ARW ENGINEERS
structural consultants

Structural Calculations

For

OWTC MT Bldg Bridge Replacement

23358.A

April 11, 2025

PROFESSIONAL ENGINEER
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4/11/2025
STATE OF UTAH

REVIEWED FOR CODE COMPLIANCE
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06/16/2025
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STRUCTURAL CALCULATIONS

FOR

OWTC MT Bldg Bridge Replacement

Client: Sanders Associates Architects

Project Number: 23358.A

DESIGN CRITERIA

GOVERNING CODE: IBC 2021

GENERAL: Risk Category = II

SEISMIC: Seismic Design Category = D
I_E = 1.0 R = 1.0
S_{DS} = 1.121 g S_{D1} = 0.614 g

WIND: Basic Wind Speed = 115 mph
Exposure Classification = C

SOILS: Site Class: D (Default)
Design Allowable Soil Bearing Pressure = 1,500 psf (Default)
Soils report by: N/A
Report Number: NA
Dated: N/A

DESIGN LOADS

FLOORS:	DL =	40 psf	LL =	100 psf
ROOFS:	DL =	7 psf	SL =	27 psf

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Project Number 23358.A | Sheet Number
Project OWTL BRIDGE REPLACEMENT
Prepared By JWM | Date 1/20/2025

DESIGN CRITERIA:

RISK CATEGORY = II

NO SOILS REPORT → 1,500 psf SOIL BEARING PRESSURE.
D-DEFAULT FOR SEISMIC.

DEAD:

FLOOR:

CONCRETE FILL, 18 GA. DECK, PONDING ALLOWANCE

4" SLAB OVER B DECK = 27.8 psf + 2.3 psf + 6 psf
= 36 psf

ALLOW RISA TO CALCULATE SELF-WT OF FRAMING

CARPET + SOFFIT = 2 psf
LIGHTING + MISC. = 2 psf
TOTAL = 40 psf

ROOF:

16 GA B-DECK = 3.5 psf
5" RIGID INSULATION = 1 psf
MISC/MEP = 2.5 psf
TOTAL = 7 psf

WALLS:

6" METAL STUD W/ DENSGLASS ; GLASS = 10 psf

SNOW LOAD:

P_g = 38 psf
C_e = 1.0
C_t = 1.0
I_s = 1.0
P_f = 0.7 + 38 psf = 27 psf

SNOW DRIFT AGAINST BLDG.:
Pd = 73 psf FOR W = 16'-0"
USE 18 GA. ROOF DECK TYPICALLY AND 16 GA @ DRIFT.

B-36 FormLok® Composite Steel Deck-Slab (ASD)
with 4 in. 110 pcf 3000 psi LWC

VERCO DECKING, INC. NUCOR

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	6'-7"	7'-9"	7'-10"
20	7'-10"	9'-2"	9'-4"
18	$\boxed{9'-0"}$	10'-9"	11'-1"
16	9'-8"	11'-11"	11'-9"

> 6'-9" ∴ ok

Maximum Unshored Span based on:

Construction Live Load w/ Concrete	20.00	psf			
Constructio	50.00	psf	Minimum end bearing	3.00	in.
Concentrated Construction Load	150.00	plf	Minimum interior bearing	5.50	in.
Concrete Ponding Allowance	6.00	psf	Maximum Deflection L/	180	≤ 0.75 in.
Concrete Volume	0.94	yd^3 / 100 ft^2 (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	Fy ksi	wdd psf	S_e^+ $in.^3/ft$	S_e^- $in.^3/ft$	I_d^+ $in.^4/ft$	I_d^- $in.^4/ft$	Vn/Ω kip/ft
22	50	1.90	0.176	0.188	0.178	0.192	2.687
20	50	2.30	0.230	0.237	0.219	0.231	3.220
18	50	2.90	0.314	0.331	0.302	0.306	4.264
16	50	3.50	0.399	0.410	0.381	0.381	5.302

Superimposed Allowable Load, Wn/Ω, Limited by L/360, psf [1]

Gage	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"
22	462	332	247	186	135	102	78	61	49
20	543	391	285	200	146	109	84	66	53
18	693	480	321	226	164	123	95	75	60
16	772	527	353	248	180	135	104	82	65

100 psf LIVE + CARPET < 480 psf ∴ ok

Notes: [1] For high loads, long term concrete creep should be considered.

Composite Steel Deck-Slab Properties							Min. Temperature & Shrinkage	
Gage	w_1 psf	Ic $in.^4/ft$	Iu $in.^4/ft$	Id [1] $in.^4/ft$	Mno/Ω kip-ft/ft	Vno/Ω kip/ft	As min [2] $in.^2/ft$	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	29.7	2.41	3.81	3.11	2.22	2.41	0.028	25
20	30.1	2.71	3.99	3.35	2.58	2.41	0.028	25
18	30.7	3.22	4.32	3.77	3.26	2.41	0.028	25
16	31.3	3.65	4.62	4.14	3.89	2.41	0.028	25

Notes: [1] Id = (Ic + Iu)/2

[2] Minimum area of steel for temperature and shrinkage

Composite Deck-Slab V4.0 is based on:
ANSI/SDI C-2017, IAPMO UES ER-2018, and IAPMO UES ER-0423

Date: 1/21/2025

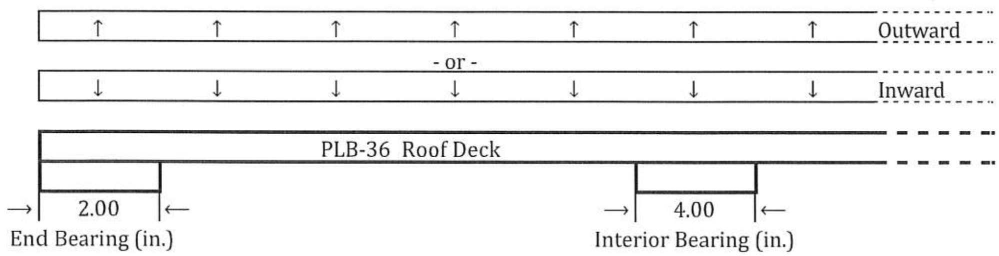
18 Gage PLB™-36 Grade 50

Uniform Allowable Load Table, ASD (psf)

For End Lapped Deck

36/7 Connection Pattern to Supports with
Hilti X-HSN 24 PAF

Support Member A572 GR50
$0.125 \leq t_2$ (in.) ≤ 0.375



86.5 psf

27 psf + 7 psf = 34 psf < 86.5 psf
∴ 18 Ga works.

Inward Uniform Allowable Load Table, ASD (psf)

6'-9"

Span	Span	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"
1	Wn/Ω	251	207	174	148	128	111	98	87
	L/180	211	159	122	96	77	63	52	43
2	Wn/Ω	259	215	181	155	134	116	102	91
	L/180	-	-	-	-	-	-	-	-
3	Wn/Ω	322	267	225	192	166	145	128	113
	L/180	-	-	-	184	147	120	99	82

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

Span	Span	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"
1	Wn/Ω	264	218	183	156	135	117	103	91
	Rn/Ω	203	185	169	156	145	135	127	119
	L/180	-	161	124	97	78	63	52	44
2	Wn/Ω	247	204	172	147	127	111	97	86
	Rn/Ω	162	148	135	125	116	108	102	96
	L/180	-	-	-	-	-	-	-	-
3	Wn/Ω	306	254	214	183	158	138	121	108
	Rn/Ω	185	168	154	142	132	123	115	109
	L/180	-	-	-	-	-	118	97	81

Steel Deck Properties

t	Design 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
0.0478	50	2.90	0.302	0.306	0.314	0.331	783	826	4264

Where: $W \leq Wn/\Omega$

W = Required strength of the governing ASD load combination
Wn/Ω = Allowable strength governed by the steel deck
Rn/Ω = Allowable strength governed by connection tension

Bare Deck Uniform Load V1.0.7 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-2018

Date: 1/21/2025

16 Gage PLB™-36 Grade 50

Uniform Allowable Load Table, ASD (psf)

For End Lapped Deck

36/7 Connection Pattern to Supports with
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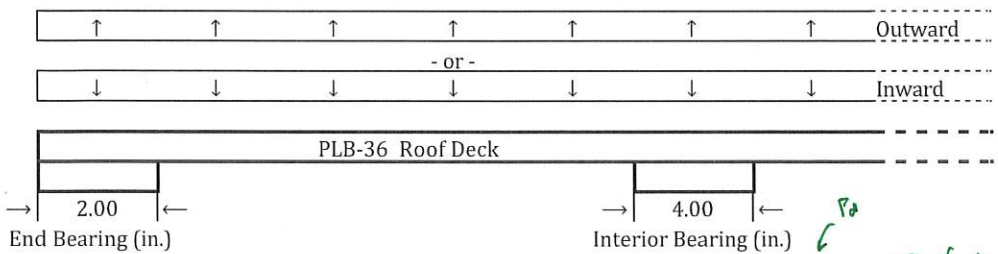


