

ENGINEERING ANALYSIS FOR:
NORTH TRASH ENCLOSURE
EAST TOWN CROSSING
APARTMENTS
3002 E PIONEER WAY
PUYALLUP, WA 98372
PARCEL NO: 0420264053



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EAST TOWN CROSSING
NORTH TRASH ENCLOSURE
PIONEER AND SHAW PUYALLUP WA

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) WIND DESIGN DATA (ASCE 7-22)
FLAT SNOW LOAD: N/A BASIC WIND SPEED (ASD) V= 85MPH
SNOW EXPOSURE FACTOR, Ce=1.0, ULTIMATE WIND SPEED V= 110MPH
SNOW IMPORTANCE FACTOR, Is=1.0, RISK CATEGORY: II EXPOSURE: B
THERMAL FACTOR, Ct=1.2 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: 5,543#
SOIL PROPERTIES:
BEARING CAPACITY: 2,000 PSF
LATERAL CAPACITY: 250 PSF/FT

City of Puyallup
Building
REVIEWED
FOR
COMPLIANCE

SKinnear
09/10/2025
10:11:50 AM

City of Puyallup
Development & Permitting Services
ISSUED PERMIT:

Building	Planning
Engineering	Public Works
Fire	Traffic

PRCNC20251159

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

REVISIONS	
1	---
REVISIONS	
ENGINEER:	CP
CHECKED BY:	CP
DATE:	2025.08.11
TITLE:	STRUCTURAL ANALYSIS
PROJECT #:	----

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:

- Uncoated steel thickness. Thickness is for carbon sheet steel.
- Minimum thickness represents 95% of the design thickness and is the minimum acceptable thickness.
- Per ASTM C955 & A1003, Table 1.
- 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

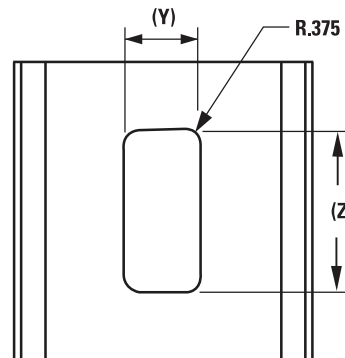
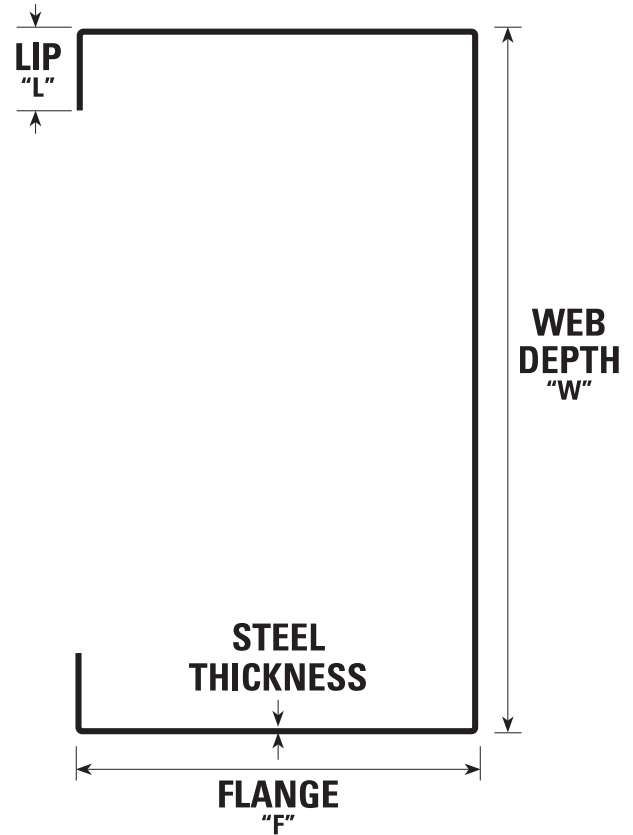
LEED v4 for Building and Design Construction

- 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)	β		
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 unpunched only.

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

$0.054 \leq \text{Support Thickness (in.)} \leq 0.175$

1.63 in. Minimum Deck End Bearing Length

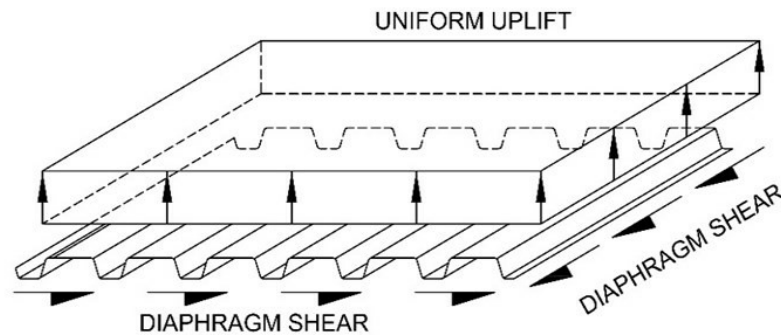
ASD Allowable Combined Wind Uplift & Diaphragm Shear Strength S_n/Ω (plf)

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
 $0.054 \leq \text{Support Thickness (in.)} \leq 0.175$
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 S_n/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

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22 Gage 1.5B-36 Grade 50

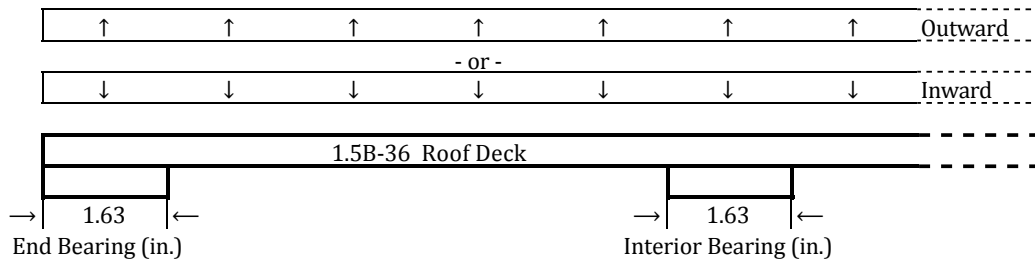
Uniform Allowable Load Table, ASD (psf)

For End Lapped Deck



36 / 4 Connection Pattern to Supports with
#10 Screw

Support Member A572 GR50
 $0.0538 \leq t_2 \text{ (in.)} \leq 0.175$



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

Where: $W \leq W_n/\Omega$

W = Required strength of the governing ASD load combination

W_n/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

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Project:
 Engineer:
 Descrip: Grid 3 (A-B) Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation		HSS12X4X1/4	
Span	Length	Support	Type
①	3.00 ft	①	Pinned
②	12.60 ft	②	Pinned
③	3.70 ft	③	Pinned
④	N.A.	④	Pinned
⑤	N.A.	⑤	N.A.
		⑥	N.A.

