PRCNC20251084

ENGINEERING ANALYSIS FOR:
TRASH ENCLOSURE
EAST TOWN CROSSING
COMMERCIAL LOTS 1&2
3002 E PIONEER WAY
PUYALLUP, WA 98372
PARCEL NO: 0420264053

DESIGN CRITERIA

BUILDING CODE: 2021 INTERNATIONAL BUILDING CODE (IBC) AS AMENDED BY THE LOCAL JURISDICTION.

VERTICAL LOADS

ROOF LIVE LOAD: 30 PSF (SNOW) - LOW SLOPE ROOF
ROOF DEAD LOAD: 15 PSF

SNOW DESIGN DATA (ASCE 7-22)
FLAT SNOW LOAD: N/A
SNOW EXPOSURE FACTOR, Ce=1.0,
SNOW IMPORTANCE FACTOR, Is=1.0,
THERMAL FACTOR, Ct=1.2

WIND DESIGN DATA (ASCE 7-22)
BASIC WIND SPEED (ASD) V= 85MPH
ULTIMATE WIND SPEED V= 110MPH
RISK CATEGORY: II EXPOSURE: B
IMPORTANCE FACTOR, Iw= 1.0
TOPOGRAPHIC FACTOR, Kzt= 1.0

SEISMIC DESIGN DATA (ASCE7-22)
SEISMIC RESPONSE SYSTEM: STEEL ORDINARY CONCENTRICALLY BRACED FRAMES
EQUIVALENT LATERAL FORCE PROCEDURE (ASCE 7-22) R=3.25
RISK CATEGORY: II SEISMIC IMPORTANCE FACTOR, Ie= 1.0
MAPPED SPECTRAL RESPONSE ACCELERATION: Ss=1.42, S1=0.43
DESIGN SPECTRAL RESPONSE ACCELERATION: Sds=1.03, Sd1=0.61
SITE CLASS: D SEISMIC DESIGN CATEGORY: D
SEISMIC RESPONSE COEFFICIENT: Cs= 0.23
DESIGN BASE SHEAR: 3,508#

SOIL PROPERTIES:
BEARING CAPACITY: 2,000 PSF
LATERAL CAPACITY: 250 PSF/FT

City of Puyallup
Building
REVIEWED FOR COMPLIANCE
SKinnear
09/04/2025
8:58:30 AM

Calculations required to be provided by the Permittee on site for all Inspections

PIERUCCIONI E&C, LLC
CHON PIERUCCIONI, PE
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EAST TOWN CROSSING
COMMERCIAL LOTS 1&2 - TRASH ENCLOSURE
3002 PIONEER WAY PUYALLUP WA 98372
P/N: 0420264053

REVISIONS	
01	----

REVISIONS

ENGINEER:	CP
CHECKED BY:	CP
DATE:	2025.08.07
TITLE:	STRUCTURAL ANALYSIS
PROJECT # :	----

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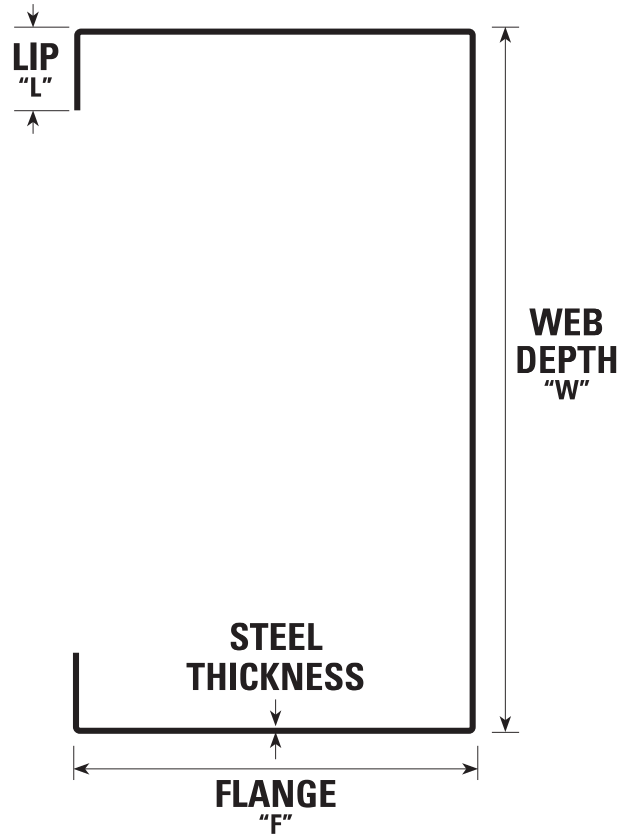
Expanding Your Solutions

1000S162-54 C-STUDS 54 MIL. (16 GA. STRUCTURAL)

Geometric Properties

1000S162-54 "S" structural load-bearing studs are produced from hot-dipped galvanized steel in standard CP60 coating. CP90 is available upon special request, and may require up-charges and extended lead times.



Physical Properties

Model No.	Design Thickness (in)	Minimum Thickness (in)	Yield (ksi)	Coating[3,4]	Web Depth (in)	Flange Size (in)	Lip (in)
1000S162-54	0.0566	0.0538	50	CP60	10	1-5/8	1/2

Notes:
1. Uncoated steel thickness. Thickness is for carbon sheet steel.
2. Minimum thickness represents 95% of the design thickness and is the minimum acceptable thickness.
3. Per ASTM C955 & A1003, Table 1.
4. CP90 available upon request. Will require extended lead time and upcharge.

Color Code (painted on ends): 54-mil: Green

ASTM & Code Standards:

- ASTM A653/A653M, A924/A924M, A1003/1003, C955 & C1007
- ICC-ES & SFIA Code Compliance Certification Program
- ICC ESR-3016
- ATI CCRR-0224
- IBC: 2015, 2018, 2021
- CBC: 2019, 2022
- AISI: S100, S200, S240

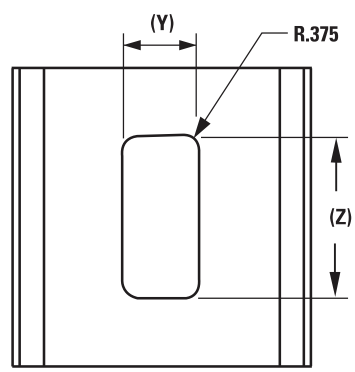
LEED v4 for Building and Design Construction

- MR Prerequisite: Construction and Demolition Waste Management Planning.
- MR Credit: Construction and Demolition Waste Management.
- MR Credit: Building Product Disclosure and Optimization – Sourcing of Raw Materials, Option 2.
- MR Credit: Building Product Disclosure and Optimization – Environmental Product Declarations, Options 1 & 2.
- MR Credit: Building Product Disclosure and Optimization – Material Ingredients, Option 1.
- MR Credit: Building Life-Cycle Impact Reduction, Option 4.

CEMCO cold-formed steel framing products contain 30% to 37% recycled steel.

- Total Recycled Content: 36.9%
- Post-Consumer: 19.8%
- Pre-Consumer: 14.4%

CSI Division: 05.40.00 – Cold-Formed Metal Framing



Hole Detail

Standard Hole Centers are 24"	(Z) (in)	(Y) (in)
2-1/2" studs	2.000	0.750
3-1/2" to 14" studs	3.250	1.500

1000S162-54 Section Properties

Design Thickness (in.)	Fy (ksi)	Gross[3]					Effective Properties[2]						Torsional Properties						Lu (in)
		Ix (in^4)	Sx (in^3)	Rx (in)	Iy (in^4)	Ry (in)	Ix (in^4)	Sx (in^3)	Ma (in-k)	Vag (lb)	Vanet (lb)	Mad (in-k)	Jx1000 (in^4)	Cw (in^6)	Xo (in)	m (in)	Ro (in)	ß	
0.0566	50	9.950	1.990	3.565	0.204	0.511	9.391	1.572	47.07	1661	1661	40.37	0.836	4.198	-0.812	0.538	3.692	0.952	31.3

Notes: 1. Web depth for track sections equals nominal depth plus 2 times the design thickness plus bend radius. **2.** The values are for members with punch-outs. **3.** Gross properties are based on the full, unreduced cross-section, away from web punchouts. **4.** Use the effective moment of inertia for deflection calculation. **5.** Allowable moment is lesser of Ma and Mad. Distortional buckling is based on an assumed $K\phi = 0$. **6.** These members are available un-punched only.

Check the updated list of Certified Production Facilities at Intertek's website at http://www.intertek.com/building/sfia

This technical information reflects the most current information available and supersedes any and all previous publications effective May 1, 2024.
05/01/2024 AT

22 ga 1.5B-36 Grade 50 Roof Deck

Diaphragm Shear & Wind Uplift Interaction

For Both Ends Lapped Deck
with MWFRS Allowable Net Wind Uplift (ASD) of 20 psf

#10 Screw Connections to Supports
36 / 4 Perpendicular Connection Pattern to Supports
#10 Screw Sidelap Connections

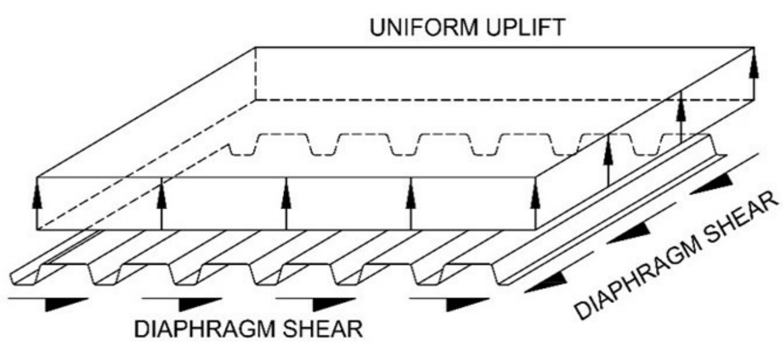
A572 GR50 Support Member or Equivalent
0.054 ≤ Support Thickness (in.) ≤ 0.175
1.63 in. Minimum Deck End Bearing Length

ASD Allowable Combined Wind Uplift & Diaphragm Shear Strength Sn/Ω (plf) Generic 3 Span Condition