Pd

109 psf   73 psf + 27 psf + 7 psf = 107 psf

6'-9"

Inward Uniform Allowable Load Table, ASD (psf)

Span	Span	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"
1	Wn/Ω	319	263	221	189	163	142	124	110
	L/180	266	200	154	121	97	79	65	54
2	Wn/Ω	321	266	224	192	165	144	127	113
	L/180	-	-	-	-	-	-	-	-
3	Wn/Ω	399	331	279	238	206	180	158	140
	L/180	-	-	-	229	183	149	123	102

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

Span	Span	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"
1	Wn/Ω	327	270	227	194	167	145	128	113
	Rn/Ω	203	185	169	156	145	135	127	119
	L/180	-	-	154	121	97	79	65	54
2	Wn/Ω	313	260	219	187	161	141	124	110
	Rn/Ω	162	148	135	125	116	108	102	96
	L/180	-	-	-	-	-	-	-	-
3	Wn/Ω	389	322	272	232	201	175	154	137
	Rn/Ω	185	168	154	142	132	123	115	109
	L/180	-	-	-	-	-	-	-	102

Steel Deck Properties

t in	Design 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
0.0598	50	3.50	0.381	0.381	0.399	0.410	996	1023	5302

Where: $W \leq Wn/\Omega$

W = Required strength of the governing ASD load combination
Wn/Ω = Allowable strength governed by the steel deck
Rn/Ω = Allowable strength governed by connection tension

Bare Deck Uniform Load V1.0.7 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-2018

Date: 1/21/2025

ENGINEERS

ASCE 7-16 SNOW DRIFT ANALYSIS

Version Date: May 17, 2021 Author: TMD

21-Jan-25

10:30 AM

JOB TITLE:

DRIFT LOCATION:

JOB #:

DATA USED IN ANALYSIS

Ground Snow Load	P_g :	38	psf
Exposure Factor	C_e :	1.0	
Thermal Factor	C_t :	1.0	
Importance Factor	I_s :	1.0	
Roof Dead Load		20	psf
Joist Span		60	ft
Distance from P_m to P_z		0	ft
Joist Spacing		5.5	ft

Horizontal Dimension of High Roof	L_u :	105	ft
Horizontal Dimension of Low Roof	L_l :	50	ft
Vertical Distance Between Upper and Lower Roof	h_r :	7	ft
Type of Roof Step :		Normal	

RESULTS OF ANALYSIS

Roof Snow Load	P_f :	26.6	psf	
Depth of Uniform Load	h_b :	1.40	ft	
Density of Snow	D :	18.94	pcf	
	$h_c = (h_r - h_b)$:	5.60	ft	
	h_c/h_b :	3.98	**> 0.2 Drifting Needs to be Considered**	

Height of Snow Drift

h_d :	3.84	ft	Leeward	**Controls Width Drift Calc**
h_d :	2.00	ft	Windward	

Length of Snow Drift	w :	15.4	ft	**Equation used (4hd)**
Length of Drift on Joist	w_j :	**15.4**	ft	
Height of Snow Drift	h_d :	**3.84**	ft	
	P_d :	**72.7**	psf	

Intensity of Drift at	P_z :	**72.7**	psf
Intensity of Drift at	P_e :	**0.0**	psf

DRIFT CAUSING TRIANGULAR LOAD ON JOISTS

	Joist Reactions (kips) based on 5.5 foot spacing		Joist Reactions (kips) based on 1 foot spacing	
	Left End	Right End	Left End	Right End
DL	3.30	3.30	0.60	0.60
SL	4.39	4.39	0.80	0.80
DRIFT	2.81	0.26	0.51	0.05
TOTAL	10.50	7.95	1.91	1.45

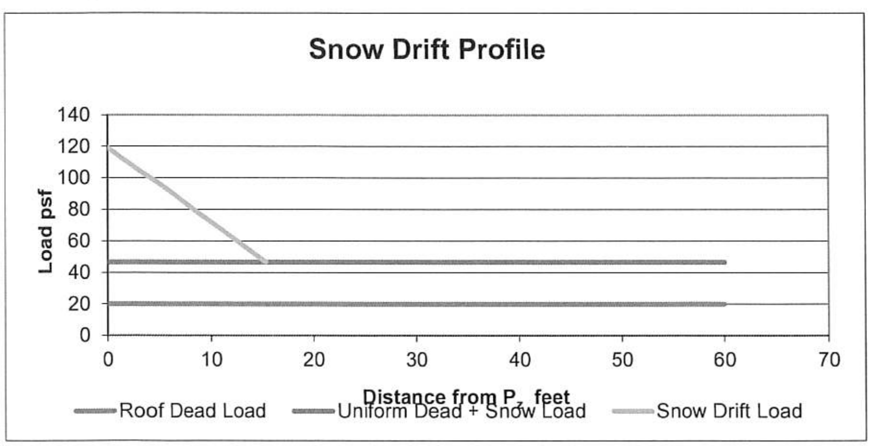
Maximum Moment	123.33	k-ft
occurring @	28.80	ft from Left End of Joist

Equivalent Uniform Joist Load (plf)

DL	110	plf
SL	164	plf
TOTAL	274	plf

Drift Information

Maximum weight of drift :	3071	lbs
center of gravity from Left End :	5.12	feet



Definitions

P_m : Maximum Magnitude of Snow Drift

P_z : Maximum Magnitude of Snow Drift at 0 feet away from Pm

P_e : Magnitude of Drift at End of Joist

Equation of Snow Drift Line

y = -4.735 * X + 72.7219790318758

Distance X is in feet from P_m

ARW ENGINEERS

PROJECT DESIGN CRITERIA

Version Date: May 30, 2022 Updated by: Scott VanderDoes

Governing Building Code : 2021 IBC
Building Risk Category : II

Street:	200 S Washington Blvd	Latitude :	41.263786
City:	Ogden	Longitude :	-111.961541
State:	UT	Elevation :	4320 ft

(Elevation based on USGS 3DEP 1/3 arc-second layer hosted at the NGTOC)

WIND DESIGN

Basic Wind Speed, V_{3s} :	115	mph
ASD Wind Speed, V_{asd} :	89	mph
Wind Exposure :	C	
K_e :	0.86	

SEISMIC DESIGN

USGS Design Code: ASCE 7-16
Site Class : D-default

Seismic Importance Factor, I_e : 1.00
Overall Structural Height : 20 ft
Structure Type : Steel Moment Frames
Approximate Period : 0.308 sec $1.5T_s$ = 0.821 sec

Structure meets exception 2 of 11.4.8

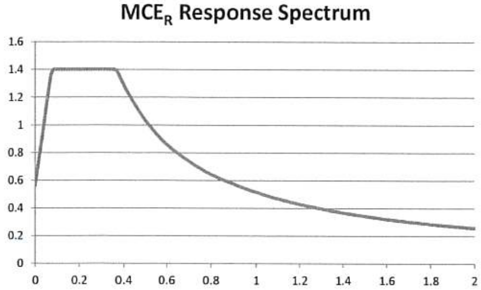
Design Category : D
Basic Seismic Force Resisting System :
Response Modification Factor, R :
Type of Analysis : STATIC
C_s : C_{s_max} : C_{s_min} :

☐ Project meets the requirements of 12.8.1.3

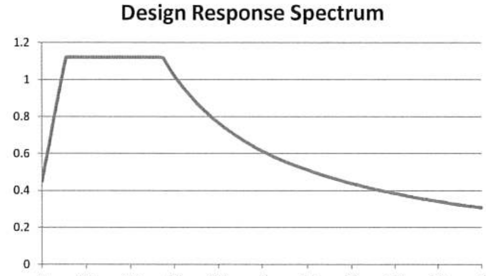
USGS-Provided Output

S_s = 1.401 g	F_a = 1.200	S_{MS} = 1.682 g	S_{DS} = 1.121 g
S_1 = 0.516 g	F_v = 1.784	S_{M1} = 0.921 g	S_{D1} = 0.614 g

MCE_R Response Spectrum



Design Response Spectrum



LATERAL ANALYSIS:

SEISMIC:

$S_{DS} = 1.121g$

$S_{D1} = 0.614g$

SITE CLASS: D - DEFAULT.

SDC: D

ALTHOUGH THE BRIDGE MEETS THE DEFINITION OF "BUILDING" AND COULD USE AN OMF SYSTEM FROM CH. 12, CALL IT A "NON-BUILDING" STRUCTURE SIMILAR TO BUILDINGS. USE "STEEL ORDINARY MOMENT FRAMES WITH UNLIMITED HEIGHT" (I.E. NO SEISMIC DETAILING) PER TABLE 15.4-1. THIS IS CONSERVATIVE.

