torsion (ft-k) $(+GC_{pi})$	Torsion (ft-k) $(-GC_{pi})$
1	440	1.84	3.95	41	89
2	1038	-12.07	-7.08	-73	-43
3	1038	-8.72	-3.72	53	23
4	440	-3.35	-1.23	75	28
1E	110	0.83	1.36	37	61
2E	260	-4.34	-3.09	-53	-37
3E	260	-2.86	-1.61	35	20
4E	110	-1.12	-0.59	50	26
1T	550	0.57	1.24	-14	-31
2T	1298	-3.77	-2.21	25	15
3T	1298	-2.72	-1.16	-18	-8
4T	550	-1.05	-0.38	-26	-10
Total Horiz. Torsional Load, M_T				132	132

Torsional Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$	Torsion (ft-k) $(+GC_{pi})$	Torsion (ft-k) $(-GC_{pi})$
1	239	0.70	1.85	5	13
2	1558	-18.11	-10.62	122	72
3	1558	-11.45	-3.96	-77	-27
4	239	-1.50	-0.35	11	3
1E	124	0.71	1.31	14	26
2E	519	-8.67	-6.18	58	42
3E	519	-4.93	-2.43	-33	-16
4E	124	-1.01	-0.41	20	8
1T	363	0.27	0.70	-3	-8
2T	2077	-6.04	-3.54	-81	-48
3T	2077	-3.82	-1.32	51	18
4T	363	-0.57	-0.13	-7	-2
Total Horiz. Torsional Load, M_T				80.5	80.5

Design pressures for components and cladding

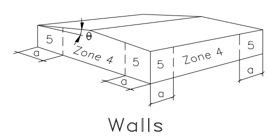
$p = q_h[(G\ C_p) - (G\ C_{pi})]$

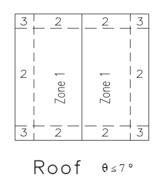
where: p = pressure on component. (Eq. 6-22, pg 28)

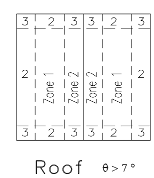
p_{min} = 10.00 psf (Sec. 6.1.4.2, pg 21)

G C_p = external pressure coefficient.

see table below. (Fig. 6-11, page 55~58)







	Effective Area (ft^2)	Zone 1 GC_P	Zone 1 $-GC_P$	Zone 2 GC_P	Zone 2 $-GC_P$	Zone 3 GC_P	Zone 3 $-GC_P$	Zone 4 GC_P	Zone 4 $-GC_P$	Zone 5 GC_P	Zone 5 $-GC_P$
Comp.	28	0.41	-0.86	0.41	-1.48	0.41	-2.33	0.92	-1.02	0.92	-1.24

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 2 Positive	Zone 2 Negative	Zone 3 Positive	Zone 3 Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
	10.00	-13.83	10.00	-22.14	10.00	-33.56	14.71	-16.05	14.71	-19.00

Note: If the effective area is roof Solar Panel area, the only zone 1, 2 , or 3 apply.

Daniel T. Li
Engineering International

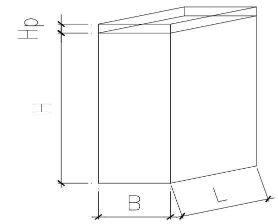
PROJECT :
CLIENT :
JOB NO. : DATE :

PAGE :
DESIGN BY :
REVIEW BY :

Wind Analysis for Building with h > 60 ft, Based on ASCE 7-05 / IBC 2009 / CBC 2010

INPUT DATA

Exposure category (B, C or D)		C	
Importance factor (0.87, 1.0 or 1.15)	I =	1.00	Category II, page 77
Basic wind speed (IBC Tab 1609.3.1 V_{3S})	V =	90	mph
Topographic factor (Sec.6.5.7.2)	K_{zt} =	1	Flat, page 26 & 45
Building height to roof	H =	157	ft
Parapet height	H_P =	4	ft
Building length	L =	300	ft
Building width	B =	180	ft
Natural frequency (Sec.6.2 & 6.5.8.2)	n_1 =	0.95541	Hz, (1 / T)
Effective area of mullion	A_M =	55	ft^2
Effective area of panel	A_P =	27	ft^2



DESIGN SUMMARY

Max building horizontal force normal to building length, L, face	=	1194.9	kips
Max overturning moment at wind normal to building length, L, face	=	180021.1	ft - kips
Max building horizontal force normal to building length, B, face	=	636.0	kips
Max overturning moment at wind normal to building length, B, face	=	159648.2	ft - kips
Max building upward force	=	1334.7	kips
Max building torsion force	=	40327.6	ft - kips

ANALYSIS

Velocity pressures

$$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I$$

where: q_z = velocity pressure at height, z. (Eq. 6-15, page 27) p_{min} = 10 psf (Sec. 6.1.4.1 & 6.1.4.2)

K_z = velocity pressure exposure coefficient evaluated at height, z. (Tab. 6-3, Case 2, page 79)

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) = 0.85

z = height above ground

z (ft)	0 - 15	20	25	30	40	50	60	70	80	90	100	120
K_z	0.85	0.90	0.94	0.98	1.04	1.09	1.13	1.17	1.21	1.24	1.26	1.31
q_z (psf)	14.98	15.86	16.57	17.27	18.33	19.21	19.92	20.62	21.33	21.86	22.21	23.09
z (ft)	140	160	161	161	161	161	161	161	161	161		
K_z	1.36	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39		
q_z (psf)	23.97	24.50	24.53	24.53	24.53	24.53	24.53	24.53	24.53	24.53		

Design pressures for MWFRS

$$p = q\, G\, C_p - q_h\, (G\, C_{pi})$$

where: p = pressure on surface for rigid building with all h. (Eq. 6-17, page 28).

q = q_z for windward wall at height z above the ground, see table above.

G $C_{p\,i}$ = internal pressure coefficient. (Fig. 6-5, Enclosed Building, page 47) = 0.18 or -0.18

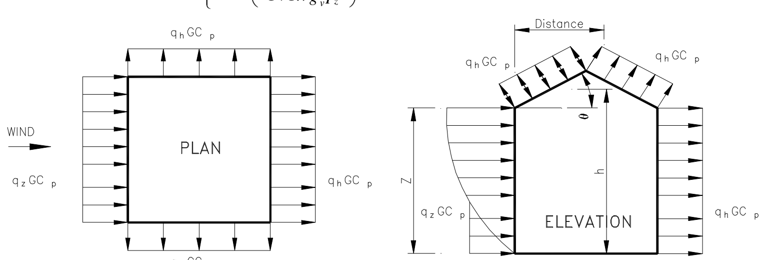
q_h = q_z value at mean roof height, h, for leeward wall, side walls, and roof.

C_p = external pressure coefficient, see right down tables.

G = gust effect factor (Sec. 6.5.8.1 & 6.5.8.2, Page 26)

$$G = \begin{cases} 0.925\left(\dfrac{1+1.7 I_z\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_z}\right), & for\ \ n_1 < 1.0 \\ 0.925\left(\dfrac{1+1.7 g_Q I_z Q}{1+1.7 g_v I_z}\right), & for\ \ n_1 \geq 1.0 \end{cases} = 0.854$$

I_z =	0.17	z =	94.2	Q =	0.84
z_{min} =	15	g_Q =	3.4	L_z =	617
c =	0.2	g_R =	4.18	β =	0.05
R_h =	0.135	R_B =	0.119	R_L =	0.023
N_1 =	5.84	R_n =	0.046	R =	0.090
h =	157	g_v =	3.4	V_z =	100.8



q G C_p Figure for Gable, Hip Roof, page 48

Fig. 6-6 fo θ < 10°, page 48

Roof	**h / B**	**Distance**	**Cp**
To L Face	0.89	80.5	-1.01
To L Face	0.89	161	-0.74
To L Face	0.89	180	-0.66
To L Face	0.89	180	
Roof	**h / L**	**Distance**	**Cp**
To B Face	0.54	80.5	-0.91
To B Face	0.54	161	-0.89
To B Face	0.54	300	-0.51
To B Face	0.54	300	

Fig. 6-6, page 48

Wall	**Direction**	**L / B**	**Cp**
Windward Wall	All	All	0.80
Leeward Wall	To L Dir	0.60	-0.50
Leeward Wall	To B Dir	1.67	-0.37
Side Wall	All	All	-0.70

(cont'd)

Hence, MWFRS Net Pressures are given by following tables (Sec. 6.5.12.2.1, Page 28)

Surface	z (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$
Windward Wall	0 - 15	5.81	14.65
	20	6.42	15.25
	25	6.90	15.73
	30	7.38	16.21
	40	8.10	16.93
	50	8.70	17.54
	60	9.18	18.02
	70	9.67	18.50
	80	10.15	18.98
	90	10.51	19.34
	100	10.75	19.58
	120	11.35	20.18
	140	11.95	20.79
	160	12.31	21.15
	161	12.34	21.17

Surface	z (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$
Side Wall	All	-19.08	-10.24

Normal to L Face		P (psf) with		Normal to B Face		P (psf) with	
Surface	z (ft)	GC_{Pi}	$-GC_{Pi}$	Surface	z (ft)	GC_{Pi}	$-GC_{Pi}$
Leeward	All	-14.89	-6.06	Leeward	All	-12.10	-3.26

Normal to L Face		P (psf) with		Normal to B Face		P (psf) with	
Surface	Dist. (ft)	GC_{Pi}	$-GC_{Pi}$	Surface	Dist. (ft)	GC_{Pi}	$-GC_{Pi}$
Roof	0 - 80.5	-25.58	-16.75	Roof	0 - 80.5	-23.48	-14.65
	161	-19.96	-11.13		161	-22.96	-14.13
	180	-18.19	-9.36		300	-15.20	-6.36

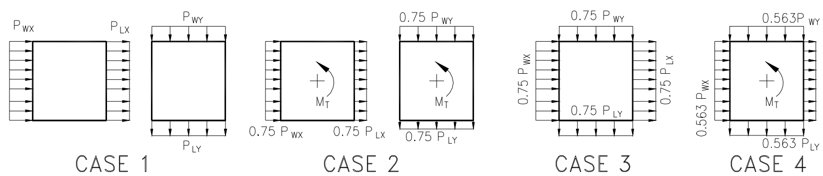


Figure 6-9, page 52

Base Forces		Normal to L Face Case 1	Normal to L Face Case 2	Normal to B Face Case 1	Normal to B Face Case 2	Wind with Angle Case 3	Wind with Angle Case 4	ASCE-7
V_{Base}	(kips)	1195	896	636	477	1373	762	
M_{Base}	(ft - kips)	180021	135016	159648	119736	254752	135466	Fig. 6-9
M_T	(ft - kips)	0	40328	0	12879	0	39941	Page 52
F_{Upward}	(kips)	965	724	815	611	1335	711	
V_{min}	(kips)	483	483	290	290	580	563	Min. wind
$F_{Up,min}$	(kips)	540	540	540	540	540	540	Sec. 6.1.4.1

Design pressures for components and cladding

$$p = q\,(G\,C_p) - q_i\,(G\,C_{pi})$$

where: p = pressure on component for building with h > 60 ft. (Eq. 6-23, page 29).

p_{min} = 10.00 psf (Sec. 6.1.4.2, pg 21)

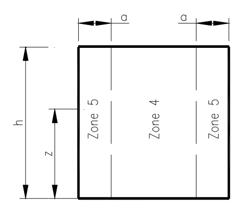
$q = q_z$ for windward wall at height z above the ground, see table above.

$q_h = q_z$ value at mean roof height, h, for leeward wall, side walls, and roof.

$G\,C_{p\,i}$ = internal pressure coefficient. (Fig. 6-5) = **0.18** or **-0.18**

a = Zone width = MAX[MIN(0.1B, 0.1L), 3] = **18.0** ft, (Fig 6-17 note 8, pg 65)

$G\,C_p$ = external pressure coefficient. (Fig. 6-17, page 65)



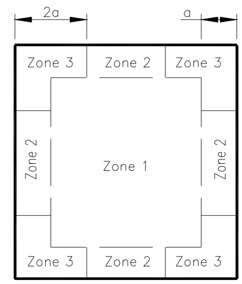
WALL ELEVATION

Wall Comp.	Actual Effective Area (ft^2)	Zone 4 GC_P	Zone 4 $-GC_P$	Zone 5 GC_P	Zone 5 $-GC_P$
Mullion	55	0.81	-0.84	0.81	-1.55
Panel	27	0.87	-0.88	0.87	-1.73

z (ft)	Mullion Pressure (psf) Zone 4 Positive	Mullion Zone 4 Negative	Mullion Zone 5 Positive	Mullion Zone 5 Negative	Panel Pressure (psf) Zone 4 Positive	Panel Zone 4 Negative	Panel Zone 5 Positive	Panel Zone 5 Negative
0 - 15	14.77	-24.96	14.77	-42.41	15.76	-26.04	15.76	-46.75
20	15.64	-24.96	16.69	-42.41	16.69	-26.04	16.69	-46.75
25	16.33	-24.96	17.43	-42.41	17.43	-26.04	17.43	-46.75
30	17.03	-24.96	18.17	-42.41	18.17	-26.04	18.17	-46.75
40	18.07	-24.96	19.28	-42.41	19.28	-26.04	19.28	-46.75
50	18.94	-24.96	20.21	-42.41	20.21	-26.04	20.21	-46.75
60	19.63	-24.96	20.95	-42.41	20.95	-26.04	20.95	-46.75
70	20.33	-24.96	21.69	-42.41	21.69	-26.04	21.69	-46.75
80	21.02	-24.96	22.44	-42.41	22.44	-26.04	22.44	-46.75
90	21.54	-24.96	22.99	-42.41	22.99	-26.04	22.99	-46.75
100	21.89	-24.96	23.36	-42.41	23.36	-26.04	23.36	-46.75
120	22.76	-24.96	24.29	-42.41	24.29	-26.04	24.29	-46.75
140	23.63	-24.96	25.22	-42.41	25.22	-26.04	25.22	-46.75
160	24.15	-24.96	25.77	-42.41	25.77	-26.04	25.77	-46.75
161	24.18	-24.96	25.81	-42.41	25.81	-26.04	25.81	-46.75

cont'd

Roof	Effective Area (ft^2)	Zone 1 - GC_p	Zone 2 - GC_p	Zone 3 - GC_p
Components and Cladding	0	-1.40	-2.30	-3.20
	10	-1.40	-2.30	-3.20
	59	-1.17	-1.98	-2.79
	108	-1.10	-1.87	-2.65
	157	-1.05	-1.81	-2.57
	206	-1.01	-1.76	-2.50
	255	-0.99	-1.72	-2.45
	304	-0.96	-1.69	-2.41
	353	-0.94	-1.66	-2.38
	402	-0.93	-1.64	-2.35
	451	-0.91	-1.62	-2.32
	500	-0.90	-1.60	-2.30
	38016	-0.90	-1.60	-2.30



ROOF PLAN

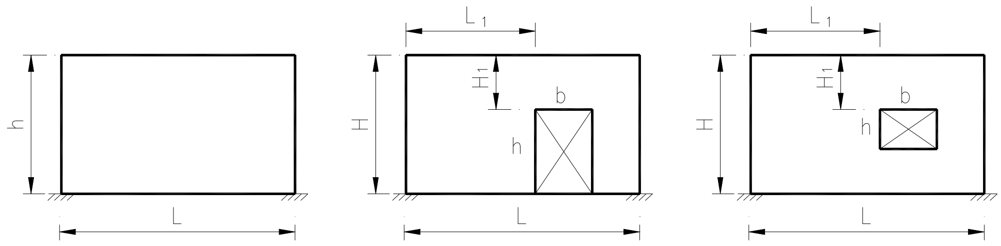
Roof	Effective Area (ft^2)	Net Pressure (psf)		
		Zone 1	Zone 2	Zone 3
Components and Cladding	0	-38.77	-60.85	-82.93
	10	-38.77	-60.85	-82.93
	59	-33.20	-53.05	-72.91
	108	-31.30	-50.40	-69.50
	157	-30.13	-48.76	-67.38
	206	-29.28	-47.56	-65.85
	255	-28.61	-46.63	-64.65
	304	-28.06	-45.86	-63.65
	353	-27.59	-45.20	-62.81
	402	-27.18	-44.63	-62.08
	451	-26.82	-44.12	-61.43
	500	-26.50	-43.67	-60.85
	38016	-26.50	-43.67	-60.85

Relative Rigidity Determination for Shear Wall with New Opening

INPUT DATA & DESIGN SUMMARY

DIMENSIONS	L =	**20**	ft
	H =	**20**	ft
OPENING	b =	**5**	ft
	h =	**10**	ft
LOCATION	L_1 =	**10**	ft
	H_1 =	**5**	ft
ONE STORY ?		**Yes**	

(Cantilever Wall with Fixed Base Only)



R = 100% **Not Apply** **R = 82%**

The new opening reduced the lateral load capacity by 18%.

ANALYSIS (Per California SE Exam 1984)

$R = 1 / \Delta$, $\Delta = \Delta_{Flexure} + \Delta_{Shear}$

Where $\Delta_{Flexure} = 4\,H^3 / (E\,L^3\,t)$, for cantilever

$\Delta_{Flexure} = H^3 / (E\,L^3\,t)$, for fixed at top and base

$\Delta_{Shear} = 1.2\,H / (G\,A) = 3\,H / (E\,L\,t)$

Solid Wall

Pier	H	L	Type	$E\,t\,\Delta_{Flexure}$	$E\,t\,\Delta_{Shear}$	$E\,t\,\Delta$	R / (E t)
Wall	20	20	Cantilever	4.00	3.00	7.00	0.143

R = 100%

Wall with Opening

Pier	H	L	Type	$E\,t\,\Delta_{Flexure}$	$E\,t\,\Delta_{Shear}$	$E\,t\,\Delta$	R / (E t)
Wall	20	20	Cantilever	4.00	3.00	7.00	
Bottom	10	20	Fixed	-0.13	-1.50	-1.63	
Left	10	10	Fixed	1.00	3.00	***4.00***	0.250
Right	10	5	Fixed	8.00	6.00	***14.00***	0.071
L & R						3.11	0.321
Total						8.49	0.118

R = 82%

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Engineering International

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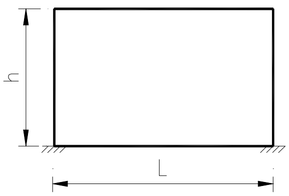
Rigidity Determination for Shear Wall & Shear Wall with Opening Using Finite Element Method

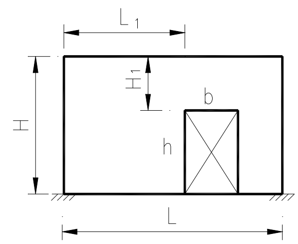
INPUT DATA & DESIGN SUMMARY

DIMENSIONS	L =	30	ft
	H =	20	ft
	thk =	8	in
OPENING	b =	5	ft
	h =	10	ft
LOCATION	L_1 =	10	ft
	H_1 =	5	ft
PROPERTIES	E =	1350	ksi
	υ =	0.25	(Poisson's ratio)

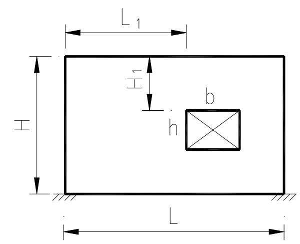
ONE STORY ? **Yes**
(Cantilever Wall with Fixed Base Only)



R = 4403 kips



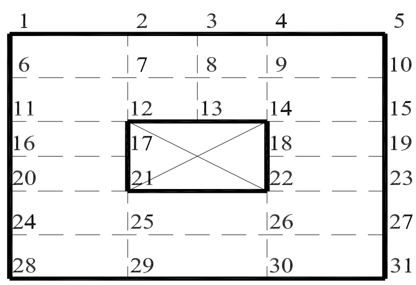
Not Apply



R = 3787 kips

The opening reduced the lateral load capacity by 14%.

ANALYSIS



Element		Dimension (ft)		Thick.
No.	Joints	X	Y	(in)
1	1, 2, 6, 7	10.0	2.5	8
2	2, 3, 7, 8	2.5	2.5	8
3	3, 4, 8, 9	2.5	2.5	8
4	4, 5, 9, 10	15.0	2.5	8
5	6, 7, 11, 12	10.0	2.5	8
6	7, 8, 12, 13	2.5	2.5	8
7	8, 9, 13, 14	2.5	2.5	8
8	9, 10, 14, 15	15.0	2.5	8
9	11, 12, 16, 17	10.0	5.0	8
10	14, 15, 18, 19	15.0	5.0	8
11	16, 17, 20, 21	10.0	5.0	8
12	18, 19, 22, 23	15.0	5.0	8
13	20, 21, 24, 25	10.0	2.5	8
14	21, 22, 25, 26	5.0	2.5	8
15	22, 23, 26, 27	15.0	2.5	8
16	24, 25, 28, 29	10.0	2.5	8
17	25, 26, 29, 30	5.0	2.5	8
18	26, 27, 30, 31	15.0	2.5	8

	Solid Wall with Fixed at Top & Base				Solid Wall with Fixed at Base only			
	Reaction (kips)		Deflection (in)		Reaction (kips)		Deflection (in)	
Joint	X	Y	X	Y	X	Y	X	Y
1	982	-1823	1	0.00	605	0	1	0.39
2	1157	-105	1	0.00	843	0	1	0.08
3	456	-41	1	0.00	422	0	1	0.03
4	1537	59	1	0.00	1494	0	1	-0.02
5	1655	1982	1	0.00	1038	0	1	-0.33
6	0	0	0.87	0.06	0	0	0.85	0.39
7	0	0	0.89	0.00	0	0	0.85	0.08
8	0	0	0.89	0.00	0	0	0.85	0.03
9	0	0	0.89	0.00	0	0	0.85	-0.02
10	0	0	0.88	-0.04	0	0	0.87	-0.33
11	0	0	0.75	0.11	0	0	0.71	0.37
12	0	0	0.76	0.01	0	0	0.70	0.08
13	0	0	0.80	-0.01	0	0	0.73	0.03
14	0	0	0.76	0.00	0	0	0.70	-0.02
15	0	0	0.76	-0.08	0	0	0.73	-0.31
16	0	0	0.49	0.14	0	0	0.45	0.29
17	0	0	0.50	0.02	0	0	0.42	0.06
18	0	0	0.50	0.00	0	0	0.42	-0.01
19	0	0	0.51	-0.10	0	0	0.47	-0.25
20	0	0	0.24	0.10	0	0	0.22	0.17
21	0	0	0.23	0.01	0	0	0.18	0.04
22	0	0	0.24	0.00	0	0	0.18	0.00
23	0	0	0.25	-0.08	0	0	0.22	-0.14
24	0	0	0.12	0.06	0	0	0.11	0.09
25	0	0	0.11	0.01	0	0	0.08	0.02
26	0	0	0.11	0.00	0	0	0.08	0.00
27	0	0	0.13	-0.05	0	0	0.11	-0.08
28	-1149	-1811	0.00	0.00	-1123	-2568	0.00	0.00
29	-1202	-261	0.00	0.00	-722	-571	0.00	0.00
30	-1895	28	0.00	0.00	-1309	27	0.00	0.00
31	-1541	1973	0.00	0.00	-1249	3112	0.00	0.00
R =	-5788	kips		R =	-4403	kips		

(cont'd)

Joint	Opening Wall with Fixed at Top & Base				Opening Wall with Fixed at Base only			
	Reaction (kips)		Deflection (in)		Reaction (kips)		Deflection (in)	
	X	Y	X	Y	X	Y	X	Y
1	664	-1608	*1*	0.00	393	0	*1*	0.33
2	1040	708	*1*	0.00	691	0	*1*	0.02
3	483	-31	*1*	0.00	595	0	*1*	0.03
4	1224	-831	*1*	0.00	1216	0	*1*	0.04
5	1504	1888	*1*	0.00	892	0	*1*	-0.30
6	0	0	0.90	0.06	0	0	0.87	0.33
7	0	0	0.91	-0.02	0	0	0.88	0.01
8	0	0	0.90	0.00	0	0	0.88	0.03
9	0	0	0.90	0.02	0	0	0.87	0.04
10	0	0	0.89	-0.04	0	0	0.87	-0.30
11	0	0	0.78	0.10	0	0	0.74	0.32
12	0	0	0.82	-0.05	0	0	0.76	0.00
13	0	0	0.84	-0.01	0	0	0.79	0.02
14	0	0	0.79	0.04	0	0	0.74	0.05
15	0	0	0.77	-0.07	0	0	0.75	-0.29
16	0	0	0.48	0.14	0	0	0.44	0.27
17	0	0	0.51	-0.07	0	0	0.44	-0.03
18	0	0	0.48	0.06	0	0	0.42	0.07
19	0	0	0.51	-0.11	0	0	0.47	-0.24
20	0	0	0.21	0.10	0	0	0.19	0.15
21	0	0	0.19	-0.03	0	0	0.14	-0.02
22	0	0	0.19	0.04	0	0	0.15	0.04
23	0	0	0.23	-0.08	0	0	0.20	-0.14
24	0	0	0.10	0.06	0	0	0.09	0.08
25	0	0	0.09	-0.02	0	0	0.07	-0.01
26	0	0	0.09	0.02	0	0	0.07	0.02
27	0	0	0.11	-0.04	0	0	0.10	-0.07
28	***-974***	-1643	0.00	0.00	***-944***	-2268	0.00	0.00
29	***-883***	638	0.00	0.00	***-491***	364	0.00	0.00
30	***-1765***	-978	0.00	0.00	***-1329***	-999	0.00	0.00
31	***-1292***	1857	0.00	0.00	***-1023***	2903	0.00	0.00
R =	***-4915***	**kips**		**R =**	***-3787***	**kips**		

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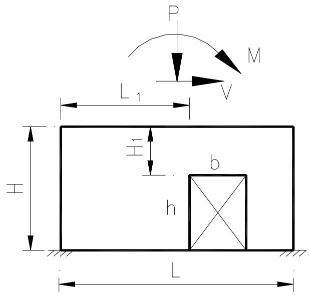
Shear Wall Analysis for Shear Wall with Opening Using Finite Element Method

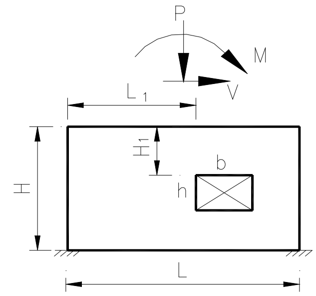
INPUT DATA & DESIGN SUMMARY

DIMENSIONS	L =	30	ft
	H =	20	ft
	thk =	8	in
OPENING	b =	5	ft
	h =	10	ft
LOCATION	L_1 =	10	ft
	H_1 =	5	ft
PROPERTIES	E =	1350	ksi
	υ =	0.25	(Poisson's ratio)
DIAPHRAGM	EA =	0	kips, (flexible diaphragm)
TOP LOADS	P =	62.5	kips
	M =	225	ft-kips
	V =	11.25	kips

(0 = flexible, ∞ = rigid, 0~∞ = semirigid. ASCE 7 12.3.1)



Not Apply

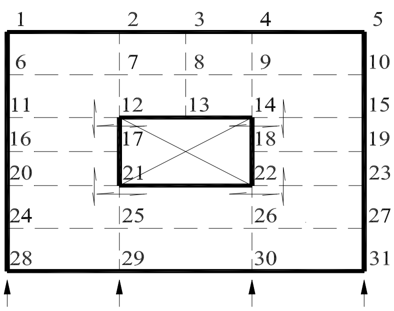


Drift = 0.005 in

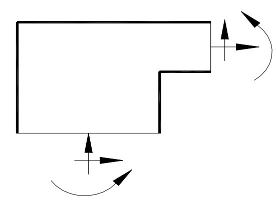
DRAG FORCE AROUND OPENING

Joint No.	Tension Positive (kips) Horiz	Vert
12	0.16	-10.20
14	-1.40	-23.71
21	-1.29	-12.84
22	0.54	-17.97

Joint No.	Reaction (kips) Horiz	Vert
28	-0.39	0.08
29	-0.02	11.89
30	-2.54	16.49
31	-8.29	34.04



Section	Section Force (kips, ft-kips) Axial	Shear	Moment
2 - 12	1.57	-0.50	3.55
3 - 13	2.51	4.08	7.15
4 - 14	-2.51	4.08	-3.05
11-12	13.42	-2.64	-67.09
14-15	49.08	-8.61	368.11
16-17	13.42	-2.64	-67.09
18-19	49.08	-8.61	368.11
20-21	13.42	2.64	-67.09
22-23	49.08	8.61	368.11
28-29	8.15	0.64	-40.74
30-31	46.24	9.54	346.77



POSITIVE DIRECTION

ANALYSIS

EQUIVALENT JOINT LOADS

Joint No.	Load (kips) Horiz	Vert
1	1.88	-4.58
2	2.34	-8.33
3	0.94	-4.58
4	3.28	-21.88
5	2.81	-23.13

Element No.	Joints	Dimension (ft) X	Y	Thick. (in)
1	1, 2, 6, 7	10.0	2.5	8
2	2, 3, 7, 8	2.5	2.5	8
3	3, 4, 8, 9	2.5	2.5	8
4	4, 5, 9, 10	15.0	2.5	8
5	6, 7, 11, 12	10.0	2.5	8
6	7, 8, 12, 13	2.5	2.5	8
7	8, 9, 13, 14	2.5	2.5	8
8	9, 10, 14, 15	15.0	2.5	8
9	11, 12, 16, 17	10.0	5.0	8
10	14, 15, 18, 19	15.0	5.0	8
11	16, 17, 20, 21	10.0	5.0	8
12	18, 19, 22, 23	15.0	5.0	8
13	20, 21, 24, 25	10.0	2.5	8
14	21, 22, 25, 26	5.0	2.5	8
15	22, 23, 26, 27	15.0	2.5	8
16	24, 25, 28, 29	10.0	2.5	8
17	25, 26, 29, 30	5.0	2.5	8
18	26, 27, 30, 31	15.0	2.5	8

Joint	Force (kips) X	Y	Deflection (in) X	Y
1	1.88	-4.58	0.004	-0.001
2	2.34	-8.33	0.004	-0.003
3	0.94	-4.58	0.004	-0.004
4	3.28	-21.88	0.004	-0.004
5	2.81	-23.13	0.005	-0.006
6	0	0	0.003	-0.001
7	0	0	0.003	-0.003
8	0	0	0.003	-0.003
9	0	0	0.003	-0.004
10	0	0	0.004	-0.006
11	0	0	0.002	0.000
12	0	0	0.003	-0.003
13	0	0	0.003	-0.003
14	0	0	0.003	-0.003
15	0	0	0.004	-0.005
16	0	0	0.001	0.000
17	0	0	0.001	-0.002
18	0	0	0.001	-0.002
19	0	0	0.003	-0.003
20	0	0	0.000	0.000
21	0	0	0.000	-0.001
22	0	0	0.000	-0.001
23	0	0	0.001	-0.002
24	0	0	0.000	0.000
25	0	0	0.000	0.000
26	0	0	0.000	0.000
27	0	0	0.001	-0.001
28	-0.39	0.08	0	0
29	-0.02	11.89	0	0
30	-2.54	16.49	0	0
31	-8.29	34.04	0	0

(cont'd)

Element Joints					**Corner Forces** (kips)							
					i		j		k		l	
No.	i	j	k	l	**X**	**Y**	**X**	**Y**	**X**	**Y**	**X**	**Y**
1	1	2	6	7	1.87	-4.58	1.04	-6.46	-1.95	3.85	-0.96	7.19
2	2	3	7	8	1.31	-1.88	-1.09	-2.20	0.43	1.66	-0.65	2.41
3	3	4	8	9	2.02	-2.39	-0.45	-2.93	-0.92	0.82	-0.65	4.50
4	4	5	9	10	3.74	-18.95	2.81	-23.13	-3.62	17.86	-2.93	24.22
5	6	7	11	12	1.95	-3.85	0.58	-7.48	-2.22	3.22	-0.31	8.11
6	7	8	12	13	-0.05	-1.36	-0.67	-1.25	-0.12	2.08	0.84	0.53
7	8	9	13	14	2.24	-1.98	0.27	-2.51	-0.84	-0.53	-1.68	5.02
8	9	10	14	15	4.00	-19.84	2.93	-24.22	-3.80	18.69	-3.13	25.37
9	11	12	16	17	2.22	-3.22	0.42	-10.20	-2.46	1.90	-0.18	11.52
10	14	15	18	19	5.48	-23.71	3.13	-25.37	-4.72	20.84	-3.88	28.24
11	16	17	20	21	2.46	-1.90	0.18	-11.52	-1.55	0.58	-1.10	12.84
12	18	19	22	23	4.72	-20.84	3.88	-28.24	-3.47	17.97	-5.13	31.11
13	20	21	24	25	1.55	-0.58	-0.19	-8.21	-0.99	0.24	-0.36	8.55
14	21	22	25	26	1.29	-4.63	0.54	-3.99	-0.28	3.71	-1.55	4.91
15	22	23	26	27	2.94	-13.98	5.13	-31.11	-1.83	12.64	-6.24	32.45
16	24	25	28	29	0.99	-0.24	-0.35	-7.91	-0.39	0.08	-0.25	8.07
17	25	26	29	30	0.99	-4.36	0.08	-3.76	0.23	3.82	-1.30	4.30
18	26	27	30	31	3.30	-13.78	6.24	-32.45	-1.24	12.19	-8.29	34.04

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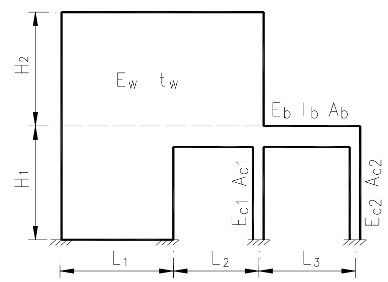
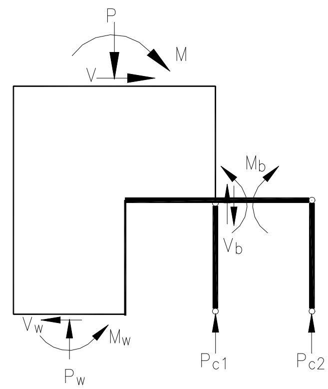
Discontinuous Shear Wall Analysis Using Finite Element Method

INPUT DATA & DESIGN SUMMARY

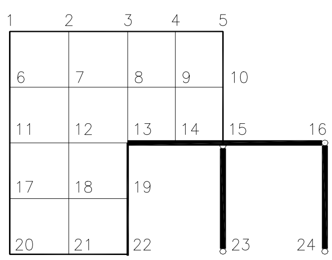
DIMENSIONS	L_1 =	15	ft
	L_2 =	15	ft
	L_3 =	10	ft
	H_1 =	16	ft
	H_2 =	14	ft
BEAM PROPERTIES	E_b =	3156	ksi
	G_b =	1372	ksi
	I_b =	106667	in^4
	A_b =	800	in^2
WALL PROPERTIES	thk =	8	in
	E =	1350	ksi
	υ =	0.25	(Poisson's ratio)
COLUMN PROPERTIES	E_{c1} =	3156	ksi
no column input 0 ==>	A_{c1} =	576	in^2
	E_{c2} =	3156	ksi
no column input 0 ==>	A_{c2} =	576	in^2
TOP LOADS	P =	62.5	kips
	M =	20	ft-kips
	V =	11.25	kips





The bottom shear wall reactions	P_w =	**34.25**	kips
	M_w =	**4.17**	ft-kips
	V_w =	**11.25**	kips
The max beam section forces	V_b =	**0.47**	kips
	M_b =	**4.68**	ft-kips
The beam deflection at end wall	Δ_b =	**0.003**	in, downward
The column axial forces	P_{c1} =	**28.71**	kips
	P_{c2} =	**-0.47**	kips

ANALYSIS



EQUIVALENT JOINT LOADS

Joint No.	Load (kips) Horiz	Vert
1	1.41	-7.40
2	2.81	-15.13
3	2.81	-15.63
4	2.81	-16.13
5	1.41	-8.23

Element No.	Joints	Dimension (ft) X	Y	Thick. (in)
1	1, 2, 6, 7	*7.5*	*7.0*	*8*
2	2, 3, 7, 8	*7.5*	*7.0*	*8*
3	3, 4, 8, 9	*7.5*	*7.0*	*8*
4	4, 5, 9, 10	*7.5*	*7.0*	*8*
5	6, 7, 11, 12	*7.5*	*7.0*	*8*
6	7, 8, 12, 13	*7.5*	*7.0*	*8*
7	8, 9, 13, 14	*7.5*	*7.0*	*8*
8	9, 10, 14, 15	*7.5*	*7.0*	*8*
9	11, 12, 17, 18	*7.5*	*8.0*	*8*
10	12, 13, 18, 19	*7.5*	*8.0*	*8*
11	17, 18, 20, 21	*7.5*	*8.0*	*8*
12	18, 19, 21, 22	*7.5*	*8.0*	*8*
13	13, 14	*7.5*	*0*	*Beam*
14	14, 15	*7.5*	*0*	*Beam*
15	15, 16	*10.0*	*0*	*Beam*
16	15, 23	*0*	*16.0*	*Column*
17	16, 24	*0*	*16.0*	*Column*

Joint	Force (kips) X	Y	Deflection (in) X	Y
1	1.41	-7.40	0.009	-0.004
2	2.81	-15.13	0.010	-0.006
3	2.81	-15.63	0.009	-0.007
4	2.81	-16.13	0.009	-0.008
5	1.41	-8.23	0.009	-0.008
6	0	0	0.007	-0.002
7	0	0	0.008	-0.005
8	0	0	0.008	-0.006
9	0	0	0.008	-0.007
10	0	0	0.008	-0.006
11	0	0	0.005	-0.001
12	0	0	0.006	-0.003
13	0	0	0.007	-0.006
14	0	0	0.007	-0.006
15	0	0	0.008	-0.003
16	0	0	0.008	0.000
17	0	0	0.002	0.000
18	0	0	0.002	-0.001
19	0	0	0.003	-0.003
20	-3.52	-3.72	0	0
21	-2.53	15.65	0	0
22	-5.20	22.33	0	0
23	0.00	28.71	0	0
24	0.00	-0.47	0	0

Note:

1. This is pure Finite Element Method without rigidity reduction factor. For cracked wall the rigidity reduction factor, such as ($1 / 2^{0.5}$), can be input by wall thickness.
2. For beam and columns design, the section forces must consider UBC 1630.8.2.1 or ASCE 7-05 12.3.3.3.

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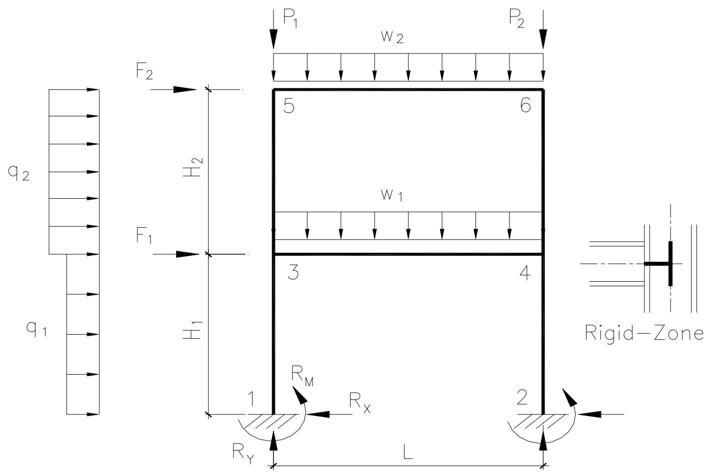
Two Story Moment Frame Analysis using Finite Element Method

INPUT DATA & SUMMARY



COLUMN SIZE (WF, Tube, or Pipe)

1- 5 =	HSS12.500X0.500	Pipe
ORIENTATION =	x-x	
2 - 6 =	W12X65	WF
ORIENTATION =	x-x	

BEAM SIZE (WF, Tube, or Pipe)

3 - 4 =	W18X65	WF
ORIENTATION =	x-x	
5 - 6 =	HSS12X6X5/8	Tube
ORIENTATION =	y-y	,(flat)

DIMENSIONS

H_1 =	14	ft
H_2 =	16	ft
L_1 =	22	ft

LOADS

F_1 =	19.5	kips, (horiz. to right)
F_2 =	12	kips, (horiz. to right)
P_1 =	5	kips, (downward)
P_2 =	5	kips, (downward)
q_1 =	0.26	kips / ft, (horiz. to right)
q_2 =	0.32	kips / ft, (horiz. to right)
w_1 =	0.64	kips / ft, (downward)
w_2 =	0.45	kips / ft, (downward)

RIGID-ZONE LENGTH FACTOR 75 % of (d / 2)

DESIGN SECTION FORCES

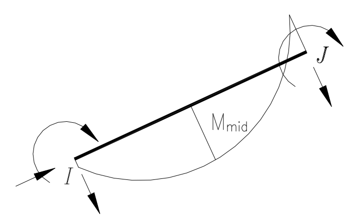
Column	N_{15} =	-6.2	k,(compre.)	N_{26} =	-38.2	k,(compression)
	V_{15} =	12.7	kips	V_{26} =	25.7	kips
	M_{15} =	91.3	ft-kips	M_{26} =	207.6	ft-kips
Beam	V_{34} =	24.4	kips	V_{56} =	8.6	kips
	M_{34} =	237.1	ft-kips	M_{56} =	57.7	ft-kips
	$M_{34,\ mid}$ =	0.0	ft-kips	$M_{56,\ mid}$ =	9.2	ft-kips

HORIZONTAL DEFLECTION

$\Delta_{H,3}$ =	0.9927	in, (horiz. to right)
$\Delta_{H,4}$ =	0.9849	in, (horiz. to right)
$\Delta_{H,5}$ =	2.3865	in, (horiz. to right)
$\Delta_{H,6}$ =	2.3820	in, (horiz. to right)

ANALYSIS

Joint	Coordinates (ft)		Reaction (k, ft-k)			Joint J Deflection (in,deg)		
	X	Y	R_X	R_Y	R_M	X	Y	θ
1	0	0	12.72	-4.20	91.28	0.000	0.000	0.000
2	22	0	25.72	38.18	207.55	0.000	0.000	0.000
3	0	14				0.993	-0.001	0.117
4	22	14				0.985	0.011	0.250
5	0	30				2.386	0.001	0.391
6	22	30				2.382	0.016	0.370

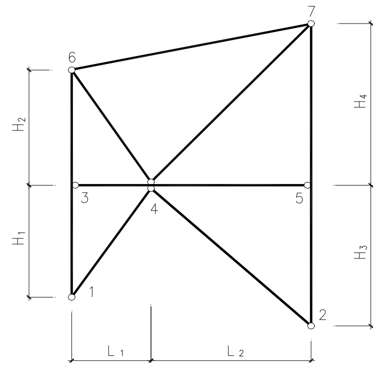


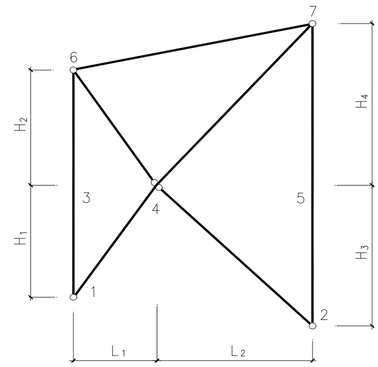
TYPICAL POSITIVE FORCES

End Joint		A	I	E	Length	Moment (ft-k)			Axial	Shear (k)	
I	J	(in^2)	(in^4)	(ksi)	(ft)	I	Mid	J	(k)	I	J
1	3	17.2	191	29000	13.43	91.28	-1.98	75.60	4.20	-12.72	10.98
2	4	19.1	533	29000	13.43	207.55	-34.91	137.72	-38.18	-25.72	25.72
3	4	19.1	1070	29000	21.65	-141.00	-10.52	-237.06	16.46	10.53	-24.39
3	5	17.2	191	29000	15.62	59.71	-8.03	23.16	-6.21	-7.80	2.81
4	6	19.1	533	29000	15.62	86.64	-14.37	57.90	-13.69	-9.25	9.25
5	6	18.7	107	29000	21.65	-23.23	9.16	-57.66	9.25	-1.14	-8.61

(The length & forces not included Rigid-Zone.)