Sidelap Connections per Span	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
1	271	198	147	111	85	-	-	-	-
2	302	235	184	145	114	89	67	44	-
3	319	258	208	167	134	105	78	48	10
4	330	273	224	183	147	115	84	50	10
5	337	283	235	193	156	122	87	51	10
6	341	290	243	200	162	126	90	52	10
7	344	294	248	206	166	129	91	52	10

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
1	24	18	24	20	24	-	-	-	-
2	18	18	24	20	24	21	19	15	-
3	12	18	16	20	18	17	16	14	12
4	11	17	16	15	14	17	14	14	12
5	9	14	16	15	14	14	14	14	12
6	8	12	16	15	14	14	14	14	12
7	7	10	14	15	14	14	14	12	12



22 ga 1.5B-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck

#10 Screw Connections to Supports
36 / 4 Perpendicular Connection Pattern to Supports
#10 Screw Sidelap Connections

A572 GR50 Support Member or Equivalent
0.054 ≤ Support Thickness (in.) ≤ 0.175
1.63 in. Minimum Deck End Bearing Length

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.) — Generic 3 Span Condition

Sidelap Connections per Span	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
1	125	101	85	73	64	-	-	-	-
2	142	118	101	88	78	70	64	58	-
3	155	131	114	101	90	82	75	69	64
4	164	142	125	111	101	92	84	78	72
5	172	151	134	121	110	101	93	86	80
6	178	158	142	129	118	108	100	94	88
7	184	164	149	136	125	115	107	100	94

ASD Allowable Seismic Diaphragm Shear Strength Sn/Ω (plf) — Generic 3 Span Condition

Sidelap Connections per Span	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
1	309	245	198	165	139	-	-	-	-
2	344	287	240	204	176	154	134	119	-
3	366	317	273	237	207	183	164	148	133
4	381	340	300	264	235	210	189	171	157
5	392	357	321	288	258	233	211	193	177
6	399	370	338	307	279	254	232	213	196
7	404	380	352	323	297	272	250	231	214

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
1	24	36	24	20	24	-	-	-	-
2	18	18	24	20	24	21	24	27	-
3	14	18	24	20	24	21	24	27	24
4	11	17	16	20	24	21	24	27	24
5	9	14	16	20	24	21	24	27	24
6	8	12	16	20	18	21	24	27	24
7	7	10	14	15	18	21	24	22	24

Bare Deck Diaphragm V5.4 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-0652
CAN/CSA-S136 (R2021) for Canadian references

Date: 5/29/2025

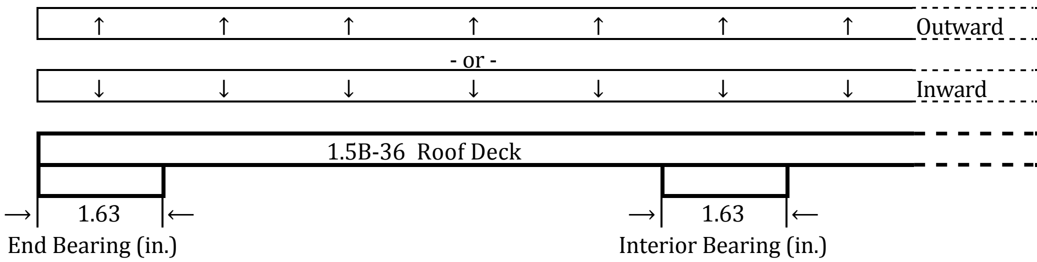
22 Gage 1.5B-36 Grade 50

Uniform Allowable Load Table, ASD (psf)

For End Lapped Deck

36 / 4 Connection Pattern to Supports with
#10 Screw

Support Member A572 GR50
$0.0538 \le t_2$ (in.) ≤ 0.175



Inward Uniform Allowable Load Table, ASD (psf)

Span	(ft-in)	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"
1	Wn/Ω	828	540	375	275	211	167	135	111
	L/240	-	-	-	237	159	112	81	61
2	Wn/Ω	494	395	329	282	219	173	141	117
	L/240	-	-	-	-	-	-	-	-
3	Wn/Ω	561	449	374	321	271	215	175	145
	L/240	-	-	-	-	-	-	-	132

Uplift (Outward) Uniform Allowable Load Table, ASD (psf)

Span	(ft-in)	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"
1	Wn/Ω	94	76	63	54	47	42	38	34
	Rn/Ω	94	76	63	54	47	42	38	34
	L/240	-	-	-	-	-	-	-	-
2	Wn/Ω	76	60	50	43	38	34	30	27
	Rn/Ω	76	60	50	43	38	34	30	27
	L/240	-	-	-	-	-	-	-	-
3	Wn/Ω	86	69	57	49	43	38	34	31
	Rn/Ω	86	69	57	49	43	38	34	31
	L/240	-	-	-	-	-	-	-	-

Steel Deck Properties

t in	Fy ksi	wdd psf	Id+ $in.^4/ft$	Id- $in.^4/ft$	Se+ $in.^3/ft$	Se- $in.^3/ft$	Mn+/Ω lbs-ft/ft	Mn-/Ω lbs-ft/ft	Vn/Ω lbs/ft
0.0295	50	1.6	0.155	0.178	0.169	0.179	422	447	2654

Where: $W \le Wn/\Omega$

W = Required strength of the governing ASD load combintaion

Wn/Ω = Allowable Strength

Bare Deck Uniform Load V3.1 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-0652
CAN/CSA-S136 (R2021) for Canadian references

Date: 5/29/2025

ASDIP Steel 5.6.4.1 **STEEL BEAM DESIGN** *www.asdipsoft.com*

GEOMETRY

Beam Designation HSS8X6X1/4

Span	Length		Support	Type
1	9.50	ft	1	Pinned
2	5.50	ft	2	Pinned
3	4.50	ft	3	Pinned
4	5.50	ft	4	Pinned
5	9.50	ft	5	Pinned
			6	Pinned

PROPERTIES

Weight	22	lb/ft	Sx ...	14.2	in³
Area .	6.2	in²	Zx ...	16.9	in³
Depth	8.0	in	rx	3.03	in
Width	6.0	in	Iy	36.4	in⁴
tnom .	0.25	in	Sy ...	12.1	in³
tdes ..	0.23	in	Zy ...	13.9	in³
Ix	56.6	in⁴	ry	2.43	in
			J	70.30	in⁴

ASD SUPPORT REACTIONS (kip)

Load Comb.						
D+L	0.3	0.6	0.8	0.8	0.6	0.3
D+Lr	0.3	0.6	0.8	0.8	0.6	0.3
D+S	0.6	1.2	3.0	3.0	1.2	0.6
D+0.75L+0.75Lr	0.3	0.6	0.8	0.8	0.6	0.3
D+0.75L+0.75S	0.5	1.1	2.5	2.5	1.1	0.5
D+0.6W	0.3	0.6	0.8	0.8	0.6	0.3
D+0.7E	0.3	0.6	0.8	0.8	0.6	0.3
D+0.75L+0.75Lr+0.45W	0.3	0.6	0.8	0.8	0.6	0.3
D+0.75L+0.75S+0.45W	0.5	1.1	2.5	2.5	1.1	0.5
D+0.75L+0.75S+0.525E	0.5	1.1	2.5	2.5	1.1	0.5
0.6D+0.6W	0.2	0.4	0.5	0.5	0.4	0.2
0.6D+0.7E	0.2	0.4	0.5	0.5	0.4	0.2
CD	0.1	0.2	0.1	0.1	0.2	0.1

DESIGN FOR SHEAR

Maximum Shear Force V = 2.5 kip (Comb: D+S)

$h = Ht - 3 * t =$ 8.0 - 3 * 0.2 = 7.3 in

$Aw = 2 * h * t =$ 2 * 7.3 * 0.2 = 3.4 in² $kv = 5$ AISC G4

$h / t =$ 7.3 / 0.2 = 31.3 $< 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}}$ = 59.2

$Cv = 1.00$ AISC Eq. (G2-3)

- Shear Yielding

Nominal strength $Vn = 0.6 * Fy * Aw =$ 0.6 * 50.0 * 3.4 = 102.1 kip

- Shear Buckling

Nominal strength $Vn = 0.6 * Fy * Aw * Cv =$ 0.6 * 50.0 * 3.4 * 1.00 = 102.1 kip AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

Shear allowable strength = $Vn / \Omega =$ 102.1 / 1.67 = 61.1 kip

Shear design ratio = $\frac{V}{Vn / \Omega} = \frac{2.5}{61.1}$ = 0.04 < 1.0 OK AISC G1

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing .... Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment M = 3.7 k-ft (Comb: D+S)

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* AISC Eq F1-1

= 12.5 * 3.7 * 1.0 / (2.5 * 3.7 + 3 * 1.5 + 4 * 1.5 + 3 * 1.5) = 3.00

- Yielding AISC F7.1

Plastic moment *Mpx = Fy * Zx =* 50.0 * 16.9 = 845.0 k-in AISC Eq. F7-1

Nominal strength *Mnx = Mpx =* 845.0 / 12 = 70.4 k-ft

- Flange Local Buckling AISC F7.2

Nominal strength *Mnx = N.A.* (compact flanges) AISC F7.2(a)

- Web Local Buckling AISC F7.3

Nominal strength *Mnx = N.A.* (compact webs) AISC F7.3(a)

- Controlling limit state: Yielding

Flexural allowable strength = *Mnx / Ω =* 70.4 / 1.67 = 42.2 k-ft

Flexural design ratio = $\frac{Mx}{Mnx / \Omega} = \frac{3.7}{42.2}$ = 0.09 < 1.0 OK AISC F1

DEFLECTIONS

Stiffness factor 1.0

Required Camber .... 0.00 in

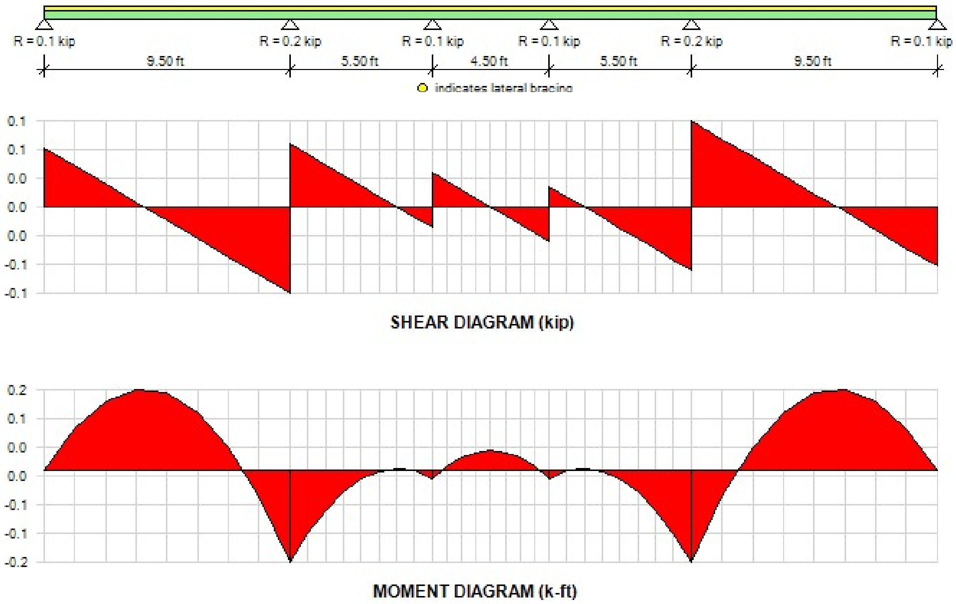
Long-term Deflection .... N.A.