$R = 1$

$\Omega_0 = 1$

$C_d = 1$

SECTION 15.4.1 REQUIREMENTS:

1) SYSTEM HAS BEEN SELECTED FROM TABLE 15.4-1
2) DOES NOT APPLY
3) $I_e = 1.0$
4) VERT. DISTRIBUTION DOES NOT MAKE SENSE. VERY LIMITED DYNAMIC INTERACTION BETWEEN LEVELS.
5) NO EXCEPTIONS APPLY ∴ NEED ACCIDENTAL TORSION
6) DOES NOT APPY
7) ✓
8) NOT USED / NEEDED
9) GRAVITY AND SEISMIC COMBOS ✓
10) USE OMEGA WHERE NEEDED. ✓

LATERAL ANALYSIS - CONT'D:

SEISMIC - CONT'D:

BY OBSERVATION, THE BRIDGE WILL HAVE $\rho = 1.3$ DUE TO SINGLE LFRS/ATTACHMENT EACH END.

CHECK FLOOR ASPECT RATIO: EVEN IF SUPPORTED FROM RETAIN. WALL TO STAIR TOWER WALL,

$$\text{ASPECT RATIO} = \frac{32'\text{-}0''}{7'\text{-}0''} = 4.6 \quad \therefore \text{ RIGID}$$

DIAPHRAGM ASSUMPTION IS NOT VALID SINCE > 3.

USE FLEXIBLE DIAPHRAGM AND APPLY LOADS TO RISA MODEL AS UNIFORM LOADS @ ROOF AND FLOOR.

PRELIMINARY TRUSS WEIGHT FROM RISA = 23.2k

FACTOR UP BY 15% FOR POTENTIAL SIZE CHANGES

$W_T = 1.15 \times 23k = 26.5k$

ROOF WT = 50'-0" x 6'-9" x 7 psf = 2.4k

FLOOR WT = 50'-0" x 6'-9" x 40 psf = 13.5k

WALL WT = 12'-0" x 50'-0" x 2 x 10 psf = 12 K (1/2 TO ROOF, 1/2 TO FLOOR)

TOTAL WT = 26.5k + 2.4k + 13.5k + 12k = 54.4k

ROOF WT ONLY = 21.7k

FLOOR WT ONLY = 32.8k

$$C_s = \frac{S_{DS}}{R/I_e} = \frac{1.121\,g}{1/1.0} = 1.121g$$

TOTAL BASE SHEAR = 61 K

ROOF BASE SHEAR = 24.3k

FLOOR BASE SHEAR = 36.8k

LATERAL ANALYSIS - CONT'D:

WIND:

BASIC WIND SPEED = 115 mph
WIND EXPOSURE = C
ELEVATION FACTOR, K_e = 0.86
DIRECTIONALITY FACTOR, K_d = 0.85
TOPOGRAPHIC FACTOR, K_{zt} = 1.0
EXPOSURE COEFFICIENT, K_z = 0.98

NOTE: WIND PARALLEL TO BRIDGE SPAN WILL NOT GOVERN WIND DESIGN BY INSPECTION

$$q_z = 0.00256\, K_z K_{zt} K_d K_e V^2$$
$$= 0.00256 \times 0.98 \times 1.0 \times 0.85 \times 0.86 \times (115\text{ mph})^2$$
$$= 24.3 \text{ psf}$$

GUST EFFECT FACTOR, G = 0.85
INTERNAL PRESSURE COEFFICIENT, GC_{pi} = +/- 0.18

TREAT LATERAL LOAD TO THE STRUCTURE AS A "SOLID FREESTANDING WALL", SEE SECTION 29.3.1 OF ASCE 7-16.

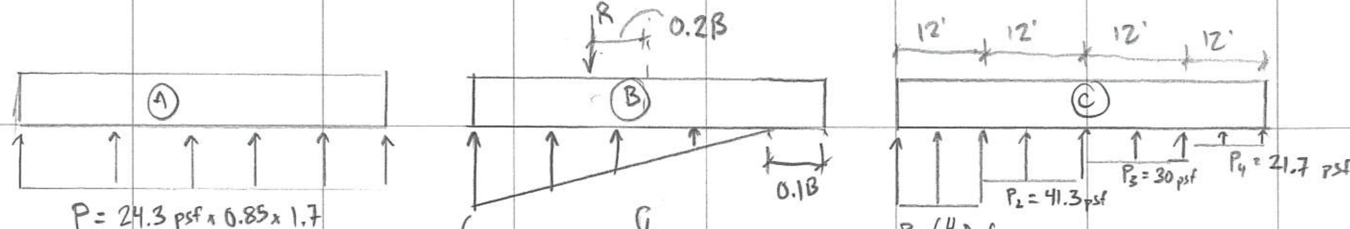
h = 24'-0"
s = 12'-0"
B = 50'-0"

CASE A & B:

B/s = 4.2
s/h = 0.5
C_f = 1.70

CASE C:

0 to s = 0 to 12'-0" C_f = 3.10
s to 2s = 12' to 24' C_f = 2.00
2s to 3s = 24' to 36' C_f = 1.45
3s to 4s = 36' to END C_f = 1.05



P = 24.3 psf × 0.85 × 1.7
= 35.2 psf

P = 2.22 × 35.2 psf = 78.2 psf

V = 21 k

V = 23 k

SEISMIC GOVERNS...

18 ga PLB™-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck

3/4" Visible Dia. Arc Spot Weld Connections to Supports
36 / 7 Perpendicular Connection Pattern to Supports
PunchLok II Connection (VSC2) Sidelap Connections

A572 GR50 Support Member or Equivalent
0.188 ≤ Support Thickness (in.)
3 in. Minimum Deck End Bearing Length

Note: Support welds at interlocking sidelaps may be 3/8" x 1 1/4" arc seam welds in lieu of arc spot welds.

ASD Allowable Seismic Diaphragm Shear Strength Sn/Ω (plf) — 1 Span Condition

Sidelap Connection Spacing (in.)	Span								
	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
4	3133	3092	3063	3042	2388	1886	1528	1263	1061
6	2802	2729	2677	2637	2388	1886	1528	1263	1061
8	2546	2511	2376	2373	2280	1886	1528	1263	1061
12	2199	2066	1971	1899	1843	1799	1528	1263	1061
18	1983	1880	1631	1600	1577	1429	1427	1263	1061
24	1736	1674	1440	1435	1269	1286	1149	1178	1061
36	1736	1446	1215	1246	1079	949	997	902	827

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

Sidelap Connection Spacing (in.)	Span								
	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
4	4	4	4	4	4	4	4	4	4
6	6	6	6	6	6	6	6	6	6
8	8	8	8	8	8	8	8	8	8
12	12	12	12	12	12	12	12	12	12
18	16	18	18	17	18	18	18	18	18
24	24	20	24	21	24	22	24	22	24
36	24	20	24	21	24	27	24	26	29

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.) — 1 Span Condition

Sidelap Connection Spacing (in.)	Span								
	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
4	326	324	323	322	322	321	321	321	320
6	306	303	301	300	299	298	297	296	296
8	290	290	283	284	280	281	277	279	276
12	267	261	257	254	251	249	248	246	245
18	251	247	230	231	231	219	221	222	213
24	230	230	213	216	203	206	196	200	191
36	230	209	191	198	185	174	181	172	164

18 ga PLB™-36 Grade 50 Roof Deck

Wind Diaphragm Shear

For Both Ends Lapped Deck

VERCO DECKING, INC.
A NUCOR COMPANY

3/4" Visible Dia. Arc Spot Weld Connections to Supports
36 / 7 Perpendicular Connection Pattern to Supports
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0.188 ≤ Support Thickness (in.)
3 in. Minimum Deck End Bearing Length

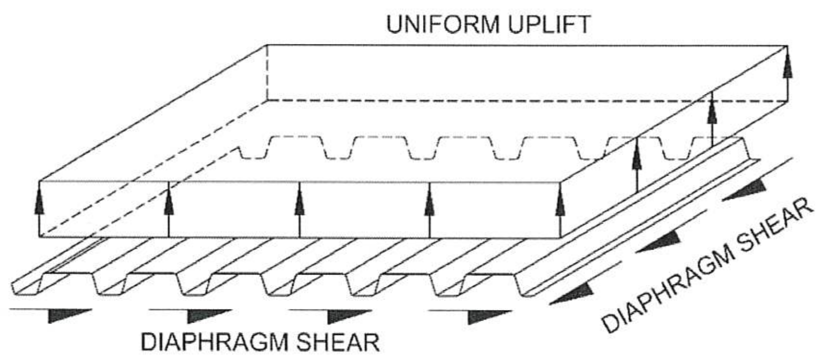
Note: Support welds at interlocking sidelaps may be 3/8" x 1 1/4" arc seam welds in lieu of arc spot welds.