PROPERTIES

Weight	26 lb/ft	Sx ...	19.9 in³
Area .	7.1 in²	Zx ...	25.6 in³
Depth	12.0 in	rx	4.10 in
Width	4.0 in	ly	21.0 in⁴
tnom .	0.25 in	Sy ...	10.5 in³
tdes ..	0.23 in	Zy ...	11.7 in³
Ix	119.0 in⁴	ry	1.72 in
		J	59.80 in⁴

ASD SUPPORT REACTIONS (kip)

Load Comb.	Δ	Δ	Δ	Δ
D+L	-0.1	0.7	2.7	1.4
D+Lr	-0.1	0.7	2.7	1.4
D+S	-0.2	1.5	7.1	3.9
D+0.75L+0.75Lr	-0.1	0.7	2.7	1.4
D+0.75L+0.75S	-0.1	1.3	6.0	3.3
D+0.6W	-0.1	0.7	2.7	1.4
D+0.7E	-0.1	0.7	2.7	1.4
D+0.75L+0.75Lr+0.45W	-0.1	0.7	2.7	1.4
D+0.75L+0.75S+0.45W	-0.1	1.3	6.0	3.3
D+0.75L+0.75S+0.525E	-0.1	1.3	6.0	3.3
0.6D+0.6W	-0.1	0.4	1.6	0.8
0.6D+0.7E	-0.1	0.4	1.6	0.8
CD	-0.1	0.3	0.3	0.0

DESIGN FOR SHEAR

Maximum Shear Force $V = 5.9$ kip (Comb: D+S)

$$h = Ht - 3 * t = 12.0 - 3 * 0.2 = 11.3 \text{ in}$$

$$Aw = 2 * h * t = 2 * 11.3 * 0.2 = 5.3 \text{ in}^2 \quad kv = 5$$

AISC G4

$$h/t = 11.3 / 0.2 = 48.5 < 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}} = 59.2$$

$$Cv = 1.00$$

AISC Eq. (G2-3)

- Shear Yielding

$$\text{Nominal strength } Vn = 0.6 * Fy * Aw = 0.6 * 50.0 * 5.3 = 158.0 \text{ kip}$$

- Shear Buckling

$$\text{Nominal strength } Vn = 0.6 * Fy * Aw * Cv = 0.6 * 50.0 * 5.3 * 1.00 = 158.0 \text{ kip}$$

AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

$$\text{Shear allowable strength} = Vn / \Omega = 158.0 / 1.67 = 94.6 \text{ kip}$$

$$\text{Shear design ratio} = \frac{V}{Vn / \Omega} = \frac{5.9}{94.6} = 0.06 < 1.0 \text{ OK}$$

AISC G1

Project:
 Engineer:
 Descrip: Grid 3 (A-B) Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment $M = 7.0$ k-ft (Comb: D+S)

$$C_b = \text{Min} (3.0, 12.5 M_{max} * R_m / (2.5 M_{max} + 3 M_a + 4 M_b + 3 M_c))$$

$$= 12.5 * 7.0 * 1.0 / (2.5 * 7.0 + 3 * 3.5 + 4 * 3.5 + 3 * 3.5) = 3.00$$

AISC Eq F1-1

- Yielding

AISC F7.1

Plastic moment $M_{px} = F_y * Z_x = 50.0 * 25.6 = 1280.0$ k-in

AISC Eq. F7-1

Nominal strength $M_{nx} = M_{px} = 1280.0 / 12 = 106.7$ k-ft

- Flange Local Buckling

AISC F7.2

Nominal strength $M_{nx} = N.A.$ (compact flanges)

AISC F7.2(a)

- Web Local Buckling

AISC F7.3

Nominal strength $M_{nx} = N.A.$ (compact webs)

AISC F7.3(a)

- Controlling limit state: Yielding

Flexural allowable strength $= M_{nx} / \Omega = 106.7 / 1.67 = 63.9$ k-ft

Flexural design ratio $= \frac{M_x}{M_{nx} / \Omega} = \frac{7.0}{63.9} = 0.11 < 1.0$ OK

AISC F1

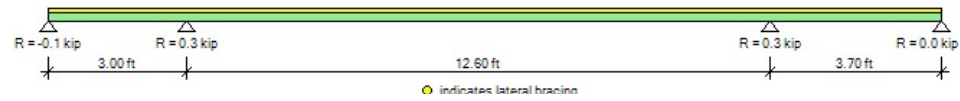
DEFLECTIONS

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	3600	360	0.10	OK
CD+CL	0.00	3600	240	0.07	OK
L	0.00	3600	360	0.10	OK
D+L	0.00	3600	240	0.07	OK

DESIGN CODES

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

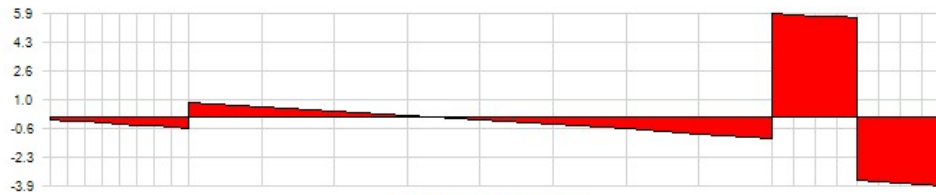
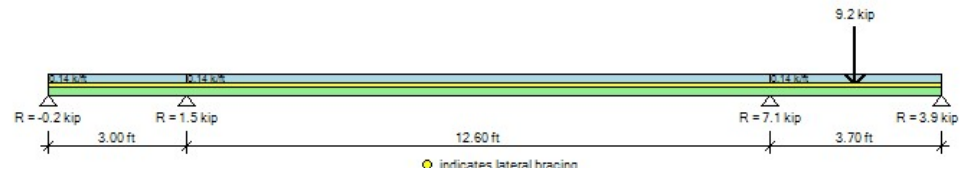


SHEAR DIAGRAM (kip)

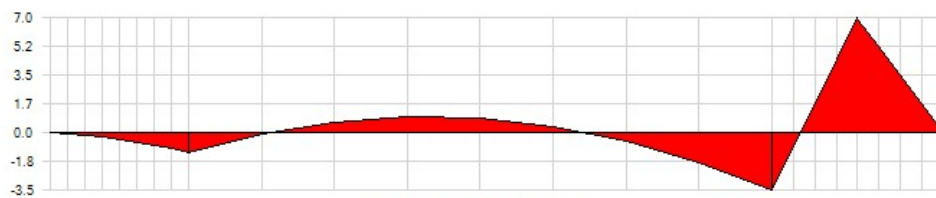


MOMENT DIAGRAM (k-ft)

(Comb: CD)



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

(Comb: D+S)

Project:
 Engineer:
 Descrip: Grid 3 (A-B) Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

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	3.00	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 2									
w1	0.00	12.60	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 3									
w1	0.00	3.70	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 3	Dist	Dead	Live	RLive	Snow	Wind	Seismic		
P1	1.83	3.3	0.0	0.0	5.9	0.0	0.0		

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

	Start	End	Width	Dead	Live		Dist	Dead	Live
--	-------	-----	-------	------	------	--	------	------	------

Project:
 Engineer:
 Descrip: Grid 3 (B-C) Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation	HSS12X4X1/4		
Span	Length	Support	Type
①	12.60 ft	①	Pinned
②	3.00 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	26 lb/ft	Sx ...	19.9 in³
Area .	7.1 in²	Zx ...	25.6 in³
Depth	12.0 in	rx	4.10 in
Width	4.0 in	ly	21.0 in⁴
tnom .	0.25 in	Sy ...	10.5 in³
t _{des} ..	0.23 in	Zy ...	11.7 in³
Ix	119.0 in⁴	ry	1.72 in
		J	59.80 in⁴

ASD SUPPORT REACTIONS (kip)

Load Comb.			
D+L	0.4	1.0	-0.3
D+Lr	0.4	1.0	-0.3
D+S	0.8	2.3	-0.6
D+0.75L+0.75Lr	0.4	1.0	-0.3
D+0.75L+0.75S	0.7	2.0	-0.5
D+0.6W	0.4	1.0	-0.3
D+0.7E	0.4	1.0	-0.3
D+0.75L+0.75Lr+0.45W	0.4	1.0	-0.3
D+0.75L+0.75S+0.45W	0.7	2.0	-0.5
D+0.75L+0.75S+0.525E	0.7	2.0	-0.5
0.6D+0.6W	0.2	0.6	-0.2
0.6D+0.7E	0.2	0.6	-0.2
CD	0.1	0.4	-0.1