X Braced Frame Analysis using Finite Element Method





INPUT DATA & SUMMARY

COLUMN SIZE

1- 6 = **W10X45** WF

ORIENTATION = **x-x**

2 - 7 = **W10X45** WF

ORIENTATION = **y-y** ,(weak axis)

BEAM SIZE

3 - 5 = **W14X22** WF

ORIENTATION = **y-y** ,(flat)

6 - 7 = **W16X40** WF

BRACE SIZE

HSS8X8X1/2 Tube

DIMENSIONS

H_1 = **14** ft H_3 = **18** ft

H_2 = **14** ft H_4 = **16** ft

L_1 = **6.5** ft

L_2 = **8** ft

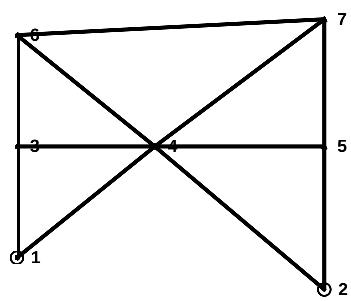
THE MAX SECTION FORCES

Column	N_{16} =	14.4	k,(tension)	N_{27} =	-50.2	k,(compression)
	V_{16} =	0.0	kips	V_{27} =	0.0	kips
	M_{16} =	-0.4	ft-kips	M_{27} =	-0.1	ft-kips
Brace	T_{max} =	68.5	k,(tension)	C_{min} =	30.3	k,(tension)
Beam	N_{67} =	2.5	k,(tension)	M_{35} =	0.004779	ft-kips

EQUIVALENT JOINT LOADS

F_3 =	**10**	kips, (horiz. to right)	P_3 =	**-12.5**	kips, (downward)	Δ_3 =	0.0870	in, (horiz. to right)
F_5 =	**11**	kips, (horiz. to right)	P_5 =	**-13**	kips, (downward)	Δ_5 =	0.0884	in, (horiz. to right)
F_6 =	**15**	kips, (horiz. to right)	P_6 =	**-11**	kips, (downward)	Δ_6 =	0.1862	in, (horiz. to right)
F_7 =	**16**	kips, (horiz. to right)	P_7 =	**-9.8**	kips, (downward)	Δ_7 =	0.1963	in, (horiz. to right)

ANALYSIS



Joint	Coordinates (ft) X	Y	Load & Reaction (k) F_X / R_X	F_Y / R_Y
1	0	0	-24.19	-66.52
2	14.5	-4	-27.81	112.82
3	0	14	10	-12.5
4	6.5	14	0	0
5	14.5	14	11	-13
6	0	28	15	-11
7	14.5	30	16	-9.8

Element	End Joint		Joint I Deflection (in, rad)			Joint J Deflection (in,rad)		
	I	J	X	Y	θ	X	Y	θ
A	1	3	0	0	-0.0005	0.0870	0.0063	-0.0006
B	1	4	0	0	-0.0004	0.0828	-0.0085	-0.0004
C	2	4	0	0	0.0003	0.0828	-0.0085	0.0003
D	2	5	0	0	-0.0004	0.0884	-0.0281	-0.0005
E	3	4	0.0870	0.0063	-0.0002	0.0828	-0.0085	-0.0002
F	4	5	0.0828	-0.0085	-0.0002	0.0884	-0.0281	-0.0002
G	3	6	0.0870	0.0063	-0.0006	0.1862	0.0180	-0.0006
H	4	6	0.0828	-0.0085	0.0006	0.1862	0.0180	0.0006
I	4	7	0.0828	-0.0085	-0.0006	0.1963	-0.0467	-0.0006
J	5	7	0.0884	-0.0281	-0.0005	0.1963	-0.0467	-0.0006
K	6	7	0.1862	0.0180	-0.0004	0.1963	-0.0467	-0.0004

Element	End Joint I	J	A (in^2)	I (in^4)	E (ksi)	Length (ft)	Moment (ft-k) I	J	Axial (k)	Shear (k)
A	1	3	13.3	248	29000	14.00	0.0	-0.4	14.4	0.0
B	1	4	13.5	125	29000	15.44	0.0	0.0	57.5	0.0
C	2	4	13.5	125	29000	19.70	0.0	0.0	-68.5	0.0
D	2	5	13.3	53.4	29000	18.00	0.0	-0.1	-50.2	0.0
E	3	4	6.49	7	29000	6.50	0.0	-0.0048	-10.1	0.0
F	4	5	6.49	7	29000	8.00	0.0048	0.0	11.0	0.0
G	3	6	13.3	248	29000	14.00	0.4	0.0	26.9	0.0
H	4	6	13.5	125	29000	15.44	0.0	0.0	41.4	0.0
I	4	7	13.5	125	29000	17.89	0.0	0.0	-30.3	0.0
J	5	7	13.3	53.4	29000	16.00	0.1	0.0	-37.2	0.0
K	6	7	11.8	518	29000	14.64	0.0	0.0	2.5	0.0

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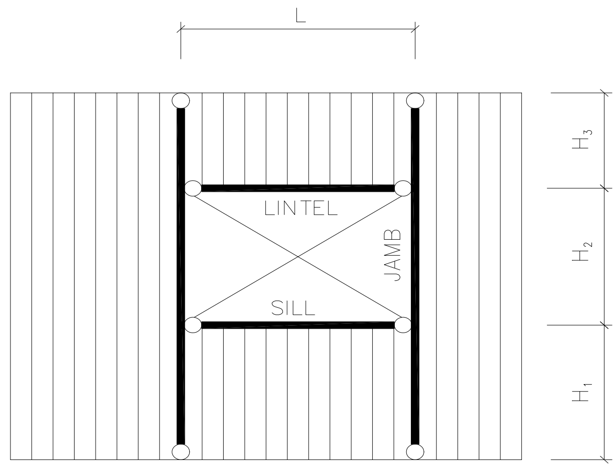
Wind Girt Deflection Analysis of Wood, Metal Stud, and/or Steel Tube

INPUT DATA & ANALYSIS RESULTS

DIMENSIONS

L =	**12**	ft
H_1 =	**4**	ft
H_2 =	**6**	ft
H_3 =	**8**	ft



GIRT SECTIONS

LINTEL = **6 x 8 - Douglas Fir-Larch No. 1**

SILL = **(2) - 2 x 6 - No. 1**

JAMB = **HSS6X2X1/4**

WIND LOAD

I =	**1**	(ASCE 7-05 Tab 6-1)
EXPOSURE =	**C**	
MEAN ROOF HEIGHT =	**36**	ft, (ASCE 7-05 Tab 6-3)
V =	**90**	mph, (IBC 09 Tab 1609.3.1)
BUILDING CORNER ?	**Yes**	(Zone 5, ASCE 7-05 Fig 6-11& 6-17)

THE DESIGN IS ADEQUATE.

DEFLECTION LIMITATION (IBC 09 Tab 1604.3) **L / 240**

ANALYSIS

OUT-OF-PLANE WIND FORCE (ASCE 7-05, Eq.6-22)

$$p = q_h\left[(GC_P)-(GC_{Pi})\right] = \left(0.00256K_hK_{Zt}K_dV^2I\right)\left[(GC_P)-(GC_{Pi})\right] = 22.4 \text{ psf}$$

USE* 16 *psf
(IBC 09 Table 1604.3 note: f)

Where K_h	=	1.02	, (ASCE Tab. 6-3)
K_d	=	0.85	, (ASCE Tab. 6-4)
Effective Area =		84.00	ft^2, (ASCE Fig. 6-11A or 6-17)
GC_p	=	-1.07	, (ASCE Fig. 6-11A or 6-17)
GC_{pi}	=	0.18	, (ASCE Fig. 6-5)
K_{zt}	=	1	Flat, (ASCE 6.5.7.2)

CHECK LINTEL DEFLECTION

$$\Delta = \frac{5wL^4}{384EI} = 0.31 \text{ in} < L / 240 = 0.60 \text{ in}$$ **[Satisfactory]**

Where w	=	0.11	kips / ft
E	=	1600	ksi
I	=	104	in^4

CHECK SILL DEFLECTION

$$\Delta = \frac{5wL^4}{384EI} = 0.53 \text{ in} < L / 240 = 0.60 \text{ in}$$ **[Satisfactory]**

Where w	=	0.08	kips / ft
E	=	1700	ksi
I	=	42	in^4

CHECK JAMB DEFLECTION

Δ_{max} = 0.53 in, at 9.00 ft from top, **<** H / 240 = 0.90 in **[Satisfactory]**

Where R_{lintel}	=	0.67	kips
R_{sill}	=	0.48	kips
E	=	29000	ksi
I	=	13	in^4

NOTE : Since girt size controlled by deflection, not by capacity, the wind coefficient w, 1.3, does not apply. (IBC 09 Section 1605.3.2)

Purchaser's LOGO

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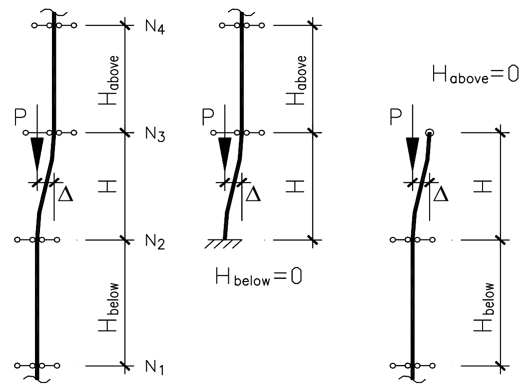
Column Deformation Compatibility Design using Finite Element Method

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION (WF, HSS tube & pipe) **W10X60**



STORY HEIGHTS H_{above} = 14 ft

H = 14 ft, continued top & bot

H_{below} = 16 ft

COLUMN AXIAL LOAD, ASD P_{DL} = 20 kips

P_{LL} = 18 kips

EXPECTED DEFORMATION Δ = 4.2 in

(ASCE 7-10 12.12.4)

STRONG AXIS BENDING ? (1=Yes, 0=No)

=> 1 yes, strong axis, x-x, bending.

THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE JOINT DEFLECTIONS AND SECTION FORCES

F_y = 50 ksi E = 29000 ksi G = 11154 ksi

A = 17.6 in^2 I = 341 in^4

JOINT DEFLECTIONS UNDER Δ ONLY

Note	**X** (in)	**Y** (in)	**θ** (deg)
N_1	0	0	0.496192
N_2	0	0.0000	-1.007175
N_3	4.2	0.0000	-0.932621
N_4	4.2	0.0000	0.457379

THE DESIGN COLUMN SECTION FORCES UNDER Δ ONLY

Note	**V** (k)	**P** (k)	**M** (ft-k)
N_2	-33.09	0.000	225.24
N_3	33.09	0.000	238.00

P_u = $1.2\,P_{DL} + 1.0\,P_{LL}$ = 42.00 kips

M_u = $(P_{DL} + P_{LL})\,\Delta + M$ = 251.30 ft-kips

V_u = V = 33.09 kips

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-10, H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) , & for\ \dfrac{P_r}{P_c}\ge 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) , & for\ \dfrac{P_r}{P_c}< 0.2 \end{cases} = 1.00 \quad < \quad 1.0$$

[Satisfactory]

Where KL_x = 14 ft, for x-x axial load. KL_y = 14 ft, for y-y axial load.

$(KL / r)_{max}$ = 65 < 200 **[Satisfactory]**

P_r = 42.00 kips

M_{rx} = 251.30 ft-kips

M_{ry} = 0.00

$P_c = \phi\,P_n$ = 0.9 x 643.43 = 579.09 kips, (AISC 360-10 Chapter E)

> P_r **[Satisfactory]**

$M_{cx} = \phi\,M_n$ = 0.9 x 289.99 = 260.99 ft-kips, (AISC 360-10 Chapter F)

> M_{rx} **[Satisfactory]**

$M_{cy} = \phi\,M_n$ = 0.9 x 145.83 72.92 ft-kips, (AISC 360-10 Chapter F)

> M_{ry} **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-10, G)

$\phi\,V_n$ = 115.67 kips > V_u **[Satisfactory]**

Note: The governing case is always that only the design story column has Story Drift and other stories no drift, since the other story at same direction drifts, including PD effects, will reduce the section forces of design story column. The opposite story drifts are not Expected Deformation, because Expected Deformation can not be developed from Linear Modes Analysis ***after*** CQC or SRSS.

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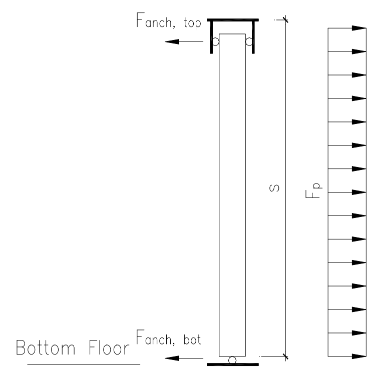
Interior Wall Lateral Forces Based on IBC 09 / CBC 10

INPUT DATA

AVERAGE ROOF HEIGHT FROM GROUND	h	=	48	ft
DESIGN WALL BOTTOM FLOOR HEIGHT	z	=	24	ft
STORY HEIGHT	s	=	14	ft
PLAIN (UNREINFORCED) MASONRY WALL ? (0=no, 1=yes)			1	Yes, (ASCE Tab. 13.5-1 Item 1)
WALL OPERATING WEIGHT	W_p	=	86	psf
SEISMIC PARAMETER	S_{DS}	=	0.96	(ASCE 7-05 Sec 11.4.4)
SEISMIC IMPORTANCE FACTOR	I_p	=	1.5	(ASCE 7-05 Tab 11.5-1)
MASONRY ANCHOR TO TOP FLEXIBLE DIAPHRAGM?			0	No, (ASCE Tab. 13.5-1 Footnote b)



DESIGN SUMMARY

Out-of Plane Force for Wall Design	F_p	=	54	psf, ASD
Top Anchorage Force	$F_{anch,\ top}$	=	555	lbs/ft, ASD
Bottom Anchorage Force	$F_{anch,\ bot}$	=	429	lbs/ft, ASD

ANALYSIS

Out-of Plane Force for Wall Design (ASCE 7-05 Sec. 13.3 & IBC/CBC 1607.13)

$$\begin{cases} F_p = Max\left[0.5\left(F_{p,\ top}\ ,\ F_{p,\ bot}\right)\ ,\ 5\ \ psf\right]\ ,\ ASD\ \ Level \\ F_{p,\ top\ or\ bot} = \frac{1}{1.4}Max\left[0.3S_{DS}I_pW_p\ ,\ Min\left(\frac{0.4a_pS_{DS}I_pW_p}{R_p}\left(1+2\frac{z}{h}\right)\ ,\ 1.6S_{DS}I_pW_p\right)\right] \end{cases}$$

	z, attachment height, (ft)	F_p (psf, ASD)	
Top	38.0	61	(0.71 W_p)
Bot	24.0	47	(0.55 W_p)
	Wall Design	***54***	(0.63 W_p)

Where : a_p = 1.0 (ASCE Tab. 13.5-1) R_p = 1.5 (ASCE Tab. 13.5-1)

Out-of-plane seismic force for anchorage design (ASCE 7-05 Sec. 13.3 & 13.4)

$$F_{anch} = \begin{cases} \frac{1.3}{1.4}Max\left[0.3S_{DS}I_pW_p\ ,\ Min\left(\frac{0.4a_pS_{DS}I_pW_p}{R_p}\left(1+2\frac{z}{h}\right)\ ,\ 1.6S_{DS}I_pW_p\right)\right] \\ \frac{1}{1.4}0.8S_{DS}I_pW_p\ ,\ for\ \ concrete\ \ or\ \ masonry\ \ to\ \ flexible\ \ diaphragm \end{cases}$$

	z, attachment height, (ft)	F_{anch} (lbs/ft, ASD)	
Top	38.0	***555***	(0.92 W_p)
Bot	24.0	***429***	(0.71 W_p)

(If diaphragm flexible, no factor 1.3 apply, ASCE 13.4.2.a)

Where : a_p = 1.0 (ASCE Tab. 13.5-1) R_p = 1.5 (ASCE 13.4.2)

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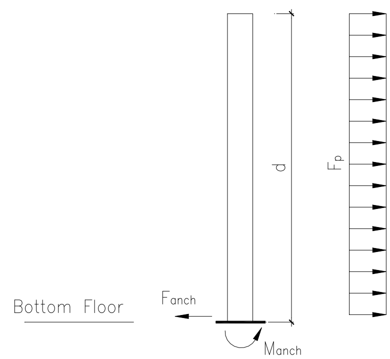
Interior Wall Lateral Forces Based on IBC 09 / CBC 10

INPUT DATA

AVERAGE ROOF HEIGHT FROM GROUND	h	=	48	ft
DESIGN WALL BOTTOM FLOOR HEIGHT	z	=	24	ft
WALL HEIGHT	d	=	10	ft
WALL OPERATING WEIGHT	W_p	=	86	psf
SEISMIC PARAMETER	S_{DS}	=	0.96	(ASCE 7-05 Sec 11.4.4)
SEISMIC IMPORTANCE FACTOR	I_p	=	1.5	(ASCE 7-05 Tab 11.5-1)



DESIGN SUMMARY

Out-of Plane Force for Wall Design	F_p	=	71	psf, ASD
Shear Anchorage Force	F_{anch}	=	1533	lbs/ft, ASD
Anchorage Moment	M_{anch}	=	7666	ft-lbs/ft, ASD

ANALYSIS

Out-of Plane Force for Wall Design (ASCE 7-05 Sec. 13.3 & IBC/CBC 1607.13)

$$F_p = Max\left[\frac{1}{1.4}0.3S_{DS}I_pW_p \;,\; \frac{1}{1.4}Min\left(\frac{0.4a_pS_{DS}I_pW_p}{R_p}\left(1+2\frac{z}{h}\right) \;,\; \frac{1}{1.4}1.6S_{DS}I_pW_p\right) \;,\; 5 \;\; psf\right] = \quad 0.82\ W_p$$

$= \quad 71 \quad$ psf, ASD Level

Where : a_p = 2.5 (ASCE Tab. 13.5-1) $\qquad R_p$ = 2.5 (ASCE Tab. 13.5-1)

Out-of-plane seismic force for anchorage design (ASCE 7-05 Sec. 13.3 & 13.4)

$$F_{anch} = \frac{1.3}{1.4}Max\left[0.3S_{DS}I_pW_p \;,\; Min\left(\frac{0.4a_pS_{DS}I_pW_p}{R_p}\left(1+2\frac{z}{h}\right) \;,\; 1.6S_{DS}I_pW_p\right)\right] = \quad 1.78\ W_p \quad = \quad 1533$$ lbs/ft, ASD Level

$\boldsymbol{M_{anch}}$ = $\boldsymbol{0.5\,d\,F_{anch}}$ **=** 7666 ft-lbs/ft, ASD Level

Where : a_p = 2.5 (ASCE Tab. 13.5-1) $\qquad R_p$ = 1.5 (ASCE 13.4.2)

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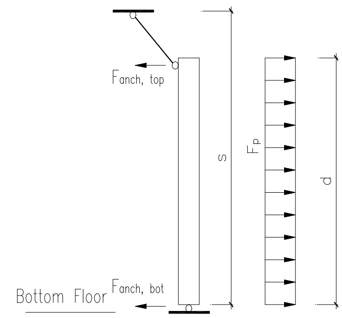
Interior Wall Lateral Forces Based on IBC 09 / CBC 10

INPUT DATA

AVERAGE ROOF HEIGHT FROM GROUND	h	=	48	ft
DESIGN WALL BOTTOM FLOOR HEIGHT	z	=	24	ft
STORY HEIGHT	s	=	14	ft
WALL HEIGHT	d	=	9.5	ft
PLAIN (UNREINFORCED) MASONRY WALL ? (0=no, 1=yes)			1	Yes, (ASCE Tab. 13.5-1 Item 1)
WALL OPERATING WEIGHT	W_p	=	86	psf
SEISMIC PARAMETER	S_{DS}	=	0.96	(ASCE 7-05 Sec 11.4.4)
SEISMIC IMPORTANCE FACTOR	I_p	=	1.5	(ASCE 7-05 Tab 11.5-1)
MASONRY ANCHOR TO TOP FLEXIBLE DIAPHRAGM?			0	No, (ASCE Tab. 13.5-1 Footnote b)



DESIGN SUMMARY

Out-of Plane Force for Wall Design	F_p	=	54	psf, ASD
Top Anchorage Force	$F_{anch,\ top}$	=	376	lbs/ft, ASD
Bottom Anchorage Force	$F_{anch,\ bot}$	=	291	lbs/ft, ASD

ANALYSIS

Out-of Plane Force for Wall Design (ASCE 7-05 Sec. 13.3 & IBC/CBC 1607.13)

$$\begin{cases} F_p = Max\left[0.5\left(F_{p,\ top}\ ,\ F_{p,\ bot}\right)\ ,\ 5\ \ psf\right]\ ,\ ASD\ \ Level \\ F_{p,\ top\ or\ bot} = \frac{1}{1.4}Max\left[0.3S_{DS}I_pW_p\ ,\ Min\left(\frac{0.4a_pS_{DS}I_pW_p}{R_p}\left(1+2\frac{z}{h}\right)\ ,\ 1.6S_{DS}I_pW_p\right)\right] \end{cases}$$

	z, attachment height, (ft)	F_p (psf, ASD)
Top	38.0	61 (0.71 W_p)
Bot	24.0	47 (0.55 W_p)
	Wall Design	***54*** (0.63 W_p)

(z at top must be attachment point at main structure floor)

Where : a_p = 1.0 (ASCE Tab. 13.5-1) R_p = 1.5 (ASCE Tab. 13.5-1)

Out-of-plane seismic force for anchorage design (ASCE 7-05 Sec. 13.3 & 13.4)

$$F_{anch} = \begin{cases} \frac{1.3}{1.4}Max\left[0.3S_{DS}I_pW_p\ ,\ Min\left(\frac{0.4a_pS_{DS}I_pW_p}{R_p}\left(1+2\frac{z}{h}\right)\ ,\ 1.6S_{DS}I_pW_p\right)\right] \\ \frac{1}{1.4}0.8S_{DS}I_pW_p\ ,\ for\ \ concrete\ \ or\ \ masonry\ \ to\ \ flexible\ \ diaphragm \end{cases}$$

	z, attachment height, (ft)	F_{anch} (lbs/ft, ASD)
Top	38.0	***376*** (0.92 W_p)
Bot	24.0	***291*** (0.71 W_p)

(If diaphragm flexible, no factor 1.3 apply, ASCE 13.4.2.a)

Where : a_p = 1.0 (ASCE Tab. 13.5-1) R_p = 1.5 (ASCE 13.4.2)

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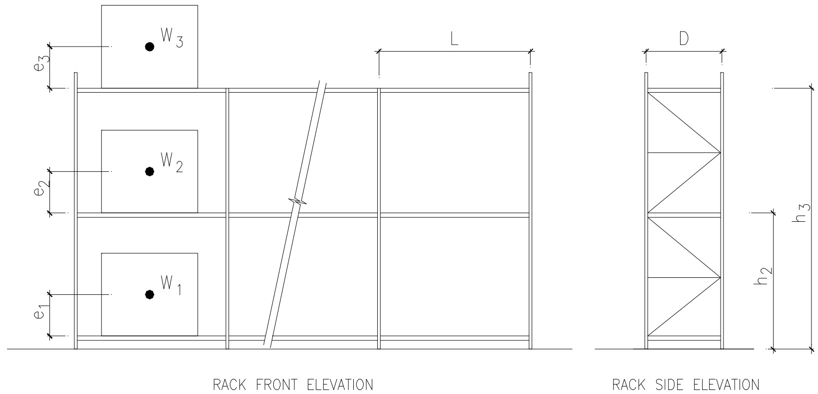
Lateral Loads of Storage Racks, with Hilti Anchorage, Based on ASCE 7-05

Dimensions

D = 3.5 ft

L = 9 ft

h_3 = 15.5 ft



Determine Rack Base Shear (Derived from ASCE 7-05 15.5.3 & 13.3)

$$V = F_p = \frac{1}{1.5} Max \left[0.3 S_{DS} I_p W_p \ , \ Min\left(\frac{0.4 a_p S_{DS} I_p W_p}{R_p}\left(1+2\frac{z}{h}\right) \ , \ 1.6 S_{DS} I_p W_p \right) \right] = \quad 0.34 \ W_p \text{, ASD}$$

= 504.5 lbs,
(ASD level force at each ***L*** rack)

Where S_{DS} = 1.121 (ASCE 7-05 11.4.4)

I_p = 1.5 (ASCE 7-05 15.5.3.1)

$W_p = W_1 + W_2 + W_3$ = 1500.0 lbs, each ***L*** rack

a_p = 2.5 (ASCE 7-05 15.5.3)

R_p = 4 (ASCE 7-05 15.5.3)

z = 0 ft, rack floor height from building ground (ASCE 7-05 13.3.3.1)

h = 36 ft, average building roof height (ASCE 7-05 13.3.3.1)

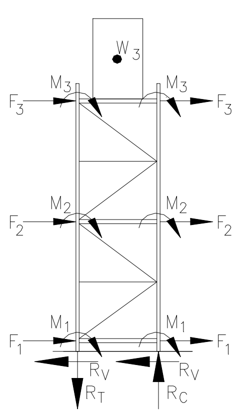
Calculate Vertical Distribution of Rack Base Shear (lbs) & Allowable Elastic Drift (ASCE 7-05 12.8.3 & 15.5.3.3)

Level	W_x	h_x	h_x^k	$W_x h_x^k$	F_x, ASD (12.8-11)		$\delta_{xe,allowable}$, in, ASD
3	500	15.5	15.5	7750	325.8	(0.65 Wx)	6.643
2	500	8	8.0	4000	168.2	(0.34 Wx)	3.429
1	500	0.5	0.5	250	10.5	(0.02 Wx)	0.214
	1500.0			12000	504.5		

Where k = 1 (ASCE 7-05 15.5.3.2)

$\delta_{xe,allowable,\ ASD}$ = 5% h_x/1.4, (ASCE 7-05 15.5.3.4)

Calculate Lateral Forces (lbs), at Each Joint of Both Directions, and Max Reactions (lbs) (ASCE 7-05 15.5.3.2)



Case a, (ASCE 7-05 15.5.3.2 a)

Level	e_x (ft)	W_x	F	M	R_T	R_C	R_V
3	3	335	109.1	327			
2	3	335	56.3	169			
1	3	335	3.5	11			
		1005.0	169.0	507.0	1012.4	2017.4	169.0

Case b, (ASCE 7-05 15.5.3.2 b)

Level	e_x (ft)	W_x	F	M	R_T	R_C	R_V
3	3	500	84.1	252			
2		0	0.0	0			
1		0	0.0	0			
		500.0	84.1	252.2	638.8	1138.8	84.1

(cont'd)

Anchorage & Summary

2 - 1/2" ϕ, w/ 2" emb in 2.5 ksi conc, KWIK Bolt-TZ required at each column base. (ICC ESR-1917)

$(P_s / P_t) + (V_s / V_t) =$ 0.895 < 1.20 **[SATISFACTORY]**

Where	$P_s =$	506.2	lbs	$P_t =$ 606	lbs / bolt, (ICC ESR-1917)
	$V_s =$	84.5	lbs,	$V_t =$ 1419.5	lbs / bolt, (ICC ESR-1917)

Notes:

1. Input W_2 & e_2 zero for only 2 level rack, or lump some levels together at their top for more than 3 levels.

2. Rack self should be designed by Manufacturers Institute Specifications (RMI), or from ICC report.

- *a.* Vertical impact load factor is 1.25 (RMI 2.4)
- *b.* Max beam deflection is 1/180 (RMI 2.5)
- *c.* Allowable soil capacity for slab on grade is 500 psf.

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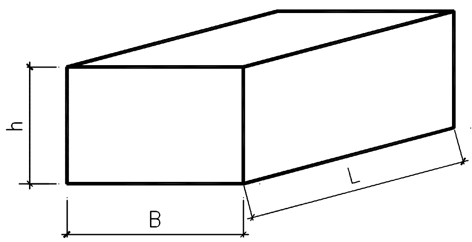
Wind Analysis for Alternate All-Heights Method, Based on ASCE 7-2010

INPUT DATA



Exposure category (B, C or D, ASCE 7-10 26.7.3)		C	
Importance factor (ASCE 7-10 Table 1.5-2)	I_w =	1.00	for all Category
Basic wind speed (ASCE 7-10 26.5.1 or 2012 IBC)	V =	136	mph
Topographic factor (ASCE 7-10 26.8 & Table 26.8-1)	K_{zt} =	1	Flat
Building height (IBC/CBC, 1609.6.1.1)	h =	22	ft
Building length	L =	100	ft
Building width (IBC/CBC, 1609.6.1.1)	B =	50	ft
Effective area of components (or Solar Panel area)	A =	28	ft^2

DESIGN SUMMARY

Max building horizontal force normal to building length, L, face	=	89.6	kips
Max overturning moment at wind normal to building length, L, face	=	3147.6	ft - kips
Max building horizontal force normal to building length, B, face	=	37.7	kips
Max overturning moment at wind normal to building length, B, face	=	3316.5	ft - kips
Max building upward force	=	136.9	kips
Max building torsion force	=	1007.6	ft - kips

ANALYSIS

Velocity pressure

$q_s = 0.00256\ V^2$ = **47.35** psf

where: q_s = wind velocity pressure at atandard height of 33 feet (IBC/CBC Table 1609.6.2(1))

Design pressures for MWFRS

$P_{net} = q_s\ K_z\ C_{net}\ [\ I\ K_{zt}\ ]$ (IBC/CBC, Equation 16-34)

where: P = pressure in appropriate zone. P_{min} = **16** psf (ASCE 7-10 28.4.4)

K_z = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) = **0.85**

$C_{net} = K_d\ [(G\ C_{pf}\ )-(G\ C_{pi}\ )]$, (IBC/CBC, 1609.6.2 / Table 1609.6.2(2). The equation used on following calcs.)

K_d = wind directionality factor. (Tab. 26.6-1, for building, page 250) = **0.85**

$G\ C_{p\,f}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 28.4-1, page 300 & 301)

$G\ C_{p\,i}$ = product of gust effect factor and internal pressure coefficient.(Tab. 26.11-1, Enclosed Building, page 258)

= **0.18** or **-0.18**

Surface	z (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$
Windward Wall	0 - 15	17.40	30.67
	20	18.81	32.08
	22	19.26	32.53

Surface	z (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$
Side Wall	All	-29.30	-16.03

Normal to L Face Surface	z (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$	Normal to B Face Surface	z (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$
Leeward	All	-22.82	-9.55	Leeward	All	-16.35	-3.08

Normal to L Face Surface	Dist. (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$	Normal to B Face Surface	Dist. (ft)	P (psf) with GC_{Pi}	P (psf) with $-GC_{Pi}$
Roof	0 - 11	-35.77	-22.50	Roof	0 - 11	-35.77	-22.50
	22	-35.77	-22.50		22	-35.77	-22.50
	44	-22.82	-9.55		44	-22.82	-9.55
	50	-16.35	-3.08		100	-16.35	-3.08

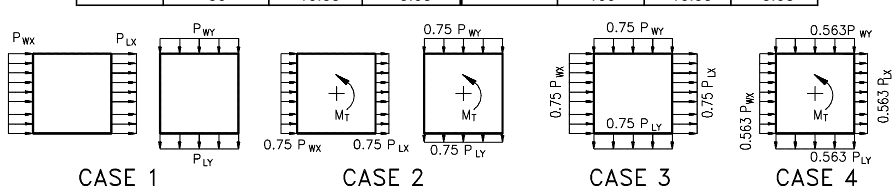
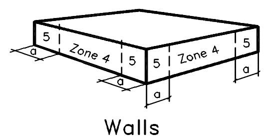


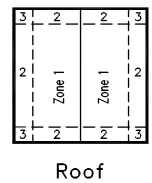
Figure 27.4-8, page 271

cont'd

Base Forces		Normal to L Face		Normal to B Face		Wind with Angle		ASCE-7
		Case 1	Case 2	Case 1	Case 2	Case 3	Case 4	
V_{Base}	(kips)	90	67	38	28	95	55	
M_{Base}	(ft - kips)	3148	2361	3316	2487	4848	2574	Fig. 27.4-8
M_T	(ft - kips)	0	1008	0	212	0	915	Page 271
F_{Upward}	(kips)	106	79	77	58	137	74	
V_{min}	(kips)	35	35	18	18	40	39	Min. wind
$F_{Up,min}$	(kips)	80	80	80	80	80	80	Sec. 6.1.4.1

Design pressures for components and cladding





a = width of edge strips, Fig 28.4-1, note 9, page 301, MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3] = **5.00 ft**

	Effective Area (ft^2)	Zone 1		Zone 2		Zone 3		Zone 4		Zone 5	
		GC_P	$-GC_P$	GC_P	$-GC_P$	GC_P	$-GC_P$	GC_P	$-GC_P$	GC_P	$-GC_P$
Comp.	28	0.26	-0.96	0.26	-1.49	0.26	-2.04	0.83	-0.92	0.83	-1.12

(Walls reduced 10 %, Fig. 6-11A note 5.)

Comp. & Cladding Pressure (psf)	Zone 1		Zone 2		Zone 3		Zone 4		Zone 5	
	Positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
	16.05	-41.85	16.05	-61.46	16.05	-81.84	37.20	-40.51	37.20	-47.85

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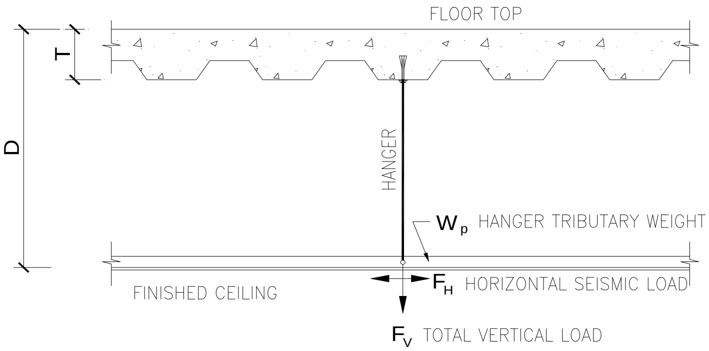
Suspended Ceiling Seismic Loads Based on ASCE 7-10

INPUT DATA & DESIGN SUMMARY



HANGER TRIBUTARY WEIGHT (4 psf min. 2010 CBC 1615A.1.16)

W_p = 3.5 kips

HEIGHT & DIMENSIONS

h =	60	ft, average roof height (ASCE 7-10 page 114)
H =	48	ft, this floor top level height
D =	6	ft
T =	8	in, from floor top to bottom anchorage point

SEISMIC COEFFICIENTS

S_{DS} =	1	(ASCE 7-10 11.4.4)
I_p =	1.5	(ASCE 7-10 13.1.3)
a_p =	1	(ASCE 7-10 Table 13.5-1 or Table 13.6-1)
R_p =	1.5	(ASCE 7-10**:** 2.5 for ceilings on Table 13.5-1, 1.5 for lighting fixtures on Table 13.6-1)

F_H =	2.40	W_p (SD)	=	1.71	W_p (ASD)	=	6.00	kips, ASD level horizontal load
F_V =	1.90	W_p (SD)	=	1.36	W_p (ASD)	=	4.75	kips, ASD level downward load

ANALYSIS

$F_H = F_p =$ (K_H) MAX{ $0.3S_{DS}I_pW_p$, MIN[$0.4a_pS_{DS}I_p(1+2z/h)/R_p\ W_p$, $1.6S_{DS}I_pW_p$] } , (ASCE 7-10 13.3.1, page 113)

= 2.5 MAX{ 0.45Wp , MIN[0.96Wp , 2.40Wp]}

= 2.40 W_p , (SD level load)

= 1.71 W_p , (ASD) = 6.00 kips, ASD level load

where z = H - D = 42.00 ft, height in structure of point of horizontal anchorage

K_H = 2.5 (ASCE 7-10 13.4.2 & 2010 CBC 1615A.1.14, but 1.0 for anchor on wood/metal or member/T-bar self)

$F_V = (1 + K_V)\ W_p$

= 1.90 W_p , (SD level load)

= 1.36 W_p , (ASD) = 4.75 kips, ASD level downward load

where z = H - T = 47.33 ft, height in structure of point of vertical anchorage

$K_V = K_H\ 0.2\ S_{DS}$ / 1.4 = 0.36 (vertical seismic factor, ASD level)

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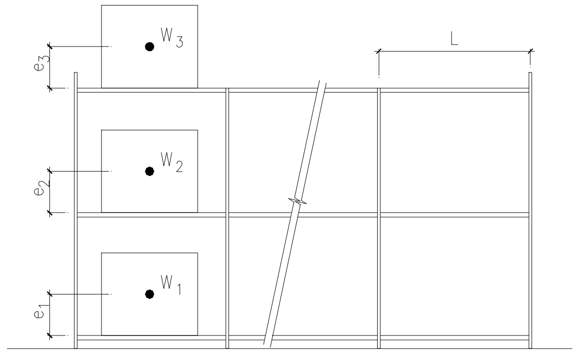
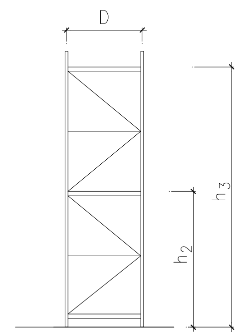
Lateral Loads of Storage Racks, with Red Head Anchorage, Based on ASCE 7-05

Dimensions

D =	3.5	ft
L =	9	ft
h_3 =	15.5	ft



RACK FRONT ELEVATION



RACK SIDE ELEVATION

Determine Rack Base Shear (Derived from ASCE 7-05 15.5.3 & 13.3)

$$V = F_p = \frac{1}{1.5} Max\left[0.3 S_{DS} I_p W_p \ , \ Min\left(\frac{0.4 a_p S_{DS} I_p W_p}{R_p}\left(1+2\frac{z}{h}\right) \ , \ 1.6 S_{DS} I_p W_p\right)\right] = \quad 0.34\ W_p\text{, ASD}$$

= 504.5 lbs,

(ASD level force at each L rack)

Where

S_{DS} =	1.121	(ASCE 7-05 11.4.4)
I_p =	1.5	(ASCE 7-05 15.5.3.1)
$W_p = W_1 + W_2 + W_3$ =	1500.0	lbs, each L rack
a_p =	2.5	(ASCE 7-05 15.5.3)
R_p =	4	(ASCE 7-05 15.5.3)
z =	0	ft, rack floor height from building ground (ASCE 7-05 13.3.3.1)
h =	36	ft, average building roof height (ASCE 7-05 13.3.3.1)

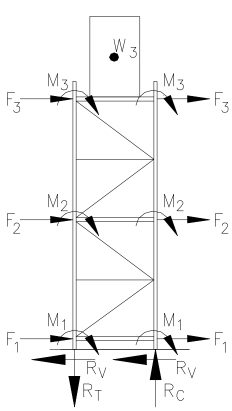
Calculate Vertical Distribution of Rack Base Shear (lbs) & Allowable Elastic Drift (ASCE 7-05 12.8.3 & 15.5.3.3)

Level	W_x	h_x	h_x^k	$W_x h_x^k$	F_x, ASD (12.8-11)		$\delta_{xe,allowable}$, in, ASD
3	500	15.5	15.5	7750	325.8	(0.65 Wx)	6.643
2	500	8	8.0	4000	168.2	(0.34 Wx)	3.429
1	500	0.5	0.5	250	10.5	(0.02 Wx)	0.214
	1500.0			12000	504.5		

Where k = 1 (ASCE 7-05 15.5.3.2)

$\delta_{xe,allowable,\ ASD} = 5\%\ h_x/1.4$, (ASCE 7-05 15.5.3.4)

Calculate Lateral Forces (lbs), at Each Joint of Both Directions, and Max Reactions (lbs) (ASCE 7-05 15.5.3.2)



Case a, (ASCE 7-05 15.5.3.2 a)

Level	e_x (ft)	W_x	F	M	R_T	R_C	R_V
3	3	335	109.1	327			
2	3	335	56.3	169			
1	3	335	3.5	11			
		1005.0	169.0	507.0	1012.4	2017.4	169.0

Case b, (ASCE 7-05 15.5.3.2 b)

Level	e_x (ft)	W_x	F	M	R_T	R_C	R_V
3	3	500	84.1	252			
2		0	0.0	0			
1		0	0.0	0			
		500.0	84.1	252.2	638.8	1138.8	84.1

(cont'd)

Anchorage & Summary

2 - 1/2" ϕ, w/ 1 7/8" emb in 2.5 ksi conc, ITW Red Head Wedge required at each column base. (ICC ESR-2251)

$(P_s / P_t) + (V_s / V_t) =$ 0.991 < 1.20 **[SATISFACTORY]**

Where $P_s =$ 506.2 lbs $P_t =$ 557 lbs / bolt, (ICC ESR-2251)

$V_s =$ 84.5 lbs, $V_t =$ 1033.5 lbs / bolt, (ICC ESR-2251)

Notes:

1. Input W_2 & e_2 zero for only 2 level rack, or lump some levels together at their top for more than 3 levels.
2. Rack self should be designed by Manufacturers Institute Specifications (RMI), or from ICC report.
   a. Vertical impact load factor is 1.25 (RMI 2.4)
   b. Max beam deflection is 1/180 (RMI 2.5)
   c. Allowable soil capacity for slab on grade is 500 psf.

Purchaser's LOGO

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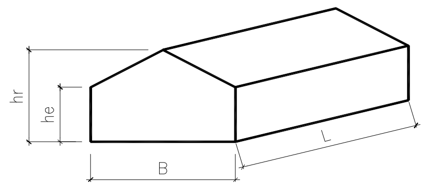
Wind Analysis for Tornado and Hurricane Based on 2012 IBC Section 423 & FEMA 361/320

INPUT DATA

Exposure category (FEMA input C only, ICC-500 C or B)		C	
Design wind speed (FEMA 361 Table 2-2)	V =	250	mph
Shelter height to eave	h_e =	8	ft
Shelter height to ridge (hr = he for flat roof)	h_r =	10	ft
Shelter length	L =	12	ft
Shelter width	B =	8	ft
Effective area of components	A =	8	ft^2



Strom Shelter

DESIGN SUMMARY

Max horizontal force normal to Shelter length, L, face	=	14.87 kips, SD level (LRFD level), Typ.
Max horizontal force normal to shelter length, B, face	=	8.37 kips
Max total horizontal torsional load	=	23.14 ft-kips
Max total upward force	=	13.21 kips
Max component pressure (see the last table for components and cladding)	=	428.40 psf

ANALYSIS

Importance factor (ASCE 7-10 Table 1.5-2)	I_w =	1.00	for all Category
Topographic factor (ASCE 7-10 26.8 & Table 26.8-1)	K_{zt} =	1.00	, (FEMA 361 Table 2-1)
Wind directionality factor (ASCE 7-10 Table 26.6-1)	K_d =	1.00	, (FEMA 361 Table 2-1)
Product of gust effect factor and internal pressure coefficient	$G\,C_{pi}$ =	0.55	or -0.55 , (FEMA 361 Table 2-1)

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2$ = **136.00** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 28.3-1, pg 299) = **0.85**

h = mean roof height = **9.00** ft

< 60 ft, [Satisfactory] (ASCE 7-10 26.2.1)

Design pressures for MWFRS

$p = q_h\,[(G\,C_{pf}) - (G\,C_{pi})]$

where: p = pressure in appropriate zone. (Eq. 28.4-1, page 298). p_{min} = **16** psf (ASCE 7-10 28.4.4)

$G\,C_{pf}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 28.4-1, page 300 & 301)

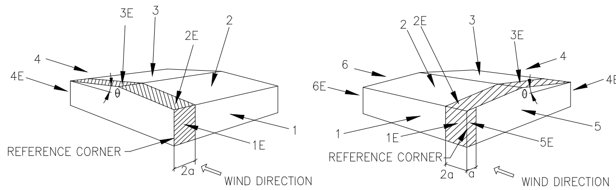
a = width of edge strips, Fig 28.4-1, note 9, page 301, MIN{MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3], 0.25B} = **2.00** **ft**

Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 26.57			Roof angle θ = 0.00		
	$G\,C_{pf}$	Net Pressure with		$G\,C_{pf}$	Net Pressure with	
		$(+GC_{pi})$	$(-GC_{pi})$		$(+GC_{pi})$	$(-GC_{pi})$
1	0.55	-0.04	149.56	-0.45	-136.00	13.60
2	-0.10	-88.28	61.32	-0.69	-168.64	-19.04
3	-0.45	-135.62	13.98	-0.37	-125.12	24.48
4	-0.39	-127.92	21.68	-0.45	-136.00	13.60
5				0.40	-20.40	129.20
6				-0.29	-114.24	35.36
1E	0.73	24.18	173.78	-0.48	-140.08	9.52
2E	-0.19	-100.68	48.92	-1.07	-220.32	-70.72
3E	-0.58	-154.35	-4.75	-0.53	-146.88	2.72
4E	-0.53	-147.55	2.05	-0.48	-140.08	9.52
5E				0.61	8.16	157.76
6E				-0.43	-133.28	16.32

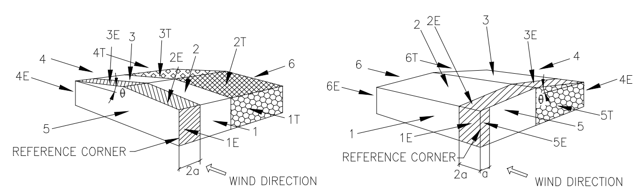
Net Pressures (psf), Torsional Load Cases

Surface	Roof angle θ = 26.57		
	$G\,C_{pf}$	Net Pressure with	
		$(+GC_{pi})$	$(-GC_{pi})$
1T	0.55	-0.01	37.39
2T	-0.10	-22.07	15.33
3T	-0.45	-33.90	3.50
4T	-0.39	-31.98	5.42
	Roof angle θ = 0.00		
Surface	$G\,C_{pf}$	Net Pressure with	
		$(+GC_{pi})$	$(-GC_{pi})$
5T	0.40	-5.10	32.30
6T	-0.29	-28.56	8.84



Load Case A (Transverse) Load Case B (Longitudinal)

Basic Load Cases



Load Case A (Transverse) Load Case B (Longitudinal)

Torsional Load Cases

Basic Load Case A (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$
1	64	0.00	9.57
2	36	-3.16	2.19
3	36	-4.85	0.50
4	64	-8.19	1.39
1E	32	0.77	5.56
2E	18	-1.80	0.88
3E	18	-2.76	-0.09
4E	32	-4.72	0.07
Σ	Horiz.	14.87	14.87
	Vert.	-11.25	3.12
Min. wind 28.4.4	Horiz.	1.92	1.92
	Vert.	-1.54	-1.54

Basic Load Case B (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$
2	36	-6.03	-0.68
3	36	-4.48	0.88
5	38	-0.78	4.91
6	38	-4.34	1.34
2E	18	-3.94	-1.27
3E	18	-2.63	0.05
5E	34	0.28	5.36
6E	34	-4.53	0.55
Σ	Horiz.	8.37	8.37
	Vert.	-13.21	5.47
Min. wind 28.4.4	Horiz.	1.15	1.15
	Vert.	-1.54	1.54

Torsional Load Case A (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$	Torsion (ft-k) $(+GC_{pi})$	Torsion (ft-k) $(-GC_{pi})$
1	16	0.00	2.39	0	5
2	9	-0.79	0.55	-1	0
3	9	-1.21	0.13	1	0
4	16	-2.05	0.35	4	-1
1E	32	0.77	5.56	3	22
2E	18	-1.80	0.88	-3	2
3E	18	-2.76	-0.09	5	0
4E	32	-4.72	0.07	19	0
1T	48	0.00	1.79	0	-5
2T	27	-0.59	0.41	1	-1
3T	27	-0.91	0.09	-1	0
4T	48	-1.54	0.26	-5	1
Total Horiz. Torsional Load, M_T				23	23

Torsional Load Case B (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$	Torsion (ft-k) $(+GC_{pi})$	Torsion (ft-k) $(-GC_{pi})$
2	36	-6.03	-0.68	-3	0
3	36	-4.48	0.88	2	0
5	2	-0.04	0.26	0	0
6	2	-0.23	0.07	0	0
2E	18	-3.94	-1.27	9	3
3E	18	-2.63	0.05	-6	0
5E	34	0.28	5.36	1	16
6E	34	-4.53	0.55	14	-2
5T	36	-0.18	1.16	0	-2
6T	36	-1.03	0.32	-2	1
Total Horiz. Torsional Load, M_T				15.1	15.1

<u>Design pressures for components and cladding</u>

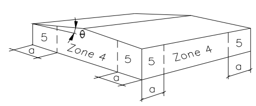
$p = q_h[(G C_p) - (G C_{pi})]$

where: p = pressure on component. (Eq. 30.4-1, pg 318)

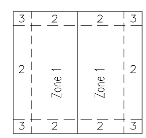
p_{min} = 16.00 psf (ASCE 7-10 30.2.2)

$G C_p$ = external pressure coefficient.