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	5400	360	0.07	OK
CD+CL	0.00	5400	240	0.04	OK
L	0.00	5400	360	0.07	OK
D+L	0.00	5400	240	0.04	OK

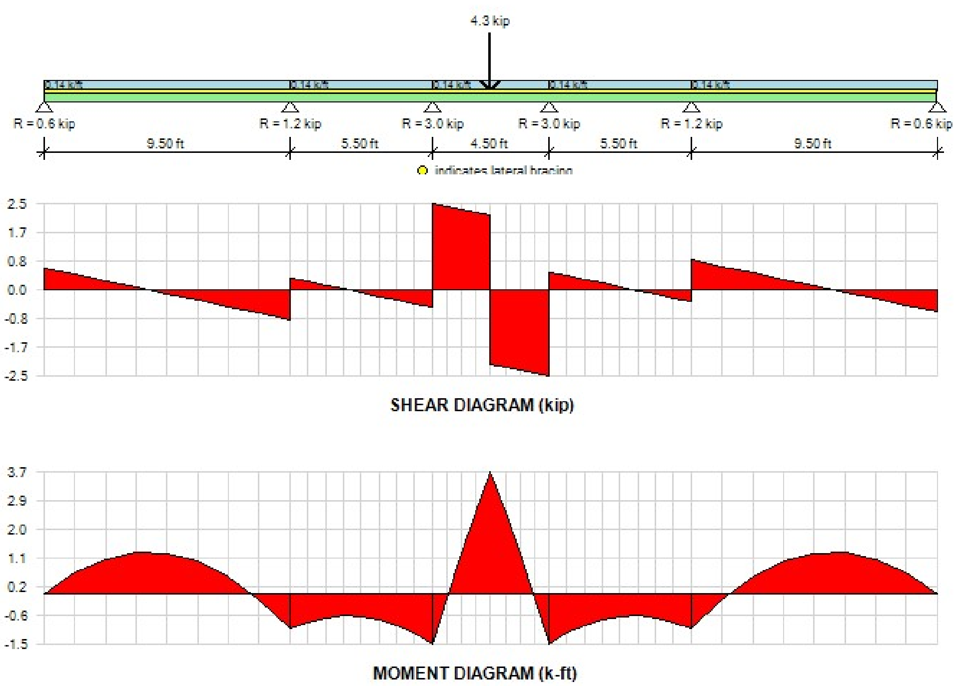
DESIGN CODES

Steel Design .... AISC 360-16

Load Combinations .... ASCE 7-10/16



(Comb: CD)



(Comb: D+S)

UNFACTORED FINAL LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

Span 1	Start	End	Width	Dead	Live	RLive	Snow	Wind	Seismic
w1	0.00	9.50	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 2									
w1	0.00	5.50	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 3									
w1	0.00	4.50	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 4									
w1	0.00	5.50	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 5									
w1	0.00	9.50	3.00	15.00	0.00	0.00	30.00	0.00	0.00

Span 3	Dist	Dead	Live	RLive	Snow	Wind	Seismic
P1	2.25	0.9	0.0	0.0	3.4	0.0	0.0

UNFACTORED CONSTRUCTION LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

	Start	End	Width	Dead	Live

	Dist	Dead	Live

GEOMETRY

Beam Designation HSS8X6X1/4

Span	Length		Support	Type
1	17.30	ft	1	Pinned
2	17.30	ft	2	Pinned
3	N.A.		3	Pinned
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

PROPERTIES

Weight	22	lb/ft	Sx ...	14.2	in^3
Area .	6.2	in^2	Zx ...	16.9	in^3
Depth	8.0	in	rx	3.03	in
Width	6.0	in	Iy	36.4	in^4
tnom .	0.25	in	Sy ...	12.1	in^3
tdes ..	0.23	in	Zy ...	13.9	in^3
Ix	56.6	in^4	ry	2.43	in
			J	70.30	in^4

ASD SUPPORT REACTIONS (kip)

Load Comb.			
D+L	0.4	1.5	0.4
D+Lr	0.4	1.5	0.4
D+S	1.0	3.4	1.0
D+0.75L+0.75Lr	0.4	1.5	0.4
D+0.75L+0.75S	0.9	2.9	0.9
D+0.6W	0.4	1.5	0.4
D+0.7E	0.4	1.5	0.4
D+0.75L+0.75Lr+0.45W	0.4	1.5	0.4
D+0.75L+0.75S+0.45W	0.9	2.9	0.9
D+0.75L+0.75S+0.525E	0.9	2.9	0.9
0.6D+0.6W	0.3	0.9	0.3
0.6D+0.7E	0.3	0.9	0.3
CD	0.1	0.5	0.1

DESIGN FOR SHEAR

Maximum Shear Force V = 1.7 kip (Comb: D+S)

$h = Ht - 3 * t =$ 8.0 - 3 * 0.2 = 7.3 in

$Aw = 2 * h * t =$ 2 * 7.3 * 0.2 = 3.4 in^2 $kv = 5$ AISC G4

$$h / t = 7.3 / 0.2 = 31.3 < 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}} = 59.2$$

$Cv = 1.00$ AISC Eq. (G2-3)

- Shear Yielding

Nominal strength $Vn = 0.6 * Fy * Aw$ = 0.6 * 50.0 * 3.4 = 102.1 kip

- Shear Buckling

Nominal strength $Vn = 0.6 * Fy * Aw * Cv$ = 0.6 * 50.0 * 3.4 * 1.00 = 102.1 kip AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

Shear allowable strength = Vn / Ω = 102.1 / 1.67 = 61.1 kip

$$\text{Shear design ratio} = \frac{V}{Vn / \Omega} = \frac{1.7}{61.1} = 0.03 < 1.0 \text{ OK}$$ AISC G1

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing .... Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment M = -5.9 k-ft (Comb: D+S)

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* AISC Eq F1-1

= 12.5 * -5.9 * 1.0 / (2.5 * -5.9 + 3 * 2.9 + 4 * 2.9 + 3 * 0.0) = 2.08

- Yielding AISC F7.1

Plastic moment *Mpx = Fy * Zx =* 50.0 * 16.9 = 845.0 k-in AISC Eq. F7-1

Nominal strength *Mnx = Mpx =* 845.0 / 12 = 70.4 k-ft

- Flange Local Buckling AISC F7.2

Nominal strength *Mnx = N.A.* (compact flanges) AISC F7.2(a)

- Web Local Buckling AISC F7.3

Nominal strength *Mnx = N.A.* (compact webs) AISC F7.3(a)

- Controlling limit state: Yielding

Flexural allowable strength = *Mnx / Ω =* 70.4 / 1.67 = 42.2 k-ft

Flexural design ratio = $\frac{Mx}{Mnx / \Omega} = \frac{-5.9}{42.2}$ = 0.14 < 1.0 OK AISC F1

DEFLECTIONS

Stiffness factor 1.0

Required Camber .... 0.00 in

Long-term Deflection .... N.A.

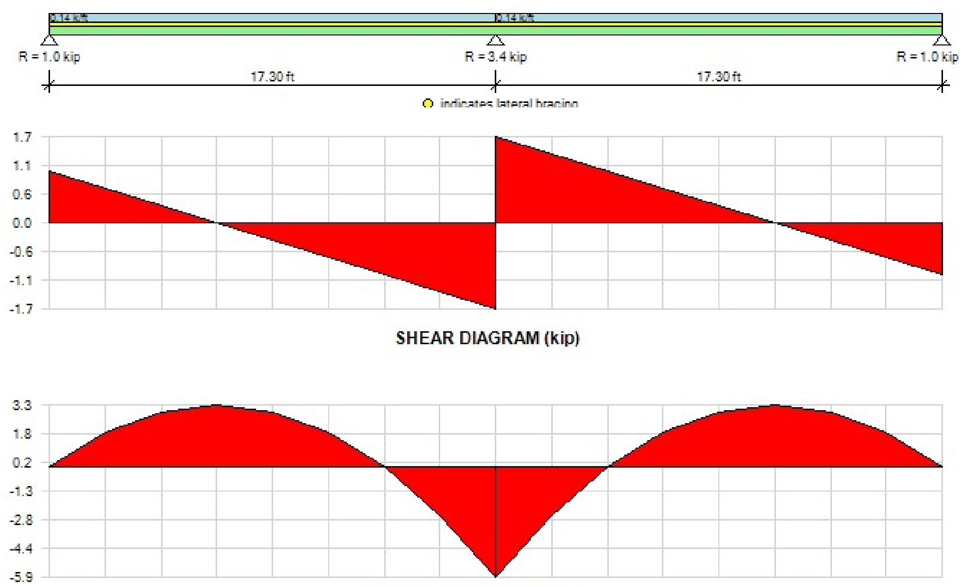
Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL	0.01	9999	240	0.02	OK
L	0.00	9999	360	0.04	OK
D+L	0.03	6114	240	0.04	OK