DESIGN FOR SHEAR

Maximum Shear Force $V = 1.2$ kip (Comb: D+S)

$$h = Ht - 3 * t = 12.0 - 3 * 0.2 = 11.3 \text{ in}$$

$$Aw = 2 * h * t = 2 * 11.3 * 0.2 = 5.3 \text{ in}^2 \quad kv = 5$$

AISC G4

$$h / t = 11.3 / 0.2 = 48.5 < 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}} = 59.2$$

$$Cv = 1.00$$

AISC Eq. (G2-3)

- Shear Yielding

$$\text{Nominal strength } V_n = 0.6 * Fy * Aw = 0.6 * 50.0 * 5.3 = 158.0 \text{ kip}$$

- Shear Buckling

$$\text{Nominal strength } V_n = 0.6 * Fy * Aw * Cv = 0.6 * 50.0 * 5.3 * 1.00 = 158.0 \text{ kip}$$

AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

$$\text{Shear allowable strength} = V_n / \Omega = 158.0 / 1.67 = 94.6 \text{ kip}$$

$$\text{Shear design ratio} = \frac{V}{V_n / \Omega} = \frac{1.2}{94.6} = 0.01 < 1.0 \text{ OK}$$

AISC G1

Project:
 Engineer:
 Descrip: Grid 3 (B-C) Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment $M = -2.6$ k-ft (Comb: D+S)

$$C_b = \text{Min} (3.0, 12.5 M_{max} * R_m / (2.5 M_{max} + 3 M_a + 4 M_b + 3 M_c))$$

$$= 12.5 * -2.6 * 1.0 / (2.5 * -2.6 + 3 * 1.7 + 4 * 1.9 + 3 * 0.4) = 1.59$$

AISC Eq F1-1

- Yielding

AISC F7.1

Plastic moment $M_{px} = F_y * Z_x = 50.0 * 25.6 = 1280.0$ k-in

AISC Eq. F7-1

Nominal strength $M_{nx} = M_{px} = 1280.0 / 12 = 106.7$ k-ft

- Flange Local Buckling

AISC F7.2

Nominal strength $M_{nx} = N.A.$ (compact flanges)

AISC F7.2(a)

- Web Local Buckling

AISC F7.3

Nominal strength $M_{nx} = N.A.$ (compact webs)

AISC F7.3(a)

- Controlling limit state: Yielding

Flexural allowable strength $= M_{nx} / \Omega = 106.7 / 1.67 = 63.9$ k-ft

Flexural design ratio $= \frac{M_x}{M_{nx} / \Omega} = \frac{-2.6}{63.9} = 0.04 < 1.0$ 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	3600	360	0.10	OK
CD+CL	0.00	3600	240	0.07	OK
L	0.00	3600	360	0.10	OK
D+L	0.00	3600	240	0.07	OK

DESIGN CODES

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

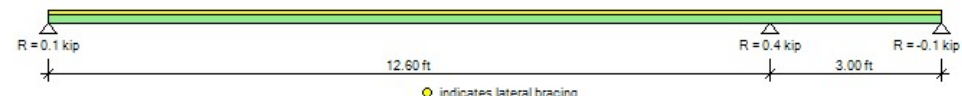
Project:
Engineer:
Descrip: Grid 3 (B-C) Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

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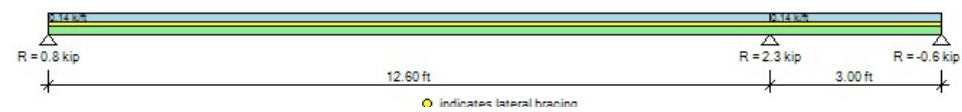


SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

(Comb: CD)



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

(Comb: D+S)

Project:
 Engineer:
 Descrip: Grid 3 (B-C) Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

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	12.60	3.00	15.00	0.00	0.00	30.00	0.00	0.00
Span 2									
w1	0.00	3.00	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
--	-------	-----	-------	------	------	--	------	------	------

Project:
 Engineer:
 Descrip: Grid A&C Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

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GEOMETRY

Beam Designation	HSS12X4X1/4		
Span	Length	Support	Type
①	5.00 ft	①	Pinned
②	18.00 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	26 lb/ft	Sx ...	19.9 in³
Area .	7.1 in²	Zx ...	25.6 in³
Depth	12.0 in	rx	4.10 in
Width	4.0 in	ly	21.0 in⁴
tnom .	0.25 in	Sy ...	10.5 in³
tdes ..	0.23 in	Zy ...	11.7 in³
Ix	119.0 in⁴	ry	1.72 in
		J	59.80 in⁴

ASD SUPPORT REACTIONS (kip)

Load Comb.			
D+L	-0.6	3.4	0.8
D+Lr	-0.6	3.4	0.8
D+S	-1.7	9.3	2.1
D+0.75L+0.75Lr	-0.6	3.4	0.8
D+0.75L+0.75S	-1.4	7.9	1.8
D+0.6W	-0.6	3.4	0.8
D+0.7E	-0.6	3.4	0.8
D+0.75L+0.75Lr+0.45W	-0.6	3.4	0.8
D+0.75L+0.75S+0.45W	-1.4	7.9	1.8
D+0.75L+0.75S+0.525E	-1.4	7.9	1.8
0.6D+0.6W	-0.4	2.1	0.5
0.6D+0.7E	-0.4	2.1	0.5
CD	-0.1	0.5	0.2

DESIGN FOR SHEAR

Maximum Shear Force $V = 5.1$ kip (Comb: D+S)

$$h = Ht - 3 * t = 12.0 - 3 * 0.2 = 11.3 \text{ in}$$

$$Aw = 2 * h * t = 2 * 11.3 * 0.2 = 5.3 \text{ in}^2 \quad kv = 5$$

AISC G4

$$h/t = 11.3 / 0.2 = 48.5 < 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}} = 59.2$$

$$Cv = 1.00$$

AISC Eq. (G2-3)

- Shear Yielding

$$\text{Nominal strength } Vn = 0.6 * Fy * Aw = 0.6 * 50.0 * 5.3 = 158.0 \text{ kip}$$

- Shear Buckling

$$\text{Nominal strength } Vn = 0.6 * Fy * Aw * Cv = 0.6 * 50.0 * 5.3 * 1.00 = 158.0 \text{ kip}$$

AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

$$\text{Shear allowable strength} = Vn / \Omega = 158.0 / 1.67 = 94.6 \text{ kip}$$

$$\text{Shear design ratio} = \frac{V}{Vn / \Omega} = \frac{5.1}{94.6} = 0.05 < 1.0 \text{ OK}$$

AISC G1

Project:
 Engineer:
 Descrip: Grid A&C Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment $M = -14.8$ k-ft (Comb: D+S)

$$C_b = \text{Min} (3.0, 12.5 M_{max} * R_m / (2.5 M_{max} + 3 M_a + 4 M_b + 3 M_c))$$

$$= 12.5 * -14.8 * 1.0 / (2.5 * -14.8 + 3 * 2.5 + 4 * 5.9 + 3 * 9.9) = 1.89$$

AISC Eq F1-1

- Yielding

AISC F7.1

$$\text{Plastic moment } M_{px} = F_y * Z_x = 50.0 * 25.6 = 1280.0 \text{ k-in}$$

AISC Eq. F7-1

$$\text{Nominal strength } M_{nx} = M_{px} = 1280.0 / 12 = 106.7 \text{ k-ft}$$

- Flange Local Buckling

AISC F7.2

$$\text{Nominal strength } M_{nx} = N.A. \quad (\text{compact flanges})$$

AISC F7.2(a)

- Web Local Buckling

AISC F7.3

$$\text{Nominal strength } M_{nx} = N.A. \quad (\text{compact webs})$$

AISC F7.3(a)

- Controlling limit state: Yielding

$$\text{Flexural allowable strength} = M_{nx} / \Omega = 106.7 / 1.67 = 63.9 \text{ k-ft}$$

$$\text{Flexural design ratio} = \frac{M_x}{M_{nx} / \Omega} = \frac{-14.8}{63.9} = 0.23 < 1.0 \text{ OK}$$