ASD Allowable Wind Diaphragm Shear Strength Sn/Ω (plf) — 1 Span Condition

Sidelap Connection Spacing (in.)	Span								
	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
4	3933	3933	3933	3118	2388	1886	1528	1263	1061
6	3910	3808	3735	3118	2388	1886	1528	1263	1061
8	3552	3503	3315	3118	2388	1886	1528	1263	1061
12	3068	2883	2750	2650	2388	1886	1528	1263	1061
18	2767	2623	2276	2233	2201	1886	1528	1263	1061
24	2422	2335	2009	2002	1770	1795	1528	1263	1061
36	2422	2018	1696	1738	1505	1324	1391	1258	1061

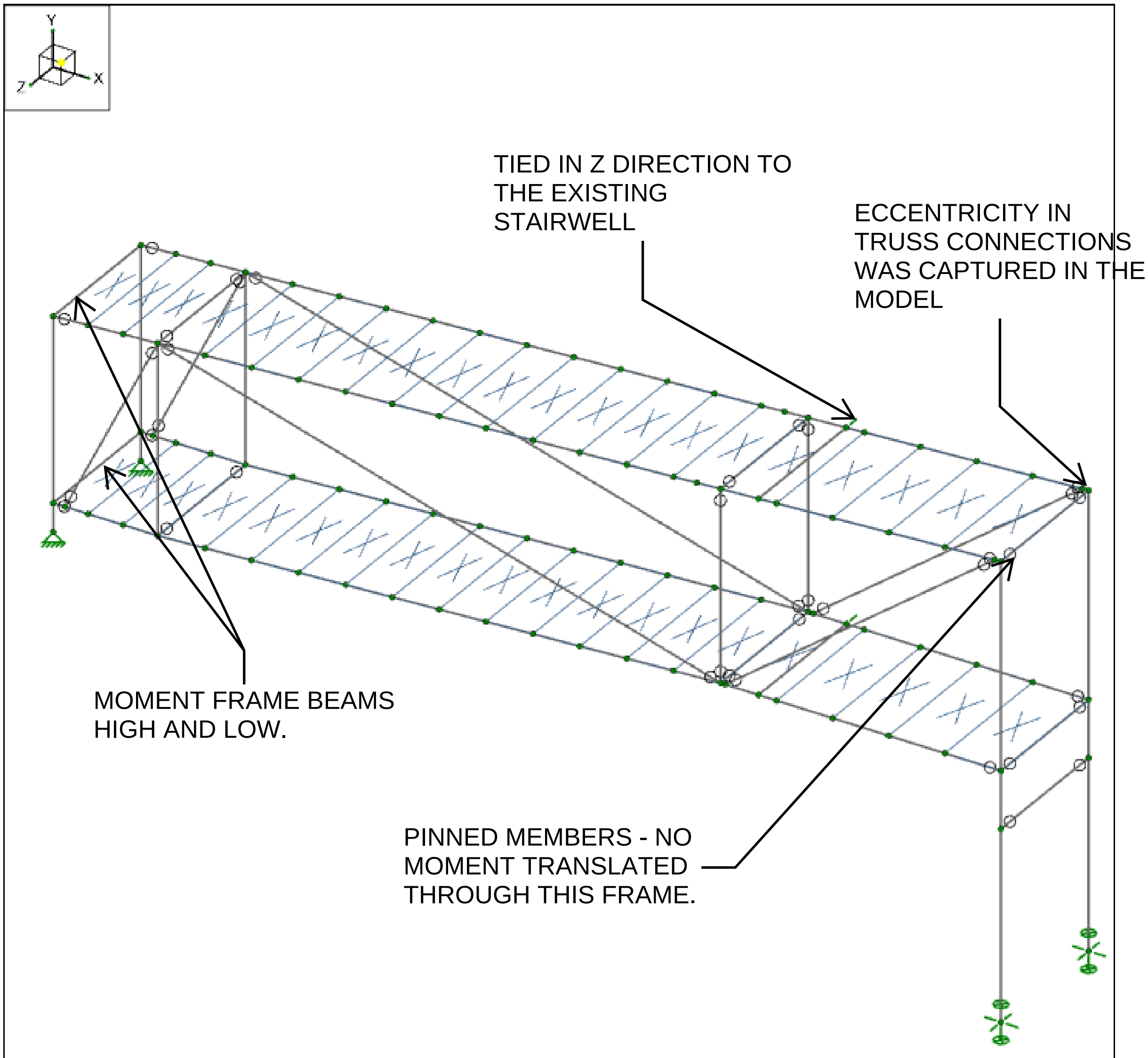
Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connection Spacing (in.)	Span								
	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
4	4	4	4	4	4	4	4	4	4
6	6	6	6	6	6	6	6	6	6
8	8	8	8	8	8	8	8	8	8
12	12	12	12	12	12	12	12	12	12
18	16	18	18	17	18	18	18	18	18
24	24	20	24	21	24	22	24	24	24
36	24	20	24	21	24	27	24	26	29

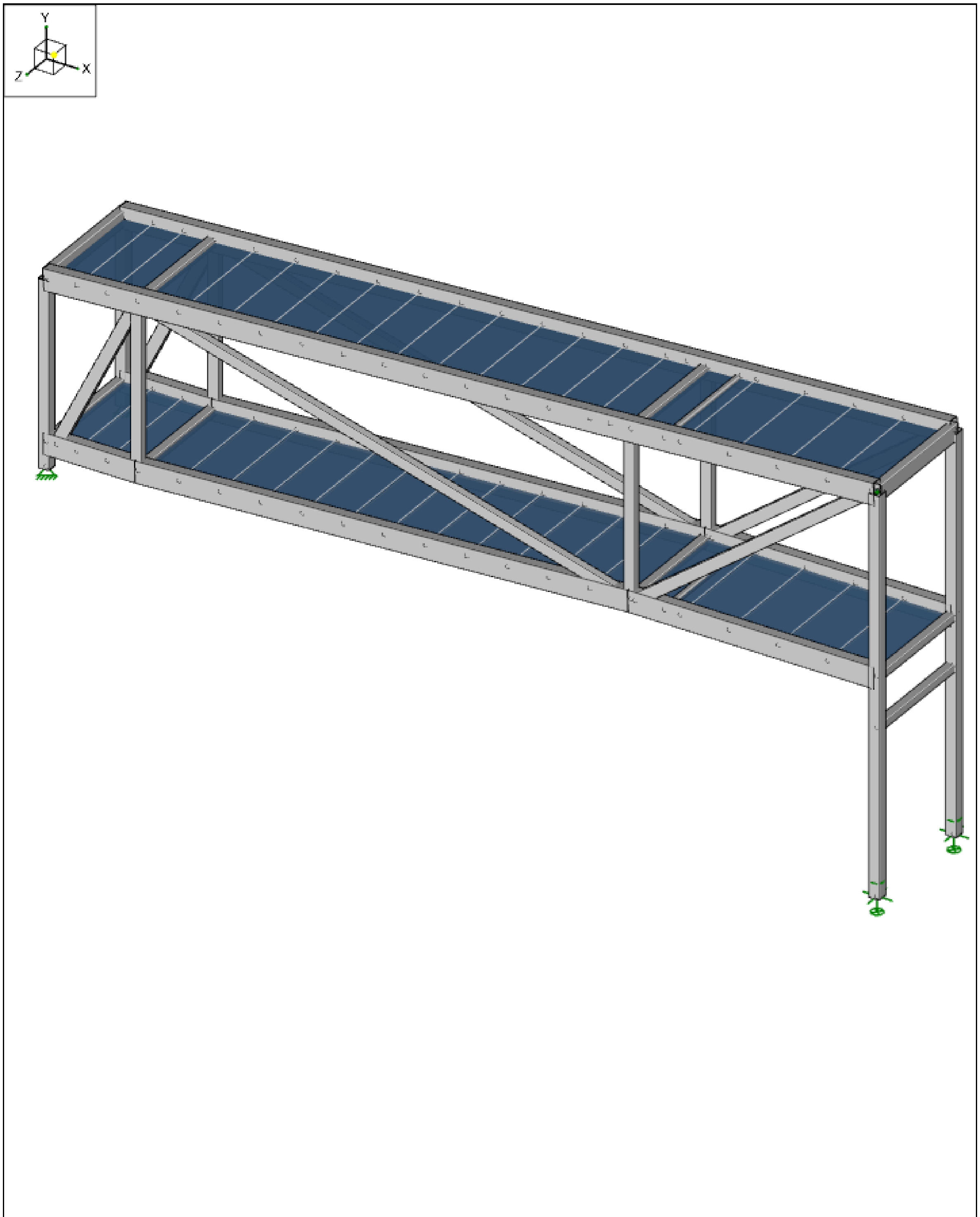


V1.0.4 of calculator based on AISI S100-16 (2020) w/S2-20 & AISI S310-16 as modified by IAPMO ER-2018 or ER-0432

Date: 2/24/2025



IIIRISA	<Licensed Company>		SK-1
	jaredm		Apr 10, 2025 at 05:18 PM
			OWTC Bridge Replacement -...