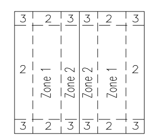
see table below. (ASCE 7-10 30.4.2)



Walls



Roof θ≤7°



Roof θ>7°

	Effective Area (ft^2)	Zone 1 GC_P	Zone 1 $-GC_P$	Zone 2 GC_P	Zone 2 $-GC_P$	Zone 3 GC_P	Zone 3 $-GC_P$	Zone 4 GC_P	Zone 4 $-GC_P$	Zone 5 GC_P	Zone 5 $-GC_P$
Comp.	8	0.50	-0.90	0.50	-1.90	0.50	-2.60	1.00	-1.10	1.00	-1.40

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 2 Positive	Zone 2 Negative	Zone 3 Positive	Zone 3 Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
	142.80	-197.20	142.80	-333.20	142.80	-428.40	210.80	-224.40	210.80	-265.20

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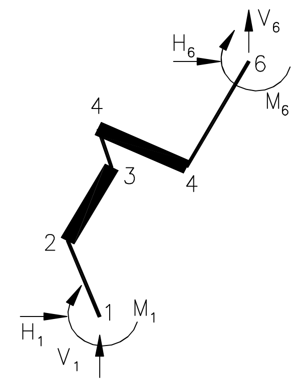
Stiffness Matrix Generator for Irregular Beam/Column

INPUT DATA & SUMMARY

	Coordinate	
Joint No.	X (in)	Y (in)
1	-5.00	0
2	-6.2125	9
3	-7.425	18
4	-8.6375	27
5	-9.85	36
6	-9.85	163.4626

Segment	Joint Number		E	A	I
Number	From	To	(ksi)	(in^2)	(in^4)
A	1	2	29000	38.443	626.1413
B	2	3	29000	40.40725	1046.096
C	3	4	29000	42.3715	1584.472
D	4	5	29000	44.33575	2247.047
E	5	6	29000	46.3	3060



POISSON'S RATIO ν = 0.3

SHEAR STRESS DISTRIBUTION FACTOR k = 1.2 ,(1.2 for Rectangle or 10/9 for Circle)

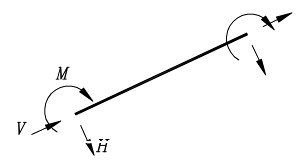
ANALYSIS

SINGLE SEGMENT STIFFNESS MATRIX

Segment Number A L = 9.1 in G = 11154 ksi

[k] =

	1	1	1	2	2	2	
1	339.6605711	-1332.480181	8647.877521	-339.6605711	1332.480181	8647.877521	V
1	-1332.480181	10050.72042	1165.061277	1332.480181	-10050.72042	1165.061277	H
1	8647.877521	1165.061277	642086.4218	-8647.877521	-1165.061277	308835.992	M
2	-339.6605711	1332.480181	-8647.877521	339.6605711	-1332.480181	-8647.877521	V
2	1332.480181	-10050.72042	-1165.061277	-1332.480181	10050.72042	-1165.061277	H
2	8647.877521	1165.061277	308835.992	-8647.877521	-1165.061277	642086.4218	M



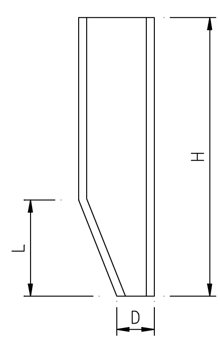
TYPICAL POSITIVE FORCES

ENTIRE ELEMENT (Irregular Beam/Column) STIFFNESS MATRIX

[K] =

	V_1	H_1	M_1	V_6	H_6	M_6
V_1	286.6279877	120.2446712	-2331.293139	-286.6279877	-120.2446712	-23301.24972
H_1	120.2446712	7494.362469	25074.47452	-120.2446712	-7494.362469	275.298004
M_1	-2331.293139	25074.47452	498818.1079	2331.293139	-25074.47452	-163980.868
V_6	-286.6279877	-120.2446712	2331.293139	286.6279877	120.2446712	23301.24972
H_6	-120.2446712	-7494.362469	-25074.47452	120.2446712	7494.362469	-275.298004
M_6	-23301.24972	275.298004	-163980.868	23301.24972	-275.298004	2296508.014

INPUT EXAMPLE



Tapered down Column

SECTION = W18X158 H = 13.62188 ft

L = 3 ft D = 10 in

	Coordinate	
Joint No.	X (in)	Y (in)
1	-5.00	0.00
2	-6.21	9.00
3	-7.43	18.00
4	-8.64	27.00
5	-9.85	36.00
6	-9.85	163.46

Segment	Joint Number		E	A	I
Number	From	To	(ksi)	(in^2)	(in^4)
A	1	2	29000	38.4	626
B	2	3	29000	40.4	1,046
C	3	4	29000	42.4	1,584
D	4	5	29000	44.3	2,247
E	5	6	29000	46.3	3,060

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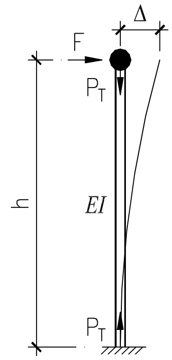
Lateral Drift Mitigation for Cantilever Column using Post-Tensioning

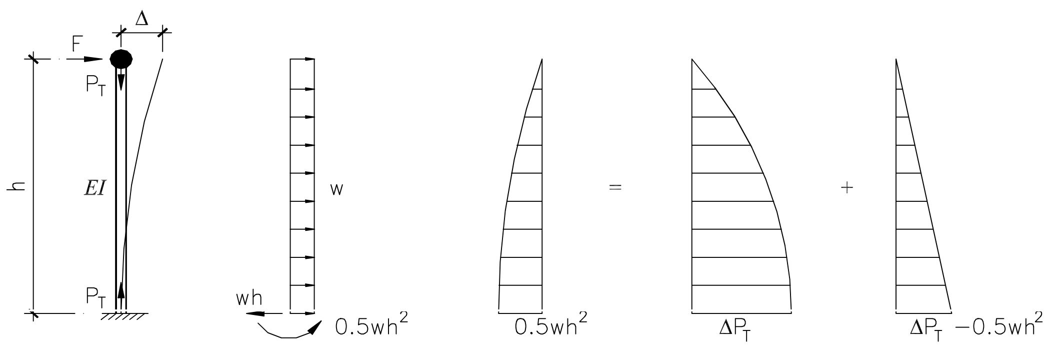
INPUT DATA & DESIGN SUMMARY

FLEXURAL STIFFNESS OF CANTILEVER COLUMN	*EI* =	**1782968**	in^2-kips
HEIGHT OF CANTILEVER COLUMN	h =	**12**	ft
AXIAL POST-TENSIONING FORCE	P_T =	**1638**	kips
LATERAL LOAD AT TOP OF COLUMN	F =	**2.1**	kips



The Lateral Drift at Top of Column **Δ =** 0.46 in
(The drift has been reduced 61% by post-tension)

ANALYSIS



Balanced Load Diagram *Balanced Moment* *Primary Moment* *Secondary Moment*

$$\begin{cases} \Delta = \dfrac{2Fh^3 + wh^4}{6EI + 2P_T h^2} \\ w = \dfrac{3\Delta P_T}{2h^2} \end{cases}$$

where Δ = 0.46 in
(1.17 in, without post-tensioning force.)
w = 0.05 kips / ft

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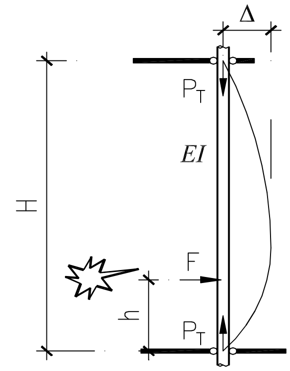
Blast Deformation Mitigation for Gravity Column using Post-Tensioning

INPUT DATA & DESIGN SUMMARY

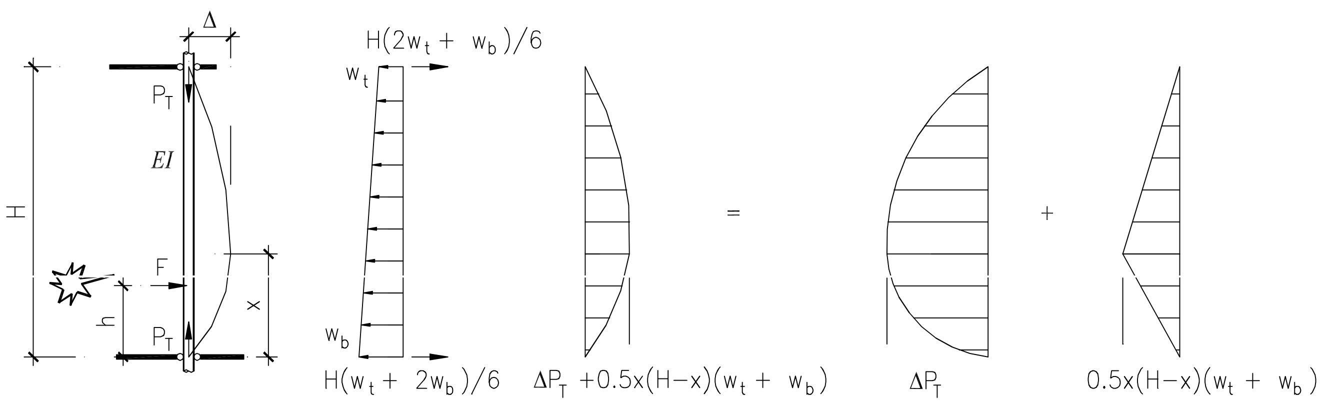
FLEXURAL STIFFNESS OF GRAVITY COLUMN	*EI* =	1782968	in^2-kips
STORY HEIGHT OF GRAVITY COLUMN	H =	10	ft
AXIAL POST-TENSIONING FORCE	P_T =	1638	kips
EQUIVALENT BLAST LOADING	F =	65	kips
BLAST LOADING LOCATION	h =	5	ft

The max deformation on Gravity Column **Δ =** 0.17 in

(The deformation has been reduced 87% by post-tension)



ANALYSIS



Balanced Load Diagram *Balanced Moment* *Primary Moment* *Secondary Moment*

$$\begin{cases} x = \sqrt{3(H+h)/3} \\ w_t + w_b = \dfrac{3\Delta P_T}{x(H-x)} \\ \Delta = \dfrac{0.06415F(H-h)}{HEI}\left(H^2-(H-h)^2\right)^{1.5} - \dfrac{H^2(H-h)^2}{6EI}(w_t+w_b) \end{cases}$$

where	Δ =	0.171	in
	(	1.312	in, at 5.00 ft from bottom, without post-tensioning force.)
	$w_t + w_b$ =	0.331	kips / ft
	x =	5.00	ft

Purchaser's LOGO

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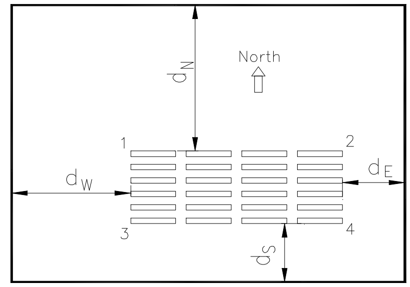
Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roof, Based on SEAOC PV2-2012

INPUT DATA & DESIGN SUMMARY

Exposure category (B, C or D, ASCE 7-10 26.7.3)		C	
Basic wind speed (ASCE 7-10 26.5.1 or 2012 IBC)	V =	110	mph
Topographic factor (ASCE 7-10 26.8 & Table 26.8-1)	K_{zt} =	1	Flat
Mean roof height or max monoslope roof height	h =	20	ft
Mean parapet height (PV2 Figure 29.9-1 Notes)	h_{pt} =	2	ft
Overall building length	W_L =	182	ft
Overall building width	W_S =	110	ft



Building Roof Plan

Array location

To North	d_N =	50	ft
To South	d_S =	10	ft
To West	d_W =	80	ft
To East	d_E =	30	ft

Effective wind area for structural element being designed

A = 3.13 ft^2

Solar Panel Net (toward & away) LRFD Pressure

North-West corner 1 =	***59.0***	***psf***
North-East corner 2 =	***81.2***	***psf***
South-West corner 3 =	***60.9***	***psf***
South-East corner 4 =	***72.0***	***psf***

Max possible pressures for any solar arrays on the roof

p_{max} = ***165.9 psf***

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2$ = **23.70** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 28.3-1, pg 299)	=	**0.90**	
K_d = wind directionality factor. (Tab. 26.6-1, for building, page 250)	=	**0.85**	
h = mean roof height	=	**20.00**	ft

< 60 ft, [Satisfactory] (ASCE 7-10 26.2.1)

< Min (L, B), [Satisfactory] (ASCE 7-10 26.2.2)

Design pressures for rooftop solar arrays

$p = q_h\,(G\,C_{rn})$

where: p = pressure on solar panels (PV2 Eq. 29.9-1).

p_{min} = 16.00 psf (ASCE 7-10 30.2.2)

$(G\,C_{rn}) = \gamma_p E\,[(G\,C_{rn})_{nom}\,(\gamma_c)]$, (PV2 Figure 29.9-1 Notes)

γ_p = 1.00 , (PV2 Figure 29.9-1 Notes)

E = arry edge factors, (PV2 Figure 29.9-1).

$(G\,C_{rn})_{nom}$ = nominal net pressure coefficient, (PV2 Figure 29.9-1).

γ_c = 0.90 , from ω and l_p .(PV2 Figure 29.9-1 Notes)

ω = **10.00** deg, angle of plane of panel to roof. (PV2 Figure 29.9-1)

l_p = **5.00** , chord length of solar panel. (PV2 Figure 29.9-1)

$a_{pv} = \mathit{Min}\,[\,h\,,\,0.5\,(h\,\mathit{Max}(W_L\,,\,W_S))^{0.5}\,]$ = 20.00 ft

$A_n = 1000\,A\,/\,[\max(a_{pv}\,,\,15\text{ ft})]^2$ = 7.83 ft^2 , (PV2 Figure 29.9-1 Notes)

h_1 = **0.50** ft, solar panel height above roof at low edge.

$h_c = \mathit{Min}\,[\,\mathit{Min}(\,h_1\,,\,1\text{ ft}\,) + l_p\,\sin\omega\,,\,0.1\,a_{pv}\,]$ = 1.37 ft, conservatively for multi-solar arrays.

Corner	No. Zone	0^o - 5^o	$(15^o$ - $35^o)\ \gamma_c$	$(G\,C_{rn})_{nom}\,\gamma_c$	E	p (psf)
1	1	1.12	1.37	1.25	2.00	**59.01**
2	2	1.49	1.94	1.71	2.00	**81.20**
3	2	1.49	1.94	1.71	1.50	**60.90**
4	3	1.70	2.35	2.02	1.50	**71.97**

Maximum possible pressures for any solar arrays on this roof

$p = q_h\,(G\,C_{rn})$ = **165.88** psf

where: γ_p = 1.00

E = 2.00

γ_c = 1.00

$(G\,C_{rn})_{nom}$ = 3.50

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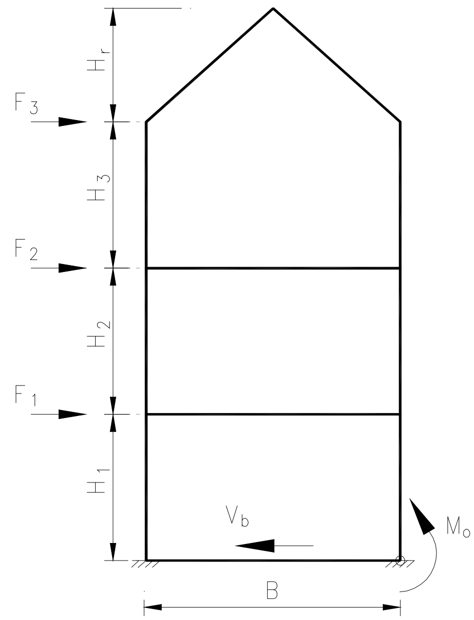
Three Story Comparison of Seismic and Wind Based on 2012 IBC / 2013 CBC

INPUT DATA & DESIGN SUMMARY



L =	150	*ft, Building Length (perpendicular, not shown)*			
B =	50	*ft, Building Width (as shown)*			
H_r =	4.5	*ft*			
H_3 =	8	*ft*	W_3 =	225	*kips, (12.8-12)*
H_2 =	8	*ft*	W_2 =	337.5	*kips*
H_1 =	9	*ft*	W_1 =	375	*kips*

Seismic

S_{DS} =	0.96	(ASCE 7 Sec 11.4.4)
S_{D1} =	0.55	(ASCE 7 Sec 11.4.4)
S_1 =	0.54	(ASCE 7 Sec 11.4.1)

Wind Exposure (B, C or D)		C	*, (ASCE 7-10 26.7.3)*
Wind Speed	V =	145	*mph, (ASCE 7-10 26.5.1)*

	Base Shear (kips)	Lateral Force (kips)			Overturning (ft-kips)
	V_b	***F_1***	***F_2***	***F_3***	***M_o***
***Seismic**, ASD*	**98.9**	22.6	**38.5**	**37.7**	1802.1
	(0.11W)	(0.06 Wx)	(0.11 Wx)	(0.17 Wx)	
***Wind**, ASD*	88.9	**25.6**	24.1	25.6	**5699.3**

Determine Seismic Factors

R =	6.5	(ASCE 7 Tab 12.2-1)
I =	1	(2012 IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)
C_t =	0.02	(ASCE 7 Tab 12.8-2)
h_n =	25.0	ft
x =	0.75	(ASCE 7 Tab 12.8-2)

$T = C_t (h_n)^x =$ 0.224 sec, (ASCE 7 Sec 12.8.2.1)

Determine Wind Factors

K_{zt} =	1	(ASCE 7-10 26.8 & Table 26.8-1)		
K_d =	0.85	(ASCE 7-10 Tab. 26.6-1, for building, page 250)		
$G\,C_{p\,i}$ =	0.18	or	-0.18	(ASCE 7-10 Tab. 26.11-1, Enclosed Building, page 258)
p_{min} =	16	psf (ASCE 7-10 28.4.4)		
a =	5.00	ft (ASCE 7-10 Fig 28.4-1, note 9, page 301)		
K_h =	0.96	(ASCE 7-10 Tab. 28.3-1, pg 299)		
q_h =	43.83	psf (ASCE 7-10 Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)		

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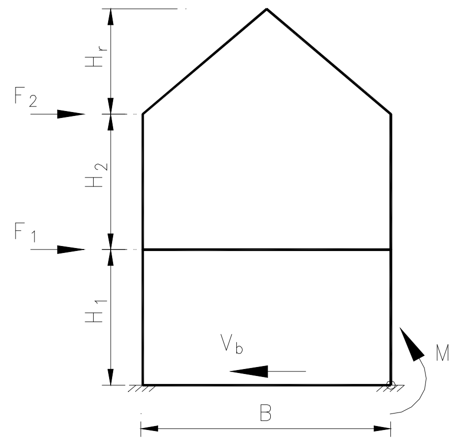
Two Story Comparison of Seismic and Wind Based on 2012 IBC / 2013 CBC

INPUT DATA & DESIGN SUMMARY



$L =$ 150 *ft, Building Length (perpendicular, not shown)*

$B =$ 50 *ft, Building Width (as shown)*

$H_r =$ 4.5 *ft*

$H_2 =$ 8 *ft* $W_2 =$ 337.5 *kips, (12.8-12)*

$H_1 =$ 9 *ft* $W_1 =$ 375 *kips*

Seismic $S_{DS} =$ 0.96 (ASCE 7 Sec 11.4.4)

$S_{D1} =$ 0.55 (ASCE 7 Sec 11.4.4)

$S_1 =$ 0.54 (ASCE 7 Sec 11.4.1)

Wind Exposure (B, C or D) C *, (ASCE 7-10 26.7.3)*

Wind Speed $V =$ 145 *mph, (ASCE 7-10 26.5.1)*

	Base Shear (kips)	Lateral Force (kips)		Overturning (ft-kips)
	V_b	F_1	F_2	M_o
***Seismic**, ASD*	**75.2**	**27.8**	**47.3**	1055.1
	(0.11W)	(0.07 Wx)	(0.14 Wx)	
***Wind**, ASD*	54.4	21.5	21.5	**4673.8**

Determine Seismic Factors

$R =$ 6.5 (ASCE 7 Tab 12.2-1)

$I =$ 1 (2012 IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)

$C_t =$ 0.02 (ASCE 7 Tab 12.8-2)

$h_n =$ 17.0 ft

$x =$ 0.75 (ASCE 7 Tab 12.8-2)

$T = C_t (h_n)^x =$ 0.167 sec, (ASCE 7 Sec 12.8.2.1)

Determine Wind Factors

$K_{zt} =$ 1 (ASCE 7-10 26.8 & Table 26.8-1)

$K_d =$ 0.85 (ASCE 7-10 Tab. 26.6-1, for building, page 250)

$G\,C_{p\,i} =$ 0.18 or -0.18 (ASCE 7-10 Tab. 26.11-1, Enclosed Building, page 258)

$p_{min} =$ 16 psf (ASCE 7-10 28.4.4)

$a =$ 5.00 ft (ASCE 7-10 Fig 28.4-1, note 9, page 301)

$K_h =$ 0.89 (ASCE 7-10 Tab. 28.3-1, pg 299)

$q_h =$ 40.83 psf (ASCE 7-10 Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

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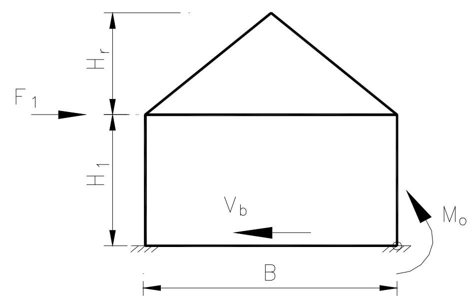
One Story Comparison of Seismic and Wind Based on 2012 IBC / 2013 CBC

INPUT DATA & DESIGN SUMMARY

L = 150 *ft, Building Length (perpendicular, not shown)*

B = 50 *ft, Building Width (as shown)*



H_r = 4.5 *ft* W_1 = 375 *kips, (12.8-12)*

H_1 = 9 *ft* (half perpendicular wall)

Seismic S_{DS} = 0.96 (ASCE 7 Sec 11.4.4)

S_{D1} = 0.55 (ASCE 7 Sec 11.4.4)

S_1 = 0.54 (ASCE 7 Sec 11.4.1)

Wind Exposure (B, C or D) C *, (ASCE 7-10 26.7.3)*

Wind Speed V = 145 *mph, (ASCE 7-10 26.5.1)*

	Base Shear (kips)	Lateral Force (kips)	Overturning (ft-kips)
	V_b	***F_1***	***M_o***
***Seismic**, ASD*	**39.6**	**39.6**	356.0
	(0.11W)	(0.11 Wx)	
***Wind**, ASD*	24.8	8.3	**4051.4**

Determine Seismic Factors

R = 6.5 (ASCE 7 Tab 12.2-1)

I = 1 (2012 IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)

C_t = 0.02 (ASCE 7 Tab 12.8-2)

h_n = 9.0 ft

x = 0.75 (ASCE 7 Tab 12.8-2)

$T = C_t (h_n)^x =$ 0.104 sec, (ASCE 7 Sec 12.8.2.1)

Determine Wind Factors

K_{zt} = 1 (ASCE 7-10 26.8 & Table 26.8-1)

K_d = 0.85 (ASCE 7-10 Tab. 26.6-1, for building, page 250)

$G C_{p\,i}$ = 0.18 or -0.18 (ASCE 7-10 Tab. 26.11-1, Enclosed Building, page 258)

p_{min} = 16 psf (ASCE 7-10 28.4.4)

a = 4.50 ft (ASCE 7-10 Fig 28.4-1, note 9, page 301)

K_h = 0.85 (ASCE 7-10 Tab. 28.3-1, pg 299)

q_h = 38.89 psf (ASCE 7-10 Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

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Self-Centering Lateral Frame Design Based on ASCE 7-10, AISC 360-10 & ACI 318-11

DESIGN CRITERIA

Self-Centering Lateral Frame (SCLF) can increase ductility capacity, reduce residual drift after earthquake, and cost less for both new and existing structures. But since post-tensioning tendon have to be used, without acceptable/accurate stiffness matrix, SCLF only can apply to flexible diaphragm structures, which the seismic loads NOT from FEM analysis.

INPUT DATA & DESIGN SUMMARY

TRIANGLE SECTION (Tube or Pipe)

=> **HSS8X8X5/8**

Tube	A	r_{min}	t	b	h
	16.40	2.98	0.63	8.00	8.00

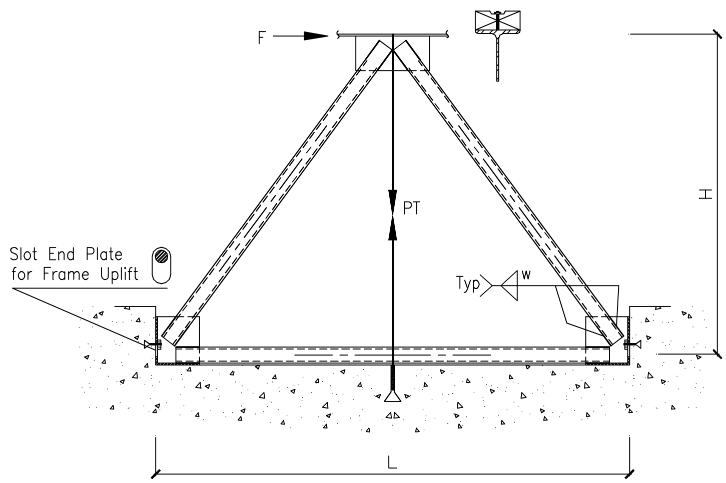
F_y = **46** ksi

DIMENSION H = **12** ft
L = **18** ft

SEISMIC LOAD (SD level)
F = **200** kips

POST-TENSIONING TENDONS
1 x **12** strands (each **0.153** in^2 area)

f_{se} = **175** ksi, (ACI 318-11 18.6)
(180 ksi suggested for 270 ksi f_{pu} / 243 ksi f_{py})



END/GUSSET PLATE AND FILLET WELD
1 in. gusset, **0.5** in. weld, **10** in. leg long, typical.

THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

CHECK SELF-CENTERING/OVERTURNING (assuming PT as Dead Load)

PT = 321.3 kips **>** (1 / 0.9) (2 F H / L) = 296.30 kips **[Satisfactory]**

Where, the factor, 0.9, is from Load Combination, which controls structural overturning design, because Load Combinations are probability. If an equation value of Load Combinations is negative, that means the structure is inadequate (probability result out-of [0, 1]).

DETERMINE TRIANGLE MEMBER AXIAL FORCES

	Bottom		Left/Tension		Right/Compression	
- T / P =	220.49	kips	34.15	kips	367.48	kips

CHECK TRIANGLE MEMBER CAPACITY (AISC 360-10 E3)

$\phi_c P_n = \phi_c A_g F_{cr}$ = 477.24 kips **>** P_{uc} = 367.48 kips **[Satisfactory]**

Where ϕ_c = 0.9

KL/r = 72.39 **<** 200 **[Satisfactory]**

F_e = $\pi^2 E / (KL / r)^2$ = 54.61 ksi

F_{cr} = 32.33 ksi

CHECK FILLET WELD SIZE (AISC 360-10 J2.2b)

w = 0.5 in **>** w_{MIN} = 0.25 in

< w_{MAX} = (ϕ 0.6 F_u t) / (ϕ 0.707 F_{EXX}) = (0.75 x 0.6 x 58 ksi) t / (0.75 x 0.707 x 70 ksi)

[Satisfactory] = 1.1795 t = 0.74 in

L_w = (1 / ϕ) P_{uc} / [(4) (0.6) F_{EXX} (0.707 w)] = 8.25 in **<** 10 in **[Satisfactory]**

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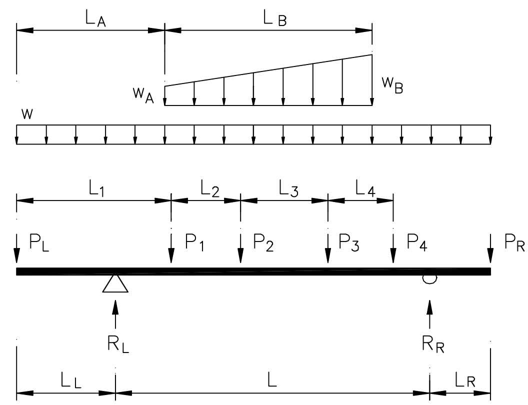
General Beam Analysis

INPUT DATA & DESIGN SUMMARY

LENGTH	L_L =	10	ft			
	L =	30	ft			
	L_R =	8	ft			
LOAD & LOCATION	P_L =	5	kips			
	P_1 =	3.5	kips	L_1 =	11	ft
	P_2 =	4	kips	L_2 =	3	ft
	P_3 =	3	kips	L_3 =	3.5	ft
	P_4 =	5	kips	L_4 =	4	ft
	P_R =	6	kips			
	w =	380	lbs / ft			
	w_A =	250	lbs / ft	L_A =	9	ft
	w_B =	600	lbs / ft	L_B =	36	ft



REACTION R_L = **32.348** *kips*

R_R = **27.692** *kips*

SECTION FORCE M_{max} = **84.080** *ft-kips, from left 22.60 ft.*

V_{max} = **23.165** *kips, from left 10.00 ft.*

LATERAL-TORSIONAL BUCKLING LENGTH OF ***TOP TENSION*** BENDING

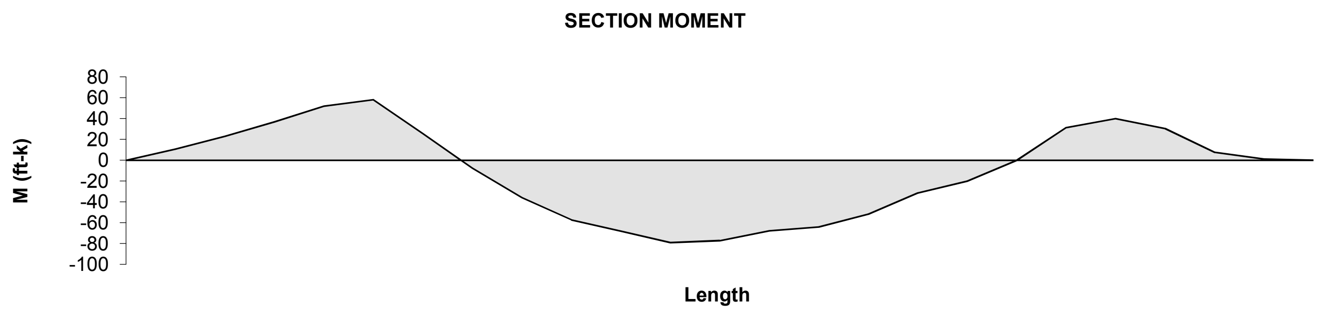
L_b = **14** ft

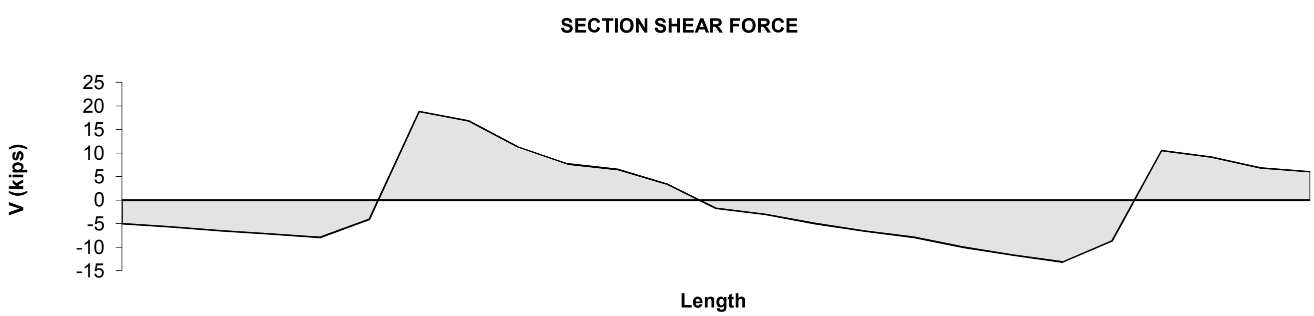
1. *If no diaphragm at top, L_b should be the bottom tension bending length (L_L + L + L_R).*
2. *Wood beam EFFECTIVE LENGTH, L_e, may conservatively be 1.87 L_b (NDS 2012 Table 3.3.3).*

DEFLECTION Δ_{Left} = **-0.618** *in* Δ_{Middle} = **0.78607** *in* Δ_{Right} = **-0.524** *in*

Where *E* = 29000 *ksi* *I* = 301 *in*^{4}

ANALYSIS





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Wind Analysis for Trussed Tower Based on ASCE 7-2010

INPUT DATA

Exposure category (B, C or D)		=	C	
Importance factor, 1.0 only, (Table 1.5-2)	I_w	=	1.00	
Basic wind speed (ASCE 7-10 26.5.1)	V	=	90	mph
Topographic factor (26.8 & Table 26.8-1)	K_{zt}	=	1	Flat
Projected area normal to the wind	A_f	=	500	ft^2
Height of the centroid of area A_f	h	=	420	ft
Ratio of solid area to gross area (ASCE 7-10 Figure 29.5-3)	Є	=	0.35	

DESIGN SUMMARY

Max horizontal wind pressure	p	=	83	psf
Max total horizontal force at centroid of base	F	=	41.56	kips
Max bending moment at centroid of base	M	=	17456.57	ft-kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h K_{zt} K_d V^2$ = **33.61** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 29.3-1 page 307 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 29.3-1, pg 310)	=	**1.71**	
K_d = wind directionality factor. (Tab. 26.6-1, conservative value, page 250)	=	**0.95**	
h = height of the centroid of area	=	**420.00**	ft

$p = q_h\, G\, C_f$ =	=	**83**	psf
$F = p\, A_f$	=	**41.56**	kips
$M = F\, h$	=	**17456.6**	ft-kips

where:

G = gust effect factor. (Sec. 26.9)	=	**0.85**	
C_f = 1.2 Max (4 $Є^2$ - 5.9 Є + 4 , 3.4 $Є^2$ - 4.7 Є + 3.4) , (conservative value, Figure 29.5-3)	=	**2.91**	
A_f =	=	**500.0**	ft^2

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Post-Tensioned Lateral Frame Analysis using Finite Element Method

DESIGN CRITERIA

The post-tensioned lateral frame (PTLF) is self-centering structure, with good hysteretic performance, which can apply to both new and existing structures of concrete, wood, or steel. The PT tendon forces are not constant, because one direction tendons are released/changed at lateral loads.

There are three kind of forces on each section of beam and column.

1. Primary equivalent loads, PT_1 & PT_2 section forces. The tendon is mentally removed and replaced with all of the loads it exerts on the structure.
2. Secondary section forces from all reactions of primary PT_1 & PT_2 , on free-body structure.
3. Balanced section forces with external loads, F_1 & F_2 . The external loads can be ASD level for serviceability design, or SD level for strength design.

INPUT DATA

DIMENSION

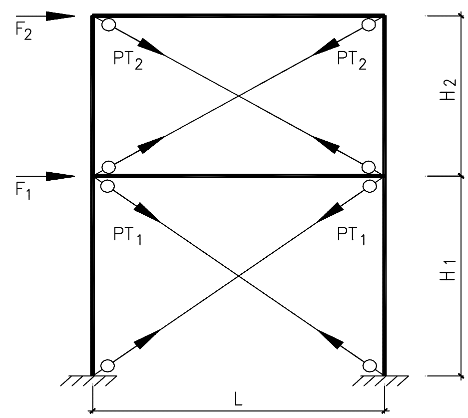
L = 22 ft

H_1 = 16 ft

H_2 = 18 ft

MODULUS OF ELASTICITY

E = 29000 ksi



Member	A (in^2)	I_x (in^4, in plane)
Column	19.1	533
Bot Beam	19.1	533
Top Beam	19.1	533

LATERAL LOADS

F_1 = 18 kips

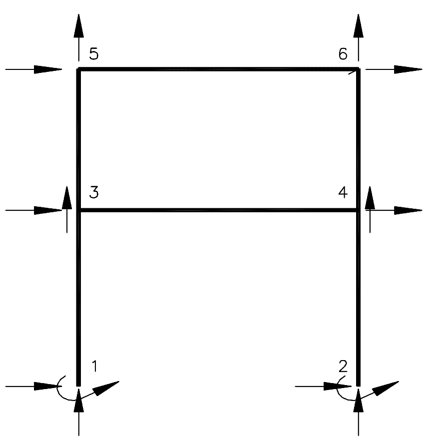
F_2 = 16 kips

TENDON FORCE AFTER ALLOWANCE LOSSES

PT_1 = 30 kips, at the first level

PT_2 = 30 kips, at the second level

ANALYSIS & DESIGN SUMMARY



Joint Equivalent Load and Deformation

Joint	***PT_1 & PT_2***		Balanced (***PT_1 , PT_2***) & (***F_1 , F_2***)			
	F_x (kips)	F_y (kips)	F_x (kips)	F_y (kips)	Δ_x (in)	Δ_y (in)
1	24.26	17.65	24.26	17.65	0	0
2	-24.26	17.65	0.00	0.00	0	0
3	47.48	1.35	41.22	19.00	0.40	0.01
4	-47.48	1.35	-27.93	-14.65	0.39	-0.01
5	23.22	-19.00	23.22	-3.00	0.58	0.01
6	-23.22	-19.00	-23.22	-19.00	0.57	-0.02

Reaction

Joint	***PT_1 & PT_2***			Balanced (***PT_1 , PT_2***) & (***F_1 , F_2***)		
	R_x (kips)	R_y (kips)	M (ft-k)	R_x (kips)	R_y (kips)	M (ft-k)
1	-0.21	-17.65	-1.90	-6.79	19.99	-63.78
2	0.21	-17.65	1.90	-6.50	-37.64	-61.08

Section Forces

Section	Primary equivalent loads, only ***PT_1 & PT_2***			Secondary section forces from primary reactions		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)	P (kips, axial)	M (ft-kips)	V (kips, in plane)
1 of 1-3	17.65	1.90	0.21	0.00	-1.90	-0.21
3 of 1-3	-17.65	1.49	-0.21	0.00	-1.49	0.21
2 of 2-4	17.65	-1.90	-0.21	0.00	1.90	0.21
4 of 2-4	-17.65	-1.49	0.21	0.00	1.49	-0.21
3 of 3-4	47.18	-0.30	0.00	0.28	0.50	0.00
4 of 3-4	-47.18	0.30	0.00	-0.28	-0.50	0.00
3 of 3-5	19.00	-1.19	-0.09	0.00	0.99	0.06
5 of 3-5	-19.00	-0.37	0.09	0.00	0.17	-0.06
4 of 4-6	19.00	1.19	0.09	0.00	-0.99	-0.06
6 of 4-6	-19.00	0.37	-0.09	0.00	-0.17	0.06
5 of 5-6	23.31	0.37	0.00	-0.06	-0.17	0.00
6 of 5-6	-23.31	-0.37	0.00	0.06	0.17	0.00

(Cont'd)

Section Forces

Section	Balanced (***PT*** $_1$ ***, PT*** $_2$) and (***F*** $_1$ ***, F*** $_2$)		
	P (kips, axial)	M (ft-kips)	V (kips, in plane)
1 of 1-3	-19.99	63.78	6.79
3 of 1-3	19.99	44.93	-6.79
2 of 2-4	37.64	61.08	6.50
4 of 2-4	-37.64	42.91	-6.50
3 of 3-4	34.37	-37.85	-3.42
4 of 3-4	-34.37	-37.36	3.42
3 of 3-5	2.42	-7.08	-0.05
5 of 3-5	-2.42	6.11	0.05
4 of 4-6	19.57	-5.55	0.05
6 of 4-6	-19.57	6.53	-0.05
5 of 5-6	23.27	-6.11	-0.57
6 of 5-6	-23.27	-6.53	0.57

Member Design Forces *(kips, ft-kips)*

Member		***Column***	***Bot Beam***	***Top Beam***
Primary	P	17.65	47.18	23.31
	M	1.90	0.30	0.37
	V	0.21	0.00	0.00
Secondary	P	0.00	0.28	-0.06
	M	1.90	0.50	0.17
	V	0.21	0.00	0.00
Balanced	P	37.64	34.37	23.27
	M	63.78	37.85	6.53
	V	6.79	3.42	0.57

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Beam Strengthening Analysis Using External Post-Tensioning Systems

INPUT DATA & DESIGN SUMMARY

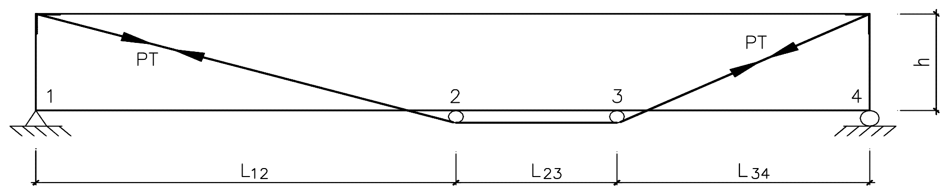
DIMENSION

L_{12} = 20 ft

L_{23} = 8 ft

L_{34} = 10 ft

h = 36 in



BEAM MODULUS OF ELASTICITY E = 3122.019 ksi

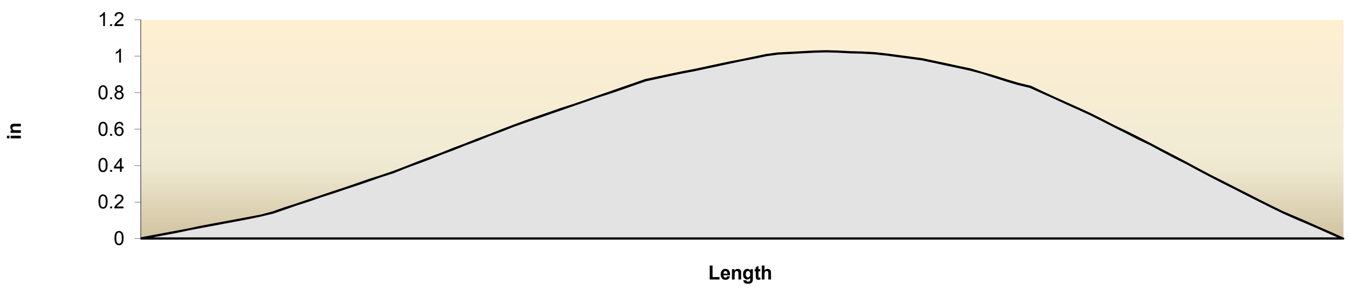
BEAM SECTION A = 720 in^2 I_x = 77760 in^4, in plane

TENDON FORCE AFTER ALLOWANCE LOSSES PT = 972 kips

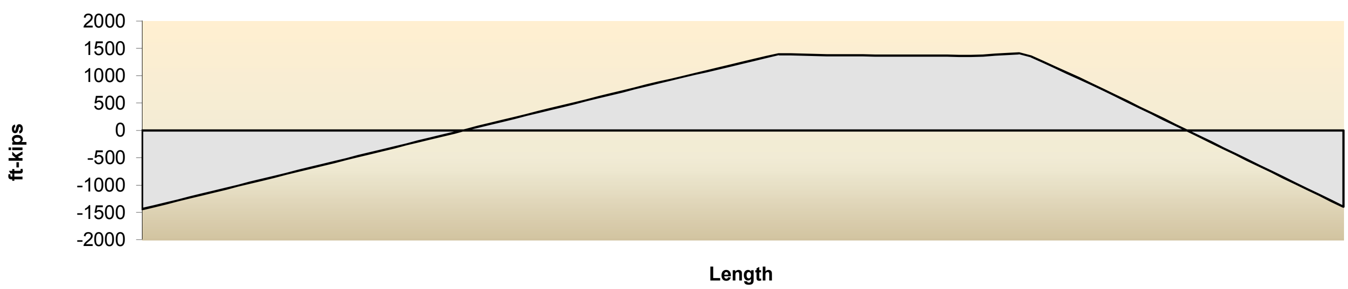
Δ_{max} =	1.03	*in @*	21.60	*ft, from left*	
M_{max} =	1420.38	*ft-kips @*	28.00	*ft, from left*	
V_{max} =	279.30	*kips, section shear force*			
P_{max} =	972.00	*kips, section axial compression*			

ANALYSIS

Camber Deflection



Section Moment



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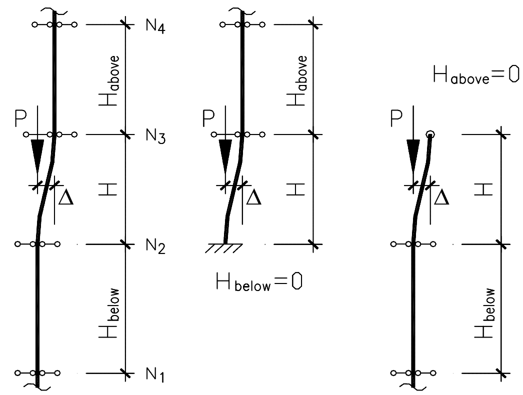
Lateral Drift Compatibility Analysis using Finite Element Method

INPUT DATA & DESIGN SUMMARY



STORY HEIGHTS H_{above} = **14** ft

H = **14** ft, continued top & bot

H_{below} = **16** ft

AXIAL LOAD, ASD P_{DL} = **20** kips

P_{LL} = **18** kips

EXPECTED STORY DRIFT Δ = **4.2** in
(ASCE 7-10 12.12.4)

GRAVITY VERTICAL MEMBER INFORMATION

A = **17.6** in^2 E = **29000** ksi

I = **341** in^4 G = **11154** ksi

ANALYSIS
DETERMINE JOINT DEFLECTIONS AND SECTION DESIGN FORCES

JOINT DEFLECTIONS UNDER Δ ONLY

Note	**X** (in)	**Y** (in)	**θ** (deg)
N_1	0	0	0.496192
N_2	0	0.0000	-1.007175
N_3	4.2	0.0000	-0.932621
N_4	4.2	0.0000	0.457379

THE DESIGN MEMBER SECTION FORCES UNDER Δ ONLY

Note	**V** (k)	**P** (k)	**M** (ft-k)
N_2	-33.09	0.000	225.24
N_3	33.09	0.000	238.00

P_u = $1.2\,P_{DL} + 1.0\,P_{LL}$ = 42.00 kips, *SD level*

M_u = $(P_{DL} + P_{LL})\,\Delta + M$ = 251.30 ft-kips, *SD level*

V_u = V = 33.09 kips, *SD level*

Note: The governing case is always that only the design vertical member has Story Drift and other stories no drift, since the other story at same direction drifts, including PD effects, will reduce the section forces of design vertical member. The opposite story drifts are not Expected Deformation, because Expected Deformation can not be developed from Linear Modes Analysis ***after*** CQC or SRSS.