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	1200	180	0.15	OK
CD+CL	0.00	1200	120	0.10	OK
L	0.00	1200	180	0.15	OK
D+L	-0.01	1064	120	0.11	OK

DESIGN CODES

Steel Design AISC 360-16

Load Combinations ASCE 7-10/16

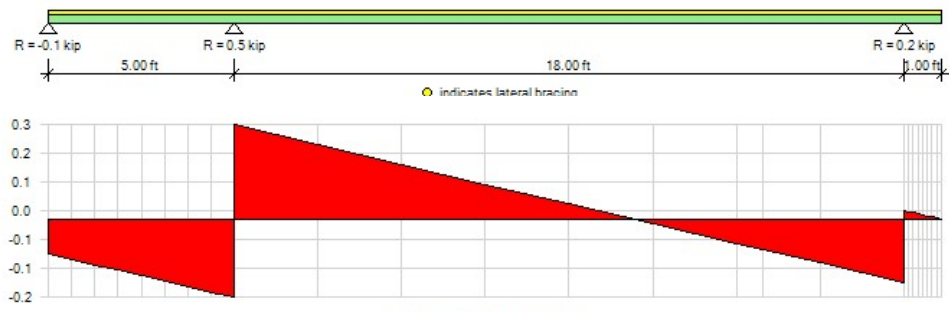
Project:
Engineer:
Descrip: Grid A&C Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

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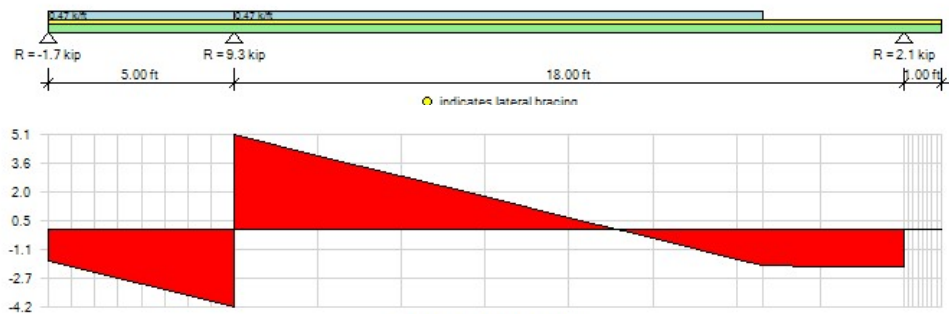


SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

(Comb: CD)



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

(Comb: D+S)

Project:
 Engineer:
 Descrip: Grid A&C Steel Beam

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

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	5.00	10.50	15.00	0.00	0.00	30.00	0.00	0.00
Span 2									
w1	0.00	14.20	10.50	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
--	-------	-----	-------	------	------	--	------	------	------

Project:
 Engineer:
 Descrip: Roof Rafter

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation	HSS12X4X1/4		
Span	Length	Support	Type
①	23.00 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	26 lb/ft	Sx ...	19.9 in³
Area .	7.1 in²	Zx ...	25.6 in³
Depth	12.0 in	rx	4.10 in
Width	4.0 in	ly	21.0 in⁴
tnom .	0.25 in	Sy ...	10.5 in³
t _{des} ..	0.23 in	Zy ...	11.7 in³
Ix	119.0 in⁴	ry	1.72 in
		J	59.80 in⁴

ASD SUPPORT REACTIONS (kip)

Load Comb.		
D+L	3.3	3.3
D+Lr	3.3	3.3
D+S	9.2	9.2
D+0.75L+0.75Lr	3.3	3.3
D+0.75L+0.75S	7.8	7.8
D+0.6W	3.3	3.3
D+0.7E	3.3	3.3
D+0.75L+0.75Lr+0.45W	3.3	3.3
D+0.75L+0.75S+0.45W	7.8	7.8
D+0.75L+0.75S+0.525E	7.8	7.8
0.6D+0.6W	2.0	2.0
0.6D+0.7E	2.0	2.0
CD	0.3	0.3

DESIGN FOR SHEAR

Maximum Shear Force $V = 9.2$ kip (Comb: D+S)

$$h = Ht - 3 * t = 12.0 - 3 * 0.2 = 11.3 \text{ in}$$

$$Aw = 2 * h * t = 2 * 11.3 * 0.2 = 5.3 \text{ in}^2 \quad kv = 5$$

AISC G4

$$h/t = 11.3 / 0.2 = 48.5 < 1.1 * \sqrt{\frac{kv * E}{Fy}} = 1.1 * \sqrt{\frac{5 * 29000}{50}} = 59.2$$

$$Cv = 1.00$$

AISC Eq. (G2-3)

- Shear Yielding

$$\text{Nominal strength } Vn = 0.6 * Fy * Aw = 0.6 * 50.0 * 5.3 = 158.0 \text{ kip}$$

- Shear Buckling

$$\text{Nominal strength } Vn = 0.6 * Fy * Aw * Cv = 0.6 * 50.0 * 5.3 * 1.00 = 158.0 \text{ kip}$$

AISC Eq. (G4-1)

- Controlling limit state: Shear Yielding

$$\text{Shear allowable strength} = Vn / \Omega = 158.0 / 1.67 = 94.6 \text{ kip}$$

$$\text{Shear design ratio} = \frac{V}{Vn / \Omega} = \frac{9.2}{94.6} = 0.10 < 1.0 \text{ OK}$$

AISC G1

Project:
 Engineer:
 Descrip: Roof Rafter

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

DESIGN FOR FLEXURE (Non-Composite)

Lateral Bracing Continuous (Top) , Unbraced (Bottom)

- Max. Bending Moment $M = 53.2$ k-ft (Comb: D+S)

$$C_b = \text{Min} (3.0, 12.5 M_{max} * R_m / (2.5 M_{max} + 3 M_a + 4 M_b + 3 M_c))$$

$$= 12.5 * 53.2 * 1.0 / (2.5 * 53.2 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0) = 3.00$$

AISC Eq F1-1

- Yielding

AISC F7.1

Plastic moment $M_{px} = F_y * Z_x = 50.0 * 25.6 = 1280.0$ k-in

AISC Eq. F7-1

Nominal strength $M_{nx} = M_{px} / 12 = 106.7$ k-ft

- Flange Local Buckling

AISC F7.2

Nominal strength $M_{nx} = N.A.$ (compact flanges)

AISC F7.2(a)

- Web Local Buckling

AISC F7.3

Nominal strength $M_{nx} = N.A.$ (compact webs)

AISC F7.3(a)

- Controlling limit state: Yielding

Flexural allowable strength $= M_{nx} / \Omega = 106.7 / 1.67 = 63.9$ k-ft

Flexural design ratio $= \frac{M_x}{M_{nx} / \Omega} = \frac{53.2}{63.9} = 0.83 < 1.0$ 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	0.05	5859	240	0.04	OK
L	0.00	9999	360	0.04	OK
D+L	0.52	530	240	0.45	OK

DESIGN CODES

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

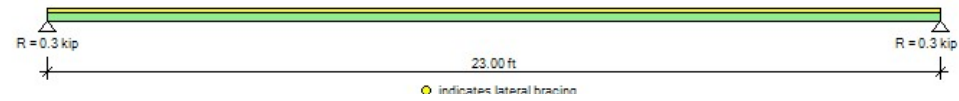
Project:
Engineer:
Descrip: Roof Rafter

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

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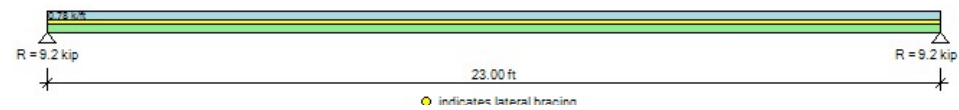