RISA	<Licensed Company>		SK-2
	jaredm		Apr 10, 2025 at 05:18 PM
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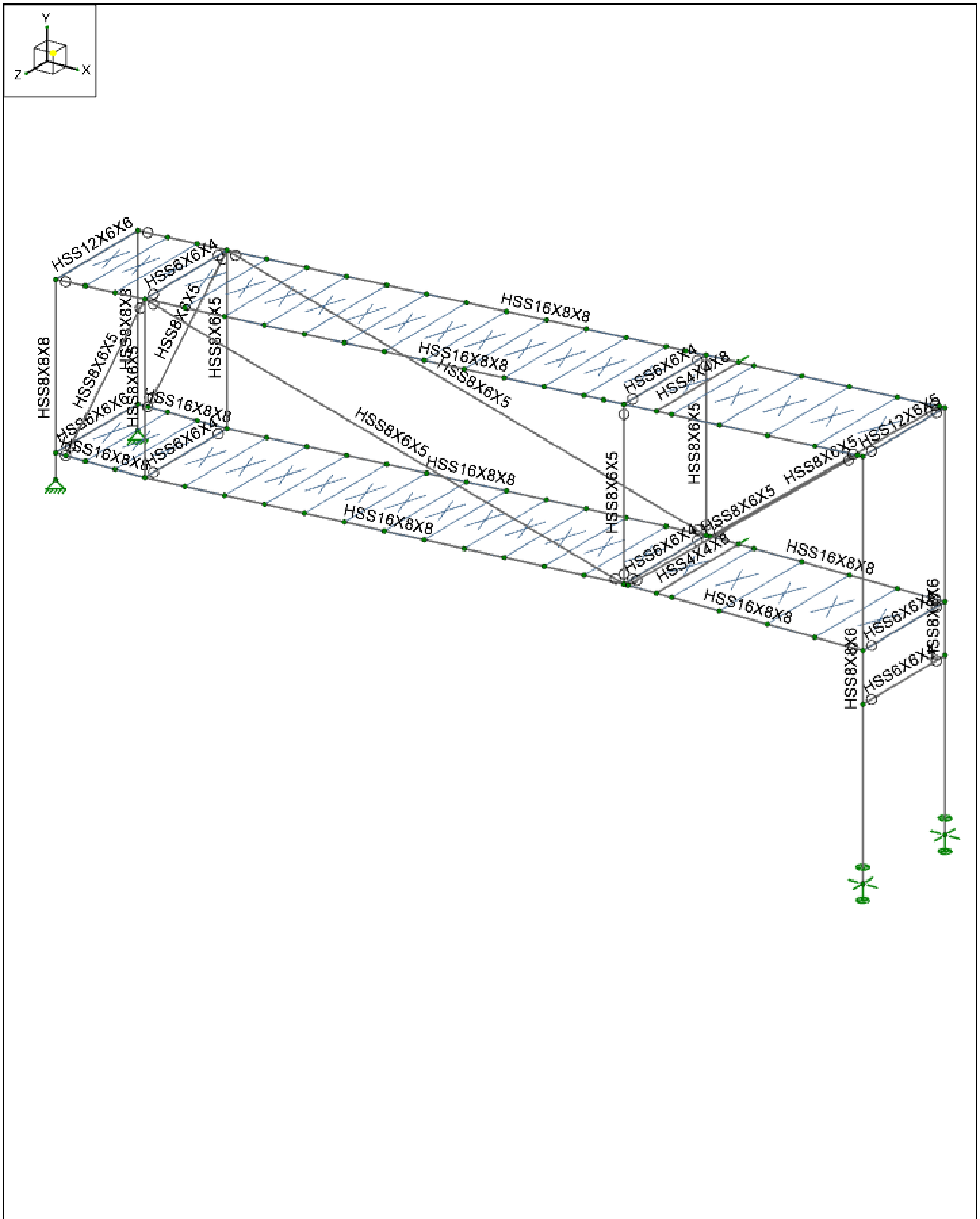

	<Licensed Company>		SK-3
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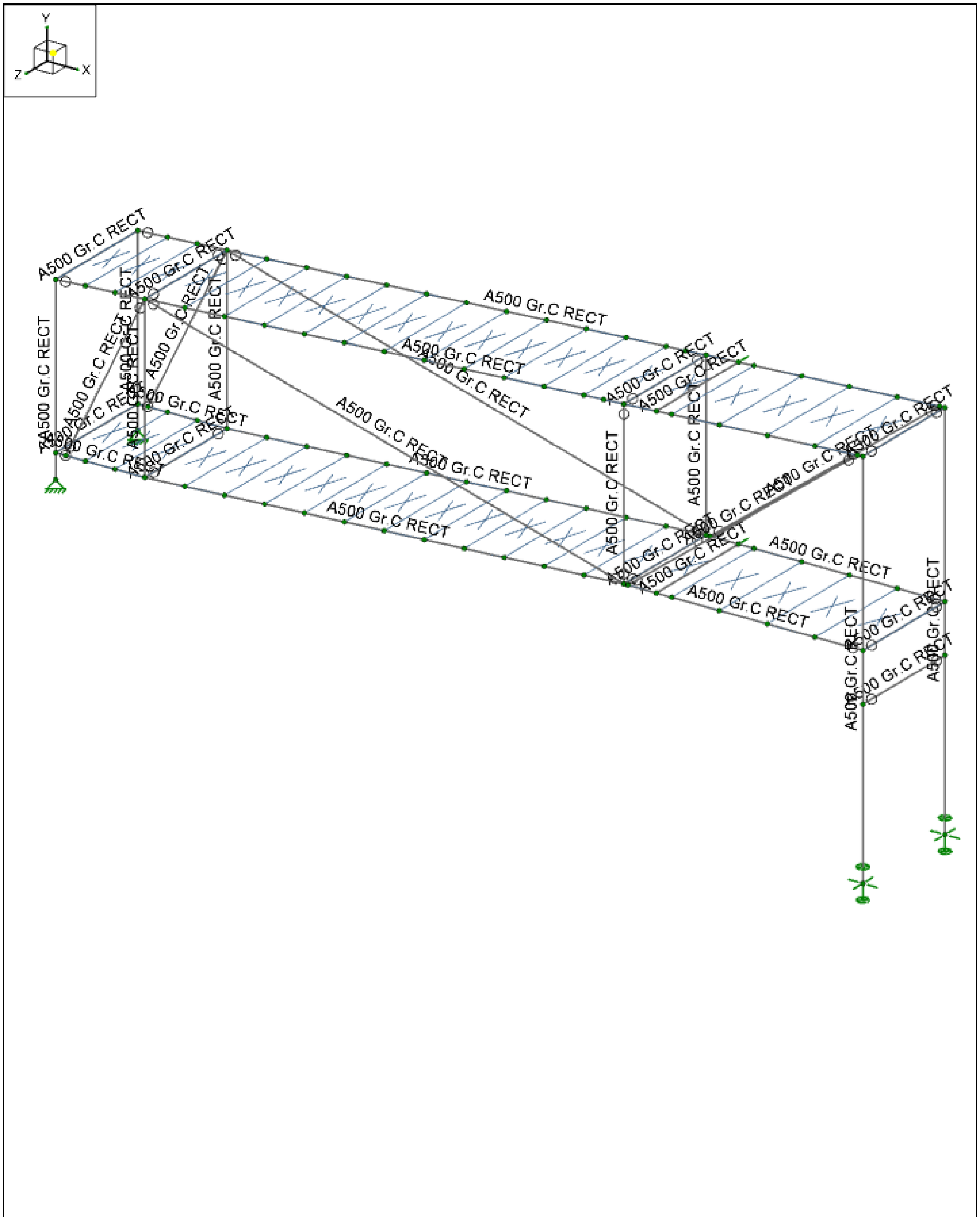
			OWTC Bridge Replacement -...