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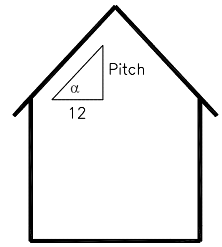
Seismic Analysis for Sloped Flexible Diaphragm

DESIGN CRITERIA

1. The most sharp sloped roof are flexible diaphragm, based on ASCE 7-10 12.3.1, no matter if concrete roof, or steel, wood roof.
2. Flexible diaphragm design can be done by horizontal projected area under equivalent tributary load. But for sharp sloped roof, the diaphragm shear stress in-plane has to include both seismic force and vertical load.
3. If using diaphragm design force as (F_{px} / Cos α + w_{px} Sin^2 α), the all analysis results (panel shear stress in-plane, chord force, & drag/collector axial force), by horizontal projected area under equivalent tributary load, are conservative.



INPUT DATA & ANALYSIS SUMMARY

ROOF PITCH 16 /12 , (α = 53.1 deg.)

DIAPHRAGM DESIGN FORCE FACTOR F_{px} = 0.24 w_{px} , (ASCE 7-10 12.10.1.1)

DIAPHRAGM EDGE CORNER POINT

Corner Point	X (ft)	Y (ft)
1	40	62
2	40	-2
3	82	-2
4	-2	62
5	-2	-2
6	122	-2
7	122	42
8	82	42
9	82	62
10	60	20
11	122	20

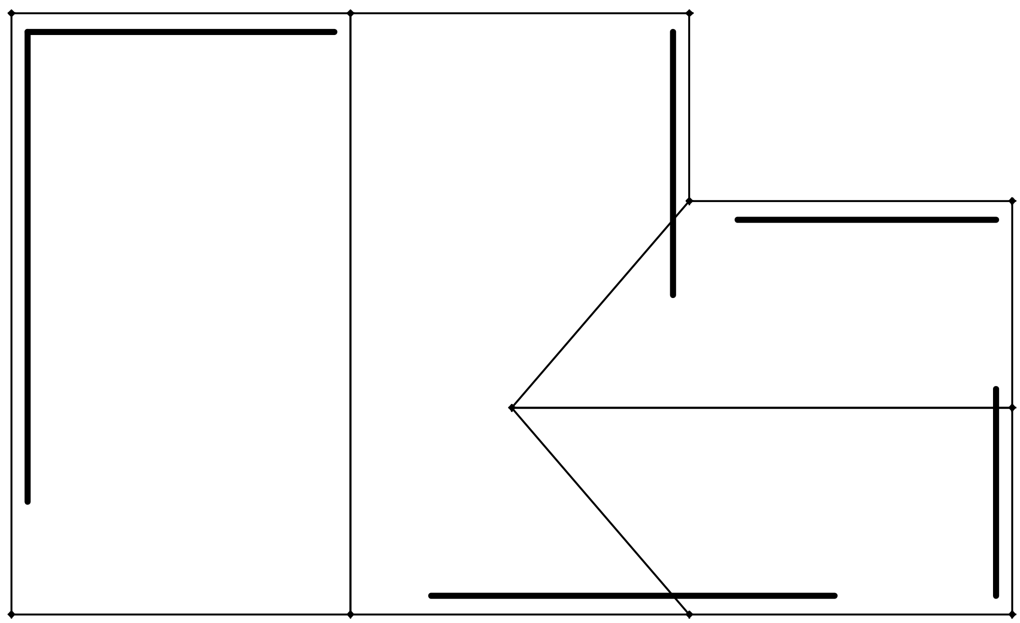
DIAPHRAGM PANEL & SEISMIC DEAD LOAD

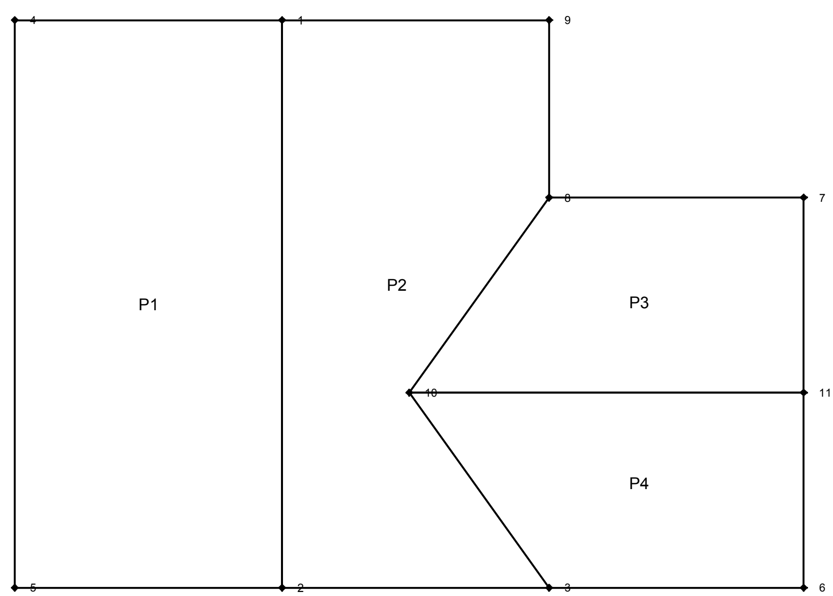
Panel No.	Corner 1	Corner 2	Corner 3	Corner 4	Corner 5	Corner 6	Corner 7	Corner 8	Corner 9	W_{px} (psf)
1	1	2	5	4						25
2	1	9	8	10	3	2				25
3	8	7	11	10						20
4	3	10	11	6						20

SHEAR WALL (LATERAL FRAME) LOCATION

SW NO.	Drag to Start (ft)	Start Point X (ft)	Start Point Y (ft)	End Point X (ft)	End Point Y (ft)	Drag fr End (ft)	Σ Drag Length	Wall Length
1	12	0	10	0	60	2	64.0	50.0
2	2	0	60	38	60	44	84.0	38.0
3	52	50	0	100	0	22	124.0	50.0
4	34	80	32	80	60	2	64.0	28.0
5	8	88	40	120	40	2	42.0	32.0
6	2	120	0	120	22	20	44.0	22.0

Tip: Input the START point of each wall at left/bottom & END at right/top, so easy to follow drag force locations. (cont'd)





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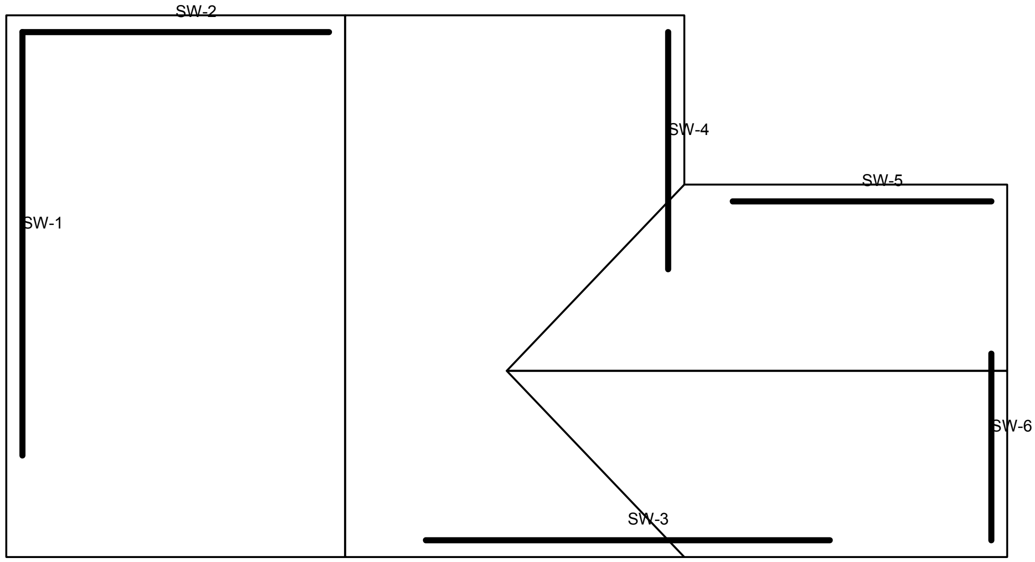

DIAPHRAGM PANEL LOAD AND CHORD FORCE

Panel No.	Area (ft^2)	Centroid X (ft)	Centroid Y (ft)	Updated F_{px} (kips)	Chord Force L (ft)	Chord Force T / C (k)
1	2688	19.0	30.0	37.2	89.8	14.0
2	2204	58.0	32.2	30.5	81.3	11.4
3	1122	96.1	30.2	12.4	58.0	4.7
4	1122	96.1	9.8	12.4	58.0	4.7
Σ	**7136**			**92.6**		

Note:
The Chord Force, T / C (kips), is from the equation,
T and/or C = Moment / Depth = (F_{px} /*Area*) L^2 / 8 ,
where L should be actual maximum supported span
and not larger than $(3\ A)^{0.5}$ since diaphragm ratio 3 max.

(cont'd)

TRIBUTARY DIAPHRAGM AREA TO **HORIZONTAL (X DIRECTION)** SHEAR WALL

Panel	Shear Wall			Shear Wall			Shear Wall			Shear Wall			Σ
No.	Number	Top	Bottom	Number	Top	Bottom	Number	Top	Bottom	Number	Top	Bottom	%
1	2		51%	3	51%								102%
2	2		51%	3	51%								102%
3	5		100%	3	5%								105%
4	3	100%		5	5%								105%

TRIBUTARY DIAPHRAGM AREA TO **VERTICAL (Y DIRECTION)** SHEAR WALL

Panel	Shear Wall			Shear Wall			Shear Wall			Shear Wall			Σ
No.	Number	Left	Right	Number	Left	Right	Number	Left	Right	Number	Left	Right	%
1	1		100%	4	5%								105%
2	4	100%		1	5%								105%
3	6	98%		4	3%	4%							105%
4	6	98%		4	3%	4%							105%

(cont'd)

DIAPHRAGM SHEAR STRESS AT EACH SIDE OF SHEAR WALL & DRAG FORCE AT TWO ENDS OF SHEAR WALL

Shear	Diaphragm								Shear Wall	
Wall	**Shear** (k)		**Length** (ft)		**Stress** (plf)		**Drag Force** (k)		**Force**	**Stress**
NO.	**L / T**	**R / B**	**L / T**	**R / B**	**L / T**	**R / B**	**Start**	**End**	(k)	(plf)
1	1.5	37.2	64.0	64.0	24	581	7.3	1.2	38.7	775
2	0.0	34.5	84.0	84.0	0	411	0.8	18.1	34.5	909
3	47.6	0.0	124.0	124.0	384	0	20.0	8.4	47.6	951
4	33.1	1.0	64.0	64.0	517	16	18.1	1.1	34.1	1218
5	0.6	12.4	42.0	42.0	15	296	2.5	0.6	13.0	408
6	24.3	0.0	44.0	44.0	553	0	1.1	11.1	24.3	1107
Max.					**553**	**581**	**20.0**	**18.1**	**47.6**	**1218**

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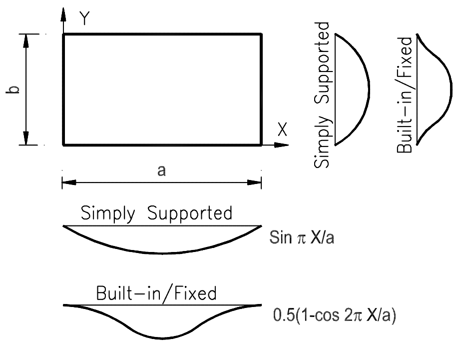
Two-Way Floor Vibration Design Based on The Structural Engineer, Vol. 94-1, 2016

DESIGN CRITERIA

1. Floor vibration can be felt strongly by human occupants of structures, if the maximum dynamic deflection of single walker with average weight more than (Diagonal Span / 35000).
2. Typically, structures with fundamental natural frequencies in a range of 1.5 to 4 Hz would be at risk. The first resonant frequencies of composite floor between 6.5 and 7.5 Hz fall in the frequency range where human occupants are most sensitive in perceiving vibration.

INPUT DATA & DESIGN SUMMARY

THICKNESS OF FLOOR	t	=	8	in
DIMENSION	a	=	30	ft
	b	=	24	ft
MODULUS OF ELASTICITY	E	=	4030.51	ksi
POISSON'S RATIO	ν	=	0.2	
DAMPING RATIO	ξ	=	0.05	, (ASCE 7-10 16.1.3 & 21.1.3)
FLOOR UNIT WEIGHT	w	=	150	lbs/ft^3
EDGE CONNECTION (1 or 2)	1	<== Simply Supported		



THE FLOOR DESIGN IS ADEQUATE.

ANALYSIS

$D = \frac{E t^3}{12\left(1-\nu^2\right)} =$ 179134 , *flexural rigidity of the uniform floor*

For Simply Supported

$m^* = \frac{wabt}{4g} =$ 0.05 , *generalized mass*

$k^* = D\pi^4\left(\frac{1}{2ab}+\frac{b}{4a^3}+\frac{a}{4b^3}\right) =$ 177 , *equivalent stiffness*

For Built-in/Fixed

$m^* = \frac{9wabt}{64g} =$ 0.03 , *generalized mass*

$k^* = D\pi^4\left(\frac{1}{2ab}+\frac{3b}{4a^3}+\frac{3a}{4b^3}\right) =$ 362 , *equivalent stiffness*

$\omega = \sqrt{\frac{k^*}{m^*}} =$ 61.578 , *circular frequency*

$f = \frac{\omega}{2\pi} =$ 9.800 Hz, *natural frequency* > 7.5 **[Satisfactory]**

$T = \frac{1}{f} =$ 0.102 Sec, *natural period*

$c^* = 2\zeta\omega m^* =$ 0.287 , *generalized damping*

$a = \frac{F\omega}{c^*} =$ 32.167 in/sec^2, *maximum acceleration* where *F* = 150 lbs, *average walker weight*

$y_{max} = \frac{F}{\omega c^*} =$ 0.008 in, *maximum dynamic deflection* < $\frac{\sqrt{a^2+b^2}}{35000} =$ 0.013 in **[Satisfactory]**

References

1. Andrew Robertson, MSc, Ceng, MIStructE, MICE: Simplified dynamic analysis of beams and slabs with tuned mass dampers. The Structural Engineer, Vol. 94-1, 2016.
2. Reza Kashani, PhD, PE: Vibration abatement of rectangular, trapezoidal and irregular-shaped joist-framed floors, using tuned mass dampers. The Structural Engineer, Vol. 94-1, 2016.

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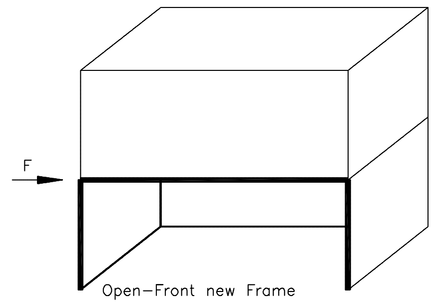
Retrofit Soft, Weak, or Open-Front Story Based on FEMA P807/ASCE 41-13

DESIGN CRITERIA & DESIGN SUMMARY

1. The new frame design is to eliminate a specific deficiency, not to bring entire building "up to new code".
2. The design force, F, based on the maximum deliverable force from the existing diaphragm, may be inadequate, because new diaphragm/horizontal brace may be required if the connector or nailer cannot be designed with the over strength factor from horizontal irregularities (ASCE 7-10 12.3.3.4, Table 12.3-1).
3. Diaphragm, to deliver shear to new frame, is to be considered flexible, conservatively.



The design force of new frame, ***F*** *=* **50% x** ***71.7*** = ***35.83*** *kips, (ASD)*

The nailer: ***1.25 x*** **0.75 x** ***33.0*** **+** **50% x** ***38.4*** = ***50.14*** *kips, (ASD)*

INPUT DATA & ANALYSIS

Determine Base Shear (Derived from ASCE 7 Sec. 12.8 & Supplement 2)

V = MAX{ MIN [S_{D1}I / (RT) , S_{DS} I / R] , MAX(0.044S_{DS}I , 0.01) , 0.5S_1 I / R } W

= MAX{ MIN[1.19W , 0.48W] , 0.04W , 0.00W }

^ (for $S_1 \geq 0.6$ g only)

= 0.48 W, (SD)

= 0.34 W, (ASD) = **71.66** kips

Where			
	S_{DS} =	0.96	(ASCE 7 Sec 11.4.4)
	S_{D1} =	0.55	(ASCE 7 Sec 11.4.4)
	S_1 =	0.54	(ASCE 7 Sec 11.4.1)
	R =	2	(ASCE 7 Tab 12.2-1, *for wall finish on the upper level*)
	I =	1	(2015 IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)
	C_t =	0.02	(ASCE 7 Tab 12.8-2)
	h_n =	26.0	ft
	x =	0.75	(ASCE 7 Tab 12.8-2)

T = $C_t\,(h_n)^x$ = 0.230 sec, (ASCE 7 Sec 12.8.2.1)

Calculate Vertical Distribution of Forces & Allowable Elastic Drift (ASCE 7, Sec 12.8.3 & 12.8.6)

Level	W_x	h_x	h_x^k	$W_x h_x^k$	F_x , ASD (12.8-11)		$\delta_{xe,allowable,\ ASD}$
Roof	80	26	26.0	2080	38.4	(0.48 Wx)	0.6
2ND	129	14	14.0	1806	33.3	(0.26 Wx)	0.8
	209.0			3886	71.7		

Where		
	k = 1	for T <= 0.5
	k = 0.5 T + 0.75	for T @ (0.5 , 2.5)
	k = 2	for T >= 2.5

$\delta_{xe,allowable,\ ASD} = \Delta_a$ I / (1.4 C_d), (ASCE 7 Sec 12.8.6)

C_d = 2 ,(ASCE 7 Tab 12.2-1)

Δ_a = 0.0125 h_{sx}, (ASCE 7 Tab 12.12-1)

Calculate Diaphragm Forces (ASCE 7, Sec 12.10.1.1)

Level	W_x	ΣW_x	F_x	ΣF_x	F_{px} , ASD, (12.10-1)	
Roof	80.0	80.0	38.4	38.4	20.5	(0.26 Wx)
2ND	129.0	209.0	33.3	71.7	33.0	(0.26 Wx)
	209.0		71.7			

Where F_{min} = 0.2 S_{DS} I W_x / 1.5 , ASD

F_{max} = 0.4 S_{DS} I W_x / 1.5 , ASD

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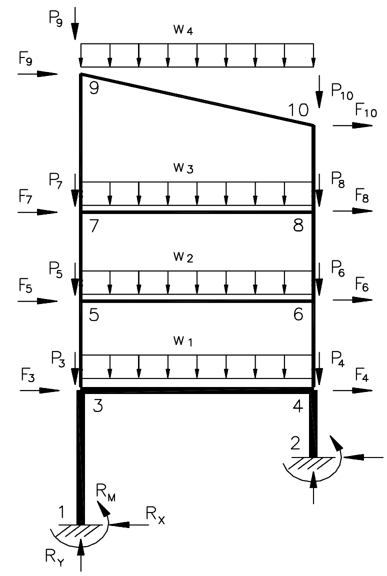
Four Story Moment Frame Analysis using Finite Element Method

INPUT DATA

JOINTS	X (ft)	Y (ft)	P (kips)	F (kips)	Δ_x (in)	Δ_y (in)	R_x (kips)	R_y (kips)	R_M (ft-k)
1	0	-18			0	0	-4.20369	22.49333	69.48815
2	24	-12			0	0	-19.7963	65.50667	177.574
3	0	0	5	3	-0.54	0.00			
4	24	0	5	3	-0.53	-0.01			
5	0	12	5	3	-1.78	-0.01			
6	24	12	5	3	-1.78	-0.02			
7	0	24	5	3	-2.99	-0.01			
8	24	24	5	3	-2.99	-0.03			
9	0	42	5	3	-3.87	-0.02			
10	24	36	5	3	-3.86	-0.03			



Element			E (ksi)	A (in^2)	I_x (in^4, in plane)	w (k/ft)	L (ft)	Drift (in)
1	Beam	3 - 4	29000	30	500	0.5	24.00	0.00
2	Beam	5 - 6	29000	17.2	191	0.5	24.00	0.01
3	Beam	7 - 8	29000	17.2	191	0.5	24.00	0.02
4	Beam	9 - 10	29000	17.2	191	0.5	24.74	0.02
5	Column	1 - 3	29000	50	800		18.00	0.54
6	Column	2 - 4	29000	50	800		12.00	0.53
7	Column	3 - 5	29000	17.2	191		12.00	1.24
8	Column	4 - 6	29000	17.2	191		12.00	1.24
9	Column	5 - 7	29000	17.2	191		12.00	1.22
10	Column	6 - 8	29000	17.2	191		12.00	1.22
11	Column	7 - 9	29000	17.2	191		18.00	0.87
12	Column	8 - 10	29000	17.2	191		12.00	0.87

ANALYSIS

Design Section Forces

Section		Left/Bottom End			Middle	Right/Top End		
		P (kips, axial)	V (kips, shear)	M (ft-kips)	M (ft-kips)	P (kips, axial)	V (kips, shear)	M (ft-kips)
Beam	3 - 4	6.97	-2.22	71.08	8.5	6.97	-14.22	-126.15
Beam	5 - 6	-0.63	0.00	50.93	15.0	-0.63	-12.00	-92.98
Beam	7 - 8	-1.90	1.63	28.36	12.0	-1.90	-10.37	-76.42
Beam	9 - 10	2.51	3.80	2.03	14.1	5.42	-7.85	-48.07
Column	1 - 3	22.49	4.20	-69.49	-31.7	22.49	4.20	6.18
Column	2 - 4	65.51	19.80	-177.57	-58.8	65.51	19.80	59.98
Column	3 - 5	19.71	8.17	-64.90	-15.9	19.71	8.17	33.17
Column	4 - 6	46.29	9.83	-66.17	-7.2	46.29	9.83	51.75
Column	5 - 7	14.71	4.55	-17.76	9.5	14.71	4.55	36.80
Column	6 - 8	29.29	7.45	-41.22	3.5	29.29	7.45	48.22
Column	7 - 9	8.07	-0.36	8.44	5.2	8.07	-0.36	2.03
Column	8 - 10	13.93	6.36	-28.20	9.9	13.93	6.36	48.07

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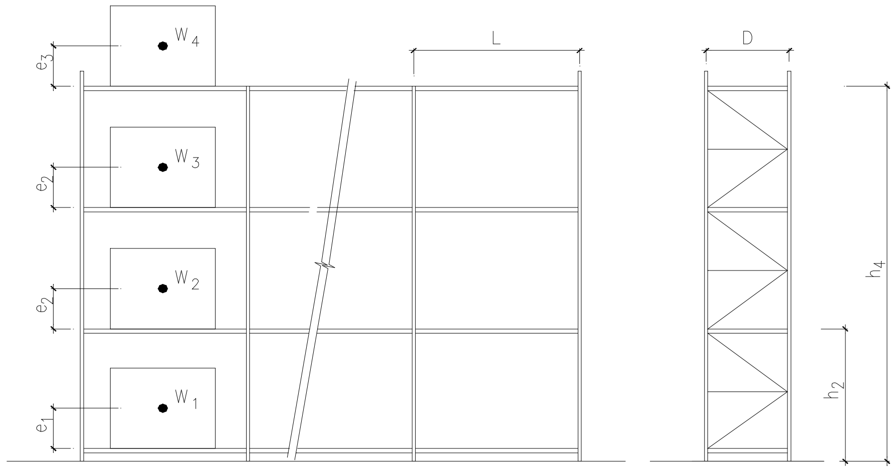
Lateral Loads of 4 Level Shelving, with Hilti Anchorage, Based on ASCE 7-10

Dimensions

D =	3.5	ft
L =	9	ft
h_4 =	18.5	ft



SHELVING FRONT ELEVATION

SIDE ELEVATION

Determine Shelving Base Shear (Derived from ASCE 7-10 15.5.3 & 13.3)

$$V = F_p = \frac{1}{1.5} Max\left[0.3 S_{DS} I_p W_p \ , \ Min\left(\frac{0.4 a_p S_{DS} I_p W_p}{R_p}\left(1+2\frac{z}{h}\right) \ , \ 1.6 S_{DS} I_p W_p\right)\right] = \quad 0.34 \ W_p, ASD$$

= 470.8 lbs,

(ASD level force at each ***L*** Shelving)

Where

S_{DS} =	1.121	(ASCE 7-10 11.4.4)
I_p =	1.5	(ASCE 7-10 15.5.3.1)
$W_p = W_1 + W_2 + W_3$ =	1400.0	lbs, each L shelving
a_p =	2.5	(ASCE 7-10 15.5.3)
R_p =	4	(ASCE 7-10 15.5.3)
z =	0	ft, shelving floor height from building ground (ASCE 7-10 13.3.3.1)
h =	36	ft, average building roof height (ASCE 7-10 13.3.3.1)

Calculate Vertical Distribution of Shelving Base Shear (lbs) & Allowable Elastic Drift (ASCE 7-10 12.8.3 & 15.5.3.3)

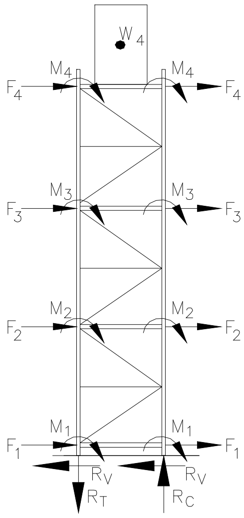
Level	$\mathbf{W_x}$	$\mathbf{h_x}$	$\mathbf{h_x^k}$	$\mathbf{W_x h_x^k}$	$\mathbf{F_x}$, ASD (12.8-11)		$\delta_{xe,allowable}$, in, ASD
4	350	18.5	18.5	6475	229.2	(0.65 Wx)	7.929
3	350	12.5	12.5	4375	154.9	(0.44 Wx)	5.357
2	350	6.5	6.5	2275	80.5	(0.23 Wx)	2.786
1	350	0.5	0.5	175	6.2	(0.02 Wx)	0.214
	1400.0			13300	470.8		

Where k = 1 (ASCE 7-10 15.5.3.2)

$\delta_{xe,allowable,\ ASD} = 5\%\ h_x/1.4$, (ASCE 7-10 15.5.3.4)

(cont'd)

Calculate Lateral Forces (lbs), at Each Joint of Both Directions, and Max Reactions (lbs) (ASCE 7-10 15.5.3.2)



Case a, (ASCE 7-10 15.5.3.2 a)

Level	e_x (ft)	W_x	F	M	R_T	R_C	R_V
4	3	234.5	76.8	230			
3	3	234.5	51.9	156			
2	3	234.5	27.0	81			
1	3	234.5	2.1	6			
		938.0	157.7	473.2	1084.5	2022.5	157.7

Case b, (ASCE 7-10 15.5.3.2 b)

Level	e_x (ft)	W_x	F	M	R_T	R_C	R_V
4	3	350	58.9	177			
3		0	0.0	0			
2		0	0.0	0			
1		0	0.0	0			
		350.0	58.9	176.6	548.1	898.1	58.9

Anchorage & Summary

2 - 1/2" ϕ, w/ 2" emb in 2.5 ksi conc, KWIK Bolt-TZ required at each column base. (ICC ESR-1917)

$(P_s / P_t) + (V_s / V_t) =$ 0.950 < 1.20 **[SATISFACTORY]**

Where $P_s =$ 542.3 lbs $P_t =$ 606 lbs / bolt, (ICC ESR-1917)

$V_s =$ 78.9 lbs, $V_t =$ 1419.5 lbs / bolt, (ICC ESR-1917)

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Box Moment Frame Analysis for Enhanced/New Wall Opening

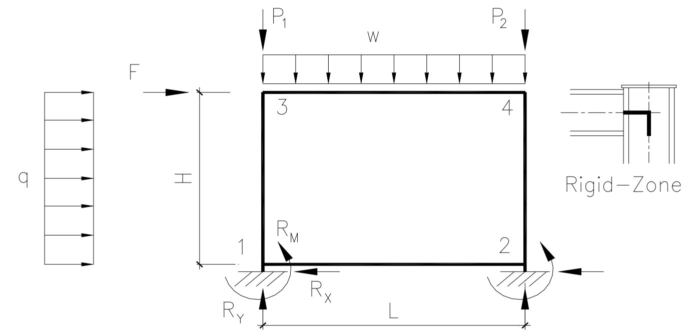
INPUT DATA & SUMMARY

COLUMN SIZE (WF, Tube, or Pipe)

1 - 5 = **HSS12.500X0.500** Pipe
ORIENTATION = **x-x**
2 - 6 = **W12X65** WF
ORIENTATION = **x-x**

HORIZONTAL MEMBER (WF, Tube, or Pipe)

3 - 4 = **HSS12X6X5/8** Tube <== Header Beam
ORIENTATION = **x-x**
1 - 2 = **W18X65** WF <== Threshold
ORIENTATION = **y-y** ,(flat)



DIMENSIONS
$H =$ **16** ft
$L_1 =$ **22** ft

LOADS
$F =$ **12** kips, (horiz. to right)
$P_1 =$ **5** kips, (downward)
$P_2 =$ **5** kips, (downward)
$q =$ **0.32** kips / ft, (horiz. to right)
$w =$ **0.45** kips / ft, (downward)

RIGID-ZONE LENGTH FACTOR
75 % of (d / 2)

DESIGN SECTION FORCES

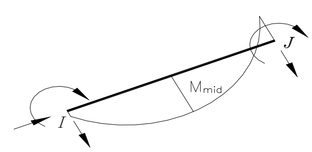
Column	$N_{13} =$	-5.9	k,(compre.)	$N_{24} =$	-14.0	k,(compression)
	$V_{13} =$	7.0	kips	$V_{24} =$	10.2	kips
	$M_{13} =$	46.8	ft-kips	$M_{24} =$	99.2	ft-kips
Horizontal Member	$V_{12} =$	0.0	kips	$V_{34} =$	8.9	kips
	$M_{12} =$	0.1	ft-kips	$M_{34} =$	63.4	ft-kips
	$M_{12,\ mid} =$	0.0	ft-kips	$M_{34,\ mid} =$	6.7	ft-kips

HORIZONTAL DEFLECTION

$\Delta_{H,3} =$ 0.6633 in, (horiz. to right)
$\Delta_{H,5} =$ 0.6584 in, (horiz. to right)

ANALYSIS

Joint	Coordinates (ft)		Reaction (k, ft-k)			Joint J Deflection (in,deg)		
	X	Y	R_X	R_Y	R_M	X	Y	θ
1	0	0.03	6.97	5.91	46.83	0.000	0.000	0.001
2	22	0.03	10.15	13.99	99.16	0.000	0.000	0.001
3	0	16.03				0.663	0.002	0.117
4	22	16.03				0.658	0.005	0.157



TYPICAL POSITIVE FORCES

End Joint		A	I	E	Length	Moment (ft-k)			Axial	Shear (k)	
I	J	(in^2)	(in^4)	(ksi)	(ft)	I	Mid	J	(k)	I	J
1	2	19.1	54.8	29000	21.65	-0.06	0.00	-0.05	0.00	0.00	0.00
1	3	17.2	191	29000	15.81	46.45	-1.36	23.25	-5.91	-6.94	1.88
2	4	19.1	533	29000	15.81	98.76	-18.55	61.66	-13.99	-10.15	10.15
3	4	18.7	321	29000	21.65	-24.04	6.71	-63.38	10.15	-0.84	-8.91

(The length & forces not included Rigid-Zone.)

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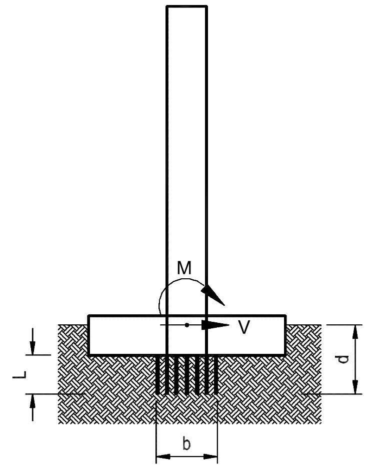
High-Rise Structural Embedded Design Based on 2016 CBC/2015 IBC



DESIGN CRITERIA

1. There are many code sections to check building lateral stability, including Load Combinations on 2015 IBC/2016 CBC 1605 with probability in [0 , 1] and Overturning Factor on 2015 IBC/ 2016 CBC 1604.4 1808.3.1 & ASCE 7-10 12.13.4.
2. This design method is the lateral stability bottom line, based on 2015 IBC/2016 CBC 1807.3. All high-rise structures have to comply with this design, including Non-Building structures ASCE 7-10 15.1.1.

INPUT DATA & DESIGN SUMMARY

SINGLE DIAMETER OF GROUP PILES D = 30 in, (762 mm)

LENGTH OF GROUP PILES L = 150 ft, (46 m)

ACTURAL EMBEDMENT d = 250 ft, (76 m)

BASE SHEAR AND OVERTURNING MOMENT, AT GRADE LEVEL, ON STRENGTH DESIGN

(If Modal Response Spectrum Analysis applied, please input the first modal only before/without CQC/SRSS)

V = 3750 kips, (16681 kN)

M = 1347459 ft-kips, (1826909 kN-m)

IS FOOTING RESTRAINED AT GRADE LEVEL ? *(1=YES, 0=NO)* 1 Yes.

(Only for big spread multi-story concrete basement/box foundation, input 1)

EFFECTIVE DIAMETER OF High-Rise Structural GROUP PILES b = 35.3553 ft, (11 m)

(Input $\boldsymbol{n^{0.5} D}$ *suggested, where* $\boldsymbol{n}$ *is total pile numbers)*

LATERAL SOIL BEARING CAPACITY S = 0.2 ksf / ft, (31.4175 [kN/m^2] / m)

ISOLATED POLE FACTOR (2015 IBC 1806.3.4) F = 2

THE EMBEDDED DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE PILE EFFECTIVE LENGTH (2015 IBC/2016 CBC 1810.3.5.2.2):

L_{eff} = 30 D = 75.00 ft, (22.86 m)

d_{eff} = (d - L) + L_{eff} = 175.00 ft, (53.34 m)

DETERMINE REQUIRD EMBEDDED DEPTH (2015 IBC/2016 CBC 1807.3):

LATERAL BEARING @ BOTTOM : $S_3 = FS \ \ Min\left(d \ \ , \ \ 12'\right)$ h = M / V = 359.32 ft, (109.52 m)

LATERAL BEARING @ d/3 : $S_1 = FS \ \ Min\left(\frac{d}{3} \ \ , \ \ 12'\right)$ P = V / 1.4 = 2678.57 kips, ASD (11915 kN)

$$A = \frac{2.34P}{bS_1}$$

REQUIRD DEPTH :

$$d = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25Ph}{bS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

= **155.25** ft, (47.32 m)

< d_{eff}

[Satisfactory]

Purchaser's LOGO	PROJECT :		PAGE :	
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Flexible Diaphragm Design with Tension Rod Cross Bracing

DESIGN CRITERIA

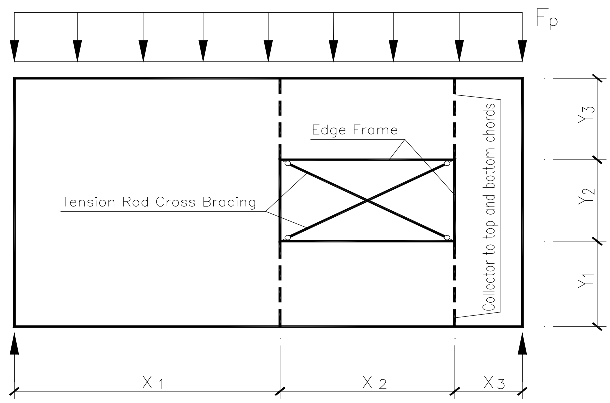
1. A Tension Rod Cross Bracing includes two cross rods and four sides frame, which can be new design or added to existing diaphragm.
2. The area of Tension Rod Cross Bracing may be open or not open, but Tension Rod Cross Bracing takes full this area lateral loads.
3. For multi span bracing, the each single threaded rod has to be anchored at both ends frame, and be designed by this Cross Bracing.

INPUT DATA

LATERAL DIAPHRAGM FORCE : F_p = 380 plf

DIMENSIONS: X_1 = 72 ft , X_2 = 24 ft , X_3 = 16 ft

Y_1 = 20 ft , Y_2 = 20 ft , Y_3 = 20 ft



DESIGN SUMMARY

THE MAXIMUM CHORD FORCE:

$T_{max, Chord} = -C_{max, Chord} =$ 9.93 kips

THE MAXIMUM SHEAR STRESS: $V_{max} =$ 354.67 plf

THE MAXIMUM COMPRESSION ON EDGE FRAME MEMBER:

$-C_{Edge} =$ 7.60 kips

THE MAXIMUM CROSS ROD TENSION FORCE:

$T_{Rod} =$ 11.87 kips

ANALYSIS

$L = X_1 + X_2 + X_3 =$ 112 ft

$B = Y_1 + Y_2 + Y_3 =$ 60 ft

$X_S = \text{Min}(X_1 , X_3) =$ 16 ft

$R = 0.5 (F_p L) =$ 21.28 kips

$V_{max} = R / B =$ 354.6667 plf

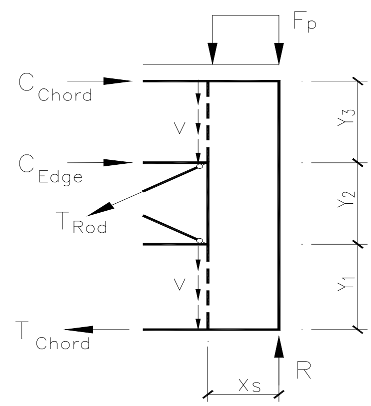
$T_{max, Chord} = F_p L^2 / (8 B) =$ 9.93 kips

$C_{max, Chord} = -T_{max, Chord} =$ -9.93 kips

$v = (R - F_p X_S) / B =$ 253.33 plf

$T_{Rod} = (1.5 v Y_2) (X_2^2 + Y_2^2)^{0.5} / Y_2 =$ 11.87 kips

$C_{Edge} = - (1.5 v Y_2) \text{Max} [1 , Y_2 / X_2] =$ -7.60 kips

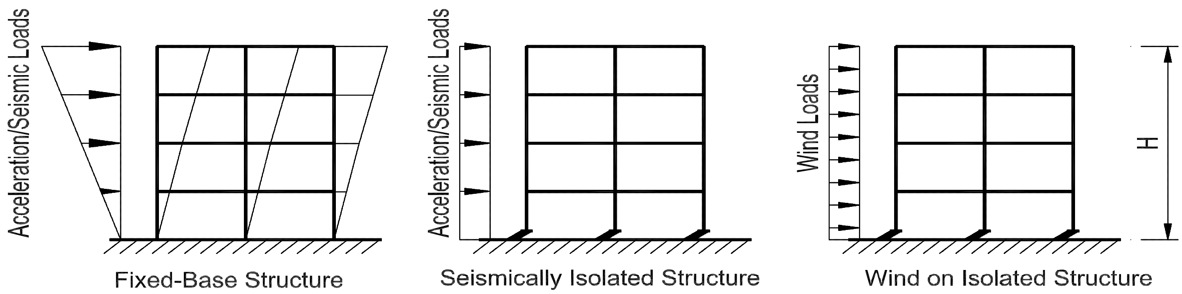


Purchaser's LOGO	PROJECT : CLIENT : JOB NO. :	DATE :	PAGE : DESIGN BY : REVIEW BY :	

Base Isolated Building Design Based on ASCE 7-10

DESIGN CRITERIA

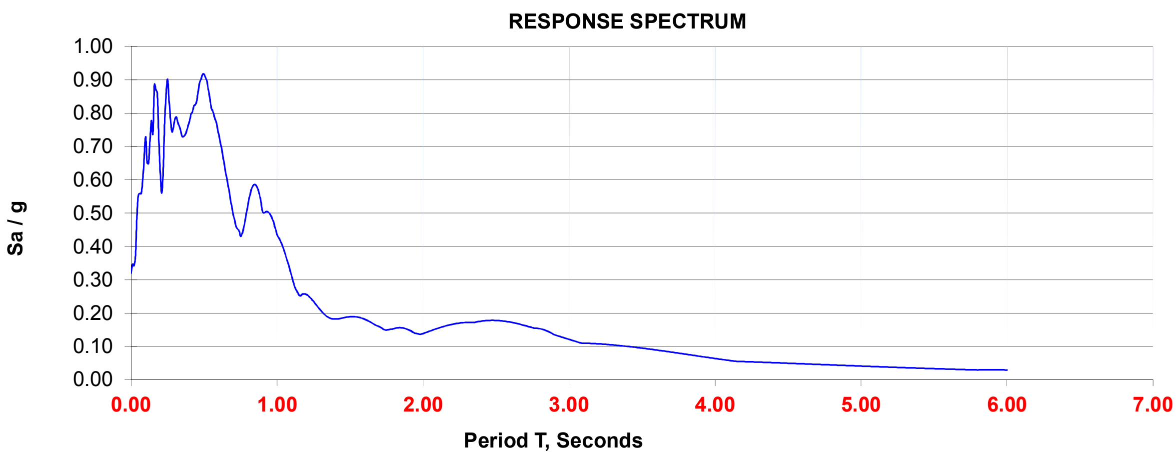
1. Base isolator system can reduce seismic loads by increasing the period/reducing the stiffness of structure. But the building wind loads are the same without change. So not all structures are adequate for base isolator system, with both seismic and wind limits.



2. The period of fixed-base structure and building wind load can be calculated by 3D finite element method (FEM) and spreadsheets. But it is very difficult to use finite elements modeling isolators, since damping and isolator stiffness matrix. This design method, using building effective stiffness concept for entire structure and isolated system, to check if the structure is adequate for base isolator system. If adequate, users can select and test isolated system by wind loads to drift limits (ASCE 7-10 17.2.4.2 & 17.5.6).

INPUT DATA & DESIGN SUMMARY

BUILDING STRUCTURAL HIGHT	H = 64	ft, (19.51 m)
BUILDING TOTAL WEIGHT	W = 6000	kips, (2721540 kg)
FIXED-BASE STRUCTURAL PERIOD	T = 0.55	Sec, (The first mode period from 3D FEM)
BUILDING TOTAL WIND LOAD (MWFRS, SD level)	P = 220	kips, (979 kN)



El Centro 1940 North South Component (ζ = 0.05)

THE DESIGN IS ADEQUATE.

ANALYSIS

DETERMINE EFFECTIVE STIFFNESS OF FIXED-BASE STRUCTURE

$k_{Fix} = \frac{4W\pi^2}{gT^2} =$ 2028.14 kips/in, (355165 kN/m). *Sa / g =* 0.81453 *, from the Response Spectrum by T value.*

$>$ *0.025 W , (ASCE 7-10 17.2.4.4)*

[Satisfactory]

$F = \frac{WSa}{g} =$ 4887.18 kips, (21738 kN), the first mode seismic force. $\Delta_{2H/3} = F / k_{Fix} =$ 2.41 in, (61 mm).

$\Delta_{Max} = \left(\frac{1}{8} - \frac{1}{30}\right)\frac{wH^4}{EI} = \frac{11FH^3}{60EI} = \left(\frac{3}{2}\right)^2 \Delta_{2H/3}$ $EI =$ 74858518679 k in in, (214819371 kN m m).

(cont'd)

DETERMINE EFFECTIVE STIFFNESS OF ISOLATOR UNIT

$$\delta_{Max} = \frac{P h_{iso}^3}{3(EI)_{iso}} + \frac{0.5 P H h_{iso}^2}{2(EI)_{iso}} = 0.015 h_{xx} =$$ 2.88 in, (73 mm).

where h_{xx} = 16 ft, (ASCE 7-10 17.5.6)

h_{iso} = 13.5 in, *isolator unit thickness*

$(EI)_{iso}$ = 2735648.438 k in in, (7850 kN m m).