SHEAR DIAGRAM (kip)

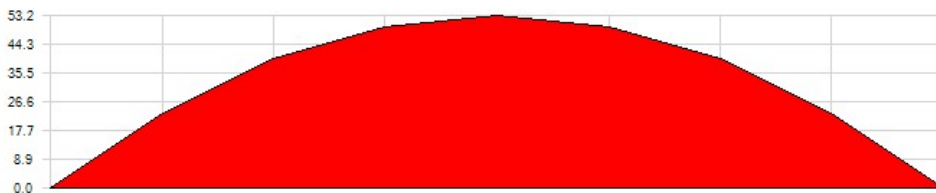


MOMENT DIAGRAM (k-ft)

(Comb: CD)



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

(Comb: D+S)

Project:
 Engineer:
 Descrip: Roof Rafter

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ASDIP Steel 5.6.4.1

STEEL BEAM DESIGN

www.asdipsoft.com

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	23.00	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
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Project:
 Engineer:
 Descrip: Steel Column

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ASDIP Steel 5.6.4.1

STEEL COLUMN DESIGN

www.asdipsoft.com

GEOMETRY		PROPERTIES			
Column Designation	HSS4X4X1/4	Weight	12 lb/ft	Sx ...	3.9 in³
Steel Yield Strength Fy	50.0 ksi	Area .	3.4 in²	Zx ...	4.7 in³
Modulus of Elasticity Es	29000 ksi	Depth	4.0 in	rx	1.52 in
Member Length L	17.00 ft	Width	4.00 in	ly	7.8 in⁴
Effective Length Kx-factor	1.00	t nom .	0.25 in	Sy ...	3.9 in³
Effective Length Ky-factor	1.00	t des .	0.23 in	Zy ...	4.7 in³
Unbraced Length Lb	N.A. ft	Ix	7.8 in⁴	ry	1.52 in
				J	12.80 in⁴

UNFACTORED LOADS (Elastic Second-Order Analysis)

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	3.3	0.0	0.0	5.9	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: 1.4D

Axial load Pr = $1.4 * 3.3 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 5.9 + 0.0 * 0.0 + 0.00 * 6.0 = 4.6$ kip

Mx gravity bot = $1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0$ k-ft

Mx lateral bot = 0.0 k-ft

Mx gravity top = $1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0$ k-ft

Mx lateral top = 0.0 k-ft

My gravity bot = $1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0$ k-ft

My lateral bot = 0.0 k-ft

My gravity top = $1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0$ 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

Project:
 Engineer:
 Descrip: Steel Column

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ASDIP Steel 5.6.4.1

STEEL COLUMN DESIGN

www.asdipsoft.com

DESIGN FOR COMPRESSION

Slender unstiffened $Q_s = 1.00$, Slender stiffened $Q_a = 1.00$, $Q = Q_s * Q_a = 1.00$ AISC E7

Slenderness ratio $K_x L / r_x = 1.00 * 17.00 / 1.52 = 134.2$

Slenderness ratio $K_y L / r_y = 1.00 * 17.00 / 1.52 = 134.2$

Maximum slenderness ratio = Max (134.2, 134.2) = 134.2 AISC E3

- Flexural Buckling

Elastic buckling $Fe = \frac{\pi^2 * E}{(K L / r)^2} = \frac{3.14^2 * 29000}{134.2^2} = 15.9 \text{ ksi}$ AISC Eq. E3-4

$Fe < 0.44 * Q * Fy$ (15.9 < 0.44 * 1.00 * 50.0 = 22.0)

Flexural buckling stress $Fcr = Fe * 0.877 = 15.9 * 0.877 = 13.9 \text{ ksi}$ AISC Eq. E3-3

- Torsional Buckling

$Fe = \left[\frac{\pi^2 * E * C_w}{(K_y L)^2} + G * J \right] \frac{1}{I_x + I_y}$ AISC Eq. E4-4
 $= \left[\frac{\pi^2 * 29000 * 6.6}{(1.00 * 17.0 * 12)^2} + 11200 * 12.8 \right] \frac{1}{7.8 + 7.8} = 9192.6 \text{ ksi}$

$Fe \geq 0.44 * Q * Fy$ (9192.6 >= 0.44 * 1.00 * 50.0 = 22.0)

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

$Fcr = 1.00 * 50.0 * (0.658)^{1.00 * 50.0 / 9192.6} = 49.9 \text{ ksi}$

Compressive strength $Pn = Fcr * Ag = 13.9 * 3.4 = 47.0 \text{ kip}$ AISC Eq. E3-1

Controlling limit state: Flexural Buckling

Compressive design ratio = $\frac{Pu}{\phi Pn} = \frac{17.8}{0.90 * 47.0} = 0.42 < 1.0 \text{ OK}$ AISC E1

Project:
Engineer:
Descrip: Steel Column

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ASDIP Steel 5.6.4.1

STEEL COLUMN DESIGN

www.asdipsoft.com

DESIGN FOR FLEXURE

$$C_b = \text{Min} (3.0, 12.5 M_{\max} * R_m / (2.5 M_{\max} + 3 M_a + 4 M_b + 3 M_c))$$
$$= 12.5 * 0.0 * 1.0 / (2.5 * 0.0 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0) = 1.00$$

AISC Eq F1-1

- Yielding

Plastic moment $M_{px} = F_y * Z_x = 50.0 * 4.7 = 234.5$ k-in

Nominal strength $M_{nx} = M_{px} = 234.5 / 12 = 19.5$ k-ft

Plastic moment $M_{py} = F_y * Z_y = 50.0 * 4.7 = 234.5$ k-in

Nominal strength $M_{ny} = M_{py} = 234.5 / 12 = 19.5$ k-ft

AISC Eq. F7-1

- Flange Local Buckling

AISC F7.2

Nominal strength $M_{nx} = N.A.$ (compact flanges)

AISC F7.2(a)

- Web Local Buckling

AISC F7.3

Nominal strength $M_{nx} = N.A.$ (compact webs)

AISC F7.3(a)

X - Controlling limit state: Yielding

$$X - \text{Flexural design ratio} = \frac{M_x}{\phi M_{nx}} = \frac{0.0}{0.90 * 19.5} = 0.00 < 1.0 \text{ OK}$$

AISC F1

Y - Controlling limit state: Yielding

$$Y - \text{Flexural design ratio} = \frac{M_y}{\phi M_{ny}} = \frac{0.0}{0.90 * 19.5} = 0.00 < 1.0 \text{ OK}$$

AISC F1

DESIGN FOR COMBINED FORCES

Design axial strength $P_c = \phi P_n = 0.90 * 47.0 = 42.3$ kip

AISC E1

Design flexural strength $M_{cx} = \phi M_{nx} = 0.90 * 19.5 = 17.6$ kip

AISC F1

Design flexural strength $M_{cy} = \phi M_{ny} = 0.90 * 19.5 = 17.6$ kip

AISC F1

$$\text{Combined forces ratio} = \frac{P}{P_c} + 8/9 \left[\frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \right]$$