DESIGN CODES

Steel Design .... AISC 360-16

Load Combinations .... ASCE 7-10/16

MOMENT DIAGRAM (k-ft)
(Comb: CD)



MOMENT DIAGRAM (k-ft)
(Comb: D+S)

UNFACTORED FINAL LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

Span 1	Start	End	Width	Dead	Live	RLive	Snow	Wind	Seismic
w1	0.00	17.30	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 2									
w1	0.00	17.30	3.00	15.00	0.00	0.00	30.00	0.00	0.00

	Dist	Dead	Live	RLive	Snow	Wind	Seismic

UNFACTORED CONSTRUCTION LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

	Start	End	Width	Dead	Live

	Dist	Dead	Live

GEOMETRY

Beam Designation HSS8X6X1/4

Span	Length		Support	Type
1	13.16	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

PROPERTIES

Weight	22	lb/ft	Sx ...	14.2	in^3
Area .	6.2	in^2	Zx ...	16.9	in^3
Depth	8.0	in	rx	3.03	in
Width	6.0	in	Iy	36.4	in^4
tnom .	0.25	in	Sy ...	12.1	in^3
tdes ..	0.23	in	Zy ...	13.9	in^3
Ix	56.6	in^4	ry	2.43	in
			J	70.30	in^4

ASD SUPPORT REACTIONS (kip)

Load Comb.		
D+L	1.2	1.2
D+Lr	1.2	1.2
D+S	3.2	3.2
D+0.75L+0.75Lr	1.2	1.2
D+0.75L+0.75S	2.7	2.7
D+0.6W	1.2	1.2
D+0.7E	1.2	1.2
D+0.75L+0.75Lr+0.45W	1.2	1.2
D+0.75L+0.75S+0.45W	2.7	2.7
D+0.75L+0.75S+0.525E	2.7	2.7
0.6D+0.6W	0.7	0.7
0.6D+0.7E	0.7	0.7
CD	0.1	0.1

DESIGN FOR SHEAR

Maximum Shear Force V = 3.2 kip (Comb: D+S)

*h = Ht - 3 * t =* 8.0 - 3 * 0.2 = 7.3 in

*Aw = 2 * h * t =* 2 * 7.3 * 0.2 = 3.4 in^2 *kv = 5* AISC G4

$h / t = 7.3 / 0.2 = 31.3 < 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}} = 59.2$

Cv = 1.00 AISC Eq. (G2-3)

- Shear Yielding

Nominal strength *Vn = 0.6 * Fy * Aw =* 0.6 * 50.0 * 3.4 = 102.1 kip

- Shear Buckling

*Nominal strength Vn = 0.6 * Fy * Aw * Cv =* 0.6 * 50.0 * 3.4 * 1.00 = 102.1 kip AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

Shear allowable strength = *Vn / Ω =* 102.1 / 1.67 = 61.1 kip

Shear design ratio = $\frac{V}{Vn / \Omega} = \frac{3.2}{61.1} = 0.05$ < 1.0 OK AISC G1

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing .... Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment M = 10.5 k-ft (Comb: D+S)

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* AISC Eq F1-1

= 12.5 * 10.5 * 1.0 / (2.5 * 10.5 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0) = 3.00

- Yielding AISC F7.1

Plastic moment *Mpx = Fy * Zx =* 50.0 * 16.9 = 845.0 k-in AISC Eq. F7-1

Nominal strength *Mnx = Mpx =* 845.0 / 12 = 70.4 k-ft

- Flange Local Buckling AISC F7.2

Nominal strength *Mnx = N.A.* (compact flanges) AISC F7.2(a)

- Web Local Buckling AISC F7.3

Nominal strength *Mnx = N.A.* (compact webs) AISC F7.3(a)

- Controlling limit state: Yielding

Flexural allowable strength = *Mnx / Ω =* 70.4 / 1.67 = 42.2 k-ft

Flexural design ratio = $\frac{Mx}{Mnx / \Omega} = \frac{10.5}{42.2}$ = 0.25 < 1.0 OK AISC F1

DEFLECTIONS

Stiffness factor 1.0

Required Camber .... 0.00 in

Long-term Deflection .... N.A.

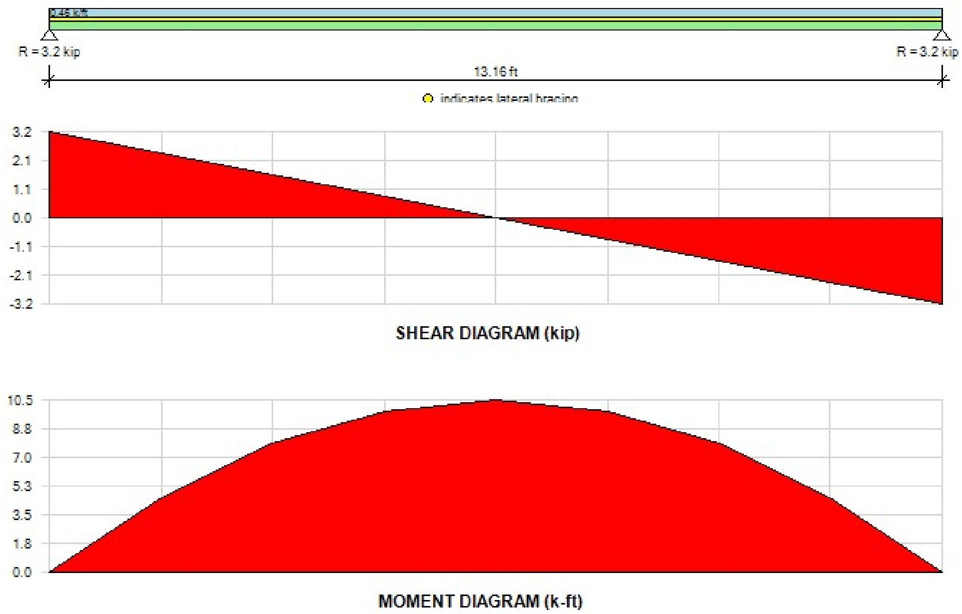
Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL	0.01	9999	240	0.02	OK
L	0.00	9999	360	0.04	OK
D+L	0.07	2171	240	0.11	OK

DESIGN CODES

Steel Design .... AISC 360-16

Load Combinations .... ASCE 7-10/16

(Comb: CD)



(Comb: D+S)

UNFACTORED FINAL LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

Span 1	Start	End	Width	Dead	Live	RLive	Snow	Wind	Seismic
w1	0.00	13.16	10.30	15.00	0.00	0.00	30.00	0.00	0.00

	Dist	Dead	Live	RLive	Snow	Wind	Seismic

UNFACTORED CONSTRUCTION LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

	Start	End	Width	Dead	Live

	Dist	Dead	Live

ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation HSS8X6X1/4

Span	Length		Support	Type
1	13.16	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

PROPERTIES

Weight	22	lb/ft	Sx ...	14.2	in³
Area .	6.2	in²	Zx ...	16.9	in³
Depth	8.0	in	rx	3.03	in
Width	6.0	in	Iy	36.4	in⁴
tnom .	0.25	in	Sy ...	12.1	in³
tdes ..	0.23	in	Zy ...	13.9	in³
Ix	56.6	in⁴	ry	2.43	in
			J	70.30	in⁴

ASD SUPPORT REACTIONS (kip)

Load Comb.		
D+L	1.9	1.9
D+Lr	1.9	1.9
D+S	5.3	5.3
D+0.75L+0.75Lr	1.9	1.9
D+0.75L+0.75S	4.4	4.4
D+0.6W	1.9	1.9
D+0.7E	1.9	1.9
D+0.75L+0.75Lr+0.45W	1.9	1.9
D+0.75L+0.75S+0.45W	4.4	4.4
D+0.75L+0.75S+0.525E	4.4	4.4
0.6D+0.6W	1.1	1.1
0.6D+0.7E	1.1	1.1
CD	0.1	0.1

DESIGN FOR SHEAR

Maximum Shear Force V = 5.3 kip (Comb: D+S)

*h = Ht - 3 * t =* 8.0 - 3 * 0.2 = 7.3 in

*Aw = 2 * h * t =* 2 * 7.3 * 0.2 = 3.4 in² *kv = 5* AISC G4

$h / t = 7.3 / 0.2 = 31.3 < 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}} = 59.2$

Cv = 1.00 AISC Eq. (G2-3)

- Shear Yielding

Nominal strength *Vn = 0.6 * Fy * Aw =* 0.6 * 50.0 * 3.4 = 102.1 kip

- Shear Buckling

*Nominal strength Vn = 0.6 * Fy * Aw * Cv =* 0.6 * 50.0 * 3.4 * 1.00 = 102.1 kip AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

Shear allowable strength = *Vn / Ω =* 102.1 / 1.67 = 61.1 kip

Shear design ratio = $\frac{V}{Vn / \Omega} = \frac{5.3}{61.1} = 0.09$ < 1.0 OK AISC G1

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing .... Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment M = 17.3 k-ft (Comb: D+S)

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* AISC Eq F1-1

= 12.5 * 17.3 * 1.0 / (2.5 * 17.3 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0) = 3.00

- Yielding AISC F7.1

Plastic moment *Mpx = Fy * Zx =* 50.0 * 16.9 = 845.0 k-in AISC Eq. F7-1

Nominal strength *Mnx = Mpx =* 845.0 / 12 = 70.4 k-ft

- Flange Local Buckling AISC F7.2

Nominal strength *Mnx = N.A.* (compact flanges) AISC F7.2(a)

- Web Local Buckling AISC F7.3

Nominal strength *Mnx = N.A.* (compact webs) AISC F7.3(a)

- Controlling limit state: Yielding

Flexural allowable strength = *Mnx / Ω =* 70.4 / 1.67 = 42.2 k-ft

Flexural design ratio = $\frac{Mx}{Mnx / \Omega} = \frac{17.3}{42.2}$ = 0.41 < 1.0 OK AISC F1