RISA	<Licensed Company>		SK-4
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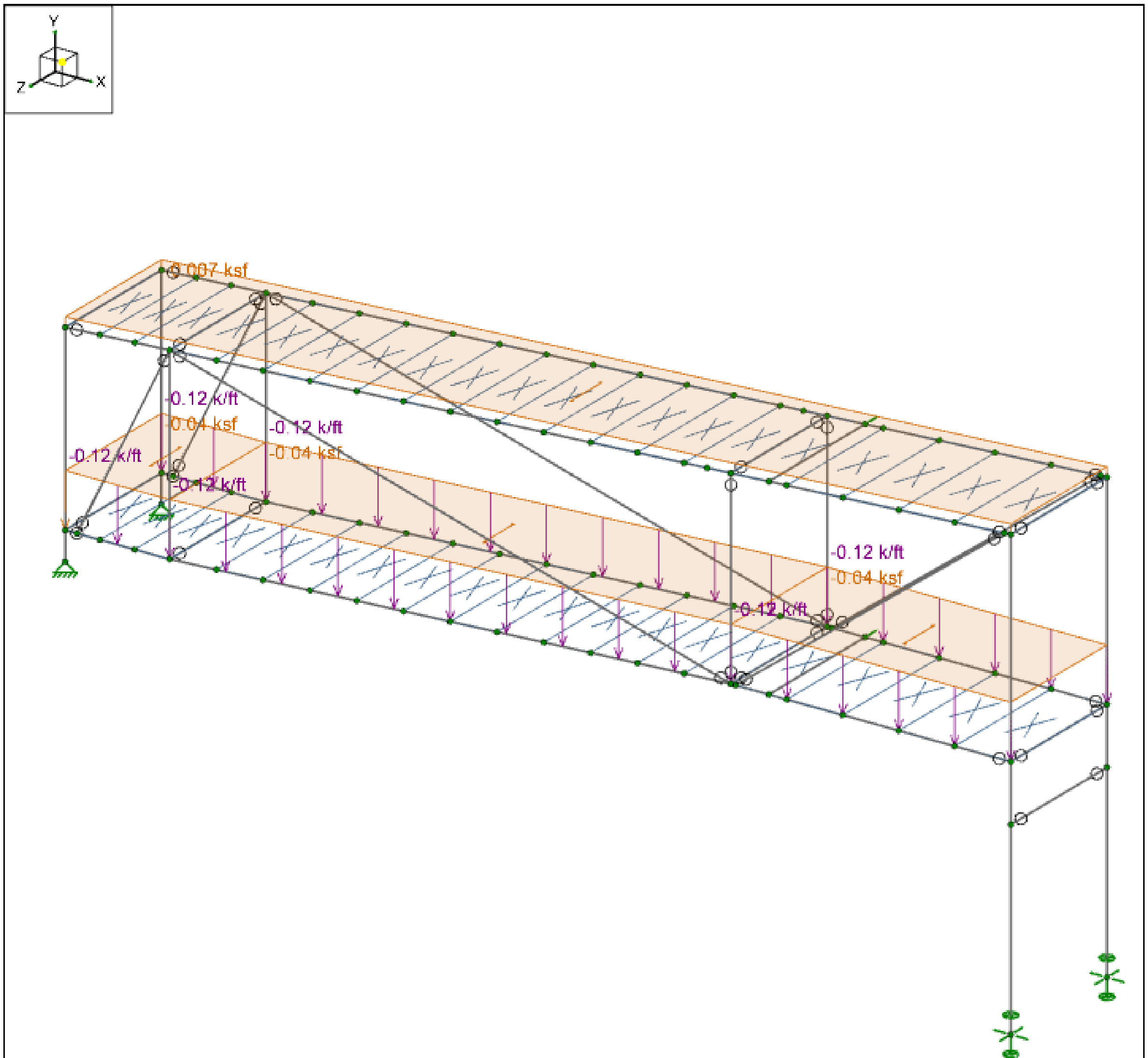
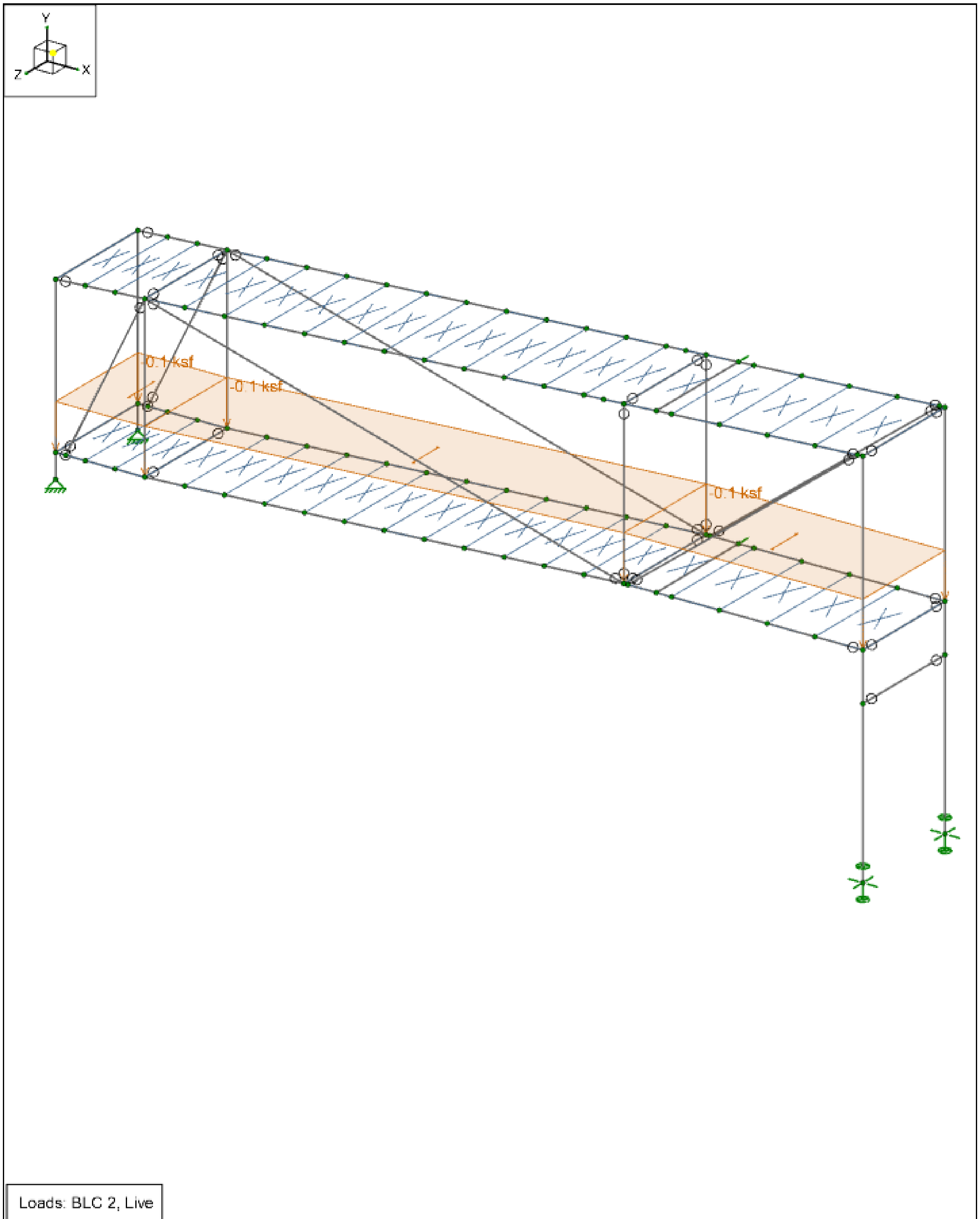
			OWTC Bridge Replacement -...