AISC Eq. H1-1a

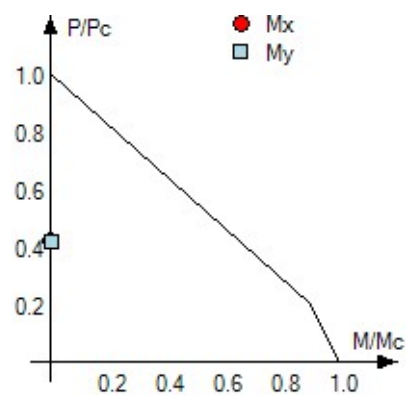
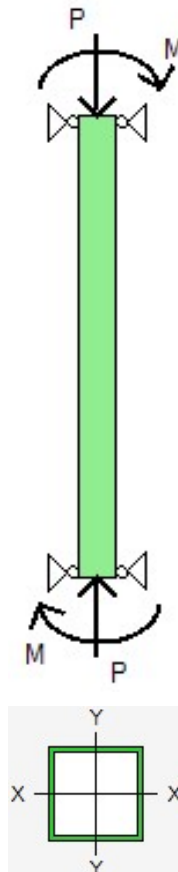
$$= \frac{17.8}{42.3} + 8/9 \left[\frac{0.0}{17.6} + \frac{0.0}{17.6} \right] = 0.42 < 1.0 \quad \text{OK}$$

DESIGN CODES

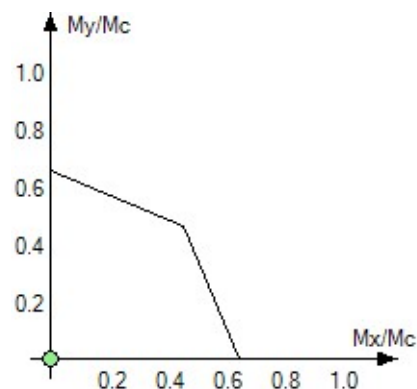
Steel Design AISC 360-16
 Load Combinations ASCE 7-10/16

SEISMIC PROVISIONS

Design Spectral Acc. S_{ds} 1.03
 Overstrength Seismic Factor Ω_o 2.00



INTERACTION DIAGRAM



BIAXIAL BENDING DIAGRAM

GEOMETRY

Footing Length (X-dir)	2.80	ft	
Footing Width (Z-dir)	1.50	ft	
Footing Thickness	8.0	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	1.9	ksf	
Soil Pressure at Corner 2	1.9	ksf	
Soil Pressure at Corner 3	1.9	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.97		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	3.3	0.0	0.0	5.9	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 = $0.00 + 8.0 / 12 = 0.67$ ft

Moment = $0.0 * 0.67 = 0.0$ k-ft

- Passive Force = 0.0 kip

Arm = 0.27 ft

Moment = 0.0 k-ft

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

- Resisting about X-X

- Footing weight = $0.6 * W * L * Thick * Density = 0.6 * 1.50 * 2.80 * 8.0 / 12 * 0.15 = 0.3$ kip

Arm = $W / 2 = 1.50 / 2 = 0.75$ ft

Moment = $0.3 * 0.75 = 0.2$ 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 = $W / 2 - Offset = 1.50 / 2 - 0.0 / 12 = 0.75$ ft

Moment = $0.0 * 0.75 = 0.0$ k-ft

- Soil cover = $0.6 * W * L * SC * Density = 0.6 * (1.50 * 2.80 - 6.0 / 12 * 6.0 / 12) * 1.0 * 110 = 0.3$ kip

Arm = $W / 2 = 1.50 / 2 = 0.75$ ft

Moment = $0.3 * 0.75 = 0.2$ k-ft

- Buoyancy = $0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 1.50 * 2.80 * 62 * (0.67) = -0.1$ kip

Arm = $W / 2 = 1.50 / 2 = 0.75$ ft

Moment = $0.1 * 0.75 = -0.1$ k-ft

- Axial force P = $0.6 * 3.3 + 0.6 * 0.0 = 2.0$ kip

Arm = $W / 2 - Offset = 1.50 / 2 - 0.0 / 12 = 0.75$ ft

Moment = $2.0 * 0.75 = 1.5$ k-ft

- Resisting moment X-X = $0.2 + 0.0 + 0.2 + 1.5 + -0.1 = 1.8$ k-ft

- Overturning safety factor X-X = $\frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{1.8}{0.0} = 17.91 > 1.50$ OK

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment } Z-Z = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 1.50 * 2.80 * 8.0 / 12 * 0.15 = 0.3 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.80 / 2 = 1.40 \text{ ft}$$

$$\text{Moment} = 0.3 * 1.40 = 0.4 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.80 / 2 - 0.0 / 12 = 1.40 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.40 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (1.50 * 2.80 - 6.0 / 12 * 6.0 / 12) * 1.0 * 110 = 0.3 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.80 / 2 = 1.40 \text{ ft}$$

$$\text{Moment} = 0.3 * 1.40 = 0.4 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 1.50 * 2.80 * 62 * (0.67) = -0.1 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.80 / 2 = 1.40 \text{ ft}$$

$$\text{Moment} = 0.1 * 1.40 = -0.1 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 3.3 + 0.6 * 0.0 = 2.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.80 / 2 - 0.0 / 12 = 1.40 \text{ ft}$$

$$\text{Moment} = 2.0 * 1.40 = 2.8 \text{ k-ft}$$

$$\text{- Resisting moment } Z-Z = 0.4 + 0.0 + 0.4 + 2.8 + -0.1 = 3.3 \text{ k-ft}$$

$$\text{- Overturning safety factor } Z-Z = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{3.3}{0.0} = 33.43 > 1.50 \text{ OK}$$

SOIL BEARING PRESSURES (Comb: D+0.7S)

$$\text{Overturning moment } X-X = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment } X-X = 0.3 + 0.0 + 0.3 + -0.1 + 5.6 = 6.1 \text{ k-ft}$$

$$\text{Overturning moment } Z-Z = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment } Z-Z = 0.6 + 0.0 + 0.6 + -0.2 + 10.4 = 11.4 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.4 + 0.0 + 0.4 - 0.2 + 7.4 = 8.1 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

$$X_p = \frac{Z\text{-Resisting moment} - Z\text{-Overturning moment}}{\text{Resisting force}} = \frac{11.4 - 0.0}{8.1} = 1.40 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Z_p = \frac{X\text{-Resisting moment} - X\text{-Overturning moment}}{\text{Resisting force}} = \frac{6.1 - 0.0}{8.1} = 0.75 \text{ ft}$$

$$X\text{-ecc} = \text{Length} / 2 - X_p = 2.80 / 2 - 1.40 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 1.50 / 2 - 0.75 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 1.50 * 2.80 = 4.2 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 2.80 * 1.50^2 / 6 = 1.1 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 1.50 * 2.80^2 / 6 = 2.0 \text{ ft}^3$$

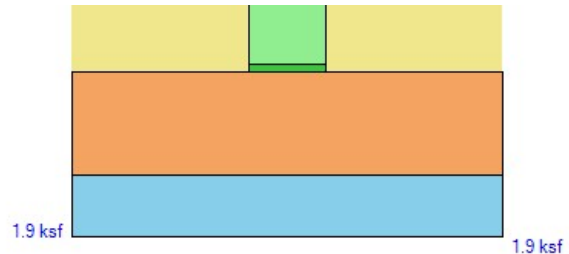
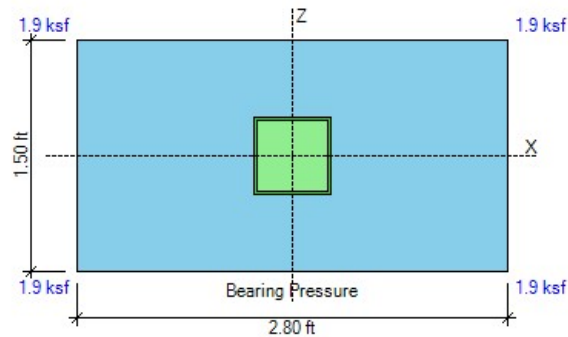
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 8.1 * (1/4.2 + 0.00 / 1.1 + 0.00 / 2.0) = 1.93 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 8.1 * (1/4.2 - 0.00 / 1.1 + 0.00 / 2.0) = 1.93 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 8.1 * (1/4.2 - 0.00 / 1.1 - 0.00 / 2.0) = 1.93 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 8.1 * (1/4.2 + 0.00 / 1.1 - 0.00 / 2.0) = 1.93 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (1.00 + 8.0 / 12 / 2) = 0.63$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.63 \cdot 8.0 / 12 \cdot 1.50 = 0.6$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.63 \cdot 8.0 / 12 \cdot 2.80 = 1.2$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 2.4 \cdot 0.35) = 0.8$ kip