DEFLECTIONS

Stiffness factor 1.0

Required Camber .... 0.00 in

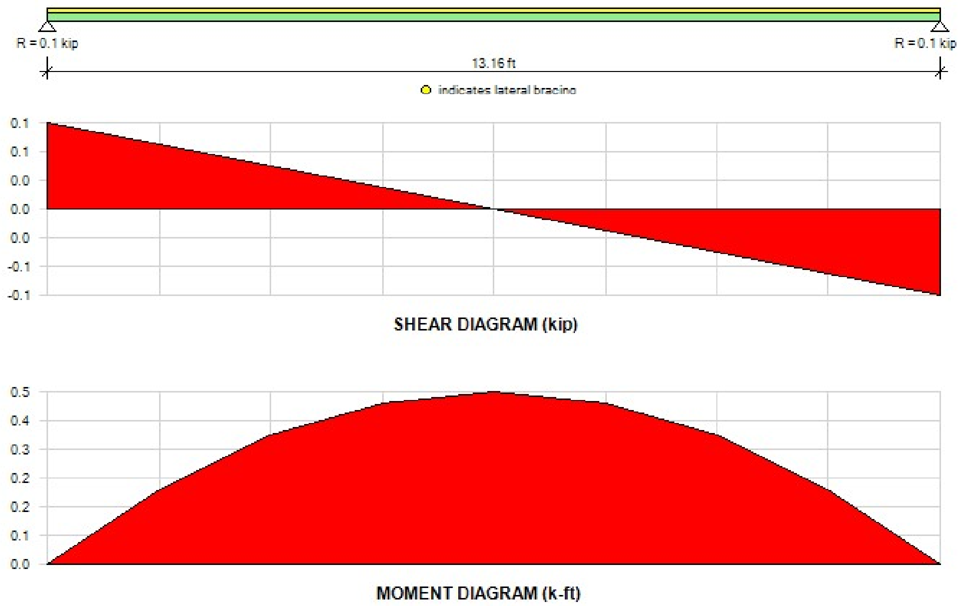
Long-term Deflection .... N.A.

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL	0.01	9999	240	0.02	OK
L	0.00	9999	360	0.04	OK
D+L	0.12	1362	240	0.18	OK

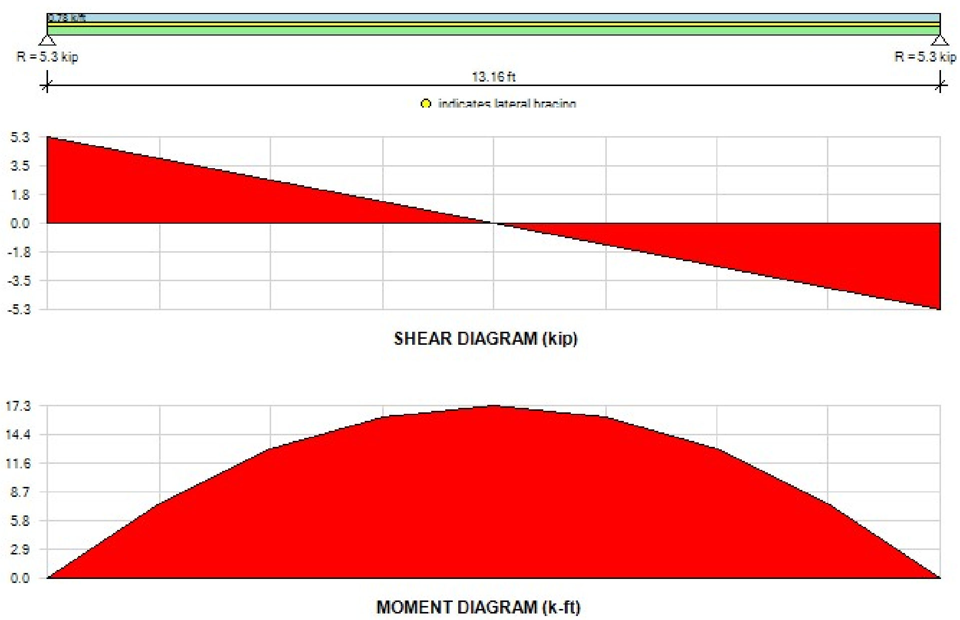
DESIGN CODES

Steel Design .... AISC 360-16

Load Combinations .... ASCE 7-10/16



(Comb: CD)



(Comb: D+S)

UNFACTORED FINAL LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

Span 1	Start	End	Width	Dead	Live	RLive	Snow	Wind	Seismic
w1	0.00	13.16	17.30	15.00	0.00	0.00	30.00	0.00	0.00

	Dist	Dead	Live	RLive	Snow	Wind	Seismic

UNFACTORED CONSTRUCTION LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

	Start	End	Width	Dead	Live

	Dist	Dead	Live

ASDIP Steel 5.6.4.1 **STEEL COLUMN DESIGN** *www.asdipsoft.com*

GEOMETRY

Column Designation	C6X10.5	
Steel Yield Strength Fy	50.0	ksi
Modulus of Elasticity Es	29000	ksi
Member Length L	4.50	ft
Effective Length Kx-factor	1.00	
Effective Length Ky-factor	1.00	
Unbraced Length Lb	4.50	ft OK

PROPERTIES

Area ..	3.1	in^2	Sx ...	5.0	in^3
Depth	6.0	in	Zx ...	6.2	in^3
bf	2.0	in	rx	2.22	in
tw	0.31	in	Iy	0.9	in^4
tf	0.34	in	Sy ...	0.6	in^3
k des .	0.81	in	Zy ...	1.1	in^3
Ix	15.1	in^4	ry	0.53	in
Cw	5.9	in^6	J	0.13	in^4

UNFACTORED LOADS (Elastic Second-Order Analysis)

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	0.0	0.0	0.0	0.0	0.0	3.6	kip
Mx at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Mx at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft

FACTORED LOADS

Controlling load combination: D+L

Axial load Pr = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 3.6 = 0.0 kip

Mx gravity bot = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral bot = 0.0 k-ft

Mx gravity top = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral top = 0.0 k-ft

My gravity bot = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral bot = 0.0 k-ft

My gravity top = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral top = 0.0 k-ft

M1x nt = 0.0 k-ft	M1x lt = 0,0 k-ft	M2x nt = 0.0 k-ft	M2x lt = 0.0 k-ft
M1y nt = 0.0 k-ft	M1y lt = 0.0 k-ft	M2y nt = 0.0 k-ft	M2y lt = 0.0 k-ft

Mrx = M2x nt + M2x lt = 0.0 + 0.0 = 0.0 k-ft Mry = M2y nt + M2y lt = 0.0 + 0.0 = 0.0 k-ft

DESIGN FOR COMPRESSION

Slender unstiffened Qs = 1.00 , Slender stiffened Qa = 1.00 , Q = Qs * Qa = 1.00 — AISC E7

Slenderness ratio *Kx L / rx* = 1.00 * 4.50 / 2.22 = 24.3

Slenderness ratio *Ky L / ry* = 1.00 * 4.50 / 0.53 = 102.1

Maximum slenderness ratio = Max (24.3, 102.1) = 102.1 — AISC E3

- Flexural Buckling

Elastic buckling $Fe = \dfrac{\pi^2 * E}{(K\,L / r)^2} = \dfrac{3.14^2 * 29000}{102.1^2} = 27.5$ ksi — AISC Eq. E3-4

*Fe >= 0.44 * Q * Fy* (27.5 >= 0.44 * 1.00 * 50.0 = 22.0)

Flexural buckling stress $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

$Fcr = 1.00 * 50.0 * (0.658)^{1.00 * 50.0 / 27.5} = 23.3$ ksi

- Torsional Buckling

$Fey = \dfrac{\pi^2 * E}{(Kx\,L / rx)^2} = \dfrac{\pi^2 * 29000}{24.3^2} = 483.7$ ksi — AISC Eq. E4-7

$Fez = [\dfrac{\pi^2 * E * Cw}{(Ky\,L)^2} + G * J]\dfrac{1}{Ag * ro^2}$ — AISC Eq. E4-9

$= [\dfrac{\pi^2 * 29000 * 5.9}{(1.00 * 4.5 * 12)^2} + 11200 * 0.13]\dfrac{1}{3.1 * 2.5^2} = 106.6$ ksi

$Fe = \dfrac{(Fey + Fez)}{(2 * H)} * (1 - \sqrt{1 - (\dfrac{4 * Fey * Fez * H}{(Fey + Fez)^2})})$ — AISC Eq. E4-5

$= \dfrac{(483.7 + 106.6)}{(2 * 0.8)} * (1 - \sqrt{1 - (\dfrac{4 * 483.7 * 106.6 * 0.8}{(483.7 + 106.6)^2})})$ Fe = 102.3 ksi

*Fe >= 0.44 * Q * Fy* (102.3 >= 0.44 * 1.00 * 50.0 = 22.0)

Flexural-torsional buckling $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

$Fcr = 1.00 * 50.0 * (0.658)^{1.00 * 50.0 / 102.3} = 40.8$ ksi

Controlling limit state: Flexural Buckling

Compressive design ratio = $\dfrac{P}{Pn / \Omega} = \dfrac{5.0}{71.7 / 1.67} = 0.12$ < 1.0 OK — AISC E1

DESIGN FOR FLEXURE

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* AISC Eq F1-1

= 12.5 * 0.0 * 1.0 / (2.5 * 0.0 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0 = 1.00