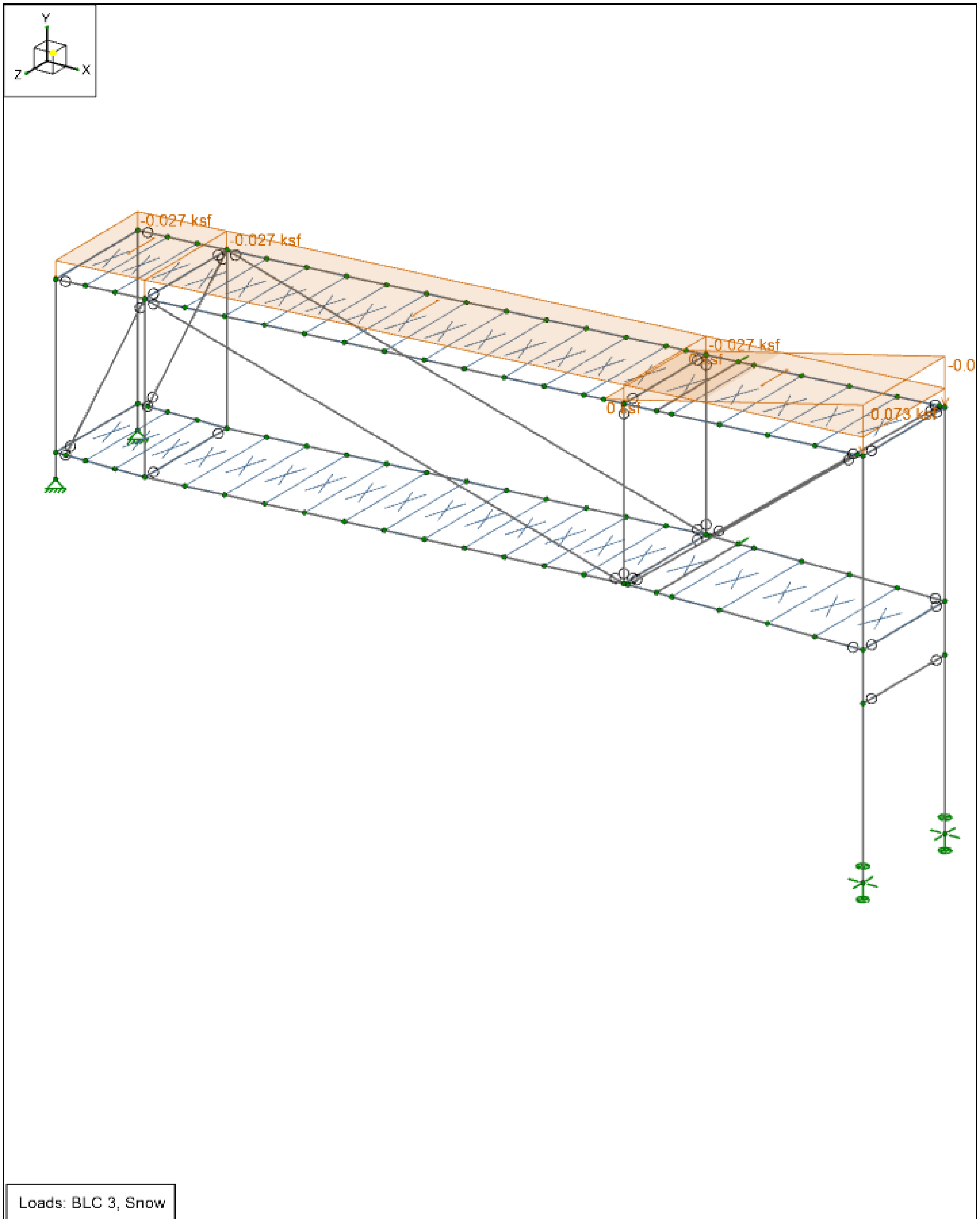
SEE WHICH LOAD CASE
IS DISPLAYED IN THIS
CORNER. TYP EACH
SHEET

Loads: LC 1, D Only

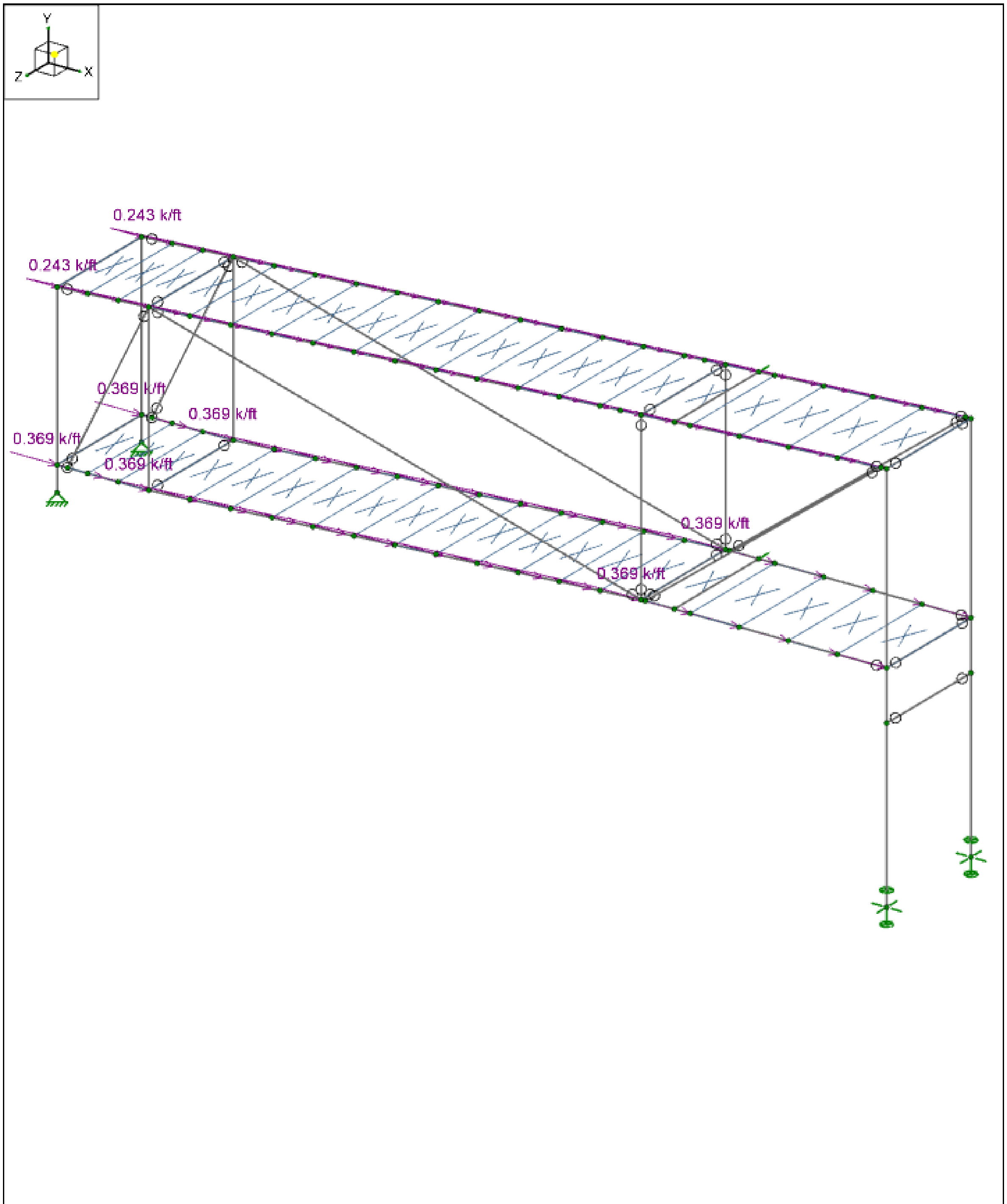
RISA	<Licensed Company>		SK-5
	jaredm		Apr 10, 2025 at 05:19 PM
			OWTC Bridge Replacement -...



<Licensed Company>			SK-6
jaredm			Apr 10, 2025 at 05:20 PM
			OWTC Bridge Replacement -...



RISA	<Licensed Company>		SK-7
	jaredm		Apr 10, 2025 at 05:20 PM
			OWTC Bridge Replacement -...



IIRISA	<Licensed Company>		SK-8
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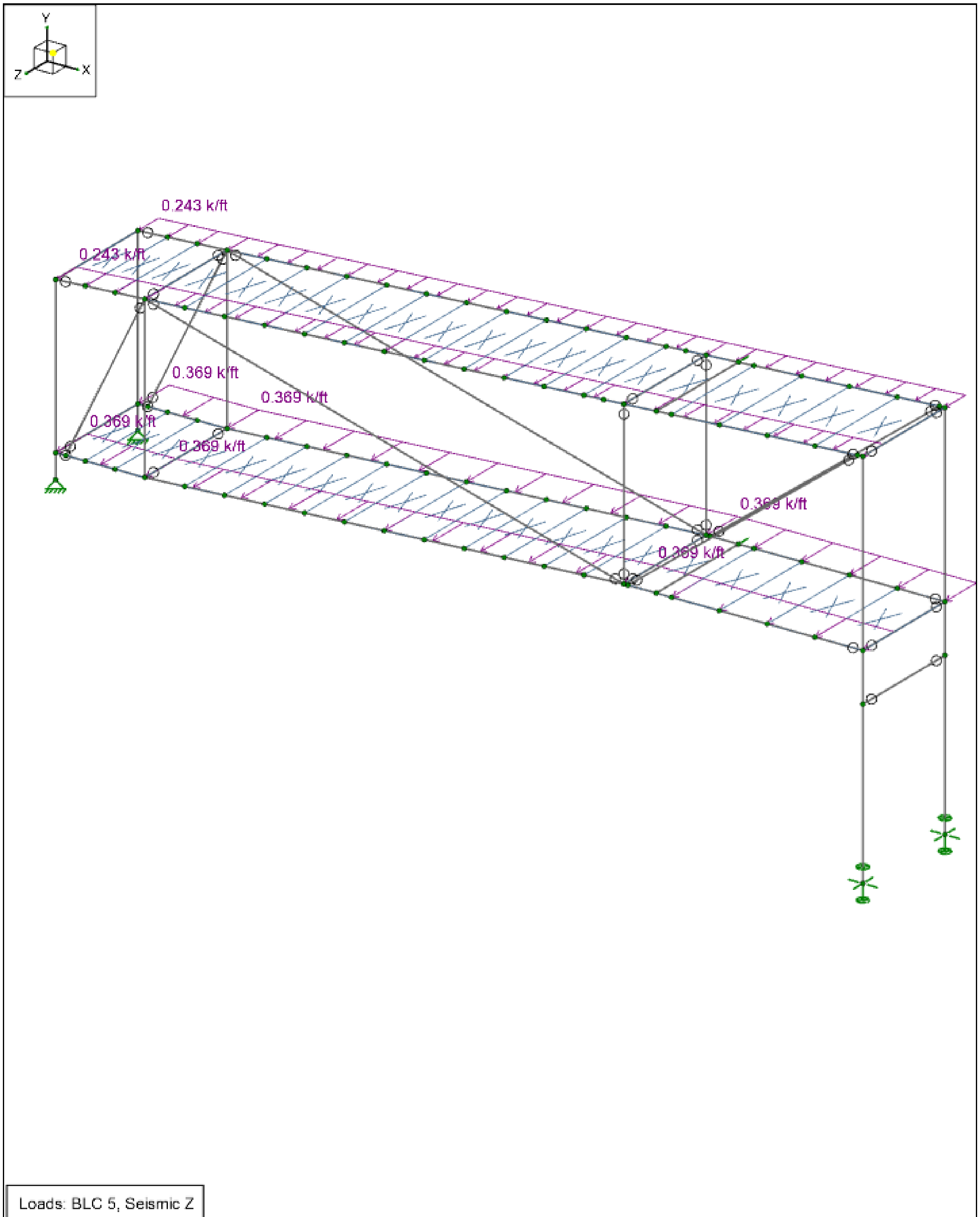
			OWTC Bridge Replacement -...