DETERMINE EFFECTIVE STIFFNESS OF ISOLATED STRUCTURE (ASCE 7-10 17.2.4.4)

$$\frac{1}{k} = \left(\frac{h_{iso}^3}{3(EI)_{iso}} + \frac{0.5 H h_{iso}^2}{2(EI)_{iso}} + \frac{0.5 H h_{iso}^2}{(EI)_{iso}} + \frac{(0.5H)^3}{3EI} \right) =$$ 0.0389 *, elastic value, without lateral restoring force.*

k_{iso} *= Max(k / 50% , k + 0.025W) =* 175.69 kips/in, (30767 kN/m).

CHECK IF SEISMIC FORCE OF ISOLATED STRUCTURE REDUCED

$$T_{iso} = 2\pi \sqrt{\frac{W}{g k_{iso}}} =$$ 1.87 Sec

(Sa) $_{iso}$ */ g =* 0.1555 *, from the Response Spectrum by* T_{iso} *value.*

$$F_{iso} = \frac{W (Sa)_{iso}}{g} =$$ 933.133 kips, (4151 kN), the first mode seismic force.

< *F* = 4887.176 kips, (21738 kN), the fixed-base seismic force.

[Satisfactory]

Purchaser's LOGO

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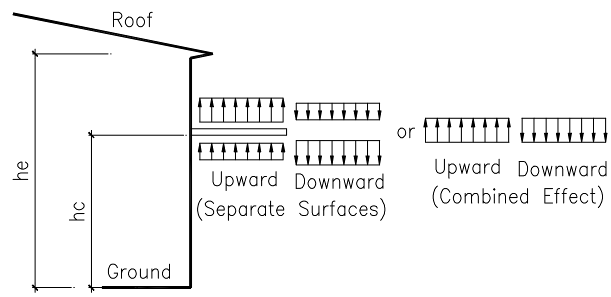
Wind Load on Canopy Based on ASCE 7-16 Section 30.11

INPUT DATA

BASIC WIND VELOCITY	V =	136	mph
EXPOSURE TYPE (B, C, D)	== >	C	
MEAN ROOF HEIGHT	h =	40	ft, (ASCE 7-16 26.10.2)
MEAN CANOPY HEIGHT	h_c =	28	ft, (ASCE 7-16 Fig. 30.11-1B)
MEAN EAVE HEIGHT	h_e =	32	ft, (ASCE 7-16 Fig. 30.11-1B)
EFFECTIVE WIND AREA	A =	40	ft^2, (ASCE 7-16 Fig. 30.11-1)
TOPOGRAPHIC FACTOR	K_{Zt} =	1	Flat, (26.8 & Table 26.8-1)



ANALYSIS & DESIGN SUMMARY

$$p = \left(0.00256 K_z K_{zt} K_d K_e V^2\right) GC_p = \quad 41.86\ \mathbf{GC_p} \quad \mathbf{==>}$$

Where : K_Z = 1.04 , (ASCE 7-16 Tab. 26.10-1)

K_d = 0.85 , (ASCE 7-16 Tab. 26.6-1)

K_e = 1 , (ASCE 7-16 Tab. 26.9-1)

GC_p	ASCE 7-16 Fig. 30.11-1A		ASCE 7-16 Fig. 30.11-1B	
	Upward	Downward	Upward	Downward
Upper	*0.9092*	*0.6796*	*0.7495*	*0.7194*
Lower	*0.6796*	*0.7398*		

p	Separate, (psf)		Combined, (psf)	
	Upward	Downward	Upward	Downward
Upper	***38.06***	***28.45***	***31.37***	***30.11***
Lower	***28.45***	***30.97***		

The Max Wind Load: 30.1 psf. (1442 N/m^2).

Purchaser's LOGO

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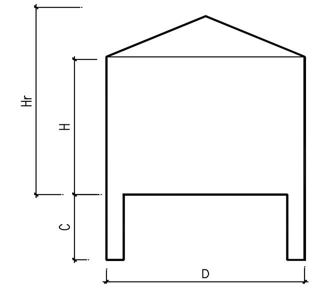
Wind Analysis for Bin or Silo, Supported by Columns, Based on ASCE 7-16



INPUT DATA

Exposure category (B, C or D, ASCE 7-16 26.7.3)		C	
Importance factor (ASCE 7-16 Table 1.5-2)	I_w =	1.00	for all Category
Basic wind speed (ASCE 7-16 26.5.1 or 2015 IBC)	V =	105	*mph, (168.98 kph)*
Topographic factor (ASCE 7-16 26.8 & Table 26.8-1)	K_{zt} =	1	Flat
Height to eave (< 120 ft)	H =	35	*ft, (10.67 m)*
Height to ridge	H_r =	42	*ft, (12.80 m)*
Diameter (< 120 ft)	D =	30	*ft, (9.14 m)*
Clearance Height (< H required)	C =	28	*ft, (8.53 m)*

DESIGN SUMMARY

Max horizontal force to building face		=	**29.9**	*kips, (133 kN), SD level (LRFD level), Typ.*
Max overturning moment at ground base end		=	**1734.3**	*ft-kips, (2351 kN-m)*
Max total upward force		=	**13.6**	*kips, (60 kN)*
Pressures for components and cladding	*from*	**-67.9** *(-3.3 kPa)*	*to*	**32.7** *psf (1.6 kPa)*

ANALYSIS

__Velocity pressure__

$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, K_e\, V^2$ = **25.61** psf

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, K_e\, V^2$ = **27.73** psf

where: $q_{z,h}$ = velocity pressure at mean roof height, h. (Eq. 26.10-1 page 268)

K_z = velocity pressure exposure coefficient evaluated at height, z, (Tab. 26.10-1, pg 268)	=	**1.07**
K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 26.10-1, pg 268)	=	**1.16**
K_d = wind directionality factor. (Tab. 26.6-1, for building, page 266)	=	**0.85**
z = height to centroid of wall	=	**45.50** ft
h = mean roof height	=	**66.50** ft

K_e = ground elevation factor. (***1.0*** per Sec. 26.9, page 268)

__Design pressures for MWFRS__

F_{wall} = Max[q_z G C_f , p_{min}] A_f = **30.18** kips $\qquad$ $M_{OT,\ wall}$ = (C + 0.5 H) F_{wall} = **1508.1** ft-kips

where: F_{wall} = wind force on wall. (Eq. 29.4-1, page 322). $\qquad$ p_{min} = **16** psf (ASCE 7-16 29.7)

G = gust effect factor (ASCE 7-16, 26.11) = **0.85**

A_f = projected area normal to the wind = **1050** ft^2

C_f = force coefficient from (Fig. 29.4-1 & Fig. 29.4-1) = **1.32**

p = Max[q_h [(G C_p)-(G C_{pi})], p_{min}]

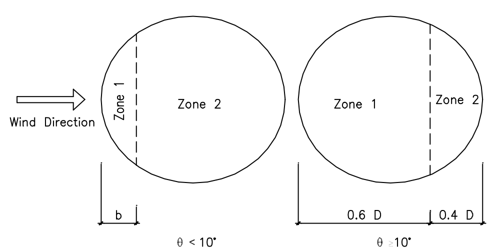
where: p = pressure in appropriate roof zone. (Eq. 29.4-4, page 327).

G $C_{p\,i}$ = product of gust effect factor and internal pressure coefficient.(Tab. 26.13-1, Enclosed Building, page 271)

= **0.18** or **-0.18**

b = width of zone 1 for C_p (ASCE 7-16, page 328) = **21.50** ft

θ = Angle of plane of roof from horizontal **25.02** °



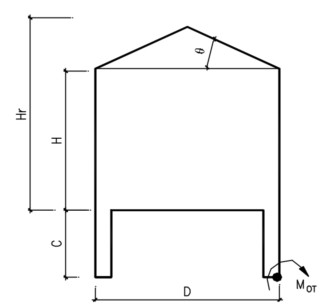


Figure 29.4-5

page 328

	Horizontal Projected Area (ft^2)	Vertical Projected Area (ft^2)	C_p	*p* (psf)	F_{Hori} (kips)	F_{Vert} (kips)	$M_{OT,\ roof}$ (ft-k)
Zone 1	442.83	114.83	-0.8	-23.85	-0.95991	9.57067	126.68
Zone 2	264.03	95.17	-0.5	-16.778	0.675287	4.01444	99.49

__Design pressures for components and cladding__

p = Max[q_h [(G C_p)-(G C_{pi})], p_{min}] = **-67.95** to **32.72** psf

where: p = pressure on component. (Eq. 30.12-1, pg 382)

p_{min} = 16.00 psf (ASCE 7-16 30.2.2)

G C_p = external pressure coefficient. (ASCE 7-16 30.12, page 383) = **1.00** to **-1.45**

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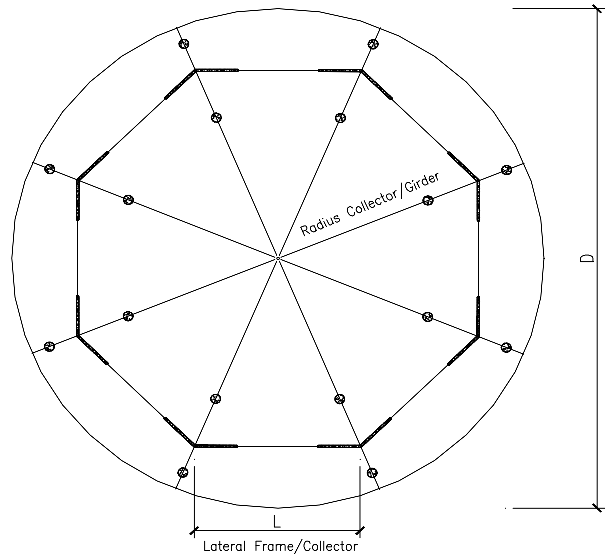
Circular Flexible Diaphragm Analysis



INPUT DATA & DESIGN SUMMARY

LATERAL DIAPHRAGM LOAD : F_p = 30 *psf, (1436 N/m²)*

DIMENSIONS: D = 214 *ft (65.23 m)*

L = 66.5 *ft (20.27 m)*

The Max Diaphragm Single Side Shear Stress =

903 *plf, (13168 N/m)*

The Max Diaphragm Both Sides Shear Stress to Lateral Frame =

1756 *plf, (25615 N/m)*

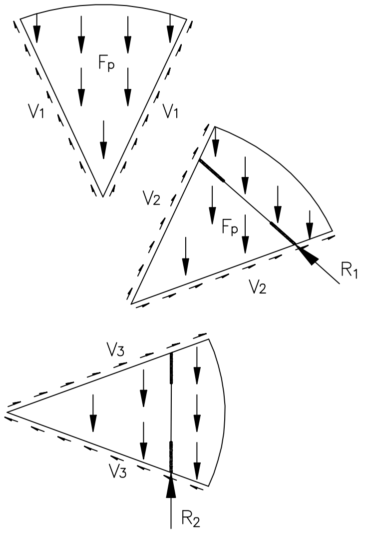
The Max Lateral Frame Force within L = **265.41** *kips, (1181 kN)*

The Max Curved Edge Chord Force = **27.93** *kips, (124 kN)*

The Max Radius Collector Force = **126.59** *kips, (563 kN)*

ANALYSIS

DETERMINE TYPICAL SECTION STRESS & FORCE



Section Area = 4496.012 in²

*(*Left Area = 2311.341 in²

Right Area = 2184.671 in²*)*

Load = 134.8804 kips

Angle = ϕ = 45 Deg

Centroid = 69.514 ft, *to Center*

V_1 = 682 plf

V_2 = 501 plf

V_3 = 221 plf

R_1 = 82.59 kips

R_2 = 116.80 kips

DETERMINE DIAPHRAGM STRESS & FORCE

The Max Diaphragm Shear Stress = Max (V_1 , V_2 , V_3 , R_1/L , R_2/L) = 1756 plf

(The Max Diaphragm ONE SIDE Shear Stress = 903 plf*)*

The Max Radius Collector Force = 0.5 D Max (V_1 + V_2 , V_2 + V_3) = 126.59 kips

The Max Curved Edge Chord Force = 0.5 D V_1 SIN (0.5 ϕ) = 27.93 kips

The Max Lateral Frame Force within L = R_2 + 2 [0.5 D (V_1 + V_2) COS(0.5 ϕ) SIN(0.5 ϕ) + 0.5 D (V_2 + V_3) SIN(0.5 ϕ)] =

265.41 kips

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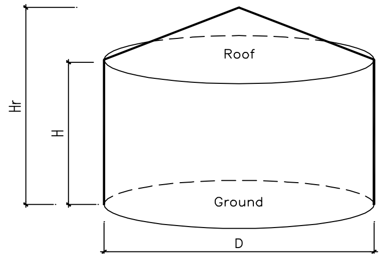
Wind Analysis for Circular Structure Based on ASCE 7-16

INPUT DATA



Exposure category (B, C or D, ASCE 7-16 26.7.3)		C	
Importance factor (ASCE 7-16 Table 1.5-2)	I_w =	1.00	for all Category
Basic wind speed (ASCE 7-16 26.5.1 or 2015 IBC)	V =	120	*mph, (193.12 kph)*
Topographic factor (ASCE 7-16 26.8 & Table 26.8-1)	K_{zt} =	1	Flat
Height to eave (< 120 ft)	H =	60	*ft, (18.29 m)*
Height to ridge	H_r =	80	*ft, (24.38 m)*
Diameter (< 120 ft)	D =	100	*ft, (30.48 m)*

DESIGN SUMMARY

Max horizontal force to building face = **200.4** *kips, (892 kN), SD level (LRFD level), Typ.*

Max overturning moment at ground base end = **18400.0** *ft-kips, (24947 kN-m)*

Max total upward force = **204.5** *kips, (909 kN)*

Pressures for components and cladding *from* **-79.2** *(-3.8 kPa)* *to* **43.3** *(2.1 kPa)* *psf*

ANALYSIS

Velocity pressure

$q_z = 0.00256\, K_z K_{zt} K_d K_e V^2$ = **30.71** psf

$q_h = 0.00256\, K_h K_{zt} K_d K_e V^2$ = **36.66** psf

where:
- $q_{z,h}$ = velocity pressure at mean roof height, h. (Eq. 26.10-1 page 268)
- K_z = velocity pressure exposure coefficient evaluated at height, z, (Tab. 26.10-1, pg 268) = **0.98**
- K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 26.10-1, pg 268) = **1.17**
- K_d = wind directionality factor. (Tab. 26.6-1, for building, page 266) = **0.85**
- z = height to centroid of wall = **30.00** ft
- h = mean roof height = **70.00** ft
- K_e = ground elevation factor. (***1.0*** per Sec. 26.9, page 268)

Design pressures for MWFRS

F_{wall} = Max[q_z G C_f , p_{min}] A_f = **203.59** kips $\qquad$ $M_{OT,\,wall}$ = 0.5 H F_{wall} = **6107.8** ft-kips

where:
- F_{wall} = wind force on wall. (Eq. 29.4-1, page 322). $\qquad$ p_{min} = **16** psf (ASCE 7-16 29.7)
- G = gust effect factor (ASCE 7-16, 26.11) = **0.85**
- A_f = projected area normal to the wind = **6000** ft^2
- C_f = force coefficient from (Fig. 29.4-1 & Fig. 29.4-1) = **1.30**

p = Max[q_h [(G C_p)-(G C_{pi})], p_{min}]

where:
- p = pressure in appropriate roof zone. (Eq. 29.4-4, page 327).
- G $C_{p\,i}$ = product of gust effect factor and internal pressure coefficient.(Tab. 26.13-1, Enclosed Building, page 271) = **0.18** or **-0.18**
- b = width of zone 1 for C_p (ASCE 7-16, page 328) = **53.20** ft
- θ = Angle of plane of roof from horizontal **21.80** °

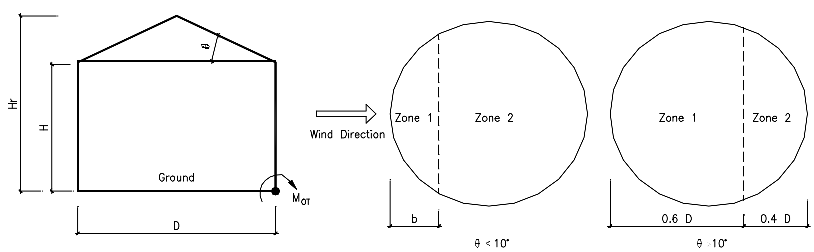


Figure 29.4-5
page 328

	Horizontal Projected Area (ft^2)	Vertical Projected Area (ft^2)	C_p	*p* (psf)	F_{Hori} (kips)	F_{Vert} (kips)	$M_{OT,\,roof}$ (ft-k)
Zone 1	4920.28	1093.59	-0.8	-31.529	-10.6136	144.035	8791.56
Zone 2	2933.70	906.41	-0.5	-22.18	7.466537	60.4156	3500.63

Design pressures for components and cladding

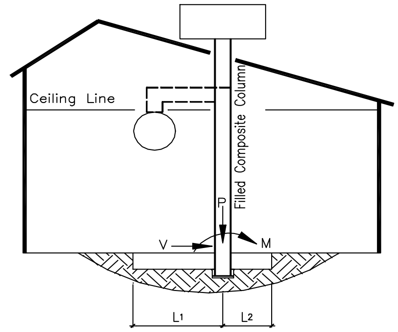
p = Max[q_h [(G C_p)-(G C_{pi})], p_{min}] = -79.19 to **43.26** psf

where:
- p = pressure on component. (Eq. 30.12-1, pg 382)
- p_{min} = 16.00 psf (ASCE 7-16 30.2.2)
- G C_p = external pressure coefficient. (ASCE 7-16 30.12, page 383) = **1.00** to **-1.16**

Support Design, for New Loads on Existing Roof, Based on ASCE 41-17, AISC 360-16 & ACI 318-14

DESIGN CRITERIA

1. The new loads on existing roof may be from equipment, solar panels, or big things hanged upon ceiling. To design the new loads directly supported by existing roof beams/trusses is inadequate, because the engineer has to assume the original beams/trussed has been over designed, and the roof diaphragm seismic design may have to be brought to current building code (ASCE 41-17).
2. Using new cantilever column (standalone or more), without connected existing structure, to fully support the new loads does not change existing structural any capacity, which may save cost.
3. The new cantilever column, which can be put within architectural wall, shall be filled composite HSS tube/pipe, to reduce drift, without steel buckling checks.



INPUT DATA & DESIGN SUMMARY

STEEL (Tube or Pipe) & SIZE **HSS8X6X1/2** <== **Tube**

STEEL YIELD STRESS F_y = **46** *ksi, (317 MPa)*

CONCRETE STRENGTH $f_c{}'$ = **4** *ksi, (28 MPa)*

AXIAL COMPRESSION FORCE P_r = **80** *kips, (356 kN), ASD*

STRONG AXIS EFFECTIVE LENGTH $L_{c,x} = KL_x$ = **24** *ft, (7.32 m)*

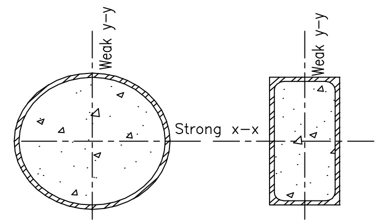
WEAK AXIS EFFECTIVE LENGTH $L_{c,y} = KL_y$ = 24 *ft, (7.32 m)*

BASE STRONG AXIS BENDING MOMENT M_{rx} = **35** *ft-kips, (47 kN-m), ASD*

STRONG AXIS BENDING UNBRACED LENGTH

L_b = 24 *ft, (7.32 m), (AISC 360 F2.2.c)*

STRONG DIRECTION SHEAR LOAD, ASD V_{strong} = **50** *kips, (222 kN)*



WEAK AXIS BENDING MOMENT M_{ry} = **20** *ft-kips, (27 kN-m), ASD*

WEAK DIRECTION SHEAR LOAD, ASD V_{weak} = **30** *kips, (133 kN)*

FOOTING THICKNESS *(12" Min.)* T = **16** *in, (406 mm).*

ALLOWABLE SOIL PRESSURE Q_a = **2.5** *ksf, (120 kPa)*

FOOTING LENGTH L_1 = **4** *ft, (1.22 m)*

L_2 = **3.5** *ft, (1.07 m)*

FOOTING LENGTH B = **6** *ft, (1.83 m)*

REINFORCING SIZE # **7**

THE FOOTING DESIGN SUMMARY:

LONGITUDINAL REINF., TOP	Not Required
LONGITUDINAL REINF., BOT.	5 # 7 @ 16 in o.c.
TRANSVERSE REINF., BOT.	6 # 7 @ 16 in o.c.

THE COLUMN DESIGN IS ADEQUATE.

ANALYSIS

CHECK FILLED COMPOSITE COLUMN LIMITATIONS (AISC 360 I2.2a)

A_{steel} / A_{total} = 0.25 **>** 1.0% **[Satisfactory]**

Where A_{steel} = 11.6 in^2 A_{total} = 46.6 in^2

b / t = 12.00 **<** $2.26\,(E / F_y)^{0.5}$ = 56.75

D / t = N/A $0.15\,E / F_y$ = 94.57 **[Satisfactory]**

Where b = 6.0 in t = 0.5 in

D = N/A E = 29000 ksi

CHECK COMPRESSION CAPACITY (AISC 360 I2.2b)

$P_c = P_n / \Omega_c$ = 107.05 kips **>** P_r **[Satisfactory]**

Where Ω_c = 2.0 P_n = 214.1 kips C_3 = 0.90

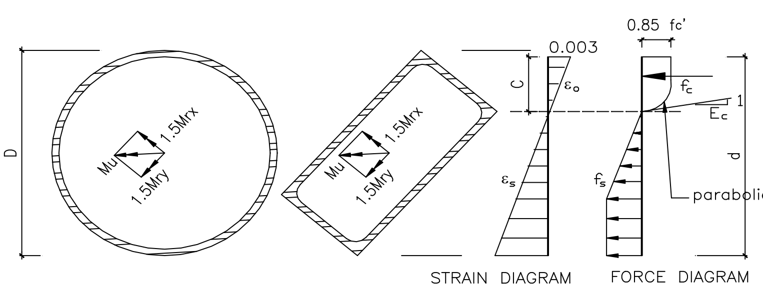
C_2 = 0.85 P_0 = 652.6 kips P_e = 244.13 kips

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360 I1.1b & I4, and ACI 318-14 Chapter 21 & 22)

$$\varepsilon_O = or\left[\frac{2\left(\beta f'_C\right)}{E_C}\ ,\ \varepsilon_{\max}\right]\ ,\ E_C = 57\sqrt{f'_C}\ ,\ E_S = 29000ksi$$

$$f_C = \begin{cases} \beta f'_C\left[2\left(\dfrac{\varepsilon_C}{\varepsilon_O}\right) - \left(\dfrac{\varepsilon_C}{\varepsilon_O}\right)^2\right] , & for\ \ 0 < \varepsilon_C < \varepsilon_O \\ \beta f'_C\ , & for\ \ \varepsilon_C \ge \varepsilon_O \end{cases}$$

$$f_S = \begin{cases} \varepsilon_S E_S\ , & for\ \ \varepsilon_S \le \varepsilon_t \\ f_y\ , & for\ \ \varepsilon_S > \varepsilon_t \end{cases}$$



C = 4.74 in

$P_c = P_n / \Omega_b$ = 80 kips

$M_c = M_n / \Omega_b$ = 43 ft-kips **>** $M_u / (\Omega_b / \phi_b) = (M_{rx}{}^2 + M_{ry}{}^2)^{0.5}$ = 40.3 ft-kips

Where Ω_b = 1.67 P_n = 133.6 kips **[Satisfactory]**

ϕ_b = 0.9 M_n = 71.4 ft-kips

CHECK SHEAR CAPACITY (AISC 360 I2.2d & G2)

$V_{n,strong} / \Omega_v$ = 147.2 / 1.67 = 88.1 kips **>** V_{strong} = 50.0 kips **[Satisfactory]**

$V_{n,weak} / \Omega_v$ = 110.4 / 1.67 = 66.1 kips **>** V_{weak} = 30.0 kips **[Satisfactory]**

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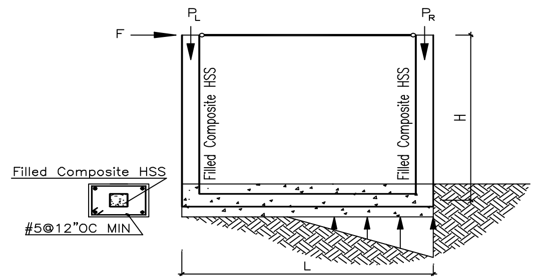
Reversed Lateral Frame Design Based on ASCE 41-17 & 7-16, AISC 360-16 & ACI 318-14

DESIGN CRITERIA

1. This is cantilevered column system (ASCE 7-16 Table 12.2-1). The columns, which fixed at bottom and pinned at top, shall be filled composite HSS tube, to reduce story drift, without steel buckling checks. The grade beam includes the same bending HSS section with filled concrete, which fully welded columns.
2. To retrofit soft, weak, or open-front existing story, this lateral frame can easily be built, without fully moment connection at top. The bottom HSS moment connections with filled concrete can be done in shop before to job site.
3. Since grade beam includes the same filled HSS section and site pour RC, the critical capacity section is at column bottom.

INPUT DATA & DESIGN SUMMARY

STEEL (HSS Tube) & SIZE		HSS14X10X1/2	< == Tube
WEAK AXIS BENDING?		No	
STEEL YIELD STRESS	F_y =	46	ksi, (317 MPa)
CONCRETE STRENGTH	f_c' =	4	ksi, (28 MPa)
DIMENSION	L =	24	ft, (7.32 m)
	H =	16	ft, (4.88 m)
GRADE BEAM	Width, b =	20	in, (508 mm)
	Height, h =	16	in, (406 mm)
LOADS	P_L =	80	kips, (356 kN), SD
	P_R =	70	kips, (311 kN), SD
	F =	18	kips, (80 kN), SD
ALLOWABLE SOIL PRESSURE	Q_a =	2.5	ksf, (120 kPa)



THE FRAME DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (2015 IBC 1605.2.1, 1808.3.1, & ASCE 7-10 12.13.4)

M_R / M_O = 4.6 > F = 0.75 / 0.9 = 0.83 **[Satisfactory]**

CHECK SLIDING (2015 IBC 1807.2.3) & SOIL BEARING CAPACITY (ACI 318 13.3.1.1)

1.1 ($V_{Lat, ASD}$) = 3.4 kips < $\mu \Sigma W$ = 21.87 kips **[Satisfactory]**

Where μ = 0.4

$$q_{Max}=\begin{cases}\dfrac{(\Sigma P)\left(1+\dfrac{6e_L}{L}\right)}{L}, & for\quad e_L\le\dfrac{L}{6}\\ \dfrac{2(\Sigma P)}{3(0.5L-e_L)}, & for\quad e_L>\dfrac{L}{6}\end{cases}$$

[Satisfactory]

CHECK STORY DRIFT (ASCE 7-16 12.8.6)

Δ_{max} = 0.94 in < $\delta_{xe,allowable,\ SD} = \Delta_a\ I\ /\ C_d$ = 3.07 in **[Satisfactory]**

CHECK FILLED COMPOSITE COLUMN LIMITATIONS (AISC 360 I2.2a)

$A_{steel}\ /\ A_{total}$ = 0.15 > 1.0% **[Satisfactory]**

Where A_{steel} = 20.9 in^2 A_{total} = 137.9 in^2

$b\ /\ t$ = 20.00 < $2.26\ (E\ /\ F_y)^{0.5}$ = 56.75

$D\ /\ t$ = N/A $0.15\ E\ /\ F_y$ = 94.57 **[Satisfactory]**

Where b = 10.0 in t = 0.5 in

D = N/A E = 29000 ksi

CHECK COLUMN COMPRESSION CAPACITY (AISC 360 I2.2b)

$P_c = P_n\ /\ \Omega_c$ = 573.20 kips > P_r **[Satisfactory]**

Where Ω_c = 2.0 P_n = 1146.4 kips C_3 = 0.90

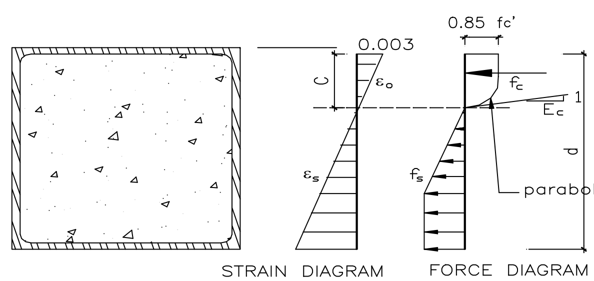
C_2 = 0.85 P_0 = 1359.2 kips P_e = 3341 kips

CHECK COLUMN COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360 I1.1b & I4, and ACI 318-14 Chapter 21 & 22)

$$\varepsilon_O = or\left[\frac{2\left(\beta f_C'\right)}{E_C}\ ,\ \varepsilon_{\max}\right]\ ,\ E_C=57\sqrt{f_C'}\ ,\ E_S=29000ksi$$

$$f_C=\begin{cases}\beta f_C'\left[2\left(\dfrac{\varepsilon_C}{\varepsilon_O}\right)-\left(\dfrac{\varepsilon_C}{\varepsilon_O}\right)^2\right]\ , & for\quad 0<\varepsilon_C<\varepsilon_O\\ \beta f_C'\ , & for\quad \varepsilon_C\ge\varepsilon_O\end{cases}$$

$$f_S=\begin{cases}\varepsilon_S E_S\ , & for\quad \varepsilon_S\le\varepsilon_t\\ f_y\ , & for\quad \varepsilon_S>\varepsilon_t\end{cases}$$



C = 7.13 in

$P_c = P_n\ /\ \Omega_b$ = 46.6666667 kips

$M_c = M_n\ /\ \Omega_b$ = 207 ft-kips > $M_u\ /\ (\Omega_b\ /\ \phi_b) = (M_{rx}^{\ 2} + M_{ry}^{\ 2})^{0.5}$ = 192.0 ft-kips

Where Ω_b = 1.67 P_n = 77.933 kips **[Satisfactory]**

ϕ_b = 0.9 M_n = 345.4 ft-kips

CHECK COLUMN SHEAR CAPACITY (AISC 360 I2.2d & G2)

$V_n\ /\ \Omega_v$ = 257.6 / 1.67 = 154.3 kips > V = 6.0 kips **[Satisfactory]**

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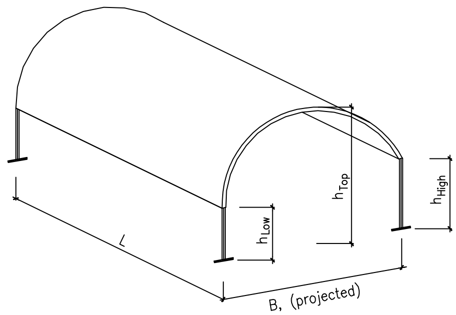
Wind Analysis for Open Arch Roof Based on ASCE 7-16

DESIGN CRITERIA

1. Arch roof building may be enclosed building, open building, or open ends building. For enclosed arch roof building, the wind loads are less than gable hip roof building, so wind analysis may conservatively be based on ASCE 7-16 Chapter 28 and 30.
2. For open arch roof building, there are no ASCE 7-16 sections can directly apply. Since the wind loads (horizontal, uplift, and overturning) of open arch roof building are critical, this software conservatively use the maximum net pressure coefficients based on ASCE 7-16 Chapter 27 and 30.



INPUT DATA

Exposure category (B, C or D)	=	C					
Importance factor, 1.0 only, (Table 1.5-2)	I_w =	1.00					
Basic wind speed (ASCE 7 26.5.1)	V =	125	mph, (201.17 kph)				
Topographic factor (26.8 & Table 26.8-1)	K_{zt} =	1	Flat				
Height of top roof	h_{Top} =	20	ft, (6.10 m)				
Height of roof edges	h_{High} =	12	ft, (3.66 m)	Height of mean roof	h =	16.00	ft, (4.88 m)
	h_{Low} =	9	ft, (2.74 m)	Radius of arch roof	R =	37.80	ft, (11.52 m)
Roof length	L =	80	ft, (24.38 m)	Angle of arch roof	α =	83	°, Deg
Roof horizontal projected width	B =	50	ft, (15.24 m)				
Effective area of component / cladding	A =	12	ft^2 <== (1.12 m^2)				

DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	27.91	kips, (124 kN), one side only, SD level.
Max overturning moment, at edge, in wind normal to building length, L, face	=	3214.70	ft-kips, (4359 kN-m)
Max overturning moment, at edge, in wind normal to building length, B, face	=	2409.67	ft-kips, (3267 kN-m)
Max total upward force	=	125.50	kips, (558 kN)

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, K_e\, V^2$ = **29.24** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 26.10-1 page 268), K_e = **1.00** , (Tab. 26.9-1 page 268)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 26.10-1, pg 268) = **0.86**

K_d = wind directionality factor. (Tab. 26.6-1, page 266) = **0.85**

h = height of mean roof = **16.00** ft

Main Wind-Force Resisting System (Sec. 27.3.2)

$p = q_h\, G\, C_N$ = **62.14** psf, *net pression in radius direction, curved, at wind normal to building length, L, face.*

= **29.82** psf, *net pression in radius direction, curved, at wind normal to building length, B, face.*

where: G = gust effect factor. (Sec. 26.11) = **0.85**

C_N = net force coefficient at wind normal to building length, L, face. (Fig. 27.3-4, page 279) = **2.50**

= net force coefficient at wind normal to building length, B, face. (Fig. 27.3-7, page 282) = **1.20**

Component and Cladding Elements (Sec. 30.7.2)

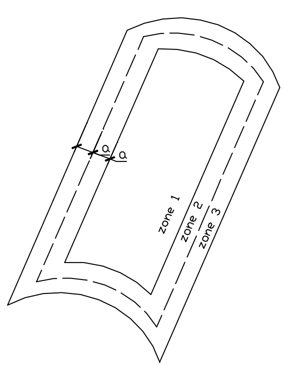
$p = q_h\, G\, C_N$

where: C_N = net force coefficients. (Fig. 30.7-1, page 376)

a = Max[Min(0.1L , 0.1B_{curve} , 0.4h) , Min(0.04L , 0.04B_{curve}) , 3ft] , (Fig. 30.7-1) = **5.5** ft

For effective area, 12 sq.ft. as given

	CLEAR WIND FLOW						OBSTRUCTED WIND FLOW					
	ZONE 3		ZONE 2		ZONE 1		ZONE 3		ZONE 2		ZONE 1	
C_N	2.40	-3.30	1.80	-1.70	1.20	-1.10	1.00	-3.60	0.80	-1.80	0.50	-1.20
p (psf)	59.65	-82.02	44.74	-42.25	29.82	-27.34	24.85	-89.47	19.88	-44.74	12.43	-29.82



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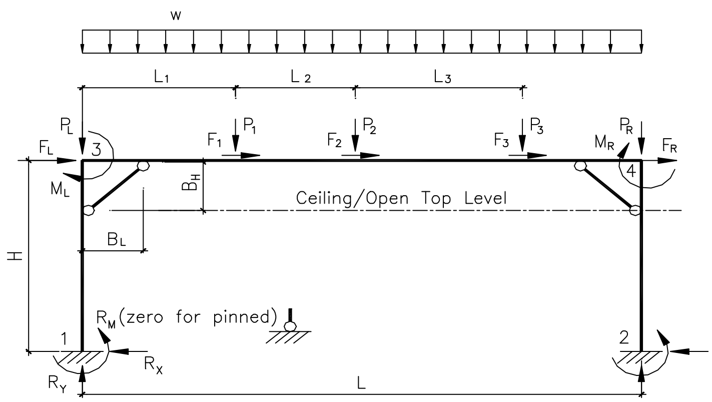
Knee Braced Moment Resisting Frame (KBRF) Analysis using Finite Element Method



DESIGN CRITERIA

1. The most moment resisting frames (SMRF/OMRF) are controlled by allowable story drift (ASCE 7-16 Tab 12.12-1), not by column/ beam capacity, so too big column and beam sizes. The KBRF is good for solving that issues.
2. The knee brace can also be added to existing frame to retrofit soft, weak, or open-front old story.

INPUT DATA & SUMMARY

DIMENSIONS
B_L = **5** ft
B_H = **5** ft
H = **15** ft
L = **40** ft
L_1 = **10** ft
L_2 = **10** ft
L_3 = **10** ft

KNEE BRACE SIZE (WF, Tube, or Pipe)
HSS7X0.500 Pipe

COLUMN SIZE (WF, Tube, or Pipe)
1- 3 = **W18x234** WF
ORIENTATION = **x-x**
2 - 4 = **W18x234** WF
ORIENTATION = **x-x**

BEAM SIZE (WF, Tube, or Pipe)
3 - 4 = **W24X250** WF
ORIENTATION = **x-x**

RIGID-ZONE LENGTH FACTOR: 0% of (d / 2)

LOADS (ASD)
P_L = **81** kips, (downward)
F_L = **80** kips, (horiz. to right)
M_L = **80** ft-kips, (clockwise)
P_1 = **0**
F_1 = **0**
P_2 = **79** kips, (downward)
F_2 = **13.5** kips, (horiz. to right)
P_3 = **145** kips, (downward)
F_3 = **-77** kips, (horiz. to left)

P_R = **0**
F_R = **75** kips, (horiz. to right)
M_R = **-100** ft-kips, (anticlockwise)
w = **0.2501** kips / ft, (downward)

BASE (1 & 2) PINNED ? **Yes** , (pinned)

HORIZONTAL STORY DRIFT

$\Delta_{H,3}$ = 0.6244 in, (horiz. to right) **[Satisfactory]**
$\Delta_{H,4}$ = 0.6130 in, (horiz. to right) **[Satisfactory]**
$\delta_{xe,allowable,\ ASD} = \Delta_a\ I / (1.4\ C_d)$ = 0.6429 in, *(ASCE 7-16 12.8.6)*
where C_d = 4 *,(ASCE 7 Tab 12.2-1)*
Δ_a = 0.02 h_{sx}, *(ASCE 7 Tab 12.12-1)*

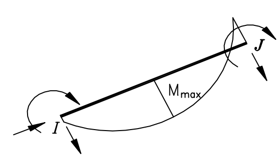
DESIGN SECTION FORCES *(ASD level)*

Knee Brace	T =	22.7	k	C =	-207.0	k,(compression)
Column	N_{13} =	-127.9	k,(compre.)	N_{24} =	-187.1	k,(compression)
	V_{13} =	11.9	kips	V_{24} =	103.4	kips
	M_{13} =	118.9	ft-kips	M_{24} =	1033.9	ft-kips
Beam	N_{34} =	-105.4	k,(compression)			
	V_{34} =	185.8	kips			
	$M_{34,\ max}$ =	792.5	ft-kips			

BEAM DEFLECTION Δ_{max} / L = 1 / 708 *, ASD level*

ANALYSIS

Joint	Coordinates (ft) X	Y	Reaction (k, ft-k) R_X	R_Y	R_M	Joint Deflection (in,deg) X	Y	θ
1	0	0	-11.89	127.94	0.00	0.000	0.000	0.175
2	40	0	103.39	187.06	0.00	0.000	0.000	0.400
3	0	15				-0.624	-0.011	0.236
4	40	15				-0.613	-0.011	-0.125
P_1/F_1	10	15				-0.620	-0.476	0.183
P_2/F_2	20	15				-0.615	-0.688	-0.011
P_3/F_3	30	15				-0.609	-0.426	-0.212



TYPICAL POSITIVE FORCES

End Joint I	J	A (in^2)	I (in^4)	E (ksi)	Length (ft)	Moment (ft-k) I	Mid	J	Axial (k)	Shear (k) I	J
1	3	68.8	4900	29000	15.00	0.00	59.45	118.89	127.94	-11.89	11.89
2	4	68.8	4900	29000	15.00	0.00	-516.95	-1033.89	187.06	103.39	-103.39
3	P_1/F_1	73.5	8490	29000	10.00	132.19	-243.28	-360.63	91.89	-45.69	45.69
P_1/F_1	P_2/F_2	73.5	8490	29000	10.00	360.63	-573.45	-792.51	91.89	-43.19	43.19
P_2/F_2	P_3/F_3	73.5	8490	29000	10.00	792.51	-597.82	-409.38	105.39	38.31	-38.31
P_3/F_3	4	73.5	8490	29000	10.00	409.38	58.28	519.69	28.39	185.81	-185.81

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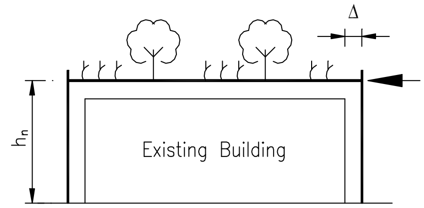
Green Roof Seismic Analysis Based on 2018 IBC, ASCE 41-17 & ASCE 7-16

DESIGN CRITERIA

1. New appurtenances cannot directly be supported by existing building roof, no matter local jurisdiction approved or not, because the original existing building not over designed for new appurtenances. The new appurtenances loads ate un-happen seismic/other capacities of existing building.
2. To save cost for both new building and existing roof, green roof appurtenances should be supported by unattached cover/sleeve of Non-Building Structure.



ANALYSIS

Determine Cover/Sleeve of Non-Building Structure Base Shear (Derived from ASCE 7-16 15.5 & 13.3)

$$V = F_p = \frac{1}{1.5} Max\left[0.3 S_{DS} I_p W_p \ , \ Min\left(\frac{0.4 a_p S_{DS} I_p W_p}{R_p}\left(1+2\frac{z}{h}\right) \ , \ 1.6 S_{DS} I_p W_p\right)\right] = \quad 0.22\ W_p\text{, ASD}$$

= 1849.7 lbs (ASD level force)

Where			
	S_{DS} =	1.121	(ASCE 7-16 11.5.4)
	I_p =	1	(ASCE 7-16 15.4.1.1)
	W_p =	8250	lbs
	a_p =	1	(ASCE 7-16 13.3.1.1)
	R_p =	2.5	(ASCE 7-16 13.3.1.1)
	z =	0	ft, (ASCE 7-16 13.3.1.1)
	$h_n = h_x = h$ =	16	ft, average height (ASCE 7-16 13.3.1.1)
		(4.9 m)	

Calculate Non-Building Vertical Forces & Allowable Building Elastic Drift (ASCE 7, Sec 12.8.3 & 12.8.6)

Level	W_x	h_x	h_x^k	$W_x h_x^k$	F_x, ASD (12.8-11)	$\delta_{xe,allowable,\ ASD}$
Green Roof	8250	16	16.0	132000	1849.7 (0.22 Wx)	0.7
	8250.0			132000	1849.7	

(1 ft = 0.305 m , 1 in = 25.4 mm , 1 kips = 4.448 kN , 1 ft-k = 1.356 kN-m)

Where
k = 1 for T <= 0.5
k = 0.5 T + 0.75 for T @ (0.5 , 2.5)
k = 2 for T >= 2.5

$\delta_{xe,allowable,\ ASD} = \Delta_a I / (1.4\ C_d)$, (ASCE 7 Sec 12.8.6)
C_d = 4 ,(ASCE 7 Tab 12.2-1)
Δ_a = 0.02 h_{sx}, (ASCE 7 Tab 12.12-1)

Calculate Green Roof Diaphragm Forces (ASCE 7, Sec 12.10.1.1)

Level	W_x	ΣW_x	F_x	ΣF_x	F_{px}, ASD, (12.10-1)
Green Roof	8250.0	8250.0	1849.7	1849.7	1849.7 (0.22 Wx)
	8250.0		1849.7		

Where $F_{min} = 0.2\ S_{DS}\ I\ W_x / 1.5$, ASD

$F_{max} = 0.4\ S_{DS}\ I\ W_x / 1.5$, ASD

Determine Gap, ASD, between Building & Non-Building Structure (ASCE 7-16 13.3.2.2 & 15.5)

$\Delta = 5\%\ h_x/1.4 + \delta_{xe,allowable,\ ASD}$ = 7.5 in

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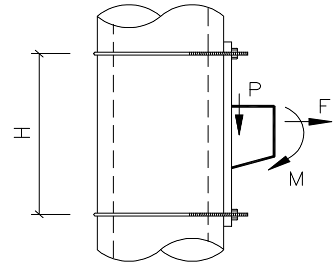
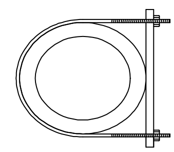
Pole Mount Clamp Design Based on ACI 318-14 & AISC 360-16





DESIGN CRITERIA

1. There are only two issues that have to be checked: One is bolt/U bar tension capacity, another is friction sliding.
2. The end plate capacities of block shear or other force do not need to be checked, because any bending deformations of the plate will change the design to friction sliding control.