- Yielding

Plastic moment *Mpx = Fy * Zx* = 50.0 * 6.2 = 309.0 k-in AISC Eq. F2-1

Nominal strength *Mnx = Mpx* = 309.0 / 12 = 25.8 k-ft

- Lateral-Torsional Buckling

$$Lp = 1.76 * ry\sqrt{\frac{E}{Fy}} = 1.76 * ry\sqrt{\frac{29000}{50}}$$ Lp = 22.4 in AISC Eq. F2-5

$$rts = \sqrt{\frac{\sqrt{Iy * Cw}}{Sx}} = \sqrt{\frac{\sqrt{0.9 * 5.9}}{5}}$$ rts = 0.7 in AISC Eq. F2-7

ho = d - tf = 6.0 - 0.3 = 5.7 in

$$c = \frac{ho}{2 * \sqrt{\frac{Iy}{Cw}}} = \frac{5.657}{2 * \sqrt{\frac{0.86}{5.91}}}$$ c = 1.1 in AISC Eq. F2-8b

$$Lr = \frac{1.95 * rts * E}{(0.7 * Fy)} * \sqrt{\frac{J * c}{(Sx * ho)} + \sqrt{(\frac{J * c}{(Sx * ho)})^2 + 6.76 * (\frac{0.7 * Fy}{E})^2}}$$ AISC Eq. F2-6

$$\frac{1.95 * 0.7 * 29000}{(0.7 * 50)} * \sqrt{\frac{0.1 * 1.1}{(5 * 5.7)} + \sqrt{(\frac{0.1 * 1.1}{(5 * 5.7)})^2 + 6.76 * (\frac{0.7 * 50}{29000})^2}}$$ Lr = 111.3 in

$$Fcr = \frac{Cb * \pi^2 * E}{(Lb / rts)^2} \sqrt{1 + \frac{0.078 * J * c}{(Sx * ho)} * (\frac{Lb}{rts})^2}$$ AISC Eq. F2-4

$$= \frac{1.00 * \pi^2 * 29000.0}{(4.5 * 12 / 0.7)^2} \sqrt{1 + \frac{0.078 * 0.1 * 1.1}{(5 * 5.7)} * (\frac{4.5}{0.7})^2}$$ Fcr = 81.7 ksi

Nominal strength *Mnx = Cb * [Mpx - (Mpx - 0.7 * Fy * Sx) * (Lb - Lp) / (Lr - Lp)]* AISC Eq. F2-2

Mnx = 1.00 * [309.0 - (309.0 - 0.7 * 50.0 * 5.0)(4.5 * 12 - 22.4) / (111.3 - 22.4)] / 12 = 21.8 k-ft

X - Controlling limit state: Lateral-Torsional Buckling

X - Flexural design ratio = $\frac{Mx}{Mnx / \Omega} = \frac{0.0}{21.8 / 1.67}$ = 0.00 < 1.0 OK AISC F1

- Yielding

*Mpy = Min (Fy * Zy, 1.6 * Fy * Sy)* =Min (50.0 * 1.1, 1.6 * 50.0 * 0.6) = 44.9 k-in

Nominal strength *Mny = Mpy* = 44.9 / 12 = 3.7 k-ft AISC Eq. F6-1

- Flange Local Buckling

Nominal strength *Mny = N.A.* (compact flanges)

Y - Controlling limit state: Yielding

Y - Flexural design ratio = $\frac{My}{Mny / \Omega} = \frac{0.0}{3.7 / 1.67}$ = 0.00 < 1.0 OK AISC F1

ASDIP Steel 5.6.4.1 **STEEL COLUMN DESIGN** *www.asdipsoft.com*

DESIGN FOR COMBINED FORCES

Allowable axial strength $Pc = \frac{Pn}{\Omega} = \frac{71.7}{1.67} = 42.9$ kip AISC E1

Allowable flexural strength $Mcx = \frac{Mnx}{\Omega} = \frac{21.8}{1.67} = 13.1$ kip AISC F1

Allowable flexural strength $Mcy = \frac{Mny}{\Omega} = \frac{3.7}{1.67} = 2.2$ kip AISC F1

Combined forces ratio $= \frac{P}{2\,Pc} + \left[\frac{Mx}{Mcx} + \frac{My}{Mcy}\right]$ AISC Eq. H1-1b

$= \frac{5.0}{2 * 42.9} + \left[\frac{0.0}{13.1} + \frac{0.0}{2.2}\right] = 0.06 \quad < 1.0$ OK

DESIGN CODES

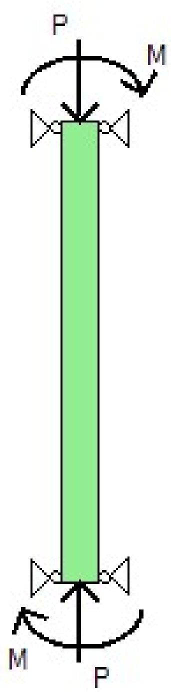
Steel Design .... AISC 360-16

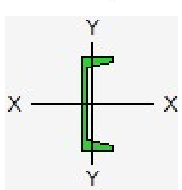
Load Combinations .... ASCE 7-10/16

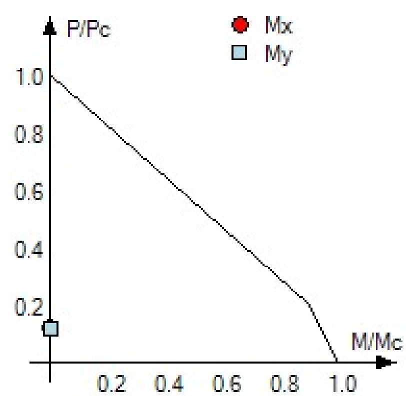
SEISMIC PROVISIONS

Design Spectral Acc. Sds 1.03

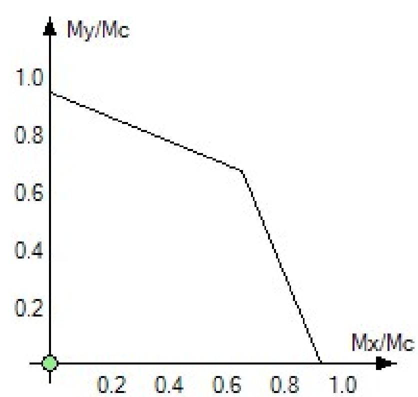
Overstrength Seismic Factor Ωo 2.00







INTERACTION DIAGRAM



BIAXIAL BENDING DIAGRAM

ASDIP Steel 5.6.4.1 **STEEL COLUMN DESIGN** www.asdipsoft.com

GEOMETRY

Column Designation	HSS6X6X1/4	
Steel Yield Strength Fy	50.0	ksi
Modulus of Elasticity Es	29000	ksi
Member Length L	17.00	ft
Effective Length Kx-factor	1.00	
Effective Length Ky-factor	1.00	
Unbraced Length Lb	N.A.	ft

PROPERTIES

Weight	19	lb/ft	Sx ...	9.5	in³
Area .	5.2	in²	Zx ...	11.2	in³
Depth	6.0	in	rx	2.34	in
Width	6.00	in	Iy	28.6	in⁴
t nom .	0.25	in	Sy ...	9.5	in³
t des .	0.23	in	Zy ...	11.2	in³
Ix	28.6	in⁴	ry	2.34	in
			J	45.60	in⁴

UNFACTORED LOADS (Elastic Second-Order Analysis)

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	2.2	0.0	0.0	3.8	0.0	6.0	kip
Mx at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Mx at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft

FACTORED LOADS

Controlling load combination: D+L

Axial load Pr = 1.0 * 2.2 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 3.8 + 0.0 * 0.0 + 0.00 * 6.0 = 2.2 kip

Mx gravity bot = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral bot = 0.0 k-ft

Mx gravity top = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral top = 0.0 k-ft

My gravity bot = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral bot = 0.0 k-ft

My gravity top = 1.0 * 0.0 + 1.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral top = 0.0 k-ft

M1x nt = 0.0 k-ft M1x lt = 0,0 k-ft M2x nt = 0.0 k-ft M2x lt = 0.0 k-ft

M1y nt = 0.0 k-ft M1y lt = 0.0 k-ft M2y nt = 0.0 k-ft M2y lt = 0.0 k-ft

Mrx = M2x nt + M2x lt = 0.0 + 0.0 = 0.0 k-ft Mry = M2y nt + M2y lt = 0.0 + 0.0 = 0.0 k-ft

DESIGN FOR COMPRESSION

Slender unstiffened Qs = 1.00 , Slender stiffened Qa = 1.00 , Q = Qs * Qa = 1.00 — AISC E7

Slenderness ratio *Kx L / rx =* 1.00 * 17.00 / 2.34 = 87.2

Slenderness ratio *Ky L / ry =* 1.00 * 17.00 / 2.34 = 87.2

Maximum slenderness ratio = Max (87.2, 87.2) = 87.2 — AISC E3

- Flexural Buckling

Elastic buckling $Fe = \dfrac{\pi^2 * E}{(K\,L / r)^2} = \dfrac{3.14^2 * 29000}{87.2^2} = 37.7$ ksi — AISC Eq. E3-4

*Fe >= 0.44 * Q * Fy* (37.7 >= 0.44 * 1.00 * 50.0 = 22.0)

Flexural buckling stress $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

$Fcr = 1.00 * 50.0 * (0.658)^{1.00 * 50.0 / 37.7} = 28.7$ ksi

- Torsional Buckling

$$Fe = \left[\frac{\pi^2 * E * Cw}{(Ky\,L)^2} + G * J\right]\frac{1}{Ix + Iy}$$ AISC Eq. E4-4

$$= \left[\frac{\pi^2 * 29000 * 15.4}{(1.00 * 17.0 * 12)^2} + 11200 * 45.6\right]\frac{1}{28.6 + 28.6} = 8930.5 \text{ ksi}$$

*Fe >= 0.44 * Q * Fy* (8930.5 >= 0.44 * 1.00 * 50.0 = 22.0)

Torsional buckling $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

$Fcr = 1.00 * 50.0 * (0.658)^{1.00 * 50.0 / 8930.5} = 49.9$ ksi

Compressive strength *Pn = Fcr * Ag =* 28.7 * 5.2 = 150.3 kip — AISC Eq. E3-1

Controlling limit state: Flexural Buckling

Compressive design ratio = $\dfrac{P}{Pn / \Omega} = \dfrac{11.6}{150.3 / 1.67} = 0.13$ < 1.0 OK — AISC E1

DESIGN FOR FLEXURE

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* AISC Eq F1-1

= 12.5 * 0.0 * 1.0 / (2.5 * 0.0 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0 = 1.00

- Yielding

Plastic moment *Mpx = Fy * Zx* = 50.0 * 11.2 = 560.0 k-in

Nominal strength *Mnx = Mpx* = 560.0 / 12 = 46.7 k-ft

Plastic moment *Mpy = Fy * Zy* = 50.0 * 11.2 = 560.0 k-in AISC Eq. F7-1

Nominal strength *Mny = Mpy* = 560.0 / 12 = 46.7 k-ft

- Flange Local Buckling AISC F7.2

Nominal strength *Mnx = N.A.* (compact flanges) AISC F7.2(a)