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

$$\text{- Sliding safety factor X-X} = \frac{\text{X-Passive force} + \text{Friction}}{\text{X-Horizontal load}} = \frac{1.00 \cdot 0.6 + 1.00 \cdot 0.8}{0.0} = 14.70 > 1.50 \quad \text{OK}$$

$$\text{- Sliding safety factor Z-Z} = \frac{\text{Z-Passive force} + \text{Friction}}{\text{Z-Horizontal load}} = \frac{1.00 \cdot 1.2 + 1.00 \cdot 0.8}{0.0} = 20.20 > 1.50 \quad \text{OK}$$

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

$$\text{- Uplift safety factor} = \frac{\text{Pedestal} + \text{Footing} + \text{Cover} - \text{Buoyancy}}{\text{Uplift load}} = \frac{0.0 + 0.3 + 0.3 - 0.1}{0.0} = 99.99 > 1.00 \quad \text{OK}$$

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

Use Plain Concrete Shear Strength

$$\phi V_{cx} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 1.5 \cdot 12 \cdot 8.0 / 1000 = 5.8 \text{ kip}$$

ACI 14.5.5.1

$$\phi V_{cz} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 2.8 \cdot 12 \cdot 8.0 / 1000 = 10.8 \text{ kip}$$

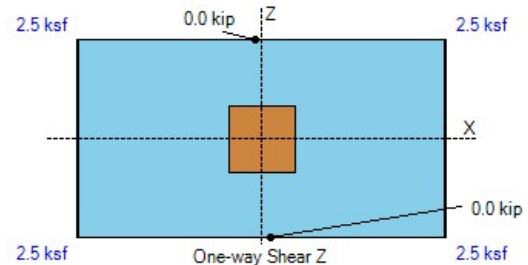
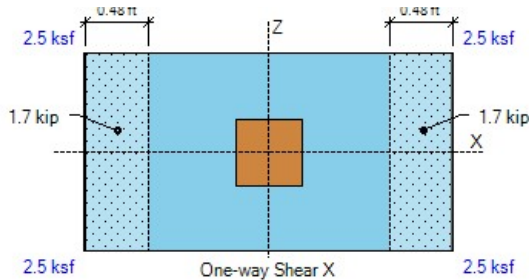
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

$$\text{One-way shear } V_{ux} \text{ (- Side)} = 1.7 \text{ kip} < 5.8 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{ux} \text{ (+ Side)} = 1.7 \text{ kip} < 5.8 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (- Side)} = 0.0 \text{ kip} < 10.8 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (+ Side)} = 0.0 \text{ kip} < 10.8 \text{ kip} \quad \text{OK}$$



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.80 * 8.0^2 / 6 / 1000 = 1.2 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 1.50 * 8.0^2 / 6 / 1000 = 0.6 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 4.5 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 4.5 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 2.4 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 2.4 \text{ k-ft OK}$$

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

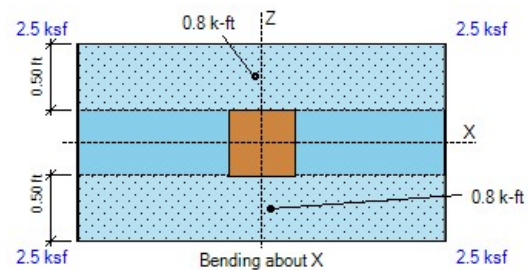
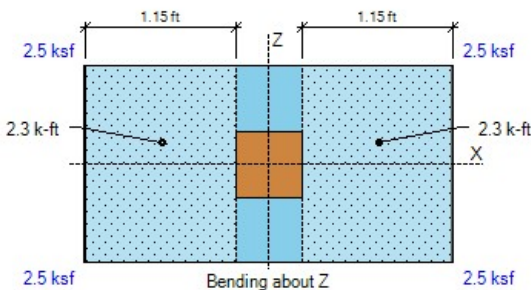
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

$$\text{Bottom moment Mux (- Side)} = 0.8 \text{ k-ft} < 4.5 \text{ k-ft OK} \quad \text{ratio} = 0.18$$

$$\text{Bottom moment Mux (+ Side)} = 0.8 \text{ k-ft} < 4.5 \text{ k-ft OK} \quad \text{ratio} = 0.19$$

$$\text{Bottom moment Muz (- Side)} = 2.3 \text{ k-ft} < 2.4 \text{ k-ft OK} \quad \text{ratio} = 0.97$$

$$\text{Bottom moment Muz (+ Side)} = 2.3 \text{ k-ft} < 2.4 \text{ k-ft OK} \quad \text{ratio} = 0.97$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 9.9 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.3 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.80 * 12 / 2 - 0.0 - 6.0 / 2, 1.50 * 12 / 2 - 0.0 - 6.0 / 2) = 6.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.80 * 12 * 1.5 * 12, (6.0 + 2 * 6.0) * (6.0 + 2 * 6.0)] = 324.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(324.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.3 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 1 / 55 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db}^{1.5})$

ACI 25.4.3

$L_{dh} = \text{Max} (8 \text{ db}, 6, 1 / 55 * 60.0 * 1000 / (2500)^{1/2} * 1.6 * 1.0 * 0.8 * 0.75^{1.5}) = 17.4 \text{ in}$

$L_d \text{ provided} = \text{Dowel length} = 3.00 * 12 = 36.0 \text{ in} > 12.0 \text{ in OK}$

$L_{dh} \text{ provided} = \text{Footing thickness} - \text{Cover} = 8.00 - 3.0 = 5.0 \text{ in} < 17.4 \text{ in NG}$

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

X-Edge = $\text{Length} / 2 - \text{Offset} - \text{Col} / 2 = 2.80 * 12 / 2 - 0.0 - 6.0 / 2 = 13.8 \text{ in}$ $\alpha_{sx} = 10$

Z-Edge = $\text{Width} / 2 - \text{Offset} - \text{Col} / 2 = 1.50 * 12 / 2 - 0.0 - 6.0 / 2 = 6.0 \text{ in}$ $\alpha_{sz} = 10$

$\alpha_s = \alpha_{sx} + \alpha_{sz} = 10 + 10 = 20$ Col type = Corner $\beta = L / W = 6.0 / 6.0 = 1.00$

ACI 22.6.5.2

Perimeter $b_o = \alpha_{sz} / 10 * (L + d / 2 + X\text{-Edge}) + \alpha_{sx} / 10 * (W + d / 2 + Z\text{-Edge})$

ACI 22.6.4.2

$b_o = 10 / 10 * (6.0 + 8.0 / 2 + 13.8) + 10 / 10 * (6.0 + 8.0 / 2 + 6.0) = 39.8 \text{ in}$

Area $A_{bo} = (L + d / 2 + X\text{-Edge}) * (W + d / 2 + Z\text{-Edge}) = (6.0 + 8.0 / 2 + 13.8) * (6.0 + 8.0 / 2 + 6.0) = 380.8 \text{ in}^2$

Use Plain Concrete Shear Strength

$\phi V_c = \phi * \text{Min} (1 + 2 / \beta, 2) * 4/3 * \sqrt{f_c}$

ACI 14.5.5.1

$\phi V_c = 0.60 * \text{Min} (1 + 2 / 1.00, 2) * 4/3 * \sqrt{2500} = 80.0 \text{ psi}$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$F = 9.9 + 0.19 * 380.8 / 144 - 3.5 = 6.9 \text{ kip}$