INPUT DATA & DESIGN SUMMARY

LOADS (ASD)	F =	**2.1**	kips (9.5 kN)
	P =	**10.6**	kips (47.3 kN)
	M =	**30.0**	ft-kips (133.4 kN-m)
BOLT/U BAR YIELD STRESS	F_y =	**50**	ksi (345 N / mm^2)
BOLT/U BAR DIAMETER	d =	**1**	in (25 mm)
DIMENTION	H =	**10**	in (254 mm)
FRICTION COEFFICIENT [see Table below]	μ =	**0.6**	

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK BOLT/U BAR TENSION CAPACITY (AISC 360-16 D)

$\phi T_n = \phi f_y A =$ 35 kips > $T_u = 1.5\,(F/4 + 0.5\,M/H) =$ 28 kips

Where ϕ = 0.9 [Satisfactory]

A = 0.79 in^2 / Bolt (U Bar)

CHECK FRICTION SLIDING

$\phi P_n = \mu\,\phi\,(M/H) =$ 16 kips > $P_u = 1.5\,P =$ 16 kips

Where ϕ = 0.75 [Satisfactory]

COEFFICIENT OF FRICTION FOR NORMAL WEIGHT CONCRETE POLE [ACI 318-14 22.9.4.2]

	μ	
Concrete place monolithically	1.40	
Concrete placed against hardened concrete with surface intentionally roughened	1.00	
Concrete placed against hardened concrete NOT intentionally roughened	0.60	<=
Concrete anchored to as-rolled structural steel by headed stud or by reinforcing bars	0.70	

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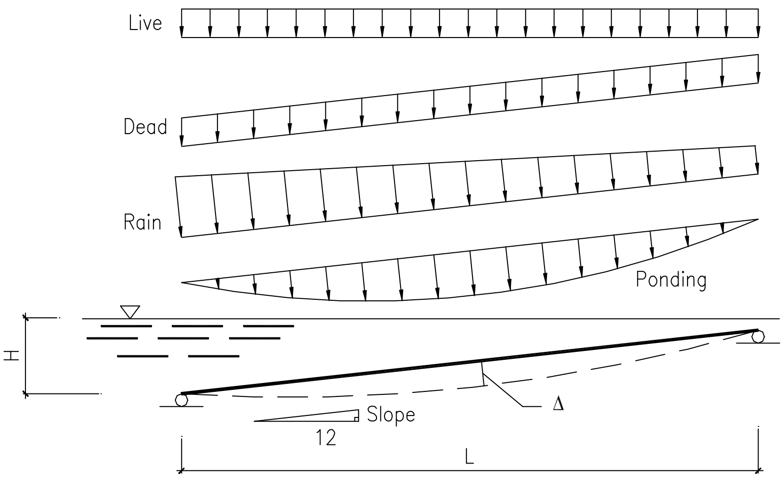
Ponding Design for Roof Beam Based on 2018 IBC, 2019 CBC, & AISC 360-16

DESIGN CRITERIA

1. Roof beam gravity design may be controlled by (Deal Load + Live Load) or (Dead Load + Rain Load plus Ponding Load) based on IBC/CBC 1605.3.2. The ponding load is from beam deflection, Δ, and contribute to the secondary bending moment.
2. The rain head, H, can be either Triangular or Trapezoid. And this analysis is not only for steel (AISC 360-16 APP. 2) but also for wood, aluminum, or other materials.



INPUT DATA & SUMMARY

DIMENSIONS $H =$ **2.5** ft
$L =$ **30** ft
Slope = **1.5** **: 12** (θ 7.13 ^{0})

BEAM SPACING/TRIBUTARY WIDTH $S =$ **8** ft

BEAM SECTION $I_x =$ **75** in^4
$E =$ **29000** ksi

DEAD LOAD (vertical, along slope) DL = **20** psf

LIVE LOAD (vertical, along horizontal) LL = **16** psf

RAIN WATER WEIGHT (perpendicular, along slope $\gamma_W =$ **62.4** pcf

Δ = 6.65 in, (perpendicular)

Beam Length = 30.23 ft, (sloped)

Design Section Forces (ASD level):

P = 1.08 kips, (axial)

M = 91.59 ft-kips, (bending)

V = 15.47 kips, (shear)

ANALYSIS

P_D = (DL S) L Sin θ / Cos θ = 0.60 kips

M_D = (DL S) L^2 / (8 Cos θ) = 18.14 ft-kips

V_D = 0.5 (DL S) L = 2.40 kips

Δ_D = 5 (DL S) L^4 / (Cos^3 θ 384 E Ix) = 1.37 in

P_L = (LL S) L Sin θ = 0.48 kips

M_L = (LL S) L^2 / 8 = 14.40 ft-kips

V_L = 0.5 (LL S) L Cos θ = 1.91 kips

Δ_L = 5 (LL S) L^4 / (Cos^2 θ 384 E Ix) = 1.09 in

		Dead	Dead + Rain + Ponding						
		Rain	1 st	2 nd	3 rd	4 th	5 th	...	Final
P_R	kips	0.60	0.60	0.60	0.60	0.60	0.60		0.60
M_R	ft-kips	70.07	86.33	90.35	91.30	91.52	91.57		91.59
V_R	kips	13.01	14.88	15.33	15.44	15.46	15.47		15.47
Δ_R	in	5.09	6.27	6.56	6.63	6.65	6.65		6.65

> $\Delta_D + \Delta_L$

(Rain & Ponding Loads Control.)

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Typical Truss Analysis by Finite Element Method Based on 2018 IBC / 2019 CBC

DESIGN CRITERIA

1. The I_x & A of Web and/or Post should be input zero for King Post or Triangle truss.
2. The loads w_1, w_2, w_3, and w_4 may be negative (outward) based on IBC/CBC 1605 wind/seismic load combination results.

INPUT DATA & DESIGN SUMMARY

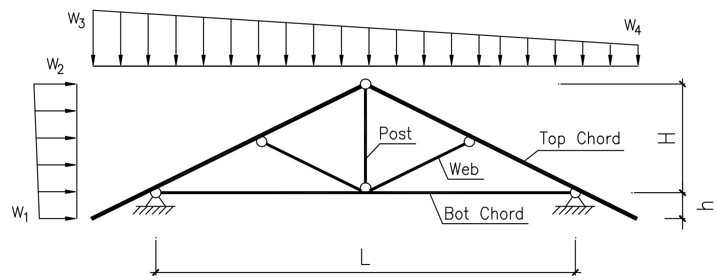
LOADS

w_1 = **0.3** *kips / ft, (4.4 kN / m)*

w_2 = **0.5** *kips / ft, (7.3 kN / m)*

w_3 = **0.2** *kips / ft, (2.9 kN / m)*

w_4 = **-0.5** *kips / ft, (-7.3 kN / m)*



DIMENSIONS

H = **10** *ft, (3.05 m)*

h = **3** *ft, (0.91 m)*

L = **60** *ft, (18.29 m)*

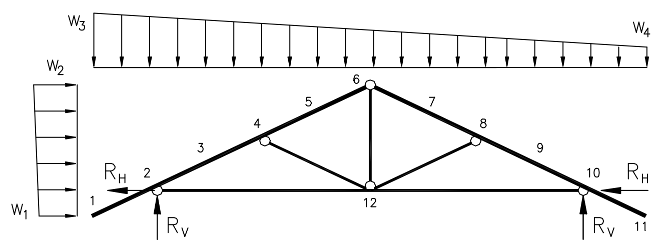
	A (in^2)	I_x (in^4)	E (ksi)	L (ft)	k L_x (ft)	Max. Section Force N (kips)	V (kips)	M(ft-kips)
Top Chord	**76.9**	**1441**	**1800**	41.11	15.81	9.86	2.86	21.63
Bottom Chord	**53.8**	**494**	**1800**	60.00	30.00	4.82	0.02	0.26
Post	**30.3**	**76**	**1600**	10.00	10.00	1.83	0	0
Web	**30.3**	**76**	**1600**	15.81	15.81	-4.53	0	0

<== Tension (Web)

(Note: Sine statically indeterminate pinned supports, the bottom chord axial force is based on reactions.)

ANALYSIS

No. Joint	Location X (ft)	Y (ft)	Drift (in) Δx	Δy
1	-39.00	-13.00	0.25	-0.76
2	-30.00	-10.00	**0**	**0**
3	-22.50	-7.50	-0.01	0.00
4	-15.00	-5.00	0.02	-0.10
5	-7.50	-2.50	0.02	-0.13
6	**0**	**0**	0.00	-0.07
7	7.50	-2.50	-0.01	-0.07
8	15.00	-5.00	-0.01	-0.06
9	22.50	-7.50	-0.02	-0.07
10	30.00	-10.00	**0**	**0**
11	39.00	-13.00	0.11	0.34
12	0.00	-10.00	0.01	-0.08



R_{H2} = -4.82 *kips* R_{H10} = 10.02 *kips*

R_{V2} = -11.45 *kips* R_{V10} = -0.25 *kips*

No. Element	Joint Start, *I*	End, *J*	*I* Sec. Forces (kips, ft-kips) Axial	Shear	Moment	*J* Sec. Forces (kips, ft-kips) Axial	Shear	Moment
1	1	2	0.00	0.00	0.00	-0.01	2.28	-21.63
2	2	3	9.71	2.86	21.63	-9.71	-2.86	0.97
3	3	4	9.86	0.03	-0.97	-9.86	-0.03	1.18
4	4	5	6.45	0.42	-1.18	-6.45	-0.42	4.54
5	5	6	6.74	-1.39	-4.54	-6.74	1.39	0.00
6	6	7	7.53	0.25	0.00	-7.53	-0.25	1.17
7	7	8	7.72	-0.34	-1.17	-7.72	0.34	-1.52
8	8	9	8.86	0.37	1.52	-8.86	-0.37	1.42
9	9	10	8.73	0.74	-1.42	-8.73	-0.74	7.28
10	10	11	0.26	-0.77	-7.28	0.00	0.00	0.00
11	2	12	-1.50	0.02	0.00	1.50	-0.02	0.26
12	10	12	1.50	-0.02	0.00	-1.50	0.02	-0.26
13	4	12	4.53	0.00	0.00	-4.53	0.00	0.00
14	6	12	-1.83	0.00	0.00	1.83	0.00	0.00
15	8	12	-1.37	0.00	0.00	1.37	0.00	0.00

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Fink Truss Analysis by Finite Element Method Based on 2018 IBC / 2019 CBC

DESIGN CRITERIA

1. The I_x & A of Webs may be input zero for Triangle truss.
2. The loads w_1, w_2, w_3, and w_4 may be negative (outward) based on IBC/CBC 1605 wind/seismic load combination results.

INPUT DATA & DESIGN SUMMARY

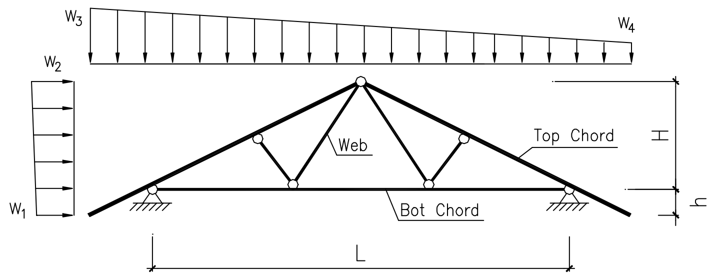
LOADS

w_1 = **0.3** kips / ft, (4.4 kN / m)

w_2 = **0.5** kips / ft, (7.3 kN / m)

w_3 = **0.2** kips / ft, (2.9 kN / m)

w_4 = **-0.5** kips / ft, (-7.3 kN / m)



DIMENSIONS

H = **10** ft, (3.05 m)

h = **3** ft, (0.91 m)

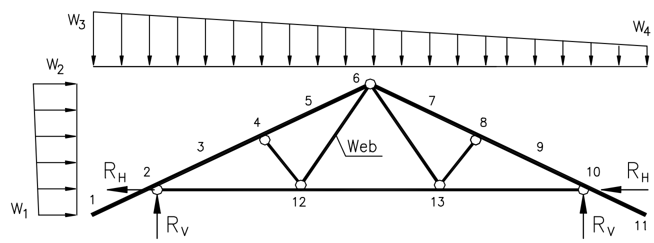
L = **60** ft, (18.29 m)

	A (in^2)	I_x (in^4)	E (ksi)	L (ft)	k L_x (ft)	Max. Section Force N (kips)	V (kips)	M(ft-kips)
Top Chord	**76.9**	**1441**	**1800**	41.11	15.81	9.82	2.85	21.63
Bottom Chord	**53.8**	**494**	**1800**	60.00	20.00	3.84	0.04	0.34
Web (4-12 & 8-13)	**30.3**	**76**	**1600**	7.07	7.07	3.06	0	0
Web (6-12 & 6-13)	**30.3**	**76**	**1600**	14.14	14.14	2.99	0	0

(Note: Sine statically indeterminate pinned supports, the bottom chord axial force is based on reactions.)

ANALYSIS

No. Joint	Location X (ft)	Y (ft)	Drift (in) Δx	Δy
1	-39.00	-13.00	0.26	-0.77
2	-30.00	-10.00	**0**	**0**
3	-22.50	-7.50	-0.01	0.01
4	-15.00	-5.00	0.02	-0.10
5	-7.50	-2.50	0.02	-0.13
6	**0**	**0**	0.00	-0.08
7	7.50	-2.50	-0.01	-0.08
8	15.00	-5.00	-0.01	-0.07
9	22.50	-7.50	-0.02	-0.07
10	30.00	-10.00	**0**	**0**
11	39.00	-13.00	0.12	0.36
12	-10.00	-10.00	0.01	-0.10
13	10.00	-10.00	0.00	-0.08



R_{H2} = -3.84 kips R_{H10} = 9.04 kips

R_{V2} = -11.45 kips R_{V10} = -0.25 kips

No. Element	Joint Start, *I*	End, *J*	*I* Sec. Forces (kips, ft-kips) Axial	Shear	Moment	*J* Sec. Forces (kips, ft-kips) Axial	Shear	Moment
1	1	2	0.00	0.00	0.00	-0.01	2.28	-21.63
2	2	3	9.67	2.85	21.63	-9.67	-2.85	0.93
3	3	4	9.82	0.02	-0.93	-9.82	-0.02	1.09
4	4	5	8.67	0.44	-1.09	-8.67	-0.44	4.56
5	5	6	8.96	-1.37	-4.56	-8.96	1.37	0.00
6	6	7	8.14	0.28	0.00	-8.14	-0.28	1.17
7	7	8	8.34	-0.31	-1.17	-8.34	0.31	-1.30
8	8	9	8.76	0.36	1.30	-8.76	-0.36	1.53
9	9	10	8.64	0.73	-1.53	-8.64	-0.73	7.28
10	10	11	0.26	-0.77	-7.28	0.00	0.00	0.00
11	2	12	-2.45	0.04	0.00	2.45	-0.04	0.34
12	12	13	1.83	-0.01	-0.34	-1.83	0.01	0.12
13	10	13	0.61	-0.02	0.00	-0.61	0.02	-0.12
14	4	12	3.06	0.00	0.00	-3.06	0.00	0.00
15	6	12	2.99	0.00	0.00	-2.99	0.00	0.00
16	6	13	-0.85	0.00	0.00	0.85	0.00	0.00
17	8	13	-0.87	0.00	0.00	0.87	0.00	0.00

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Howe Truss Analysis by Finite Element Method Based on 2018 IBC / 2019 CBC

DESIGN CRITERIA

1. The I_x & A of Web, Cripple and/or Post may be input zero for Queen Post, King Post or Triangle truss.
2. The loads w_1, w_2, w_3, and w_4 may be negative (outward) based on IBC/CBC 1605 wind/seismic load combination results.

INPUT DATA & DESIGN SUMMARY

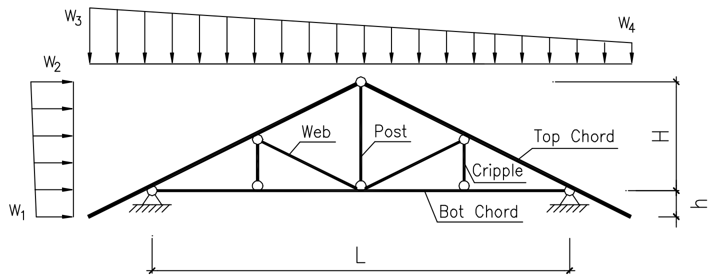
LOADS

w_1 = 0.3 kips / ft, (4.4 kN / m)

w_2 = 0.5 kips / ft, (7.3 kN / m)

w_3 = 0.2 kips / ft, (2.9 kN / m)

w_4 = -0.5 kips / ft, (-7.3 kN / m)



DIMENSIONS

H = 10 ft, (3.05 m)

h = 3 ft, (0.91 m)

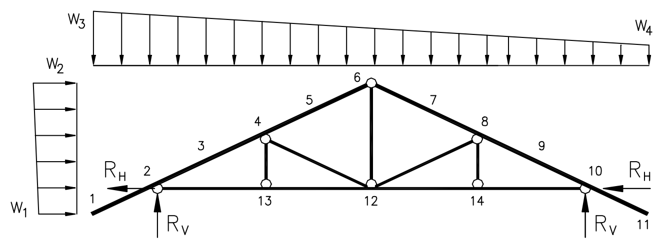
L = 60 ft, (18.29 m)

	A (in^2)	I_x (in^4)	E (ksi)	L (ft)	k L_x (ft)	Max. Section Force N (kips)	V (kips)	M(ft-kips)	
Top Chord	76.9	1441	1800	41.11	15.81	9.60	2.86	21.63	
Bottom Chord	53.8	494	1800	60.00	30.00	4.63	0.11	0.68	
Post	30.3	76	1600	10.00	10.00	1.84	0	0	
Cripple	30.3	76	1600	5.00	5.00	0.17	0	0	
Web	30.3	76	1600	15.81	15.81	-4.28	0	0	<== Tension

(Note: Sine statically indeterminate pinned supports, the bottom chord axial force is based on reactions.)

ANALYSIS

No. Joint	Location X (ft)	Y (ft)	Drift (in) Δx	Δy
1	-39.00	-13.00	0.26	-0.77
2	-30.00	-10.00	0	0
3	-22.50	-7.50	-0.01	0.01
4	-15.00	-5.00	0.02	-0.09
5	-7.50	-2.50	0.02	-0.13
6	0	0	0.00	-0.07
7	7.50	-2.50	-0.01	-0.07
8	15.00	-5.00	-0.01	-0.06
9	22.50	-7.50	-0.02	-0.06
10	30.00	-10.00	0	0
11	39.00	-13.00	0.11	0.34
12	0.00	-10.00	0.01	-0.07
13	-15.00	-10.00	0.00	-0.09
14	15.00	-10.00	0.00	-0.06



R_{H2} = -4.63 kips

R_{V2} = -11.45 kips

R_{H10} = 9.83 kips

R_{V10} = -0.25 kips

No. Element	Joint Start, ***I***	End, ***J***	***I*** Sec. Forces (kips, ft-kips) Axial	Shear	Moment	***J*** Sec. Forces (kips, ft-kips) Axial	Shear	Moment
1	1	2	0.00	0.00	0.00	-0.01	2.28	-21.63
2	2	3	9.45	2.86	21.63	-9.45	-2.86	0.94
3	3	4	9.60	0.02	-0.94	-9.60	-0.02	1.12
4	4	5	6.45	0.43	-1.12	-6.45	-0.43	4.54
5	5	6	6.74	-1.38	-4.54	-6.74	1.38	0.00
6	6	7	7.52	0.24	0.00	-7.52	-0.24	1.17
7	7	8	7.72	-0.34	-1.17	-7.72	0.34	-1.55
8	8	9	8.71	0.37	1.55	-8.71	-0.37	1.41
9	9	10	8.59	0.74	-1.41	-8.59	-0.74	7.28
10	10	11	0.26	-0.77	-7.28	0.00	0.00	0.00
11	2	13	-1.45	0.11	0.00	1.45	-0.11	0.68
12	10	14	1.45	-0.06	0.00	-1.45	0.06	-0.32
13	4	12	4.28	0.00	0.00	-4.28	0.00	0.00
14	6	12	-1.84	0.00	0.00	1.84	0.00	0.00
15	8	12	-1.23	0.00	0.00	1.23	0.00	0.00
16	12	13	-1.45	-0.06	-0.24	1.45	0.06	-0.68
17	12	14	1.45	0.04	0.24	-1.45	-0.04	0.32
18	4	13	0.17	0.00	0.00	-0.17	0.00	0.00
19	8	14	0.09	0.00	0.00	-0.09	0.00	0.00

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Attic Truss Analysis by Finite Element Method Based on 2018 IBC / 2019 CBC

DESIGN CRITERIA

1. The I_x & A of Header, Web, and/or Cripple may be input zero for Triangle trusses.
2. The loads $\boldsymbol{w_1}$, $\boldsymbol{w_2}$, $\boldsymbol{w_3}$, and $\boldsymbol{w_4}$ may be negative (outward) based on IBC/CBC 1605 wind/seismic load combination results.

INPUT DATA & DESIGN SUMMARY

LOADS

w_1 = **0.3** *kips / ft, (4.4 kN / m)*

w_2 = **0.5** *kips / ft, (7.3 kN / m)*

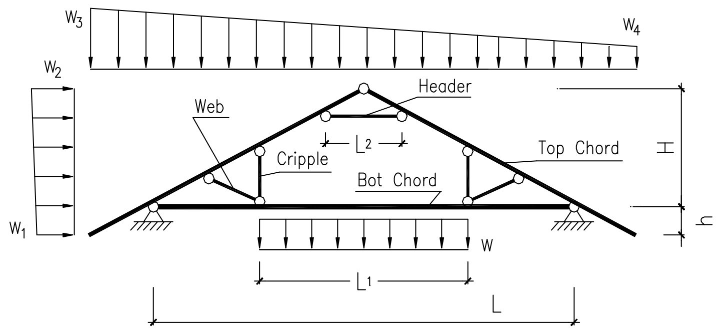
w_3 = **0.2** *kips / ft, (2.9 kN / m)*

w_4 = **-0.5** *kips / ft, (-7.3 kN / m)*

w = **0.8** *kips / ft, (11.7 kN / m)*

DIMENSIONS

H =	**10**	*ft, (3.05 m)*	L_1 =	**35**	*ft, (10.67 m)*
h =	**3**	*ft, (0.91 m)*	L_2 =	**5**	*ft, (1.52 m)*
L =	**60**	*ft, (18.29 m)*			

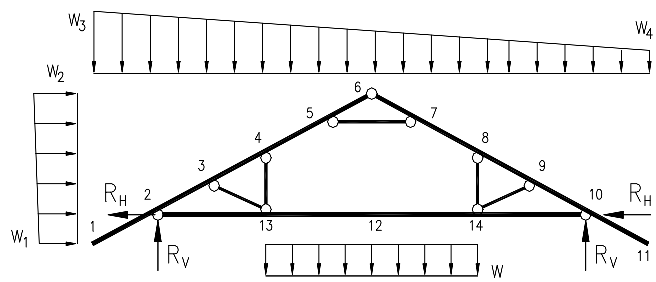


	A (in^2)	I_x (in^4)	E (ksi)	L (ft)	k L_x (ft)	Max. Section Force N (kips)	V (kips)	M(ft-kips)	
Top Chord	**76.9**	**1441**	**1800**	41.11	15.81	39.12	10.55	66.38	
Bottom Chord	**76.9**	**1441**	**1800**	60.00	27.50	-28.82	7.15	86.90	<== Tension
Header	**30.3**	**76**	**1600**	5.00	5.00	33.25	0	0	
Cripple	**30.3**	**76**	**1600**	4.17	4.17	-20.92	0	0	<== Tension
Web	**30.3**	**76**	**1600**	6.59	6.59	19.88	0	0	

(Note: Sine statically indeterminate pinned supports, the bottom chord axial force is based on reactions.)

ANALYSIS

No.	Location		Drift (in)	
Joint	X (ft)	Y (ft)	Δx	Δy
1	-39.00	-13.00	0.70	-2.09
2	-30.00	-10.00	**0**	**0**
3	-23.75	-7.92	-0.31	1.00
4	-17.50	-5.83	-0.55	1.76
5	-2.50	-0.83	-0.01	0.23
6	**0**	**0**	0.00	0.16
7	2.50	-0.83	0.03	0.24
8	17.50	-5.83	0.62	1.95
9	23.75	-7.92	0.34	1.07
10	30.00	-10.00	**0**	**0**
11	39.00	-13.00	-0.43	-1.28
12	0.00	-10.00	0.00	6.61
13	-17.50	-10.00	-0.01	1.78
14	17.50	-10.00	0.01	1.97



R_{H2} = 28.82 *kips* R_{H10} = -23.62 *kips*

R_{V2} = 2.44 *kips* R_{V10} = 13.86 *kips*

No.	Joint		***I*** Sec. Forces (kips, ft-kips)			***J*** Sec. Forces (kips, ft-kips)		
Element	Start, ***I***	End, ***J***	Axial	Shear	Moment	Axial	Shear	Moment
1	1	2	0.00	0.00	0.00	-0.01	2.28	-21.63
2	2	3	-39.12	4.27	21.63	39.12	-4.27	6.52
3	3	4	-23.10	-10.09	-6.52	23.10	10.09	-59.94
4	4	5	-16.18	6.44	59.94	16.18	-6.44	41.81
5	5	6	15.72	-6.05	-41.81	-15.72	6.05	0.00
6	6	7	16.44	4.28	0.00	-16.44	-4.28	45.28
7	7	8	-14.83	-7.06	-45.28	14.83	7.06	-66.38
8	8	9	-20.70	10.55	66.38	20.70	-10.55	3.10
9	9	10	-34.54	0.63	-3.10	34.54	-0.63	7.28
10	10	11	0.26	-0.77	-7.28	0.00	0.00	0.00
11	2	13	11.54	0.48	0.00	-11.54	-0.48	38.27
12	10	14	8.95	0.57	0.00	-8.95	-0.57	-32.94
13	4	13	20.92	0.00	0.00	-20.92	0.00	0.00
14	5	7	-33.25	0.00	0.00	33.25	0.00	0.00
15	8	14	18.69	0.00	0.00	-18.69	0.00	0.00
16	12	13	-7.32	-7.15	-86.90	7.32	7.15	-38.27
17	12	14	-7.32	6.85	86.90	7.32	-6.85	32.94
18	3	13	-19.88	0.00	0.00	19.88	0.00	0.00
19	9	14	17.14	0.00	0.00	-17.14	0.00	0.00

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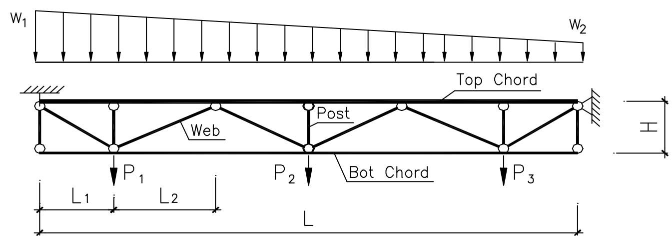
FLat Truss Analysis by Finite Element Method Based on 2018 IBC / 2019 CBC

DESIGN CRITERIA

1. The truss is symmetric on central vertical line. But the spacings, L_1, L_2, and $(0.5L - L_1 - L_2)$ may be different.
2. The loads $\boldsymbol{w_1}$, & $\boldsymbol{w_2}$ may be negative (outward), for flat roof truss, based on IBC/CBC 1605 wind load combination results.

INPUT DATA & DESIGN SUMMARY

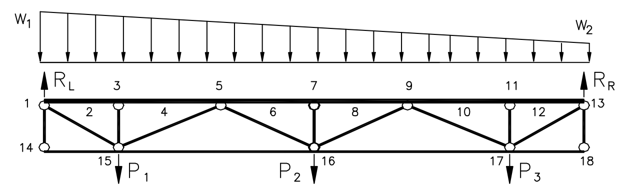
LOADS	w_1 =	**5**	kips / ft, (72.9 kN / m)
	w_2 =	**8**	kips / ft, (116.7 kN / m)
	P_1 =	**1.5**	kips, (6.7 kN)
	P_2 =	**1.3**	kips, (5.8 kN)
	P_3 =	**0.8**	kips, (3.6 kN)
DIMENSIONS			
	H =	**2**	ft, (0.61 m)
	L =	**24**	ft, (7.32 m)
	L_1 =	**3.5**	ft, (1.07 m)
	L_2 =	**4.25**	ft, (1.30 m)



	A (in^2)	I_x (in^4)	E (ksi)	L (ft)	k L_x (ft)	Max. Section Force N (kips)	V (kips)	M(ft-kips)	
Top Chord	**76.9**	**1441**	**1800**	24.00	4.25	130.81	45.17	58.40	
Bottom Chord	**53.8**	**494**	**1800**	24.00	8.50	-105.89	15.05	13.74	<== Tension
Web (diagonal)	**30.3**	**76**	**1600**	4.70	4.70	38.27	0	0	
Post (vertical)	**30.3**	**76**	**1600**	2.00	2.00	27.60	0	0	

ANALYSIS

No. Joint	Location X (ft)	Y (ft)	Drift (in) Δx	Δy
1	**0**	**0**	-0.14	**0**
2	1.75	0.00	-0.13	0.12
3	3.50	0.00	-0.13	0.37
4	5.63	0.00	-0.12	0.67
5	7.75	0.00	-0.12	0.91
6	9.88	0.00	-0.09	1.08
7	12.00	0.00	-0.07	1.13
8	14.13	0.00	-0.05	1.10
9	16.25	0.00	-0.02	0.94
10	18.38	0.00	-0.02	0.71
11	20.50	0.00	-0.01	0.39
12	22.25	0.00	-0.01	0.13
13	24.00	0.00	**0**	**0**
14	0.00	-2.00	0.04	0.01
15	3.50	-2.00	0.04	0.37
16	12.00	-2.00	-0.07	1.12
17	20.50	-2.00	-0.18	0.39
18	24.00	-2.00	-0.18	0.01



Reactions:

R_L = **73.49** *kips* R_R = **86.11** *kips*

No. Element	Joint Start, *I*	End, *J*	*I* Sec. Forces (kips, ft-kips) Axial	Shear	Moment	*J* Sec. Forces (kips, ft-kips) Axial	Shear	Moment
1	1	2	-28.06	-39.27	0.00	28.06	39.27	57.50
2	2	3	-28.06	-30.13	-57.50	28.06	30.13	4.77
3	3	4	-28.06	-13.04	-4.77	28.06	13.04	-22.95
4	4	5	-28.06	-0.93	22.95	28.06	0.93	-24.91
5	5	6	-130.81	-6.91	24.91	130.81	6.91	-39.60
6	6	7	-130.81	6.34	39.60	130.81	-6.34	-26.13
7	7	8	-130.81	-7.45	26.13	130.81	7.45	-41.95
8	8	9	-130.81	6.93	41.95	130.81	-6.93	-27.22
9	9	10	-33.22	-0.60	27.22	33.22	0.60	-28.50
10	10	11	-33.22	14.90	28.50	33.22	-14.90	3.17
11	11	12	-33.22	31.56	-3.17	33.22	-31.56	58.40
12	12	13	-33.22	45.17	-58.40	33.22	-45.17	0.00
13	14	15	0.00	-13.72	0.00	0.00	13.72	-3.09
14	15	16	99.27	-1.25	3.09	-99.27	1.25	-13.74
15	16	17	105.89	1.08	13.74	-105.89	-1.08	-4.57
16	17	18	0.00	15.05	4.57	0.00	-15.05	0.00
17	1	14	13.72	0.00	0.00	-13.72	0.00	0.00
18	3	15	6.51	0.00	0.00	-6.51	0.00	0.00
19	7	16	-27.60	0.00	0.00	27.60	0.00	0.00
20	11	17	2.05	0.00	0.00	-2.05	0.00	0.00
21	13	18	15.05	0.00	0.00	-15.05	0.00	0.00
22	1	15	32.32	0.00	0.00	-32.32	0.00	0.00
23	5	15	78.70	0.00	0.00	-78.70	0.00	0.00
24	5	16	34.85	0.00	0.00	-34.85	0.00	0.00
25	9	16	-27.53	0.00	0.00	27.53	0.00	0.00
26	9	17	-80.31	0.00	0.00	80.31	0.00	0.00
27	13	17	-38.27	0.00	0.00	38.27	0.00	0.00

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Scissor Truss Analysis by Finite Element Method Based on 2018 IBC / 2019 CBC

DESIGN CRITERIA

1. The fundamental feature of scissor truss is big horizontal reactions directly from vertical loads (arch action), which may not supported by perpendicular wall/column. So the option to release horizontal arch action from vertical loads is conservative design.
2. The loads ***w*** w_1, w_2, w_3, and w_4 may be negative (outward) based on IBC/CBC 1605 wind/seismic load combination results.

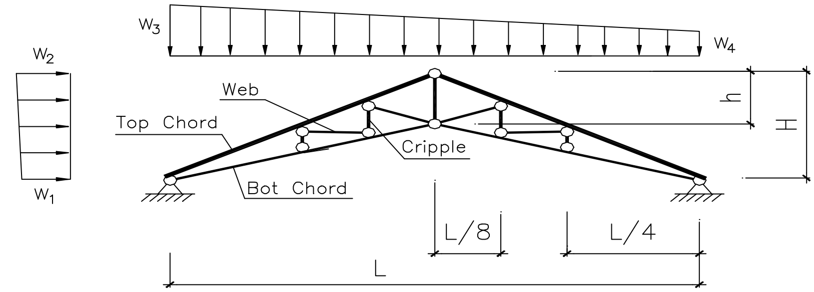
INPUT DATA & DESIGN SUMMARY

LOADS

w_1 =	0.3	*kips / ft, (4.4 kN / m)*
w_2 =	0.5	*kips / ft, (7.3 kN / m)*
w_3 =	0.2	*kips / ft, (2.9 kN / m)*
w_4 =	-0.5	*kips / ft, (-7.3 kN / m)*

DIMENSIONS

H =	20	*ft, (6.10 m)*
h =	10	*ft, (3.05 m)*
L =	80	*ft, (24.38 m)*

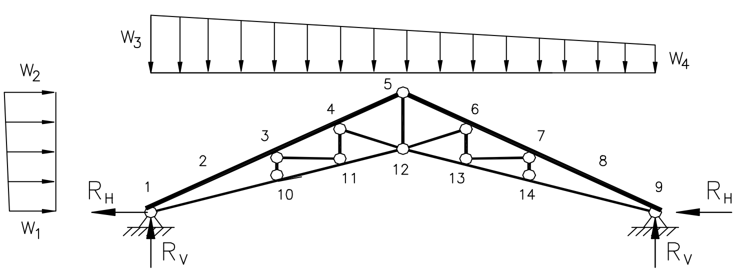


	A (in^2)	I_x (in^4)	E (ksi)	L (ft)	k L_x (ft)	Max. Section Force N (kips)	V (kips)	M(ft-kips)	
Top Chord	76.9	1441	1800	44.72	22.36	10.69	3.96	34.40	
Bottom Chord (Steel)	35	1880	29000	41.23	20.62	-6.74	3.82	29.95	<== Tension
Vertical Cripple	30.3	76	1600	10.00	10.00	6.61	0	0	
Web	30.3	76	1600	11.18	11.18	-7.16	0	0	<== Tension

RELEASE HORIZONAL ARCH ACTION FROM VERTICAL LOADS? (1 = Yes, 2 = No) 1 *Yes, released.*

ANALYSIS

No. Joint	Location X (ft)	Y (ft)	Drift (in) Δx	Δy
1	**0**	**0**	**0**	**0**
2	10.00	5.00	0.10	-0.19
3	20.00	10.00	-0.04	0.10
4	30.00	15.00	-0.06	0.17
5	40.00	20.00	-0.05	0.17
6	50.00	15.00	-0.04	0.17
7	60.00	10.00	-0.04	0.16
8	70.00	5.00	0.09	0.40
9	80.00	0.00	-0.10	0.00
10	20.00	5.00	-0.03	0.10
11	30.00	7.50	-0.04	0.16
12	40.00	10.00	-0.05	0.18
13	50.00	7.50	-0.05	0.18
14	60.00	5.00	-0.06	0.15



$R_{H,1}$ = 4.00 *kips* $R_{H,9}$ = 4.00 *kips*

$R_{V,1}$ = -0.94 *kips* $R_{V,9}$ = -11.06 *kips*

No. Element	Joint Start, *I*	End, *J*	*I* Sec. Forces (kips, ft-kips) Axial	Shear	Moment	*J* Sec. Forces (kips, ft-kips) Axial	Shear	Moment
1	1	2	-6.74	-1.96	0.00	6.74	1.96	-9.66
2	2	3	-5.68	-0.17	9.66	5.68	0.17	-11.59
3	3	4	-9.73	-3.12	-28.41	9.73	3.12	-6.51
4	4	5	-8.77	0.87	6.51	8.77	-0.87	0.00
5	5	6	-6.35	1.32	0.00	6.35	-1.32	9.84
6	6	7	-8.84	-3.96	-9.84	8.84	3.96	-34.40
7	7	8	-8.75	0.27	-12.26	8.75	-0.27	15.24
8	8	9	-10.69	-3.42	-15.19	10.69	3.42	0.00
9	1	10	3.50	1.35	0.00	-3.50	-1.35	3.43
10	10	11	3.66	0.71	-3.43	-3.66	-0.71	10.73
11	11	12	10.84	0.61	-10.73	-10.84	-0.61	0.00
12	12	13	10.49	0.16	0.00	-10.49	-0.16	3.47
13	13	14	12.96	2.57	-3.47	-12.96	-2.57	29.95
14	14	9	-6.74	-3.82	-29.67	-10.36	3.82	0.00
15	3	10	-0.66	0.00	0.00	0.66	0.00	0.00
16	3	11	7.16	0.00	0.00	-7.16	0.00	0.00
17	4	11	-3.57	0.00	0.00	3.57	0.00	0.00
18	4	12	-0.44	0.00	0.00	0.44	0.00	0.00
19	5	12	4.86	0.00	0.00	-4.86	0.00	0.00
20	6	12	-0.15	0.00	0.00	0.15	0.00	0.00
21	6	13	3.39	0.00	0.00	-3.39	0.00	0.00
22	7	13	1.87	0.00	0.00	-1.87	0.00	0.00
23	7	14	-6.61	0.00	0.00	6.61	0.00	0.00

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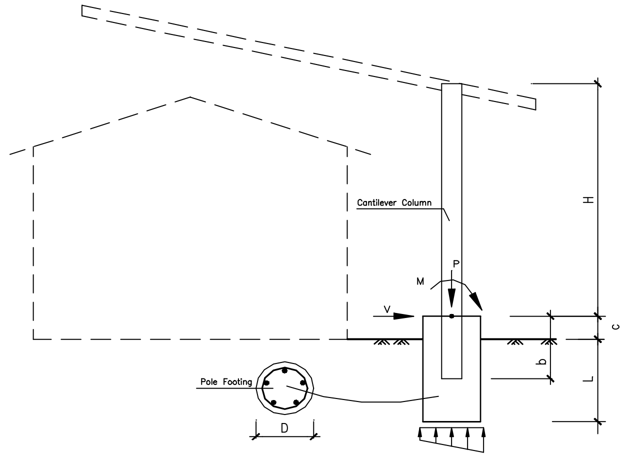
Solar Carport Pole & Footing Design Based on AISC 360-16, ACI 318-19, and 2018 IBC 1807.3



DESIGN CRITERIA

1. If solar carport covered existing building without touched, the existing building may not have to be brought to current IBC/CBC design.
2. Since the pole is cantilever column, so the maximum P, V, & M (*ASD level, wind upward may control*) at the footing top, and the design length is doubled ***2H***.
3. Since the footing is supported con-currently by lateral (*embedded, 2018 IBC 1807.3,* ***V*** *&* ***M₁*** *coupled*) and bottom soil (bearing, *ACI 318-19 13.3,* ***P*** *&* ***M₂*** *coupled*), the (***M₁ + M₂***) should be equals to total ***M***, and ***M₁*** has to be determined by iterating.

INPUT DATA & DESIGN SUMMARY

COLUMN SECTION (Tube, Pipe, or WF)		HSS8X8X5/8	**Tube**
COLUMN YIELD STRESS	F_y =	46	*ksi, (317 MPa)*
CANTILEVER HEIGHT	H =	15	*ft, (4.57 m)*
COLUMN MAX SHEAR	V =	3.2	*kips, (14 kN)*
(Strong Axis Bending only)			
COLUMN MAX AXIAL LOAD	P =	11.2	*kips, (50 kN)*
COLUMN MAX MOMENT	M =	91	*ft-kips, (123 kN-m)*
DIAMETER OF POLE FOOTING	D =	6	*ft, (1.83 m)*
ALLOW SOIL PRESSURE	Q_a =	5.6	*ksf, (268 kPa)*
LATERAL SOIL CAPACITY	P_P =	0.35	*ksf / ft, (55 kPa / m)*
RESTRAINED @ GRADE ?(1=yes,0=no)		1	Yes

THE DESIGN IS ADEQUATE.

DIMENTION L = 3 *ft, (0.91 m)* b = 2.5 *ft, (0.76 m), concrete embedment depth*

c = 12 *in, (305 mm)* *(anchor bolts are always inadequate.)*

CONCRETE f_c' = 4.5 *ksi, (31 MPa)*

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY OF COLUMN (AISC 360 H1)

$$\begin{cases} \dfrac{P_r}{P_c}+\dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) , & for \ \dfrac{P_r}{P_c}\geq 0.2 \\ \dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right) , & for \ \dfrac{P_r}{P_c}< 0.2 \end{cases} = \quad 0.92 \quad < \quad 1.0 \quad \textbf{[Satisfactory]}$$

Where P_r = 11.20 kips, M_{ry} = 0 ft-kips

M_{rx} = 91.00 ft-kips

$L_{c,y} = KL_y$ = ***30.00*** ft, weak axis unbraced axial length

$P_c = P_n / \Omega_c$ = 283 / 1.67 = 169.33 kips, (AISC 360 Chapter E)

> P_r **[Satisfactory]** (P_c / P_r) = 0.07 < 0.15 , (AISC 341 E5.4a & E6.4a)

$M_{cx} = M_n / \Omega_b$ = 171.35 / 1.67 = 102.60 ft-kips, (AISC 360 Chapter F)

> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 171.35 / 1.67 = 102.60 ft-kips > M_{ry} **[Satisfactory]**

DETERMINE SOIL MOMENT CAPACITY, ***M₁***, UNDER ***V*** & ***L*** (2018 IBC 1807.3)

Pole depth, d = L = 3.00 ft

Lateral bearing @ bottom, $S_3 = 2\, P_P$ *Min(d , 12')* = 2.10 ksf

Lateral bearing @ $d / 3$, $S_1 = 2\, P_P$ *Min(d / 3 , 12')* = 0.70 ksf

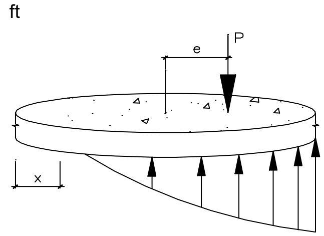
Require Depth is given by

$$d = L = \begin{cases} \dfrac{A}{2}\left[1+\sqrt{1+\dfrac{4.36h}{A}}\right] & for \ \ nonconstrained \\ \sqrt{\dfrac{4.25Ph}{bS_3}} & for \ \ constrained \end{cases} = \quad 3.0 \text{ ft} \quad \textbf{[Satisfactory]}$$

Where M_1 = 27.54 ft-kips $A = 2.34\, P / (b\, S_1)$ = 1.28

$P = V$ = 3.20 kips $h = M_1 / P + c$ = 8.61 ft

CHECK VERTICAL SOIL BEARING CAPACITY (ACI 318-19 13.3)



M_2 = *Max(* M - M_1 *, 0)* = 63.46 ft-kips

P = 11.20 kips

$e = M_2 / P$ = 5.67 ft

q_{soil} = 5.58 ksf, *(net weight of concrete footing included.)*

< Q_a **[Satisfactory]** , x = 2.65 ft

CHECK FIXED MOMENT CONDITION AT ***e*** CONCRETE EMBEDMENT DEPTH (ACI 318-19 21 & 22)

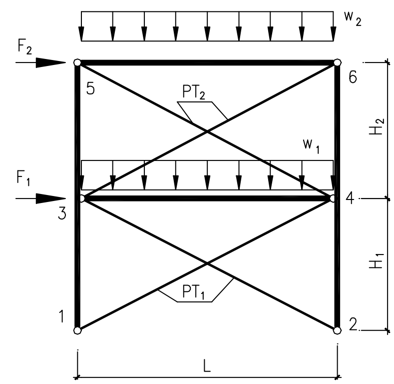
1.5 M = 136.50 ft-kips, *(LRFD level)* < $\phi\, M_n$ = 560.77 ft-kips **[Satisfactory]**

Tension-Only Braced Frame Analysis using Finite Element Method

DESIGN CRITERIA

1. The tension-only brace is easier to construct due to its light weight and simple connection details. Therefore, it is suitable for both new constructions and seismic retrofits. The beam and column can be steel, wood, and/or concrete. But the tension-only brace has to be steel cable/rod with prestressed tension forces (PT).
2. The PT are always required to reduce story drifts, because the tension-only brace, without PT, cannot be lateral frame member. The beam & column design has to include both the primary section forces and the secondary section forces. The secondary section forces are very important, which cannot directly be gotten, from 3D/2D FEM analysis with iteration, by input internal PT as external loads.