- Web Local Buckling AISC F7.3

Nominal strength *Mnx = N.A.* (compact webs) AISC F7.3(a)

X - Controlling limit state: Yielding

X - Flexural design ratio = $\frac{Mx}{Mnx / \Omega} = \frac{0.0}{46.7 / 1.67}$ = 0.00 < 1.0 OK AISC F1

Y - Controlling limit state: Yielding

Y - Flexural design ratio = $\frac{My}{Mny / \Omega} = \frac{0.0}{46.7 / 1.67}$ = 0.00 < 1.0 OK AISC F1

ASDIP Steel 5.6.4.1 | **STEEL COLUMN DESIGN** | www.asdipsoft.com

DESIGN FOR COMBINED FORCES

Allowable axial strength $Pc = \frac{Pn}{\Omega} = \frac{150.3}{1.67}$ = 90.0 kip — AISC E1

Allowable flexural strength $Mcx = \frac{Mnx}{\Omega} = \frac{46.7}{1.67}$ = 27.9 kip — AISC F1

Allowable flexural strength $Mcy = \frac{Mny}{\Omega} = \frac{46.7}{1.67}$ = 27.9 kip — AISC F1

Combined forces ratio $= \frac{P}{2\,Pc} + \left[\frac{Mx}{Mcx} + \frac{My}{Mcy}\right]$ — AISC Eq. H1-1b

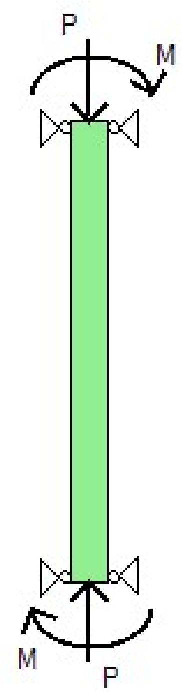
$= \frac{11.6}{2 * 90.0} + \left[\frac{0.0}{27.9} + \frac{0.0}{27.9}\right] = 0.06 < 1.0$ OK

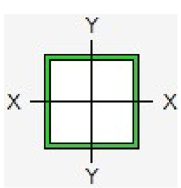
DESIGN CODES

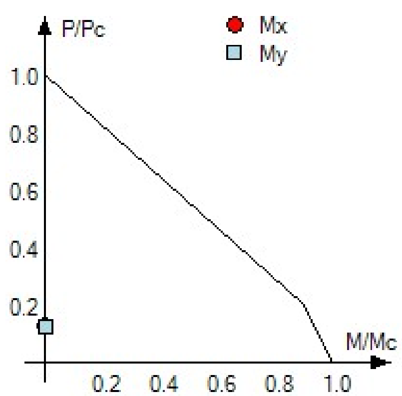
Steel Design	AISC 360-16
Load Combinations	ASCE 7-10/16

SEISMIC PROVISIONS

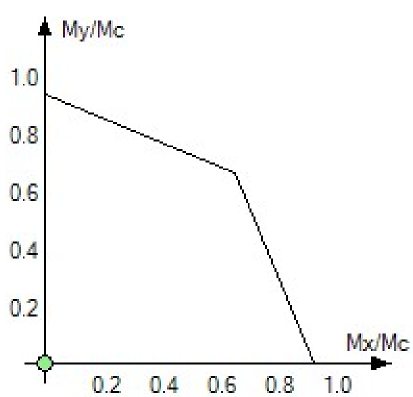
Design Spectral Acc. Sds	1.03
Overstrength Seismic Factor Ωo	2.00







INTERACTION DIAGRAM



BIAXIAL BENDING DIAGRAM

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	3.00	ft	
Footing Width (Z-dir)	3.00	ft	
Footing Thickness	10.0	in	OK
Soil Cover	1.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+0.7S)

Gross Allow. Soil Pressure	2.0	ksf
Soil Pressure at Corner 1	0.7	ksf
Soil Pressure at Corner 2	0.7	ksf
Soil Pressure at Corner 3	0.7	ksf
Soil Pressure at Corner 4	0.7	ksf
Bearing Pressure Ratio	0.36	OK
Ftg. Area in Contact with Soil	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	2.2	0.0	0.0	3.8	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

  Arm = 0.00 + 10.0 / 12 = 0.83 ft Moment = 0.0 * 0.83 = 0.0 k-ft

- Passive Force = 0.0 kip Arm = 0.33 ft Moment = 0.0 k-ft

- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 10.0 / 12 * 0.15 = 0.7 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.7 * 1.50 = 1.0 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 1.0 * 110 = 0.6 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.6 * 1.50 = 0.9 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.83) = -0.3 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.3 * 1.50 = -0.4 k-ft

- Axial force P = 0.6 * 2.2 + 0.6 * 0.0 = 1.3 kip

  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 1.3 * 1.50 = 2.0 k-ft

- Resisting moment X-X = 1.0 + 0.0 + 0.9 + 2.0 + -0.4 = 3.4 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.4}{0.0}$ = 34.38 > 1.50 OK

- Overturning about Z-Z
- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 10.0 / 12 = 0.83 ft   Moment = 0.0 * 0.83 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.33 ft   Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z
- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 10.0 / 12 * 0.15 = 0.7 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.7 * 1.50 = 1.0 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 1.0 * 110 = 0.6 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.6 * 1.50 = 0.9 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.83) = -0.3 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.3 * 1.50 = -0.4 k-ft
- Axial force P = 0.6 * 2.2 + 0.6 * 0.0 = 1.3 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 1.3 * 1.50 = 2.0 k-ft
- Resisting moment Z-Z = 1.0 + 0.0 + 0.9 + 2.0 + -0.4 = 3.4 k-ft

- Overturning safety factor Z-Z = $\dfrac{\text{Resisting moment}}{\text{Overturning moment}} = \dfrac{3.4}{0.0}$ = 34.38 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+0.7S)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 1.7 + 0.0 + 1.4 + -0.7 + 7.3 = 9.7 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 1.7 + 0.0 + 1.4 + -0.7 + 7.3 = 9.7 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 1.1 + 0.0 + 1.0 - 0.5 + 4.9 = 6.5 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\text{Z-Resisting moment - Z-Overturning moment}}{\text{Resisting force}} = \frac{9.7 - 0.0}{6.5} = 1.50 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\text{X-Resisting moment - X-Overturning moment}}{\text{Resisting force}} = \frac{9.7 - 0.0}{6.5} = 1.50 \text{ ft}$$

X-ecc = *Length / 2 - Xp =* 3.00 / 2 - 1.50 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 3.00 / 2 - 1.50 = 0.00 ft

Area = *Width * Length =* 3.00 * 3.00 = 9.0 ft^2

Sx = *Length * Width² / 6 =* 3.00 * 3.00^2 / 6 = 4.5 ft^3

Sz = *Width * Length² / 6 =* 3.00 * 3.00^2 / 6 = 4.5 ft^3

- Footing is in full bearing. Soil pressures are as follows:

P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 6.5 * (1 / 9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 0.72 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 6.5 * (1 / 9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 0.72 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 6.5 * (1 / 9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 0.72 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 6.5 * (1 / 9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 0.72 ksf

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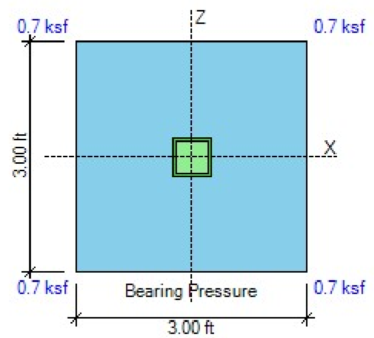
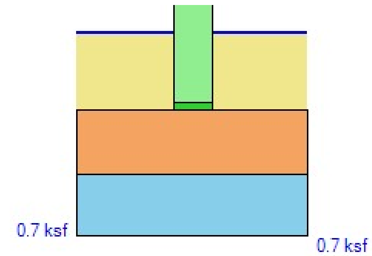




SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (1.00 + 10.0 / 12 / 2) = 0.67 ksf

X-Passive force = *Pressure * Thick * Width =* 0.67 * 10.0 / 12 * 3.00 = 1.7 kip

Z-Passive force = *Pressure * Thick * Length =* 0.67 * 10.0 / 12 * 3.00 = 1.7 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 2.3 * 0.35) = 0.8 kip

Use 100% of Passive + 100% of Friction for sliding resistance

$$\text{- Sliding safety factor X-X} = \frac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \frac{1.00 * 1.7 + 1.00 * 0.8}{0.0} = 24.87 > 1.50 \quad \text{OK}$$

$$\text{- Sliding safety factor Z-Z} = \frac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \frac{1.00 * 1.7 + 1.00 * 0.8}{0.0} = 24.87 > 1.50 \quad \text{OK}$$

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

$$\text{- Uplift safety factor} = \frac{\text{Pedestal + Footing + Cover - Buoyancy}}{\text{Uplift load}} = \frac{0.0 + 0.7 + 0.6 - 0.3}{0.0} = 99.99 > 1.00 \quad \text{OK}$$

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+S+0.5W)

Concrete f'c = 2.5 ksi | Steel fy = 40.0 ksi | Soil density = 110 pcf

d Top X-dir = *Thick - Cover - X-diameter / 2 =* 10.0 - 2.0 - 0.8 / 2 = 7.6 in

d Top Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2 =* 10.0 - 2.0 - 0.8 - 0.8 / 2 = 6.9 in

d Bot X-dir = *Thick - Cover - X-diameter / 2 =* 10.0 - 3.0 - 0.5 / 2 = 6.8 in

d Bot Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2 =* 10.0 - 3.0 - 0.5 - 0.5 / 2 = 6.3 in

$\phi V_{cx} = 8 * \rho^{1/3} * \sqrt{f'c} * Width * d$ = 8 * (0.0033)⅓ * √(2500) * 3.0 * 12 * 6.8 / 1000 = 11.1 kip   ACI 22.5.5.1

$\phi V_{cz} = 8 * \rho^{1/3} * \sqrt{f'c} * Length * d$ = 8 * (0.0036)⅓ * √(2500) * 3.0 * 12 * 6.3 / 1000 = 10.5 kip