$b_1 = L + d / 2 + X\text{-Edge} = 6.0 + 8.0 / 2 + 13.8 = 23.8 \text{ in}$ $b_2 = W + d / 2 + Z\text{-Edge} = 6.0 + 8.0 / 2 + 6.0 = 16.0 \text{ in}$

$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_2 / b_1)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(16.0 / 23.8)}} = 0.35$

ACI Eq. (8.4.4.2.2)

$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_1 / b_2)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(23.8 / 16.0)}} = 0.45$

ACI Eq. (8.4.2.3.2)

$X_{2z} = b_1^2 / 2 / (b_1 + b_2) = 23.8^2 / 2 / (23.8 + 16.0) = 7.1 \text{ in}$ $X_{2x} = b_2^2 / 2 / (b_2 + b_1) = 3.2 \text{ in}$

$J_{cz} = b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$

ACI R8.4.4.2.3

$J_{cz} = 23.8 * 8.0^3 / 12 + 23.8^3 * 8.0 / 12 + 23.8 * 8.0 * (23.8 / 2 - 7.1)^2 + 16.0 * 8.0 * 7.1^2 = 20842 \text{ in}^4$

$J_{cx} = b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$

ACI R8.4.4.2.3

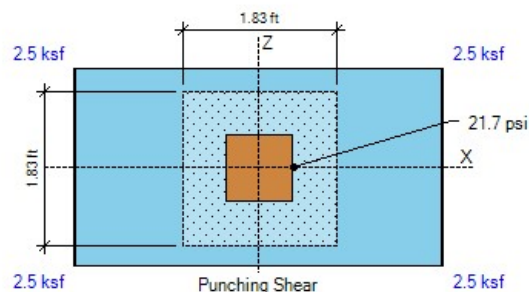
$J_{cx} = 16.0 * 8.0^3 / 12 + 16.0^3 * 8.0 / 12 + 16.0 * 8.0 * (16.0 / 2 - 3.2)^2 + 23.8 * 8.0 * 3.2^2 = 8312 \text{ in}^4$

Stress due to $P = F / (b_o * d) * 1000 = 6.9 / (39.8 * 8.0) * 1000 = 21.7 \text{ psi}$

Stress due to $M_x = \gamma_{vx} * X\text{-OTM} * X_{2x} / J_{cx} = 0.35 * 0.0 * 12 * 3.2 / 8312 * 1000 = 0.0 \text{ psi}$

Stress due to $M_z = \gamma_{vz} * Z\text{-OTM} * X_{2z} / J_{cz} = 0.35 * 0.0 * 12 * 7.1 / 20842 * 1000 = 0.0 \text{ psi}$

Punching stress = $P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 21.7 + 0.0 + 0.0 = 21.7 \text{ psi} < 80.0 \text{ psi OK}$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 9.9 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.3 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.80 * 12 / 2 - 0.0 - 6.0 / 2, 1.50 * 12 / 2 - 0.0 - 6.0 / 2) = 6.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.80 * 12 * 1.5 * 12, (6.0 + 2 * 6.0) * (6.0 + 2 * 6.0)] = 324.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(324.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

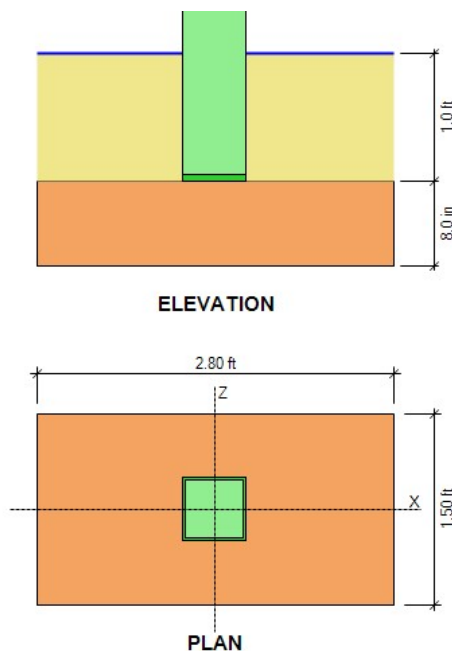
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.3 \text{ psi OK}$$

DESIGN CODES

Concrete Design ACI 318-19

Load Combinations ASCE 7-22



8/11/2025

C. P. ERULLIOWI, PE

ETC - TRASH NORTH

LATERAL ANALYSIS

1

WIND $V_{ASD} = 95 \text{ MPH}$ $V_{ULCIIORAH}$ Exp. B $K_{zt} = 1.0$ $SLOPE = 1:12$

ZONE 1E24E = 13.5 PSF 16.0 PSF MIN

ZONE 1R4 = 9.0 PSF 16.0 PSF MIN

ZONE 2E3E = 1.0 PSF 8.0 PSF MIN

ZONE 2R3 = 1.2 PSF 8.0 PSF MIN

ZONE 5E6E = 13.5 PSF 16.0 PSF MIN

ZONE 5R6 = 8.9 PSF 16.0 PSF MIN

SEISMIC $S_{DS} = 1.03$ $R = 3.75$ $I_e = 1.0$

$$C_s = (1.03 / (3.75 \times 1.0)) / 1.4 = 0.23$$

$$W_{ROOF} = (25 \text{ PSF} \times 964 \text{ SF}) = 24,100 \text{ F}$$

$$U_s = 24,100 \text{ F} \times 0.23 = 5,543 \text{ F}$$

GRID 1

$$F_w = (16 \text{ PSF} \times 202 \text{ SF}) = 3,232 \text{ F}$$

$$F_E = 5,543 \text{ F}$$

GRID 5 ABC

$$F_w = (16 \text{ PSF} \times 177 \text{ SF}) = 2,832 \text{ F}$$

$$F_E = 5,543 \text{ F} \times (482 \text{ SF} / 964 \text{ SF}) = 2,772 \text{ F}$$

8/11/2025

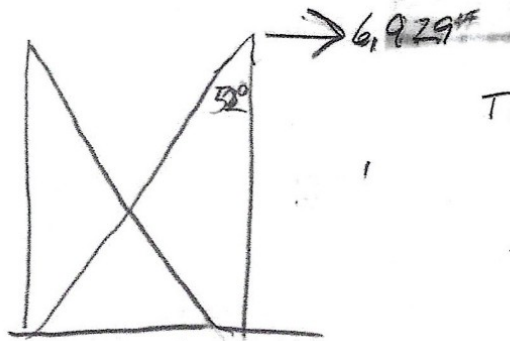
C. PIERUCCIONI, PE

ETC-TRASH NORTH

LATERAL ANALYSIS

2

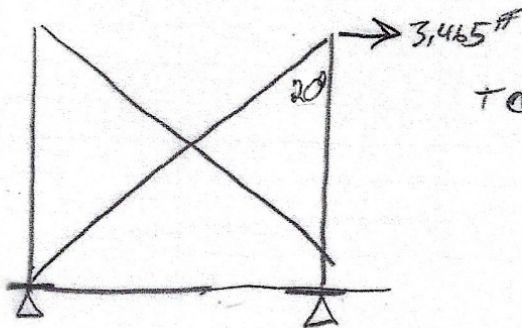
GRID 1 $F_E = 5,543^{\#} \times 1.25 = 6,929^{\#}$



$$T_{CABLE} = 6,929^{\#} / \tan 52^{\circ} = 5,414^{\#}$$

MAKE BRACING $T_E = 6,000^{\#}$

GRIDS A & C $F_E = 2,772^{\#} \times 1.25 = 3,465^{\#}$



$$T_{CABLE} = 3,465^{\#} / \tan 20^{\circ} = 9,520$$

MAKE 1 CROSS BRACING $T_E = 9,600^{\#}$