INPUT DATA & SUMMARY



DIMENSIONS

H_1 = 14 *ft, (4.3 m)* L = 20 *ft, (6.1 m)*

H_2 = 12 *ft, (3.7 m)*

LOADS

F_1 = 10 kips, (horiz. to right) w_1 = 1.1 kips/ft

F_2 = 11 kips, (horiz. to right) w_2 = 0.8 kips/ft

MEMBERS

	A (in^2)	I_x (in^4)	E (ksi)
Column (1 - 5)	13.3	248	29000
Column (2 - 6)	14	195	29000
Beam (3 - 4)	6.49	199	29000
Beam (5 - 6)	360	12000	1800

	A (in^2)	E (ksi)	PT (kips)
PT_1	0.459	29950	16.524
PT_2	0.306	29950	11.016

STORY DRIFTS

Δ_3 = ***0.35*** in, (horiz. to right)

Δ_5 = ***0.23*** in, (horiz. to right)

Δ_4 = ***0.32*** in, (horiz. to right)

Δ_6 = ***0.26*** in, (horiz. to right)

Reactions

$R_{1,X}$ = ***-9.80*** in, (horiz. to left)

$R_{1,Y}$ = ***-2.30*** in, (downward)

$R_{2,X}$ = ***-11.20*** in, (horiz. to left)

$R_{2,Y}$ = ***40.30*** in, (upward)

DESIGN SECTION FORCES

Column	$N_{(1-5)}$ =	***-13.9***	k,(compre.)	$N_{(2-6)}$ =	***-42.0***	k,(compre.)
	$V_{(1-5)}$ =	***16.0***	kips	$V_{(2-6)}$ =	***15.8***	kips
	$M_{(1-5)}$ =	***154.8***	ft-kips	$M_{(2-6)}$ =	***195.3***	ft-kips
Beam	$N_{(3-4)}$ =	***-55.8***	k,(compre.)	$N_{(5-6)}$ =	***1.3***	k,(tension)
	$V_{(3-4)}$ =	***11.0***	kips	$V_{(5-6)}$ =	***8.0***	kips
	$M_{(3-4)}$ =	***55.0***	ft-kips	$M_{(5-6)}$ =	***40.0***	ft-kips
PT	$N_{(1-4)}$ =	***28.2***	k,(tension)	$N_{(3-6)}$ =	***17.0***	k,(tension)
	$N_{(2-3)}$ =	***2.9***	k,(tension)	$N_{(4-5)}$ =	***3.8***	k,(tension)

ANALYSIS

Joint	Coordinates (ft) X	Y	Load & Reaction (k) F_x / R_x	F_y / R_y	Joint I Deflection (in) X	Y
1	0	0	-9.80	7.18	0.00	0.00
2	20	0	-11.20	49.78	0.00	0.00
3	0	14	32.98	-14.81	0.35	-0.01
4	20	14	-22.98	-14.81	0.32	-0.02
5	0	26	20.45	-13.67	0.58	-0.01
6	20	26	-9.45	-13.67	0.57	-0.02

Element	End Joint I	End Joint J	A (in^2)	I (in^4)	E (ksi)	Length (ft)	Primary M (ft-k) I	Primary M (ft-k) J	Secondary M (ft-k) I	Secondary M (ft-k) J	Axial (k) Primary	Axial (k) Secondary	Shear (k) Primary	Shear (k) Secondary
1	1	4	0.459	0	29950	24.41	0.00	0.00	0.00	0.00	11.71	0.00	0.00	0.00
2	2	3	0.459	0	29950	24.41	0.00	0.00	0.00	0.00	-13.63	0.00	0.00	0.00
3	3	6	0.306	0	29950	23.32	0.00	0.00	0.00	0.00	5.95	0.00	0.00	0.00
4	4	5	0.306	0	29950	23.32	0.00	0.00	0.00	0.00	-7.21	0.00	0.00	0.00
5	1	3	13.3	248	29000	14.00	0.00	-34.71	0.00	189.52	-13.89	0.00	-0.21	-13.54
6	2	4	14	195	29000	14.00	0.00	-5.82	0.00	-189.52	-41.96	0.00	-0.03	13.54
7	3	4	6.49	0	29000	20.00	0.00	0.00	0.00	0.00	-26.47	-29.33	0.00	0.00
8	3	5	13.3	248	29000	12.00	34.71	0.00	-189.52	0.00	-9.96	0.00	0.24	15.79
9	4	6	14	195	29000	12.00	5.82	0.00	189.52	0.00	-16.73	0.00	0.04	-15.79
10	5	6	360	0	1800	20.00	0.00	0.00	0.00	0.00	-14.51	15.79	0.00	0.00

Nonbuilding Seismic Analysis Based on ASCE 7-22 Chapter 15

Determine Base Shear (Derived from ASCE 7 Chapter 15 for *rigid and/or liquid structure*)

$V = \text{MAX}(V\ ,\ 0.3S_{DS}I_eW\ ,\ 0.03W\ ,\ 0.8S_1I_e / RW)$

$= \text{MAX}(0.36W\ ,\ 0.29W\ ,\ 0.03W\ ,\ 0.14W)$

$=$ 0.36 W, (SD)

$=$ 0.26 W, (ASD) = **30.94** *kips, (137.6 kN)*

Where			
$R =$	3	(lesser of ASCE 7 Tab 15.4.2 or the supporting structure Tab 12.2-1)	
$I_e =$	1	(ASCE 7 15.4.1.1)	
$W =$	120	*kips, (533.8 kN), rigid weight, or impulsive weight without liquid*	
$W_c =$	15	*kips, (66.7 kN), portion of the liquid weight (zero for rigid without liquid)*	
$W_i =$	$W + W_C =$	135	*kips, (600.5 kN), impulsive weight, ASCE 7 15.4.2 & 15.7.6.2*
$D =$	20	*ft, (6.10 m), diameter*	
$H =$	18	*ft, (5.49 m), liquid height (zero for rigid without liquid)*	
$T_L =$	10	(ASCE 7 Fig 22-14 to 22-17)	
$T_C =$	$2\pi\ (D / [3.68\ g \tanh(3.68\ H/D)])^{0.5} =$	2.59	sec, (ASCE 7 Eq 15.7-12)
$S_{DS} =$	0.96	(ASCE 7 11.4)	
$S_{D1} =$	0.55	(ASCE 7 11.4)	
$S_1 =$	0.54	(ASCE 7 11.4)	
$S_{ai} =$	$S_{DS} =$	0.960	(ASCE 7 Eq 15.7-7)
$S_{ac} =$	0.319	(ASCE 7 Eq 15.7-10 & Eq 15.7-11)	
$V_i =$	$S_{ai}\ I_e\ W_i / R =$	43.20	*kips, (192.2 kN), (ASCE 7 Eq 15.7-5)*
$V_c =$	$S_{ac}\ I_e\ W_c / 1.5 =$	3.19	*kips, (14.2 kN), (ASCE 7 Eq 15.7-6)*
$V =$	$(V_i^2 + V_c^2)^{0.5} =$	43.32	*kips, (192.7 kN), (ASCE 7 15.7.6.2 Note 3)*

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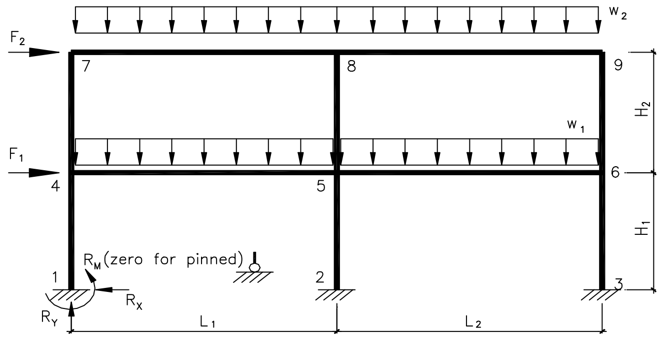
Two Span Moment Frame Analysis using Finite Element Method

DESIGN CRITERIA

1. The most seismic moment frames are governed by horizontal story drifts (ASCE 7-22 12.8.3 & 12.8.6). The seismic design of moment frame connections are based on actual column and beam capacities (sizes), not real loads, (AISC 358-16, AISC 341-16, & ACI 318-19 Chapter 18).
2. To trial F_1 or F_2, until horizontal each story drift a unit, can get each level lateral rigidity.

INPUT DATA & SUMMARY



DIMENSIONS

H_1 = **14** *ft, (4.3 m)* L_1 = **20** *ft, (6.1 m)*

H_2 = **12** *ft, (3.7 m)* L_2 = **30** *ft, (9.1 m)*

LOADS

F_1 = **10** kips, (horiz. to right) w_1 = **1.1** kips/ft

F_2 = **11** kips, (horiz. to right) w_2 = **0.8** kips/ft

MEMBERS

	A (in^2)	I_x (in^4)	E (ksi)
Column (1 - 7)	32.7	2670	29000
Column (2 - 8)	32.7	2670	29000
Column (3 - 9)	32.7	2670	29000
Beam (4 - 5)	19.1	1070	29000
Beam (5 - 6)	19.1	1070	29000
Beam (7 - 8)	19.1	1070	29000
Beam (8 - 9)	19.1	1070	29000

BASE (1, 2 & 3) PINNED ? **Yes** , (pinned)

STORY DRIFTS

Δ_4 = ***0.39*** in, (horiz. to right)

Δ_5 = ***0.39*** in, (horiz. to right)

Δ_6 = ***0.38*** in, (horiz. to right)

Δ_7 = ***0.56*** in, (horiz. to right)

Δ_8 = ***0.56*** in, (horiz. to right)

Δ_9 = ***0.57*** in, (horiz. to right)

DESIGN SECTION FORCES

Column

$N_{(1-7)}$ = ***-30.2*** k,(compre.) $N_{(2-8)}$ = ***-43.1*** k,(compre.)

$V_{(1-7)}$ = ***8.3*** kips $V_{(2-8)}$ = ***15.0*** kips

$M_{(1-7)}$ = ***116.3*** ft-kips $M_{(2-8)}$ = ***136.4*** ft-kips

$N_{(3-9)}$ = ***-21.8*** k,(compre.)

$V_{(3-9)}$ = ***11.9*** kips

$M_{(3-9)}$ = ***96.4*** ft-kips

Beam

$N_{(4-5)}$ = ***-9.5*** k,(compre.) $N_{(5-6)}$ = ***-14.8*** k,(compre.)

$V_{(4-5)}$ = ***7.6*** kips $V_{(5-6)}$ = ***4.2*** kips

$M_{(4-5)}$ = ***77.9*** ft-kips $M_{(5-6)}$ = ***68.7*** ft-kips

$N_{(7-8)}$ = ***-3.2*** k,(compre.) $N_{(8-9)}$ = ***-11.9*** k,(compre.)

$V_{(7-8)}$ = ***3.6*** kips $V_{(8-9)}$ = ***2.5*** kips

$M_{(7-8)}$ = ***37.4*** ft-kips $M_{(8-9)}$ = ***44.1*** ft-kips

Reactions

$R_{1,X}$ = ***8.31*** kips, (horiz. to left)

$R_{1,Y}$ = ***30.18*** kips, (upward)

$R_{1,M}$ = ***0.00***

$R_{2,X}$ = ***9.74*** kips, (horiz. to left)

$R_{2,Y}$ = ***43.07*** kips, (upward)

$R_{2,M}$ = ***0.00***

$R_{3,X}$ = ***2.95*** kips, (horiz. to left)

$R_{3,Y}$ = ***21.75*** kips, (upward)

$R_{3,M}$ = ***0.00***

ANALYSIS

	Coordinates (ft)		Load & Reaction (k)		Joint I Deflection (in)	
Joint	**X**	**Y**	**F_x / R_x**	**F_y / R_y**	**X**	**Y**
1	0	0	-8.31	-30.18	0.00	0.00
2	20	0	-9.74	-43.07	0.00	0.00
3	50	0	-2.95	-21.75	0.00	0.00
4	0	14	10.00	11.00	0.39	0.01
5	20	14	0.00	27.50	0.39	0.01
6	50	14	0.00	16.50	0.38	0.00
7	0	26	11.00	8.00	0.56	0.01
8	20	26	0.00	20.00	0.56	0.01
9	50	26	0.00	12.00	0.57	0.01

	End Joint		A	I	E	Length	Axial (k)		Shear (k)		Moment M (ft-k)	
Element	**I**	**J**	(in^2)	(in^4)	(ksi)	(ft)	**I**	**J**	**I**	**J**	**I**	**J**
1	1	4	32.7	2670	29000	14.00	-30.18	30.18	8.31	-8.31	0.00	116.27
2	2	5	32.7	2670	29000	14.00	-43.07	43.07	9.74	-9.74	0.00	136.39
3	3	6	32.7	2670	29000	14.00	-21.75	21.75	2.95	-2.95	0.00	41.34
4	4	5	19.1	1070	29000	20.00	9.53	-9.53	-7.57	7.57	-77.88	-73.48
5	5	6	19.1	1070	29000	30.00	14.81	-14.81	-4.20	4.20	-57.36	-68.71
6	4	7	32.7	2670	29000	12.00	-11.61	11.61	7.84	-7.84	16.61	77.44
7	5	8	32.7	2670	29000	12.00	-18.94	18.94	15.02	-15.02	63.20	116.99
8	6	9	32.7	2670	29000	12.00	-9.45	9.45	-11.85	11.85	-96.37	-45.87
9	7	8	19.1	1070	29000	20.00	3.16	-3.16	-3.61	3.61	-37.44	-34.74
10	8	9	19.1	1070	29000	30.00	-11.85	11.85	-2.55	2.55	-32.25	-44.13

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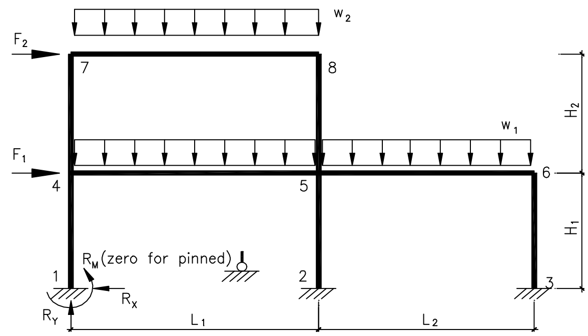
Set Back Moment Frame Analysis using Finite Element Method

DESIGN CRITERIA

1. The most seismic moment frames are governed by horizontal story drifts (ASCE 7-22 12.8.3 & 12.8.6). The seismic design of moment frame connections are based on actual column and beam capacities (sizes), not real loads, (AISC 358-16, AISC 341-16, & ACI 318-19 Chapter 18).
2. To trial F_1 or F_2, until horizontal each story drift a unit, can get each level lateral rigidity.

INPUT DATA & SUMMARY



DIMENSIONS

H_1 = **14** *ft, (4.3 m)* L_1 = **20** *ft, (6.1 m)*

H_2 = **12** *ft, (3.7 m)* L_2 = **30** *ft, (9.1 m)*

LOADS

F_1 = **10** kips, (horiz. to right) w_1 = **1.1** kips/ft

F_2 = **11** kips, (horiz. to right) w_2 = **0.8** kips/ft

MEMBERS

	A (in^2)	I_x (in^4)	E (ksi)
Column (1 - 7)	32.7	2670	29000
Column (2 - 8)	32.7	2670	29000
Column (3 - 9)	32.7	2670	29000
Beam (4 - 5)	19.1	1070	29000
Beam (5 - 6)	19.1	1070	29000
Beam (7 - 8)	19.1	1070	29000
Beam (8 - 9)	19.1	1070	29000

BASE (1, 2 & 3) PINNED ? **No** , (fixed)

STORY DRIFTS

Δ_4 = ***0.11*** in, (horiz. to right)

Δ_5 = ***0.10*** in, (horiz. to right)

Δ_6 = ***0.10*** in, (horiz. to right)

Δ_7 = ***0.19*** in, (horiz. to right)

Δ_8 = ***0.19*** in, (horiz. to right)

DESIGN SECTION FORCES

Column

$N_{(1-7)}$ = ***-24.8*** k,(compre.) $N_{(2-8)}$ = ***-32.2*** k,(compre.)

$V_{(1-7)}$ = ***10.5*** kips $V_{(2-8)}$ = ***13.7*** kips

$M_{(1-7)}$ = ***94.8*** ft-kips $M_{(2-8)}$ = ***107.5*** ft-kips

$N_{(3-6)}$ = ***-14.0*** k,(compre.)

$V_{(3-6)}$ = ***3.2*** kips

$M_{(3-6)}$ = ***78.3*** ft-kips

Beam

$N_{(4-5)}$ = ***-7.7*** k,(compre.) $N_{(5-6)}$ = ***-3.2*** k,(compre.)

$V_{(4-5)}$ = ***2.7*** kips $V_{(5-6)}$ = ***2.5*** kips

$M_{(4-5)}$ = ***30.0*** ft-kips $M_{(5-6)}$ = ***45.4*** ft-kips

$N_{(7-8)}$ = ***-2.9*** k,(compre.)

$V_{(7-8)}$ = ***3.0*** kips

$M_{(7-8)}$ = ***36.5*** ft-kips

Reactions

$R_{1,X}$ = ***10.48*** kips, (horiz. to left)

$R_{1,Y}$ = ***24.78*** kips, (upward)

$R_{1,M}$ = ***94.82*** ft-kips, (counterclockwise)

$R_{2,X}$ = ***13.69*** kips, (horiz. to left)

$R_{2,Y}$ = ***32.20*** kips, (upward)

$R_{2,M}$ = ***107.48*** ft-kips, (counterclockwise)

$R_{3,X}$ = ***-3.17*** kips, (horiz. to right)

$R_{3,Y}$ = ***14.03*** kips, (upward)

$R_{3,M}$ = ***34.03*** ft-kips, (counterclockwise)

ANALYSIS

	Coordinates (ft)		Load & Reaction (k)		Joint I Deflection (in)	
Joint	X	Y	F_x / R_x	F_y / R_y	X	Y
1	0	0	-10.48	-24.78	0.00	0.00
2	20	0	-13.69	-32.20	0.00	0.00
3	50	0	3.17	-14.03	0.00	0.00
4	0	14	10.00	11.00	0.11	0.00
5	20	14	0.00	27.50	0.10	0.01
6	50	14	0.00	16.50	0.10	0.00
7	0	26	11.00	8.00	0.19	0.01
8	20	26	0.00	8.00	0.19	0.01

	End Joint		A	I	E	Length	Axial (k)		Shear (k)		Moment M (ft-k)	
Element	I	J	(in^2)	(in^4)	(ksi)	(ft)	I	J	I	J	I	J
1	1	4	32.7	2670	29000	14.00	-24.78	24.78	10.48	-10.48	94.82	51.84
2	2	5	32.7	2670	29000	14.00	-32.20	32.20	13.69	-13.69	107.48	84.17
3	3	6	32.7	2670	29000	14.00	-14.03	14.03	-3.17	3.17	34.03	-78.34
4	4	5	19.1	1070	29000	20.00	7.66	-7.66	-2.73	2.73	-30.01	-24.50
5	5	6	19.1	1070	29000	30.00	-3.17	3.17	-2.47	2.47	-28.75	-45.41
6	4	7	32.7	2670	29000	12.00	-11.05	11.05	8.14	-8.14	33.17	64.52
7	5	8	32.7	2670	29000	12.00	-4.95	4.95	2.86	-2.86	37.83	-3.52
8												
9	7	8	19.1	1070	29000	20.00	2.86	-2.86	-3.05	3.05	-24.52	-36.48

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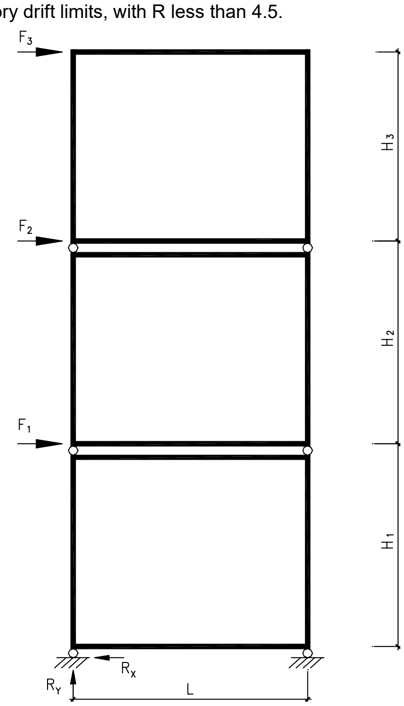
Container Building Lateral Design Based on 2021 IBC / 2022 CBC & ASCE 7-22

DESIGN CRITERIA

1. Contain has moment connection/fixed closed frame, including 4 corner vertical posts, 2 top side rails, 2 bottom side rails, 2 top end headers, and 2 bottom end sills. But the connections between containers are pinned connections. So each floor beam included two (top and bottom) rails/headers.
2. Containers, as building frames, have to be designed based on current building codes, IBC/CBC, no matter how strong on shipping loads capacity. Since the container and connections are all steel, the container moment frame design should be controlled by story drift limits, with R less than 4.5.



INPUT DATA & SUMMARY

DIMENSIONS

L = **8** *ft, (2.4 m)*

(8'-0" for end frame, or 10'-0"/20'-0"/40'-0" for long side frame)

H_3 = **8.5** *ft, (2.6 m)*

H_2 = **9.5** *ft, (2.9 m)* (8'-6" for regular, or 9'-6" for high cube)

H_1 = **9.5** *ft, (2.9 m)*

EFFECTIVE SEISMIC FLOOR WEIGHTS (for one frame ASCE 7-22 12.7.2, not container capacity)

w_3 = **15** *kips, (6803.9 kg)*

w_2 = **67.2** *kips, (30481.4 kg)*

w_1 = **67.2** *kips, (30481.4 kg)*

RESPONSE MODIFICATION COEFFICIENT (ASCE 7-22 Table 12.2-1) R = **4**

WIND LATERAL FORCES (for MWFRS ASCE 7-22 28)

$F_{3,\ wind}$ =	8.5	ft	x	**20**	ft	x	**35**	psf	=	5.95	*kips, (26.5 kN)*
$F_{2,\ wind}$ =	9.5	ft	x	20	ft	x	**30**	psf	=	5.7	*kips, (25.4 kN)*
$F_{1,\ wind}$ =	9.5	ft	x	20	ft	x	**25**	psf	=	4.75	*kips, (21.1 kN)*
	(Height)			(0.5 Length/Width)							

CONTAINER MEMBERS

	A (in^2)	I_x (in^4)	E (ksi)
Vertical Post	**23.4**	**108**	**29000**
Top Rail/Header	**24**	**256**	**29000**
Bottom Rail/Sill	**24**	**256**	**29000**

	Δ (in)		$\delta_{xe,allowable,\ ASD}$
Roof	***0.14***	<	***0.36***
3RD	***0.49***	>	***0.41***
2ND	***0.64***	>	***0.41***

[Unsatisfactory]

ANALYSIS

Determine Base Shear (Derived from ASCE 7-22 12.8)

V = MAX{ MIN [$S_{D1}I$ / (RT) , S_{DS} I / R] , MAX(0.044S_{DS}I , 0.01) , 0.5S_1 I / R } W

= MAX{ MIN[0.57W , 0.24W] , 0.04W , 0.00W }

= 0.24 W, (SD) ^ (for $S_1 \geq 0.6$ g only)

= 0.17 W, (ASD) = **25.61** *kips, (113.9 kN)*

Where						
S_{DS} =	0.96	(ASCE 7 Sec 11.4)	C_t =	0.02	(ASCE 7 Tab 12.8-2)
S_{D1} =	0.55	(ASCE 7 Sec 11.4)	h_n =	27.5	*ft, (8.38 m)*
S_1 =	0.54	(ASCE 7 Sec 11.4)	x =	0.75	(ASCE 7 Tab 12.8-2)
R =	4.0	(ASCE 7 Tab 12.2-1)	$T = C_t (h_n)^x$ =	0.240	sec, (ASCE 7 Sec 12.8.2.1)
I =	1	(2021 IBC Tab 1604.5 & ASCE 7 Tab 11.5-1)			

Calculate Vertical Distribution of Forces & Allowable Elastic Drift (ASCE 7, Sec 12.8.3 & 12.8.6)

Level	$\mathbf{W_x}$	$\mathbf{h_x}$	$\mathbf{h_x^k}$	$\mathbf{W_x h_x^k}$	$\mathbf{F_x}$, ASD (12.8-11)		$\mathbf{F_1, F_2, F_3}$		$\delta_{xe,allowable,\ ASD}$	
Roof	15.0	27.5	27.5	413	4.5	(0.30 Wx)	5.95	kips	0.36	in
3RD	67.2	19.0	19.0	1277	14.0	(0.21 Wx)	14.05	kips	0.41	in
2ND	67.2	9.5	9.5	638	7.0	(0.10 Wx)	7.02	kips	0.41	in
	149.4			2328	25.6					

Where k = 1 for T <= 0.5

k = 0.5 T + 0.75 for T @ (0.5 , 2.5)

k = 2 for T >= 2.5

$\delta_{e,allowable,\ ASD} = (\Delta_a - \delta_{di})(I / C_d)/1.4$, (ASCE 7-22 12.8.6)

C_d = 4 ,(ASCE 7 Tab 12.2-1)

Δ_a = 0.02 h_{sx}, (ASCE 7 Tab 12.12-1)

δ_{di} = 0 h_x, (ASCE 7-22 12.8.6.3)

The finite element method (FEM) results

STORY DRIFTS

Δ_{Roof} = ***0.14*** in, (horiz. to right)

Δ_{3RD} = ***0.49*** in, (horiz. to right)

Δ_{2ND} = ***0.64*** in, (horiz. to right)

REACTIONS

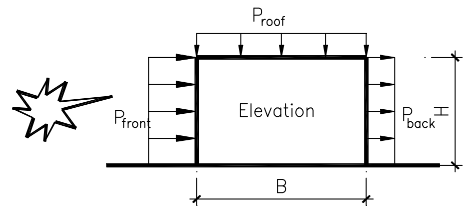
R_X = ***13.51*** kips, (horiz. to left) ***-13.51*** kips, (horiz. to right)

R_Y = ***-59.22*** kips, (downward) ***59.22*** kips, (upward)

CONTAINER CORNER ROTATIONS

Roof	Top	***-0.032***	deg	***-0.032***	deg
	Bottom	***-0.031***	deg	***-0.031***	deg
3RD	Top	***-0.081***	deg	***-0.081***	deg
	Bottom	***-0.078***	deg	***-0.078***	deg
2ND	Top	***-0.102***	deg	***-0.102***	deg
	Bottom	***-0.095***	deg	***-0.094***	deg

Determination of Blast Loads on Buildings Based on BIPS 06/FEMA 426, & UFC 3-340-02



INPUT DATA & DESIGN SUMMARY

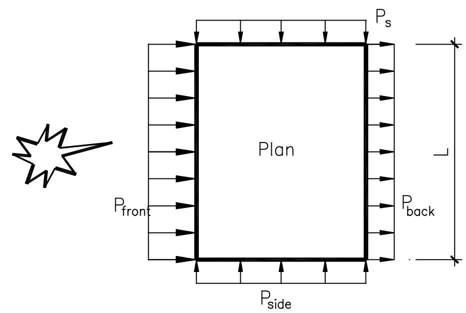
DIMENTION L = **93** *ft, (28.3 m)*
B = **67** *ft, (20.4 m)*
H = **15** *ft, (4.6 m)*

PEAK INCIDENT PRESSURE
P_{so} = **6** *psi, (41.4 kPa)*
(864.0 psf)

DURATION t_d = **0.05** *sec*

DRAG COEFFICIENT C_d = **1** *, for net overall frame loading*
-0.4 *, for components & cladding*

EFFECTIVE WIDTH OF COMPONENTS & CLADDING
L_1 = **1** *ft, (0.3 m), wall*
8 *ft, (2.4 m), roof*



Building Net Overall Base Shear (SD level): ***V =*** **1364.4** ***kips, (6068.7 kN)***

Loading of Components & Cladding:

P_{front} =	**13.80**	***psi, (95.1 kPa), (1987.2 psf), Front***
P_{roof} =	**5.12**	***psi, (35.3 kPa), (737.5 psf), Roof***
P_{side} =	**5.68**	***psi, (39.2 kPa), (818.4 psf), Side***
P_{back} =	**5.12**	***psi, (35.3 kPa), (737.5 psf), Back***

ANALYSIS

$U = 1130\,(1 + 0.058\,P_{so})^{0.5}$ = 1312 *ft / s, Shock Front Velocity*

$L_w = U\,t_d$ = 65.60 *ft, Length of Pressure Wave*

$q_o = 0.022\,P_{so}^{\,2}$ = 0.79 *psi, Peak Dynamic Wind Pressure*

$P_r = (2 + 0.05\,P_{so})\,P_{so}$ = 13.80 *psi, Reflected Overpressure*

$S = Min\,(H\,,\,L/2)$ = 15.00 *ft, Clearing distance*

$t_c = Min\,(3\,S\,/\,U\,,\,t_d)$ = 0.034 *sec, Reflected Overpressure Clearing Time*

$P_s = P_{so} + C_d\,q_o$ = 6.79 *psi, Stagnation Pressure*

$I_w = 0.5\,(P_r - P_s)\,t_c + 0.5\,P_s\,t_d$ = 0.29 *psi-s, Front Wall Impulse*

$t_e = 2\,I_w\,/\,P_r$ = 0.042 *sec, Effective Duration*

$B\,/\,L_1$ = 67 *ft, Side Wall Loading*

C_e = 1 *, equivalent load factor*

$P_a = C_e\,P_{so} + C_d\,q_o$ = 5.68 *psi, Equivalent Peak Overpressure*

$t_r = L_1\,/\,U$ = 0.001 *sec, Rise Time*

$B\,/\,L_1$ = 8.375 *ft, Roof Loading*

C_e = 0.906373 *, equivalent load factor*

$P_a = C_e\,P_{so} + C_d\,q_o$ = 5.12 *psi, Equivalent Peak Overpressure*

$t_r = L_1\,/\,U$ = 0.006 *sec, Rise Time*

$t_o = t_r + t_d$ = 0.056 *sec, otal Positive Phase Duration*

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Aluminum I or WF Member Capacity Based on Aluminum Design Manual 2010 (ASD)

INPUT DATA & DESIGN SUMMARY

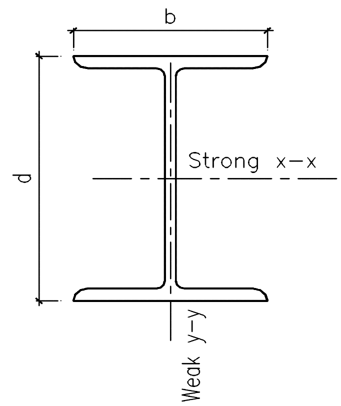
MEMBER SIZE I 8 × 6.18

d	b	A	I_x	I_y	E	**Wt** (lbs/ft)
8	5	5.26	60	7	10100	6.31

TENSILE ULTIMATE STRESS (T5 to T9, Table A.3.4) F_{tu} = **38** ksi

TENSILE YIELD STRESS (T5, T6, T7, T8, or T9) F_{ty} = **35** ksi

COMPRESSIVE YIELD STRESS (T5 to T9) F_{cy} = **35** ksi



AXIAL COMPRESSION FORCE P = **40** kips, ASD

STRONG GEOMETRIC AXIS EFFECTIVE LENGTH kL_x = **4** ft

WEAK GEOMETRIC AXIS EFFECTIVE LENGTH kL_y = **2** ft

STRONG GEOMETRIC AXIS BENDING MOMENT M_{rx} = **10** ft-kips, ASD

STRONG GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{bx} = **4** ft

WEAK GEOMETRIC AXIS BENDING MOMENT M_{ry} = **2.5** ft-kips, ASD

WEB DIRECTION SHEAR LOAD, ASD V = **22.5** kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMPRESSION STRESS IN AXIAL FORCE

$$F_{a1} = \frac{P_n}{A_g\Omega_c} = \begin{cases} Min\left(\dfrac{\left(B_c - \dfrac{D_c kL}{r}\right)}{n_u} \ , \ \dfrac{F_{cy}}{n_y}\right) \ , \ for \ \dfrac{kL}{r} < S_2 \\ \dfrac{\pi^2 E}{n_u\left(\dfrac{kL}{r}\right)^2} \ , \ for \ \dfrac{kL}{r} \geq S_2 \end{cases} = 17.70$$

ksi, (compression in column, ADM-I E3)

Where r_x = 3.370 in, (Page V-12 to V-15)

r_y = 1.180 in, (Page V-12 to V-15)

$kL / r = Max(kL_x / r_x \ , kL_y / r_y)$ = 20.34

E = 10100 ksi, (ADM-I Table A.3.4)

$B_c = F_{cy} \ [1 + (F_{cy} / 2250)^{0.5}]$ = 39.37 , (ADM-I Table B.4.2)

$D_c = (B_c / 10) \ (B_c / E)^{0.5}$ = 0.25 , (ADM-I Table B.4.2)

$C_c = 0.41 \ (B_c / D_c)$ = 65.67 , (ADM-I Table B.4.2)

$n_u = \Omega_c / 0.85$ = 1.941 , (ADM-I E.1, E.3-2 & E.3-3)

$n_y = \Omega_c$ = 1.65 , (ADM-I B.3-2, E.1, E.3-2 & E.3-3)

$S_2 = C_c$ = 65.67 , (ADM-I E.3-4)

$$F_{a2} = \frac{P_n}{A_g\Omega_c} = \begin{cases} \dfrac{F_{cy}}{n_y} \ , \ for \ \dfrac{b}{t} \leq S_1 \\ \dfrac{\left(B_p - \dfrac{5.0 D_p b}{t}\right)}{n_u} \ , \ for \ S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2\sqrt{B_p E}}{n_u \dfrac{5.0 b}{t}} \ , \ for \ \dfrac{b}{t} \geq S_2 \end{cases} = 18.57$$

ksi, (compression in flanges, ADM-I B.5.4.1)

Where b / t = 5.957 , (ADM-I Figure B.5.1)

k_1 = 0.35 , (ADM-I Table B.4.3)

(cont'd)

k_2 = 2.27 , (ADM-I Table B.4.3)

$B_p = F_{cy}\ [1 + F_{cy}\ ^{(1/3)} / 11.447]$ = 45.00 , (ADM-I Table B.4.2)

$D_p = (B_p\ /\ 10)\ (B_p\ /\ E)^{0.5}$ = 0.30 , (ADM-I Table B.4.2)

$S_1 = (B_p - n_u\ F_{cy} / n_y\ ) / (5.0\ D_p)$ = 2.55 , (ADM-I B5.4.1)

$S_2 = k_1\ B_p\ / (5.0\ D_p)$ = 10.49 , (ADM-I B5.4.1)

$$F_{a3} = \frac{P_n}{A_g \Omega_c} = \begin{cases} \dfrac{F_{cy}}{n_y} \ , \ for \ \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_p - \dfrac{1.6 D_p b}{t}\right)}{n_u} \ , \ for \ S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2 \sqrt{B_p E}}{n_u \dfrac{1.6b}{t}} \ , \ for \ \dfrac{b}{t} \ge S_2 \end{cases} = 15.97$$

ksi, (compression in web, ADM-I B.5.4.2)

Where b / t = 29.130 , (ADM-I Figure B.5.2)

$S_1 = (B_p - n_u\ F_{cy} / n_y\ ) / (1.6\ D_p)$ = 7.96 , (ADM-I B5.4.2)

$S_2 = k_1\ B_p\ / (1.6\ D_p)$ = 32.77 , (ADM-I B5.4.2)

$f_a = P / A$ = 7.60 ksi **<** $F_a = Min\ (F_{a\,1}\ , F_{a\,2}\ , F_{a\,3})$ = 15.97 ksi

[Satisfactory]

CHECK TENSION STRESS IN BENDING MOMENTS

$F = P_n/(A_g\,\Omega_t) = Min\ (F_{ty}/n_y\ ,\ F_{tu}/k_t n_u\ ,\ 1.3F_{ty}/n_y\ ,\ 1.42F_{tu}/k_t n_u)$ = 19.58 ksi, (tension bending, ADM-I D.1, D.2 & F.8.1.2)

Where k_t = 1.00 , (ADM-I Table A.3.3)

$f = Min\ (M_{rx}\ / S_x\ , M_{ry}\ / S_y)$ = 8.05 ksi **<** F = 19.58 ksi

[Satisfactory]

Where S_x = 14.90 in, (Page V-12 to V-15)

S_y = 2.92 in, (Page V-12 to V-15)

CHECK COMPRESSION STRESS IN STRONG GEOMETRIC AXIS, x - x, BENDING MOMENT

$$F_{bx1} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} \ , \ for \ \dfrac{L_b}{r_y \sqrt{C_b}} \le S_1 \\ \dfrac{\left(B_c - \dfrac{D_c L_b}{1.2 r_y \sqrt{C_b}}\right)}{n_y} \ , \ for \ S_1 < \dfrac{L_b}{r_y \sqrt{C_b}} < S_2 \\ \dfrac{C_b \pi^2 E}{n_y \left(\dfrac{L_b}{1.2 r_y}\right)^2} \ , \ for \ \dfrac{L_b}{r_y \sqrt{C_b}} \ge S_2 \end{cases} = 18.81$$

ksi, (compression in beam, ADM-I F.2.1)

Where $S_1 = 1.2\ (B_c - F_{cy}) / D_c$ = 21.31 , (ADM-I F.2.1)

$S_2 = 1.2\ C_c$ = 78.81 , (ADM-I F.2.1)

C_b = 1.00 , (ADM-I F.1.1)

$L_b\ / (r_y\ C_b\ ^{0.5})$ = 40.68

$$F_{bx2} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} \ , \ for \ \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_p - \dfrac{5.0 D_p b}{t}\right)}{n_y} \ , \ for \ S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2 \sqrt{B_p E}}{n_y \dfrac{5.0b}{t}} \ , \ for \ \dfrac{b}{t} \ge S_2 \end{cases} = 21.21$$

ksi, (compression in flanges, ADM-I B.5.4.1)

Where b / t = 5.957 , (ADM-I Figure B.5.1)

k_1 = 0.5 , (ADM-I Table B.4.3)

k_2 = 2.04 , (ADM-I Table B.4.3)

$S_1 = (B_p - F_{cy}) / (5.0\ D_p)$ = 6.66 , (ADM-I B5.4.1)

$S_2 = k_1\ B_p\ / (5.0\ D_p)$ = 14.98 , (ADM-I B5.4.1)

(cont'd)

$$F_{bx3}=\frac{M_n}{S_x\Omega_b}=\begin{cases}\dfrac{1.3F_{cy}}{n_y} &, \ for\ \dfrac{h}{t}\le S_1\\ \dfrac{\left(B_{br}-\dfrac{mD_{br}h}{t}\right)}{n_y} &, \ for\ S_1<\dfrac{h}{t}<S_2\\ \dfrac{k_2\sqrt{B_{br}E}}{n_y\dfrac{mh}{t}} &, \ for\ \dfrac{h}{t}\ge S_2\end{cases} = 26.23$$ ksi, (compression in web, ADM-I B.5.5.1)

Where $h / t = b / t =$ 31.739 , (ADM-I Figure B.5.2)

$c_c = c_o =$ 4 in

$m =$ 0.65 , (ADM-I B5.5.1)

$B_{br} = 1.3\,F_{cy}\,[1 + F_{cy}{}^{(1/3)} / 7] =$ 66.76 , (ADM-I Table B.4.2)

$D_{br} = (B_{br} / 20)\,(6\,B_{br} / E)^{(1/3)} =$ 1.14 , (ADM-I Table B.4.2)

$S_1 = (B_{br} - 1.3\,F_{cy}) / (m\,D_{br}) =$ 0.49 , (ADM-I B5.5.1)

$S_2 = k_1\,B_{br} / (m\,D_{br}) =$ 45.11 , (ADM-I B5.5.1)

$f_{bx} = M_{rx} / S_x =$ 8.05 ksi **<** $F_{bx} = Min\,(F_{bx\,1}\,,\,F_{bx\,2}\,,\,F_{bx\,3}) =$ 18.81 ksi

[Satisfactory]

CHECK COMPRESSION STRESS IN WEAK GEOMETRIC AXIS, y - y, BENDING MOMENT

$$F_{by}=\frac{M_n}{S_y\Omega_b}=\begin{cases}\dfrac{1.3F_{cy}}{n_y} &, \ for\ \dfrac{b}{t}\le S_1\\ \dfrac{\left(B_{br}-\dfrac{3.5D_{br}b}{t}\right)}{n_y} &, \ for\ S_1<\dfrac{b}{t}<S_2\\ \dfrac{\pi^2E}{n_y\left(\dfrac{3.5b}{t}\right)^2} &, \ for\ \dfrac{b}{t}\ge S_2\end{cases} = 26.08$$ ksi, (compression in web, ADM-I B.5.5.2)

Where $b / t =$ 5.957 , (ADM-I Figure B.5.3)

$C_{br} = 2B_{br} / (3\,D_{br}) =$ 39.10 , (ADM-I Table B.4.2)

$S_1 = (B_{br} - 1.3\,F_{cy}) / (3.5\,D_{br}) =$ 5.34 , (ADM-I B5.5.2)

$S_2 = C_{br} / 3.5 =$ 11.17 , (ADM-I B5.5.2)

$f_{by} = M_{ry} / S_y =$ 10.27 ksi **<** $F_{by} =$ 26.08 ksi

[Satisfactory]

CHECK COMBINED COMPRESSION AND BENDING (ADM-I H.1)

$$Max\left(\begin{array}{l}\dfrac{P_r}{P_c}+\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}=\dfrac{f_a}{F_a}+\dfrac{f_{bx}}{F_{bx}}+\dfrac{f_{by}}{F_{by}}\\ \left[\dfrac{f_a}{F_a}+\dfrac{C_{mx}f_{bx}}{F_{bx}\left(1-f_a/F_{ex}\right)}+\dfrac{C_{my}f_{by}}{F_{by}\left(1-f_a/F_{ey}\right)}\right]Check\end{array}\right)= 1.30$$ **<** 1.33 , (1.33 if IBC/CBC 1605.3.2 apply)

[Satisfactory]

Where *Check* = 1 Check if ASD 2005 Eq. 4.1.1-1 apply (1 = Yes, 0 = No)

$C_{mx} =$ 0.85 $C_{my} =$ 0.85

$F_{ex} = \pi^2 E / n_u\,(kL_x/r_x)^2 =$ 253.12 ksi

$F_{ey} = \pi^2 E / n_u\,(kL_y/r_y)^2 =$ 124.14 ksi

CHECK SHEAR STRESS (ADM-I G)

$$F_s=\frac{V_n}{A_w\Omega_v}=\begin{cases}\dfrac{F_{ty}/\sqrt{3}}{n_y} &, \ for\ \dfrac{h}{t}\le S_1\\ \dfrac{\left(B_s-\dfrac{1.25D_sh}{t}\right)}{n_y} &, \ for\ S_1<\dfrac{h}{t}<S_2\\ \dfrac{\pi^2E}{n_y\left(\dfrac{1.25h}{t}\right)^2} &, \ for\ \dfrac{h}{t}\ge S_2\end{cases} = 12.25$$ ksi, (shear, ADM-I G.2)

(cont'd)

Where $h / t = b / t =$ 31.739 , (ADM-I Figure G.2.1)

$B_s = (F_{cy} / 3^{0.5}) [1 + (F_{cy} / 3^{0.5})^{(1/3)} / 9.3] =$ 26.13 , (ADM-I Table B.4.2)

$D_s = (B_s / 10) (B_s / E)^{0.5} =$ 0.13 , (ADM-I Table B.4.2)

$C_s = 0.41 B_s / D_s =$ 80.61 , (ADM-I Table B.4.2)

$S_1 = (B_s - F_{cy} / 3^{0.5}) / (1.25 D_s) =$ 35.63 , (ADM-I G.2)

$S_2 = C_s / 1.25 =$ 64.49 , (ADM-I G.2)

$f_s = V / A_w =$ 12.23 ksi $<$ $F_s =$ 12.25 ksi

[Satisfactory]

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DESIGN BY :
REVIEW BY :

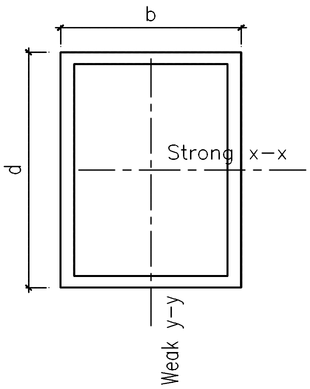
Aluminum RT Member Capacity Based on Aluminum Design Manual 2010 (ASD)

INPUT DATA & DESIGN SUMMARY

MEMBER SIZE RT 1 3/4 × 4 × 1/8

d	b	A	I_x	I_y	E	Wt (lbs/ft)
4	1.75	1.38	2.7	0.7	10100	1.62

TENSILE ULTIMATE STRESS (T5 to T9, Table A.3.4) F_{tu} = **30** ksi
TENSILE YIELD STRESS (T5, T6, T7, T8, or T9) F_{ty} = **25** ksi
COMPRESSIVE YIELD STRESS (T5 to T9) F_{cy} = **25** ksi



AXIAL COMPRESSION FORCE P = **3** kips, ASD
STRONG GEOMETRIC AXIS EFFECTIVE LENGTH kL_x = **6.4** ft
WEAK GEOMETRIC AXIS EFFECTIVE LENGTH kL_y = **3.2** ft

STRONG GEOMETRIC AXIS BENDING MOMENT M_{rx} = **1.2** ft-kips, ASD
STRONG GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{bx} = **2** ft
STRONG DIRECTION SHEAR LOAD, ASD V_{strong} = **30** kips

WEAK GEOMETRIC AXIS BENDING MOMENT M_{ry} = **0.35** ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD V_{weak} = **20** kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMPRESSION STRESS IN AXIAL FORCE

$$F_{a1}=\frac{P_n}{A_g\Omega_c}=\begin{cases}Min\left(\dfrac{\left(B_c-\dfrac{D_c kL}{r}\right)}{n_u}\ ,\ \dfrac{F_{cy}}{n_y}\right)\ ,\ for\ \dfrac{kL}{r}<S_2\\[2ex] \dfrac{\pi^2 E}{n_u\left(\dfrac{kL}{r}\right)^2}\ ,\ for\ \dfrac{kL}{r}\geq S_2\end{cases}\quad = \quad 10.18$$

ksi, (compression in column, ADM-I E3)

Where r_x = 1.410 in, (Page V-34 to V-37)
r_y = 0.730 in, (Page V-34 to V-37)
$kL / r = Max(kL_x / r_x , kL_y / r_y)$ = 54.47
E = 10100 ksi, (ADM-I Table A.3.4)
$B_c = F_{cy} [1 + (F_{cy} / 2250)^{0.5}]$ = 27.64 , (ADM-I Table B.4.2)
$D_c = (B_c / 10) (B_c / E)^{0.5}$ = 0.14 , (ADM-I Table B.4.2)
$C_c = 0.41 (B_c / D_c)$ = 78.38 , (ADM-I Table B.4.2)
$n_u = \Omega_c / 0.85$ = 1.941 , (ADM-I E.1, E.3-2 & E.3-3)
$n_y = \Omega_c$ = 1.65 , (ADM-I B.3-2, E.1, E.3-2 & E.3-3)
$S_2 = C_c$ = 78.38 , (ADM-I E.3-4)

$$F_{a2}=\frac{P_n}{A_g\Omega_c}=\begin{cases}\dfrac{F_{cy}}{n_y}\ ,\ for\ \dfrac{b}{t}\leq S_1\\[2ex] \dfrac{\left(B_p-\dfrac{1.6D_p b}{t}\right)}{n_u}\ ,\ for\ S_1<\dfrac{b}{t}<S_2\\[2ex] \dfrac{k_2\sqrt{B_p E}}{n_u\dfrac{1.6b}{t}}\ ,\ for\ \dfrac{b}{t}\geq S_2\end{cases}\quad = \quad 11.84$$

ksi, (compression in web, ADM-I B.5.4.2)