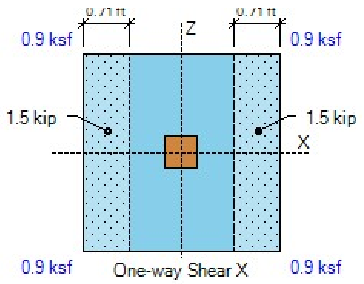
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

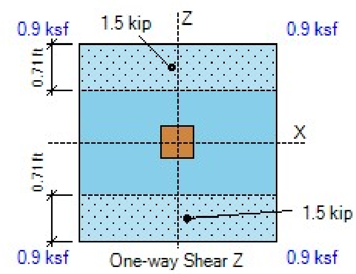
One-way shear Vux (- Side) = 1.5 kip < 11.1 kip OK

One-way shear Vux (+ Side) = 1.5 kip < 11.1 kip OK

One-way shear Vuz (- Side) = 1.5 kip < 10.5 kip OK

One-way shear Vuz (+ Side) = 1.5 kip < 10.5 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+S+0.5W)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 10.0^2 / 6 / 1000 = 2.0 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 10.0^2 / 6 / 1000 = 2.0 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 7.5 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 7.5 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 7.5 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 7.5 k-ft OK

- Bottom Bars

Use 4 #4 Z-Bars — *ρ = As / b d* = 0.8 / (3.00 * 12 * 6.3) = 0.0036 — q = 0.0036 * 40 / 2.5 = 0.057

Use 4 #4 X-Bars — *ρ = As / b d* = 0.8 / (3.00 * 12 * 6.8) = 0.0033 — q = 0.0033 * 40 / 2.5 = 0.053

β = L / W = 3.00 / 3.00 = 1.00 — *γs = 2 * β / (β + 1)* = 2 * 1.00 / (1.00 + 1) = 1.00 — ACI 13.3.3.3

Bending strength *ϕMn = ϕ * b * d² * f'c * q * (1 - 0.59 * q)* — ACI 22.2.2

ϕMnx = 0.90 * 3.00 * 12 * 6.3^2 * 2.5 * 0.057 * (1 - 0.59 * 0.057) = 14.5 k-ft

ϕMnz = 0.90 * 3.00 * 12 * 6.8^2 * 2.5 * 0.053 / 1.00 * (1 - 0.59 * 0.053 / 1.00) = 15.7 k-ft

- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

Bottom moment Mux (- Side) = 1.7 k-ft < 14.5 k-ft OK ratio = 0.12

Bottom moment Mux (+ Side) = 1.7 k-ft < 14.5 k-ft OK ratio = 0.12

Bottom moment Muz (- Side) = 1.7 k-ft < 15.7 k-ft OK ratio = 0.11

Bottom moment Muz (+ Side) = 1.7 k-ft < 15.7 k-ft OK ratio = 0.11

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.00 * 12 * 10.0 = 0.6 in² < 0.8 in² OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 10.0 = 0.6 in² < 0.8 in² OK — ACI 8.6.1.1

X-As max for fy/Es + 0.003 tension strain = 4.30 in² > 0.80 in² OK — ACI 21.2.2

Z-As max for fy/Es + 0.003 tension strain = 4.30 in² > 0.80 in² OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.4a)

X-Ld = Max (12.0, 3 / 40 * 40.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.11) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* — ACI 25.4.3

X-Ldh = Max (8 db, 6, 1 / 55 * 40.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

Z-Ld = Max (12.0, 3 / 40 * 40.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.11) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

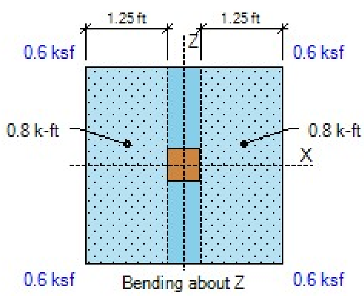
Z-Ldh = Max (8 db, 6, 1 / 55 * 40.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

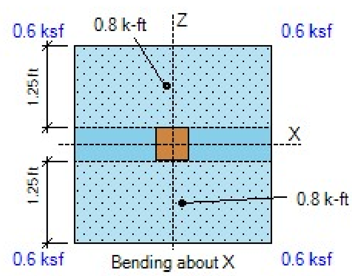
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+Z Ld provided =*(Width - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

X-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+S+0.5W)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 6.4 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.2 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.2 psi OK

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 12.0 in OK

Ldh provided = *Footing thickness - Cover =* 10.00 - 3.0 = 7.0 in < 17.4 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+0.5L+S)

X-Edge = *Length / 2 - Offset - Col / 2 =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in αsx = 10

Z-Edge = *Width / 2 - Offset - Col / 2 =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in αsz = 10

αs = *αsx + αsz =* 10 + 10 = 20 Col type = Corner β = *L / W =* 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 10 / 10 * (6.0 + 6.5 / 2 + 15.0) + 10 / 10 * (6.0 + 6.5 / 2 + 15.0) = 48.5 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 6.5 / 2 + 15.0) * (6.0 + 6.5 / 2 + 15.0) = 588.1 in²

ϕVc = *ϕ * Min (2 + 4 / β, αs * d / bo + 2, 4) * √(f'c)* ACI 22.6.5.2

ϕVc = 0.75 * Min (2 + 4 / 1.00, 20 * 6.5 / 48.5 + 2, 4) * √(2500) = 150.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 6.4 + 0.22 * 588.1 / 144 - 1.0 = 6.3 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 6.5 / 2 + 15.0 = 24.3 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 6.5 / 2 + 15.0 = 24.3 in

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \frac{1}{1 + (2/3)\sqrt{24.3 / 24.3}} = 0.40$$ ACI Eq. (8.4.4.2.2)

$$\gamma vz \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b1 / b2}} = 1 - \frac{1}{1 + (2/3)\sqrt{24.3 / 24.3}} = 0.40$$ ACI Eq. (8.4.2.3.2)

X2z = *b1² / 2 / (b1 + b2) =* 24.3² / 2 / (24.3 + 24.3) = 6.1 in X2x = *b2² / 2 / (b2 + b1) =* 6.1 in

Jcz = *b1 * d³ / 12 + b1³ * d / 12 + b1 * d * (b1 / 2 - X2z)² + b2 * d * X2z²* ACI R8.4.4.2.3

Jcz = 24.3 * 6.5³ / 12 + 24.3³ * 6.5 / 12 + 24.3 * 6.5 * (24.3 / 2 * 6.1)² + 24.3 * 6.5 * 6.1² = 19866 in⁴

Jcx = *b2 * d³ / 12 + b2³ * d / 12 + b2 * d * (b2 / 2 - X2x)² + b1 * d * X2x²* ACI R8.4.4.2.3

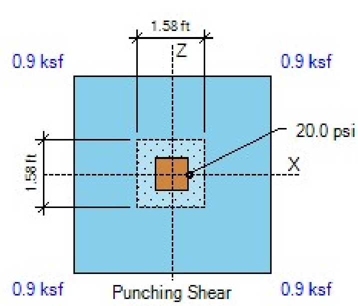
Jcz = 24.3 * 6.5³ / 12 + 24.3³ * 6.5 / 12 + 24.3 * 6.5 * (24.3 / 2 * 6.1)² + 24.3 * 6.5 * 6.1² = 19866 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 6.3 / (48.5 * 6.5) * 1000 = 20.0 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.40 * 0.0 * 12 * 6.1 / 19866 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.40 * 0.0 * 12 * 6.1 / 19866 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 20.0 + 0.0 + 0.0 = 20.0 psi < 150.0 psi OK



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+S+0.5W)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 6.4 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.2 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

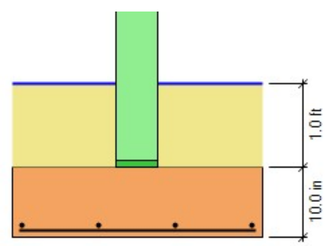
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

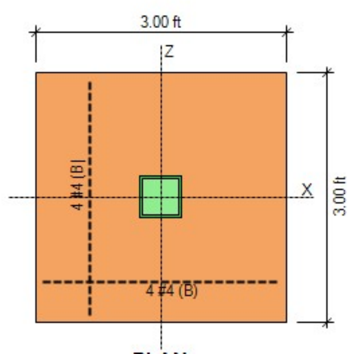
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.2 psi OK

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-22



ELEVATION



PLAN

WIND V_{ASD}=85MPH V_{ULT}=110MPH EXP. B K_{ZT}=1.0 SCALE=1:12

ZONE 1E&4E = ~~13.5PSF~~ 16.0PSF MIN

ZONE 1&4 = ~~9.0PSF~~ 16.0PSF MIN

ZONE 2E&3E = ~~1.8PSF~~ 8.0PSF MIN

ZONE 2&3 = ~~1.2PSF~~ 8.0PSF MIN

ZONE 5E&6E = ~~13.5PSF~~ 16.0PSF MIN

ZONE 5&6 = ~~8.9PSF~~ 16.0PSF MIN

SEISMIC S_{DS}=1.03 R=3.25 I_e=1.0

$C_S = ((1.03/(3.25/1.0))/1.4 = 0.23$

W_{ROOF} = (25PSF × 610SF) = 15,250#

V_S = 15,250# × 0.23 = 3,508#

GRID 1

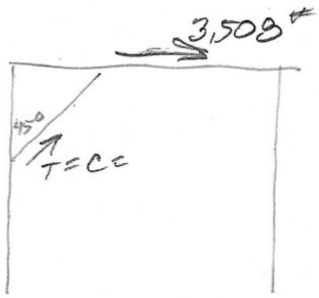
F_W = (16PSF × 133SF) = 2,128#

F_E = 3,508#

GRIDS A&C

F_W = (16PSF × 103SF) = 1,648#

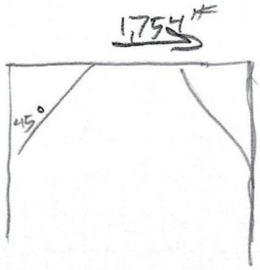
F_E = 3,508#/2 = 1,754#



$T = C = 3{,}508^{\#} / \cos 45^\circ = 4{,}961^{\#}$

CONTROLS DESIGN

GRID 1



$T = C = 1{,}754^{\#} / \cos 45^\circ = 2{,}481^{\#}$

GRIDS A & C