Where b / t = 30.000 , (ADM-I Figure B.5.2)
k_1 = 0.35 , (ADM-I Table B.4.3)
k_2 = 2.27 , (ADM-I Table B.4.3)
$B_p = F_{cy} [1 + F_{cy}^{(1/3)} / 11.447]$ = 31.39 , (ADM-I Table B.4.2)

(cont'd)

$D_p = (B_p / 10)(B_p / E)^{0.5} =$ 0.17 , (ADM-I Table B.4.2)

$S_1 = (B_p - n_u F_{cy} / n_y) / (1.6 D_p) =$ 7.05 , (ADM-I B5.4.2)

$S_2 = k_1 B_p / (1.6 D_p) =$ 39.24 , (ADM-I B5.4.2)

$f_a = P / A =$ 2.17 ksi < $F_a = Min(F_{a1}, F_{a2}) =$ 10.18 ksi

[Satisfactory]

CHECK TENSION STRESS IN BENDING MOMENTS

$F = P_n/(A_g \Omega_t) = Min(F_{ty}/n_y,\ F_{tu}/k_t n_u,\ 1.3F_{ty}/n_y,\ 1.42F_{tu}/k_t n_u) =$ 15.15 ksi, (tension bending, ADM-I D.1, D.2 & F.8.1.2)

Where $k_t =$ 1.00 , (ADM-I Table A.3.3)

$f = Min(M_{rx} / S_x,\ M_{ry} / S_y) =$ 5.02 ksi < $F =$ 15.15 ksi

[Satisfactory]

Where $S_x =$ 1.37 in, (Page V-34 to V-37)

$S_y =$ 0.84 in, (Page V-34 to V-37)

CHECK COMPRESSION STRESS IN STRONG GEOMETRIC AXIS, x - x, BENDING MOMENT

$$F_{bx1} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} , & for \quad \dfrac{L_b S_c}{C_b \sqrt{I_y J}/2} \le S_1 \\ \dfrac{1}{n_y}\left(B_c - 1.6 D_c \sqrt{\dfrac{L_b S_c}{C_b \sqrt{I_y J}/2}}\right) , & for \quad S_1 < \dfrac{L_b S_c}{C_b \sqrt{I_y J}/2} < S_2 \\ \dfrac{\pi^2 E}{2.56 n_y \left(\dfrac{L_b S_c}{C_b \sqrt{I_y J}/2}\right)} , & for \quad \dfrac{L_b S_c}{C_b \sqrt{I_y J}/2} \ge S_2 \end{cases} = \quad 15.15$$

ksi, (compression in beam, ADM-I F.3.1)

Where $S_1 = (B_c - F_{cy})^2 / (1.6 D_c)^2 =$ 129.82 , (ADM-I F.3.1)

$S_2 = (C_c / 1.6)^2 =$ 2399.9 , (ADM-I F.3.1)

$J =$ 1.8 , (Page V-34 to V-37)

$S_c = S_x =$ 1.37 in, (Page V-34 to V-37)

$C_b =$ 1.00 , (ADM-I F.1.1)

$L_b S_c / [C_b (I_y J)^{0.5} / 2] =$ 57.29

$$F_{bx2} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} , & for \quad \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_p - \dfrac{1.6 D_p b}{t}\right)}{n_y} , & for \quad S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2 \sqrt{B_p E}}{n_y \dfrac{1.6b}{t}} , & for \quad \dfrac{b}{t} \ge S_2 \end{cases} = \quad 15.15$$

ksi, (compression in flanges, ADM-I B.5.4.2)

Where $b / t =$ 12.000 , (ADM-I Figure B.5.2)

$k_1 =$ 0.5 , (ADM-I Table B.4.3)

$k_2 =$ 2.04 , (ADM-I Table B.4.3)

$S_1 = (B_p - F_{cy}) / (1.6 D_p) =$ 22.81 , (ADM-I B5.4.2)

$S_2 = k_1 B_p / (1.6 D_p) =$ 56.06 , (ADM-I B5.4.2)

$$F_{bx3} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \dfrac{1.3 F_{cy}}{n_y} , & for \quad \dfrac{h}{t} \le S_1 \\ \dfrac{\left(B_{br} - \dfrac{m D_{br} h}{t}\right)}{n_y} , & for \quad S_1 < \dfrac{h}{t} < S_2 \\ \dfrac{k_2 \sqrt{B_{br} E}}{n_y \dfrac{mh}{t}} , & for \quad \dfrac{h}{t} \ge S_2 \end{cases} = \quad 19.72$$

ksi, (compression in web, ADM-I B.5.5.1)

Where $h / t = b / t =$ 30.000 , (ADM-I Figure B.5.2)

$c_c = c_o =$ 2 in

(cont'd)

$m =$ 0.65 , (ADM-I B5.5.1)

$B_{br} = 1.3\,F_{cy}\ [1 + F_{cy}^{(1/3)} / 7] =$ 46.08 , (ADM-I Table B.4.2)

$D_{br} = (B_{br} / 20)\ (6\,B_{br} / E)^{(1/3)} =$ 0.69 , (ADM-I Table B.4.2)

$S_1 = (B_{br} - 1.3\,F_{cy}) / (m\,D_{br}) =$ 0.45 , (ADM-I B5.5.1)

$S_2 = k_1\,B_{br} / (m\,D_{br}) =$ 51.05 , (ADM-I B5.5.1)

$f_{bx} = M_{rx} / S_x =$ 10.51 ksi < $F_{bx} = Min\ (F_{bx1}, F_{bx2}, F_{bx3}) =$ 15.15 ksi

[Satisfactory]

CHECK COMPRESSION STRESS IN WEAK GEOMETRIC AXIS, y - y, BENDING MOMENT

$$F_{by} = \frac{M_n}{S_y \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} & , \ for \ \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_p - \dfrac{1.6 D_p b}{t}\right)}{n_y} & , \ for \ S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2 \sqrt{B_p E}}{n_y \dfrac{1.6b}{t}} & , \ for \ \dfrac{b}{t} \ge S_2 \end{cases} = 13.93$$

ksi, (compression in flanges, ADM-I B.5.4.2)

Where $b / t =$ 30.000 , (ADM-I Figure B.5.2)

$f_{by} = M_{ry} / S_y =$ 5.02 ksi < $F_{by} =$ 13.93 ksi

[Satisfactory]

CHECK COMBINED COMPRESSION AND BENDING (ADM-I H.1)

$$Max \left(\begin{array}{l} \dfrac{P_r}{P_c} + \dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}} = \dfrac{f_a}{F_a} + \dfrac{f_{bx}}{F_{bx}} + \dfrac{f_{by}}{F_{by}} \\ \left[\dfrac{f_a}{F_a} + \dfrac{C_{mx} f_{bx}}{F_{bx}\left(1 - f_a / F_{ex}\right)} + \dfrac{C_{my} f_{by}}{F_{by}\left(1 - f_a / F_{ey}\right)} \right] Check \end{array} \right) = 1.27 < 1.33$$

, (1.33 if IBC/CBC 1605.3.2 apply)

[Satisfactory]

Where $Check =$ 1 Check if ASD 2005 Eq. 4.1.1-1 apply (1 = Yes, 0 = No)

$C_{mx} =$ 0.85 $C_{my} =$ 0.85

$F_{ex} = \pi^2 E / n_u (kL_x / r_x)^2 =$ 17.31 ksi

$F_{ey} = \pi^2 E / n_u (kL_y / r_y)^2 =$ 18.56 ksi

CHECK SHEAR STRESS (ADM-I G)

$$F_s = \frac{V_n}{A_w \Omega_v} = \begin{cases} \dfrac{F_{ty} / \sqrt{3}}{n_y} & , \ for \ \dfrac{h}{t} \le S_1 \\ \dfrac{\left(B_s - \dfrac{1.25 D_s h}{t}\right)}{n_y} & , \ for \ S_1 < \dfrac{h}{t} < S_2 \\ \dfrac{\pi^2 E}{n_y \left(\dfrac{1.25 h}{t}\right)^2} & , \ for \ \dfrac{h}{t} \ge S_2 \end{cases} =$$

8.75 ksi, (for strong shear)
8.75 ksi, (for weak shear)
, (shear, ADM-I G.2)

Where $h / t = b / t =$ 30.000 , (for strong shear, Figure G.2)

$h / t = b / t =$ 12.000 , (for weak shear)

$B_s = (F_{cy} / 3^{0.5})\ [1 + (F_{cy} / 3^{0.5})^{(1/3)} / 9.3] =$ 18.21 , (ADM-I Table B.4.2)

$D_s = (B_s / 10)\ (B_s / E)^{0.5} =$ 0.08 , (ADM-I Table B.4.2)

$C_s = 0.41\,B_s / D_s =$ 96.55 , (ADM-I Table B.4.2)

$S_1 = (B_s - F_{cy} / 3^{0.5}) / (1.25\,D_s) =$ 39.09 , (ADM-I G.2)

$S_2 = C_s / 1.25 =$ 77.24 , (ADM-I G.2)

$f_s = V_{strong} / A_w =$ 3.75 ksi < $F_s =$ 8.75 ksi, (for strong shear)

$f_s = V_{weak} / A_w =$ 5.71 ksi < $F_s =$ 8.75 ksi, (for weak shear)

[Satisfactory]

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PROJECT :			PAGE :	
CLIENT :			DESIGN BY :	
JOB NO. :		DATE :	REVIEW BY :	

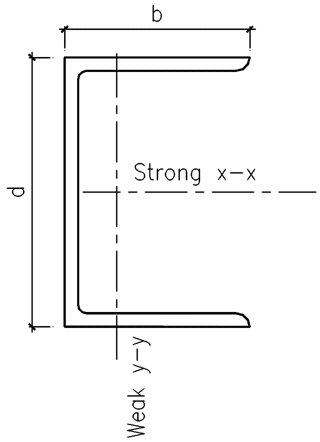
Aluminum C or CS Member Capacity Based on Aluminum Design Manual 2010 (ASD)

INPUT DATA & DESIGN SUMMARY

MEMBER SIZE CS 8 × 5.79

d	b	A	I_x	I_y	E	**Wt** (lbs/ft)
8	3.75	4.92	53	7	10100	5.90

TENSILE ULTIMATE STRESS (T5 to T9, Table A.3.4) F_{tu} = **38** ksi
TENSILE YIELD STRESS (T5, T6, T7, T8, or T9) F_{ty} = **35** ksi
COMPRESSIVE YIELD STRESS (T5 to T9) F_{cy} = **35** ksi



AXIAL COMPRESSION FORCE P = **40** kips, ASD
STRONG GEOMETRIC AXIS EFFECTIVE LENGTH kL_x = **4** ft
WEAK GEOMETRIC AXIS EFFECTIVE LENGTH kL_y = **2** ft

STRONG GEOMETRIC AXIS BENDING MOMENT M_{rx} = **8** ft-kips, ASD
STRONG GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{bx} = **4** ft

WEAK GEOMETRIC AXIS BENDING MOMENT M_{ry} = **2.4** ft-kips, ASD

WEB DIRECTION SHEAR LOAD, ASD V = **22.5** kips

THE DESIGN IS ADEQUATE.

ANALYSIS
CHECK COMPRESSION STRESS IN AXIAL FORCE

$$F_{a1}=\frac{P_n}{A_g\Omega_c}=\begin{cases} Min\left(\dfrac{\left(B_c-\dfrac{D_c kL}{r}\right)}{n_u}\ ,\ \dfrac{F_{cy}}{n_y}\right)\ ,\ for\ \dfrac{kL}{r}<S_2 \\ \dfrac{\pi^2 E}{n_u\left(\dfrac{kL}{r}\right)^2}\ ,\ for\ \dfrac{kL}{r}\geq S_2 \end{cases} = 17.75 \text{ ksi, (compression in column, ADM-I E3)}$$

Where r_x = 3.270 in, (Page V-7 to V-10)
r_y = 1.200 in, (Page V-7 to V-10)
$kL / r = Max(\, kL_x / r_x\ ,\ kL_y / r_y\,)$ = 20.00
E = 10100 ksi, (ADM-I Table A.3.4)
$B_c = F_{cy}\,[1 + (F_{cy} / 2250)^{0.5}]$ = 39.37 , (ADM-I Table B.4.2)
$D_c = (B_c / 10)\,(B_c / E)^{0.5}$ = 0.25 , (ADM-I Table B.4.2)
$C_c = 0.41\,(B_c / D_c)$ = 65.67 , (ADM-I Table B.4.2)
$n_u = \Omega_c / 0.85$ = 1.941 , (ADM-I E.1, E.3-2 & E.3-3)
$n_y = \Omega_c$ = 1.65 , (ADM-I B.3-2, E.1, E.3-2 & E.3-3)
$S_2 = C_c$ = 65.67 , (ADM-I E.3-4)

$$F_{a2}=\frac{P_n}{A_g\Omega_c}=\begin{cases} \dfrac{F_{cy}}{n_y}\ ,\ for\ \dfrac{b}{t}\leq S_1 \\ \dfrac{\left(B_p-\dfrac{5.0D_p b}{t}\right)}{n_u}\ ,\ for\ S_1<\dfrac{b}{t}<S_2 \\ \dfrac{k_2\sqrt{B_p E}}{n_u\dfrac{5.0b}{t}}\ ,\ for\ \dfrac{b}{t}\geq S_2 \end{cases} = 17.55 \text{ ksi, (compression in flanges, ADM-I B.5.4.1)}$$

Where b / t = 7.293 , (ADM-I Figure B.5.1)
k_1 = 0.35 , (ADM-I Table B.4.3)

(cont'd)

k_2 = 2.27 , (ADM-I Table B.4.3)

$B_p = F_{cy}\ [1 + F_{cy}^{(1/3)} / 11.447]$ = 45.04 , (ADM-I Table B.4.2)

$D_p = (B_p / 10)\ (B_p / E)^{0.5}$ = 0.30 , (ADM-I Table B.4.2)

$S_1 = (B_p - n_u\ F_{cy} / n_y) / (5.0\ D_p)$ = 2.57 , (ADM-I B5.4.1)

$S_2 = k_1\ B_p / (5.0\ D_p)$ = 10.48 , (ADM-I B5.4.1)

$$F_{a3} = \frac{P_n}{A_g \Omega_c} = \begin{cases} \dfrac{F_{cy}}{n_y} \ , \ for \ \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_p - \dfrac{1.6 D_p b}{t} \right)}{n_u} \ , \ for \ S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2 \sqrt{B_p E}}{n_u \dfrac{1.6b}{t}} \ , \ for \ \dfrac{b}{t} \ge S_2 \end{cases} = 16.78 \text{ ksi, (compression in web, ADM-I B.5.4.2)}$$

Where b / t = 25.920 , (ADM-I Figure B.5.2)

$S_1 = (B_p - n_u\ F_{cy} / n_y) / (1.6\ D_p)$ = 8.03 , (ADM-I B5.4.2)

$S_2 = k_1\ B_p / (1.6\ D_p)$ = 32.76 , (ADM-I B5.4.2)

$f_a = P / A$ = 8.13 ksi **<** $F_a = Min\ (F_{a1}, F_{a2}, F_{a3})$ = 16.78 ksi

[Satisfactory]

CHECK TENSION STRESS IN BENDING MOMENTS

$F = P_n/(A_g\ \Omega_t) = Min\ (F_{ty}/n_y\ ,\ F_{tu}/k_t n_u\ ,\ 1.3F_{ty}/n_y\ ,\ 1.42F_{tu}/k_t n_u)$ = 19.58 ksi, (tension bending, ADM-I D.1, D.2 & F.8.1.2)

Where k_t = 1.00 , (ADM-I Table A.3.3)

$f = Min\ (M_{rx} / S_x\ ,\ M_{ry} / S_y)$ = 7.27 ksi **<** F = 19.58 ksi

[Satisfactory]

Where S_x = 13.20 in, (Page V-7 to V-10)

S_y = 2.82 in, (Page V-7 to V-10)

CHECK COMPRESSION STRESS IN STRONG GEOMETRIC AXIS, x - x, BENDING MOMENT

$$F_{bx1} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} \ , \ for \ \dfrac{L_b}{r_y \sqrt{C_b}} \le S_1 \\ \dfrac{\left(B_c - \dfrac{D_c L_b}{1.2 r_y \sqrt{C_b}} \right)}{n_y} \ , \ for \ S_1 < \dfrac{L_b}{r_y \sqrt{C_b}} < S_2 \\ \dfrac{C_b \pi^2 E}{n_y \left(\dfrac{L_b}{1.2 r_y} \right)^2} \ , \ for \ \dfrac{L_b}{r_y \sqrt{C_b}} \ge S_2 \end{cases} = 18.89 \text{ ksi, (compression in beam, ADM-I F.2.1)}$$

Where $S_1 = 1.2\ (B_c - F_{cy}) / D_c$ = 21.31 , (ADM-I F.2.1)

$S_2 = 1.2\ C_c$ = 78.81 , (ADM-I F.2.1)

C_b = 1.00 , (ADM-I F.1.1)

$L_b / (r_y\ C_b^{0.5})$ = 40.00

$$F_{bx2} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} \ , \ for \ \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_p - \dfrac{5.0 D_p b}{t} \right)}{n_y} \ , \ for \ S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2 \sqrt{B_p E}}{n_y \dfrac{5.0b}{t}} \ , \ for \ \dfrac{b}{t} \ge S_2 \end{cases} = 20.65 \text{ ksi, (compression in flanges, ADM-I B.5.4.1)}$$

Where b / t = 7.293 , (ADM-I Figure B.5.1)

k_1 = 0.5 , (ADM-I Table B.4.3)

k_2 = 2.04 , (ADM-I Table B.4.3)

$S_1 = (B_p - F_{cy}) / (5.1\ D_p)$ = 6.68 , (ADM-I B5.4.1)

$S_2 = k_1\ B_p / (5.1\ D_p)$ = 14.97 , (ADM-I B5.4.1)

(cont'd)

$$F_{bx3} = \frac{M_n}{S_x\Omega_b} = \begin{cases} \dfrac{1.3F_{cy}}{n_y} & , \; for \;\; \dfrac{h}{t} \le S_1 \\ \dfrac{\left(B_{br} - \dfrac{mD_{br}h}{t}\right)}{n_y} & , \; for \;\; S_1 < \dfrac{h}{t} < S_2 \\ \dfrac{k_2\sqrt{B_{br}E}}{n_y \dfrac{mh}{t}} & , \; for \;\; \dfrac{h}{t} \ge S_2 \end{cases} = \quad 27.58 \quad \text{ksi, (compression in web, ADM-I B.5.5.1)}$$

Where *h / t = b / t =* 28.720 , (ADM-I Figure B.5.2)

$c_c = c_o =$ 4 in

$m =$ 0.65 , (ADM-I B5.5.1)

$B_{br} = 1.3\, F_{cy}\; [1 + F_{cy}{}^{(1/3)} / 7] =$ 66.76 , (ADM-I Table B.4.2)

$D_{br} = (B_{br} / 20)\; (6\, B_{br} / E)^{(1/3)} =$ 1.14 , (ADM-I Table B.4.2)

$S_1 = (B_{br} - 1.3\, F_{cy}) / (m\, D_{br}) =$ 0.49 , (ADM-I B5.5.1)

$S_2 = k_1\, B_{br} / (m\, D_{br}) =$ 45.11 , (ADM-I B5.5.1)

$f_{bx} = M_{rx} / S_x =$ 7.27 ksi **<** $F_{bx} = Min\,(F_{bx\,1}\,,\, F_{bx\,2}\,,\, F_{bx\,3}) =$ 18.89 ksi

[Satisfactory]

CHECK COMPRESSION STRESS IN WEAK GEOMETRIC AXIS, y - y, BENDING MOMENT

$$F_{by} = \frac{M_n}{S_y\Omega_b} = \begin{cases} \dfrac{1.3F_{cy}}{n_y} & , \; for \;\; \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_{br} - \dfrac{3.5D_{br}b}{t}\right)}{n_y} & , \; for \;\; S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{\pi^2 E}{n_y\left(\dfrac{3.5b}{t}\right)^2} & , \; for \;\; \dfrac{b}{t} \ge S_2 \end{cases} = \quad 22.85 \quad \text{ksi, (compression in web, ADM-I B.5.5.2)}$$

Where *b / t =* 7.293 , (ADM-I Figure B.5.3)

$C_{br} = 2B_{br} / (3\, D_{br}) =$ 39.10 , (ADM-I Table B.4.2)

$S_1 = (B_{br} - 1.3\, F_{cy}) / (3.5\, D_{br}) =$ 5.34 , (ADM-I B5.5.2)

$S_2 = C_{br} / 3.5 =$ 11.17 , (ADM-I B5.5.2)

$f_{by} = M_{ry} / S_y =$ 10.21 ksi **<** $F_{by} =$ 22.85 ksi

[Satisfactory]

CHECK COMBINED COMPRESSION AND BENDING (ADM-I H.1)

$$Max\left(\begin{array}{c} \dfrac{P_r}{P_c} + \dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}} = \dfrac{f_a}{F_a} + \dfrac{f_{bx}}{F_{bx}} + \dfrac{f_{by}}{F_{by}} \\ \left[\dfrac{f_a}{F_a} + \dfrac{C_{mx} f_{bx}}{F_{bx}\left(1 - f_a / F_{ex}\right)} + \dfrac{C_{my} f_{by}}{F_{by}\left(1 - f_a / F_{ey}\right)} \right] Check \end{array} \right) = \quad 1.32$$

< 1.33 , (1.33 if IBC/CBC 1605.3.2 apply)

[Satisfactory]

Where *Check =* 1 Check if ASD 2005 Eq. 4.1.1-1 apply (1 = Yes, 0 = No)

$C_{mx} =$ 0.85 $C_{my} =$ 0.85

$F_{ex} = \pi^2 E / n_u\, (kL_x / r_x)^2 =$ 238.32 ksi

$F_{ey} = \pi^2 E / n_u\, (kL_y / r_y)^2 =$ 128.38 ksi

CHECK SHEAR STRESS (ADM-I G)

$$F_s = \frac{V_n}{A_w\Omega_v} = \begin{cases} \dfrac{F_{ty}/\sqrt{3}}{n_y} & , \; for \;\; \dfrac{h}{t} \le S_1 \\ \dfrac{\left(B_s - \dfrac{1.25D_s h}{t}\right)}{n_y} & , \; for \;\; S_1 < \dfrac{h}{t} < S_2 \\ \dfrac{\pi^2 E}{n_y\left(\dfrac{1.25h}{t}\right)^2} & , \; for \;\; \dfrac{h}{t} \ge S_2 \end{cases} = \quad 12.25 \quad \text{ksi, (shear, ADM-I G.2)}$$

(cont'd)

Where $h / t = b / t =$ 28.720 , (ADM-I Figure G.2.1)

$B_s = (F_{cy} / 3^{0.5}) [1 + (F_{cy} / 3^{0.5})^{(1/3)} / 9.3] =$ 26.13 , (ADM-I Table B.4.2)

$D_s = (B_s / 10) (B_s / E)^{0.5} =$ 0.13 , (ADM-I Table B.4.2)

$C_s = 0.41 B_s / D_s =$ 80.61 , (ADM-I Table B.4.2)

$S_1 = (B_s - F_{cy} / 3^{0.5}) / (1.25 D_s) =$ 35.63 , (ADM-I G.2)

$S_2 = C_s / 1.25 =$ 64.49 , (ADM-I G.2)

$f_s = V / A_w =$ 11.25 ksi **<** $F_s =$ 12.25 ksi

[Satisfactory]

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PROJECT :
CLIENT :
JOB NO. : DATE :

PAGE :
DESIGN BY :
REVIEW BY :

Aluminum Pipe Member Capacity Based on Aluminum Design Manual 2010 (ASD)

INPUT DATA & DESIGN SUMMARY

MEMBER SIZE **PIPE 6 No. 40**

D	t	A	I_x	E	Wt (lbs/ft)
6.625	0.28	5.58	28.1	10100	6.70

TENSILE ULTIMATE STRESS (T5 to T9, Table A.3.4) F_{tu} = **38** ksi

TENSILE YIELD STRESS (T5, T6, T7, T8, or T9) F_{ty} = **35** ksi

COMPRESSIVE YIELD STRESS (T5 to T9) F_{cy} = **35** ksi

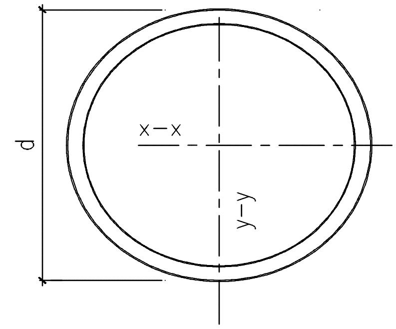
AXIAL COMPRESSION FORCE P = **40** kips, ASD

MAX GEOMETRIC AXIS EFFECTIVE LENGTH kL = **1.5** ft

GEOMETRIC AXIS BENDING MOMENT M_{rx} = **13.75** ft-kips, ASD

GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{bx} = **2** ft

MAX SHEAR LOAD, ASD V = **30** kips



THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMPRESSION STRESS IN AXIAL FORCE

$$F_{a1}=\frac{P_n}{A_g\Omega_c}=\begin{cases} Min\left(\dfrac{\left(B_c-\dfrac{D_c kL}{r}\right)}{n_u}\ ,\ \dfrac{F_{cy}}{n_y}\right)\ ,\ for\ \dfrac{kL}{r}<S_2 \\ \dfrac{\pi^2 E}{n_u\left(\dfrac{kL}{r}\right)^2}\ ,\ for\ \dfrac{kL}{r}\geq S_2 \end{cases} = 19.27$$

ksi, (compression in column, ADM-I E3)

Where r = 2.250 in, (Page V-28 to V-33)

kL / r = 8.00

E = 10100 ksi, (ADM-I Table A.3.4)

$B_c = F_{cy}\ [1 + (F_{cy} / 2250)^{0.5}]$ = 39.37 , (ADM-I Table B.4.2)

$D_c = (B_c / 10)\ (B_c / E)^{0.5}$ = 0.25 , (ADM-I Table B.4.2)

$C_c = 0.41\ (B_c / D_c)$ = 65.67 , (ADM-I Table B.4.2)

$n_u = \Omega_c / 0.85$ = 1.941 , (ADM-I E.1, E.3-2 & E.3-3)

$n_y = \Omega_c$ = 1.65 , (ADM-I B.3-2, E.1, E.3-2 & E.3-3)

$S_2 = C_c$ = 65.67 , (ADM-I E.3-4)

$$F_{a2}=\frac{P_n}{A_g\Omega_c}=\begin{cases} \dfrac{F_{cy}}{n_y}\ ,\ for\ \dfrac{R_b}{t}\leq S_1 \\ \dfrac{\left(B_t-D_t\sqrt{\dfrac{R_b}{t}}\right)}{n_u}\ ,\ for\ S_1<\dfrac{R_b}{t}<S_2 \\ \dfrac{\pi^2 E}{1.6n_u\left(\dfrac{R_b}{t}\right)\left(1+\dfrac{1}{35}\sqrt{\dfrac{R_b}{t}}\right)^2}\ ,\ for\ \dfrac{R_b}{t}\geq S_2 \end{cases} = 19.55$$

ksi, (compression in curved element, ADM-I B.5.4.5)

Where R_b / t = 11.300 , (ADM-I Page I-13)

k_1 = 0.35 , (ADM-I Table B.4.3)

k_2 = 2.27 , (ADM-I Table B.4.3)

$B_t = F_{cy}\ [1 + F_{cy}^{(1/5)} / 8.7055]$ = 43.19 , (ADM-I Table B.4.2)

$D_t = (B_t / 4.5)\ (B_t / E)^{(1/3)}$ = 1.56 , (ADM-I Table B.4.2)

C_t = 141 , (ADM-VI Table 1-1)

$S_1 = (B_t - n_u\ F_{cy} / n_y)^2 / (D_t)^2$ = 1.66 , (ADM-I B5.4.5)

$S_2 = C_t$ = 141.00 , (ADM-I B5.4.5)

(cont'd)

$f_a = P / A =$ 7.17 ksi < $F_a = Min\,(F_{a1}, F_{a2}) =$ 19.27 ksi
[Satisfactory]

CHECK TENSION STRESS IN BENDING MOMENTS

$F = P_n/(A_g\,\Omega_t) = Min\,(1.17F_{ty}/n_y\,,\ 1.24F_{tu}/k_t n_u) =$ 24.27 ksi, (tension bending, ADM-I D.1, D.2 & F.6.1)

Where $k_t =$ 1.00 , (ADM-I Table A.3.3)

$f = M_{rx} / S_x =$ 19.41 ksi < $F =$ 24.27 ksi
[Satisfactory]

Where $S_x =$ 8.50 in, (Page V-28 to V-33)

CHECK COMPRESSION STRESS IN BENDING MOMENT

$$F_{bx} = \frac{M_n}{S_x\Omega_b} = \begin{cases} \dfrac{1.17F_{cy}}{n_y} \ , \ for \ \dfrac{R_b}{t} \le S_1 \\ \dfrac{1}{n_y}\left(B_{tb} - D_{tb}\sqrt{\dfrac{R_b}{t}}\right) \ , \ for \ S_1 < \dfrac{R_b}{t} < S_2 \\ \dfrac{\pi^2 E}{1.6n_u\left(\dfrac{R_b}{t}\right)\left(1+\dfrac{1}{35}\sqrt{\dfrac{R_b}{t}}\right)^2} \ , \ for \ \dfrac{R_b}{t} \ge S_2 \end{cases} = 24.82$$

ksi, (compression in beam, ADM-I F.6)

Where $R_b / t =$ 11.300 , (Page I-13)

$B_{tb} = 1.5\,F_{cy}\,[1 + F_{cy}{}^{(1/5)} / 8.7055] =$ 64.78 , (ADM-I Table B.4.2)

$D_{tb} = (B_{tb} / 2.7)\,(B_{tb} / E)^{(1/3)} =$ 4.46 , (ADM-I Table B.4.2)

$S_2 = (B_{tb} - B_t)^2 / (D_{tb} - D_t)^2 =$ 55.44 , (ADM-I F.6.2)

$S_2 = C_t = (n_u\,B_{tb} / n_y - B_t)^2 / (n_u\,D_{tb} / n_y - D_t)^2 =$ 80.24 , (ADM-I F.6.2 & ADM 2005 Eq. 3.4.12-4)

$f_{bx} = M_{rx} / S_x =$ 19.41 ksi < $F_{bx} =$ 24.82 ksi
[Satisfactory]

CHECK COMBINED COMPRESSION AND BENDING (ADM-I H.1)

$$Max\left(\begin{array}{l} \dfrac{P_r}{P_c} + \dfrac{M_{rx}}{M_{cx}} = \dfrac{f_a}{F_a} + \dfrac{f_{bx}}{F_{bx}} \\ \left[\dfrac{f_a}{F_a} + \dfrac{C_{mx} f_{bx}}{F_{bx}\left(1 - f_a / F_{ex}\right)}\right] Check \end{array}\right) = 1.15$$

< 1.33 , (1.33 if IBC/CBC 1605.3.2 apply)
[Satisfactory]

Where *Check* = 1 Check if ASD 2005 Eq. 4.1.1-1 apply (1 = Yes, 0 = No)

$F_{ex} = \pi^2 E / n_u (kL_x / r_x)^2 =$ 802.37 ksi, $C_{mx} =$ 0.85

CHECK SHEAR STRESS (ADM-I G)

$$F_s = \frac{V_n}{A_w\Omega_v} = \begin{cases} \dfrac{F_{ty}/\sqrt{3}}{n_y} \ , \ for \ \dfrac{h}{t} \le S_1 \\ \dfrac{\left(B_s - \dfrac{1.25 D_s h}{t}\right)}{n_y} \ , \ for \ S_1 < \dfrac{h}{t} < S_2 \\ \dfrac{\pi^2 E}{n_y\left(\dfrac{1.25h}{t}\right)^2} \ , \ for \ \dfrac{h}{t} \ge S_2 \end{cases} = 12.25$$

ksi, (shear, ADM-I G.2)

Where $h / t = b / t =$ 23.66 , (for strong shear, Figure G.2)

$B_s = (F_{cy} / 3^{0.5})\,[1 + (F_{cy} / 3^{0.5})^{(1/3)} / 9.3] =$ 26.13 , (ADM-I Table B.4.2)

$D_s = (B_s / 10)\,(B_s / E)^{0.5} =$ 0.13 , (ADM-I Table B.4.2)

$C_s = 0.41\,B_s / D_s =$ 80.61 , (ADM-I Table B.4.2)

$S_1 = (B_s - F_{cy} / 3^{0.5}) / (1.25\,D_s) =$ 35.63 , (ADM-I G.2)

$S_2 = C_s / 1.25 =$ 64.49 , (ADM-I G.2)

$f_s = V_{strong} / A_w =$ 2.26 ksi < $F_s =$ 12.25 ksi, (for strong shear)
[Satisfactory]

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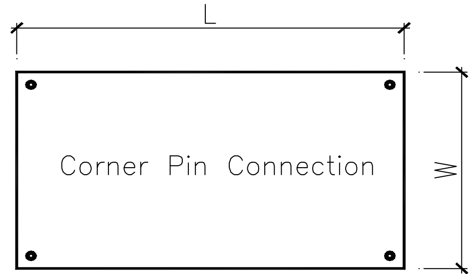
Structural Glass Design, Based on ASTM E1300 & CAN/CGSB-12.20, using Finite Element Method

INPUT DATA & DESIGN SUMMARY

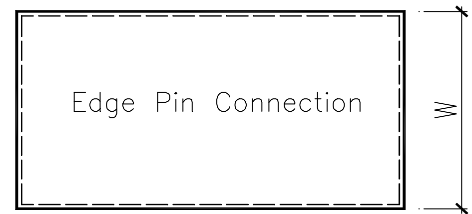
GLASS PANEL SIZE W = **1.5** *ft, (457 mm)*
L = **5** *ft, (1524 mm)*



GLASS PANEL THICKNESS t = **0.8661** *in, (22 mm)*
Weight = 85 *lbs, (38 kg)*

CONNECTION TYPE (0 or 1) **0** *, only corner pinned.*

ALLOWABLE GLASS STRENGTH σ_{allow} = **3.5** *ksi,* *(24 N /* mm^2)
(Annealed: 3.5 ksi, Heat-strengthened: 6.5 ksi, Tempered: 14.5 ksi - verify reqd.)



ALLOWABLE DEFLECTION ***L*** / **240**
(The max value suggested: L / 60 for window or curtain wall, L / 180 for stair.)

UNIFORM AREA LOAD (Perpendicular to Plane) D = **10** *psf, ASD level*
POINT LOAD (Including Impact Factor) P = **0.3** *kips, ASD level*

THE DESIGN IS ADEQUATE.

ANALYSIS

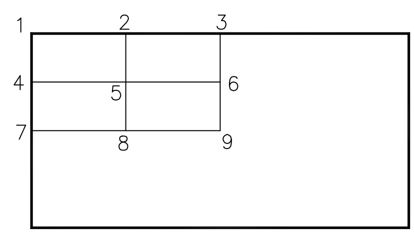
GLASS PROPERTIES

γ = 156 *lbs / ft*^{3} *(2500 kg / m*^{3}*)*
E = 10150 *ksi (70 kN / mm*^{2}*)*
υ = 0.22 *, Poisson's ratio*

JOINT DEFLECTIONS, REACTIONS, & PLATE SECTION FORCES
P = 0.3 kips,(*Point load at Joint 9.)*

Joint Number	Δ in	R kips
1	0	-0.09
2	0.11	
3	0.16	
4	0.00	
5	0.11	
6	0.15	
7	0.00	
8	0.11	
9	0.15	

Bending Section	M ft-k/ft
7 - 8	0.03
8 - 9	0.04
3 - 6	0.42
6 - 9	0.42



CHECK BENDING CAPACITY

$M_n / \Omega_b = \sigma_{allow}\ d\ t^2 / 6$ = 0.44 *ft-k/ft* **>** M = 0.42 *ft-k/ft*
[Satisfactory]

Where d = 12 *in, (1 ft)*
$M = (M_{6\text{-}9}{}^2 + M_{8\text{-}9}{}^2)^{0.5}$ = 0.42 *ft-k/ft*

CHECK DEFLECTION

Δ_{max} = 0.15 *in* **<** *L* / 240 = 0.25 in
[Satisfactory]

Where $L = Max\ (L\ ,\ W)$ = 60.0 *in*

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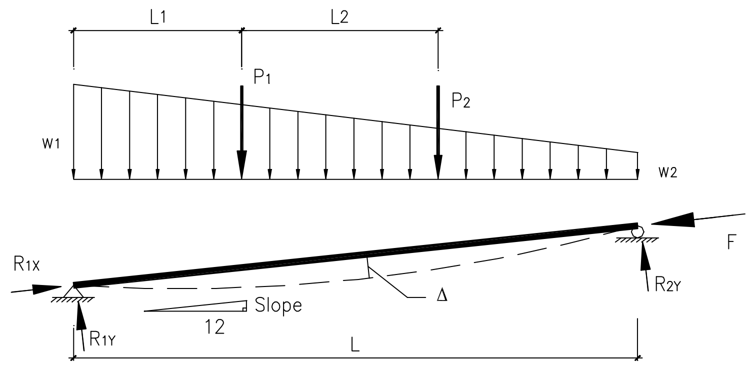
P-Delta Effect Analysis by Finite Element Method



DESIGN CRITERIA

1. The P-Delta effect always exists when the structural member concurrently supported bending and axial loads, not only for column but also for beam, brace, and diaphragm compression collector.
2. This analysis can be applied to any pinned ends building or bridge structural member of aluminum, steel, wood, or even concrete.
3. The finite element deflections are based on $1 / \rho = M / (EI)$.

INPUT DATA & SUMMARY

DIMENSIONS L = **30** ft

L_1 = **12** ft

L_2 = **8** ft

Slope = **2** **: 12** (θ = 9.46 ^{0})

MEMBER SECTION I_x = **96** in^4

E = **29000** ksi

LOADS (vertical, along horizontal, ASD)

w_1 = **0.23** kips/ft , w_2 = **0.194** kips/ft

P_1 = **3** kips , P_2 = **1.3** kips

MEMBER AXIAL FORCE F = **12** kips, ASD

***Δ* = 3.03 in, (perpendicular)**

***Member Length* = 30.41 ft, (sloped)**

Design Section Forces (ASD level):

***P* = 13.75 kips, (axial)**

***M* = 52.92 ft-kips, (bending)**

***V* = 5.43 kips, (shear)**

Reactions (ASD level):

***R_{1X}* = 13.75 kips, (axial)**

***R_{1Y}* = 5.43 kips, (perpendicular)**

***R_{2Y}* = 5.09 kips, (perpendicular)**

THE DEFLECTION IS ADEQUATE.

ANALYSIS

		No	P-Δ Effect with w, P, F							
		P-Δ	1 st	2 nd	3 rd	4 th	5 th	…	Final	X (ft)
P	kips	13.75	13.75	13.75	13.75	13.75	13.75		13.75	0.00
M	ft-kips	49.83	52.72	52.90	52.92	52.92	52.92		52.92	12.44
V	kips	5.43	5.43	5.43	5.43	5.43	5.43		5.43	0.00
Δ	in	2.84	3.02	3.03	3.03	3.03	3.03		3.03	15.14

CHECK DEFLECTION

Δ_{max} = 3.03 *in* < *L /* ***120*** = 3.04 in

[Satisfactory]

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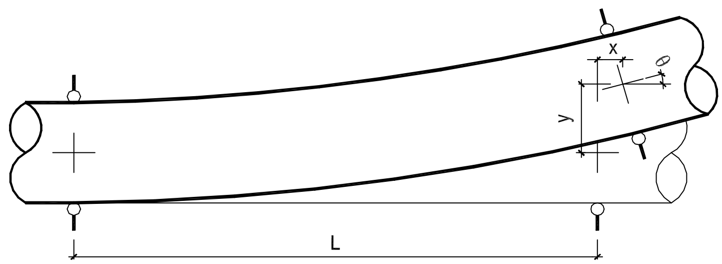
Copper Pipe Design using Finite Element Method

DESIGN CRITERIA

1. The copper pipes of gas or water, which attached on building structure or embedded under ground, may be broken early due to story drifts, foundation settlement, and/or seismic joint movement.
2. This software loads/forces are from the deformations between the two copper pipe connections/clamps, not from gravity weight.

INPUT DATA & SUMMARY



LENGTH BETWEEN CLAMPS

L = **10** *ft, (3.0 m)*

DEFORMATIONS (service level, ASCE 7-22 allowable drifts / 1.4)

x = **0.06** *in, (1.5 mm)*

y = **5.5** *in, (139.7 mm)*

θ = **4** *deg, (0.070 rad)*

COPPER SECTION (outside diameter and wall thickness)

D = **1.125** *in, (28.6 mm)*

t = **0.05** *in, (1.3 mm)*

THE DESIGN IS ADEQUATE.

COPPER MODULUS OF ELASTICITY

E = **17000** *ksi, (117.2 Gpa)*

COPPER POISSION'S RATIO

υ = **0.3**

YIELD STRENGTH

F_{ty} = **20** *ksi, (138 MPa)*

SHEAR STRENGTH

F_v = **25** *ksi, (172 MPa)*

ANALYSIS

Joint	Coordinates (ft) X	Y	Joint I Deflection Δ_x (in)	Δ_y (in)	Δ_θ (deg)
1	0	0	0.00	0.00	0.00
2	1.25	0	0.01	0.12	0.91
3	2.50	0	0.02	0.47	1.70
4	3.75	0	0.02	1.00	2.38
5	5.00	0	0.03	1.70	2.94
6	6.25	0	0.04	2.53	3.38
7	7.50	0	0.05	3.46	3.70
8	8.75	0	0.05	4.46	3.91
9	10	0	0.06	5.50	4.00

Max (N / A + 0.5 D M / I) = ***19.3*** ksi
< ***20.0*** ksi
[Satisfactory]

Max (V / A) = ***0.022*** ksi
< ***25.0*** ksi
[Satisfactory]

Element	End Joint I	J	A (in^2)	I (in^4)	E (ksi)	Length (ft)	N, Axial (k) I	J	V, Shear (k) I	J	M, Moment (ft-k) I	J
1	1	2	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.04	-0.03
2	2	3	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.03	-0.03
3	3	4	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.03	-0.02
4	4	5	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.02	-0.02
5	5	6	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.02	-0.02
6	6	7	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.02	-0.01
7	7	8	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.01	-0.01
8	8	9	0.168861	0.024445	17000	1.25	-1.44	1.44	0.00	0.00	0.01	0.00

Daniel T. Li
Engineering International

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Unit Conversions between U.S. Customary System & Metric System

U.S. Customary System to Metric System

	W14X26		==>	W360X39	
Rebar #	**6**	(0.75 in)	==>	19.1	mm, diameter.
Metal Mils	**18**	(0.0188 in)	==>	0.478	mm, thickness

Multiply		*By*	*To Obtain*	
10	kips	4.448	44.482	kN
20	lbf (lbs)	4.448	88.964	N
30	ft-kips	1.356	40.674	kN•m
40	in-lbf	112.984	4519.371	N•mm
100	psf (lbf / ft^2)	47.882	4788.159	N / m^2
1.3	ksf (kips / ft^2)	47.880	62.244	kPa
4000	psi (lbf / in^2)	0.007	27.579	MPa
5	ksi (kips / in^2)	6.895	34.474	N / mm^2
29000	ksi (kips / in^2)	0.007	199.948	GPa
16	ft (192 in)	0.305	4.877	m
20	in	25.400	508.000	mm
100	ft^2	0.093	9.290	m^2
1	yd (3 ft)	0.914	0.914	m
150	yd^3	0.765	114.683	m^3
1	lbm (lbs)	0.454	0.454	kg
20	psf (lbs / ft^2)	4.883	97.651	kg / m^2
150	pcf (lbs / ft^3)	16.018	2402.757	kg / m^3
145	pcf (lbf / ft^3)	0.157	22.780	kN / m^3
10	kips-sec^2/ ft	1488	14882	kg-s^2 / m
1	*g*	9.807	9.807	m / s^2
1	gal water	8.34 lbs / gal	3.783	kg
1	ft^3 water	62.4 lbs / ft^3	28.304	kg
100	^0F	(^{0}C - 32) / 1.8	37.778	^0C
1	acre	4047	4047	m^2
1	ton	2000	2000	lbs

Metric System to U.S. Customary System

HSS203.2X203.2X6.4		==>	HSS8X8X1/4
25	mm, diameter rebar,		# 7, (0.875 in)
1.15	mm, thickness,		Metal Mils 43, (0.0451 in)

Multiply		*By*	*To Obtain*	
100	kN	0.225	22.481	kips
200	N	0.225	44.962	lbf (lbs)
300	kN•m	0.738	221.272	ft-kips
400	N•mm	0.009	3.540	in-lbf
1000	N / m^2	0.021	20.885	psf (lbf / ft^2)
65	kPa	0.021	1.358	ksf (kips / ft^2)
30	MPa	145.038	4351.153	psi (lbf / in^2)
500	N / mm^2	0.145	72.519	ksi (kips / in^2)
200	GPa	145.038	29007.548	ksi (kips / in^2)
1	m	3.281	3.281	ft (39 in)
1	mm	0.039	0.039	in
10	m^2	10.764	107.643	ft^2
1	m	1.094	1.094	yd
115	m^3	1.308	150.414	yd^3
0.5	kg	2.205	1.102	lbm (lbs)
100	kg / m^2	0.205	20.481	psf (lbs / ft^2)
2500	kg / m^3	0.062	156.071	pcf (lbs / ft^3)
24	kN / m^3	6.365	152.769	pcf (lbf / ft^3)
15000	kg-s^2 / m	0.000672	10.080	kips-sec^2/ ft
1	g	32.174	32.174	ft / sec^2
			(386	in / sec^2)
1	m^3 water	1000 kg / m^3	2204.634	lbs
37	^0C	1.8 ^{0}C + 32	98.600	^0F
1	acre	43560	43560	ft^2
1	ton	907.180	907	kg

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