IIRISA	<Licensed Company>		SK-9
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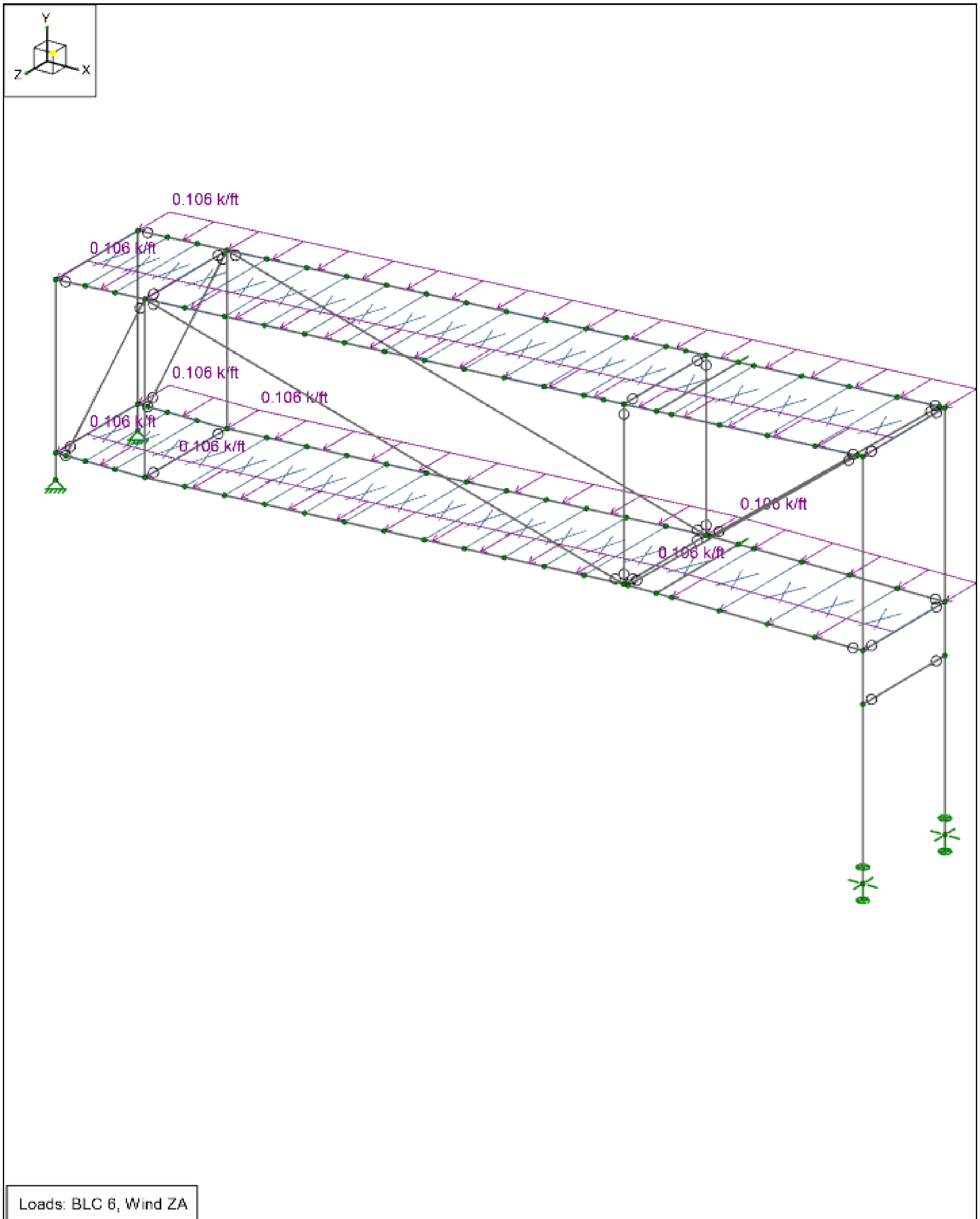
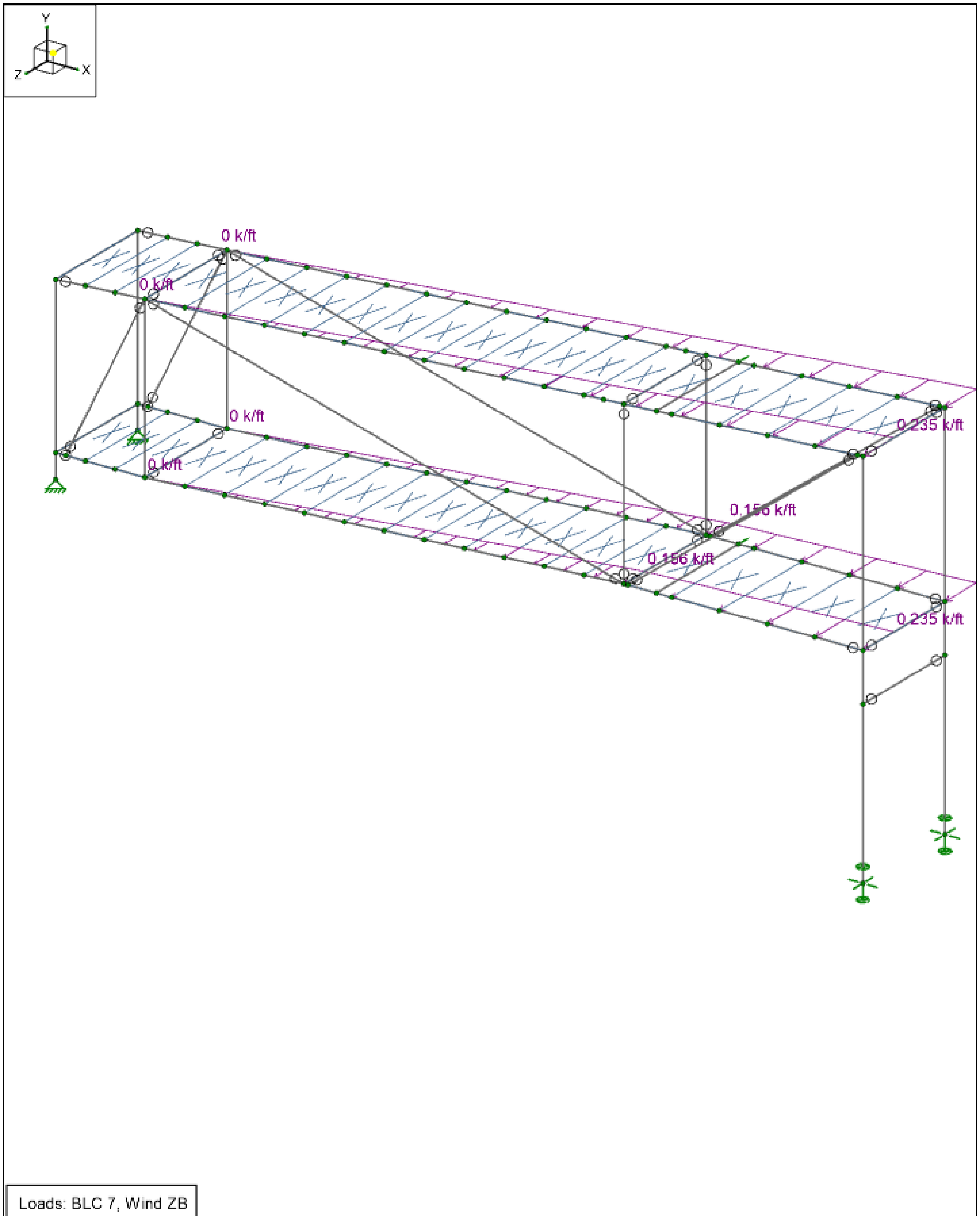


IIRISA	<Licensed Company>		SK-10
	jaredm		Apr 10, 2025 at 05:20 PM
			OWTC Bridge Replacement -...



RISA	<Licensed Company>		SK-11
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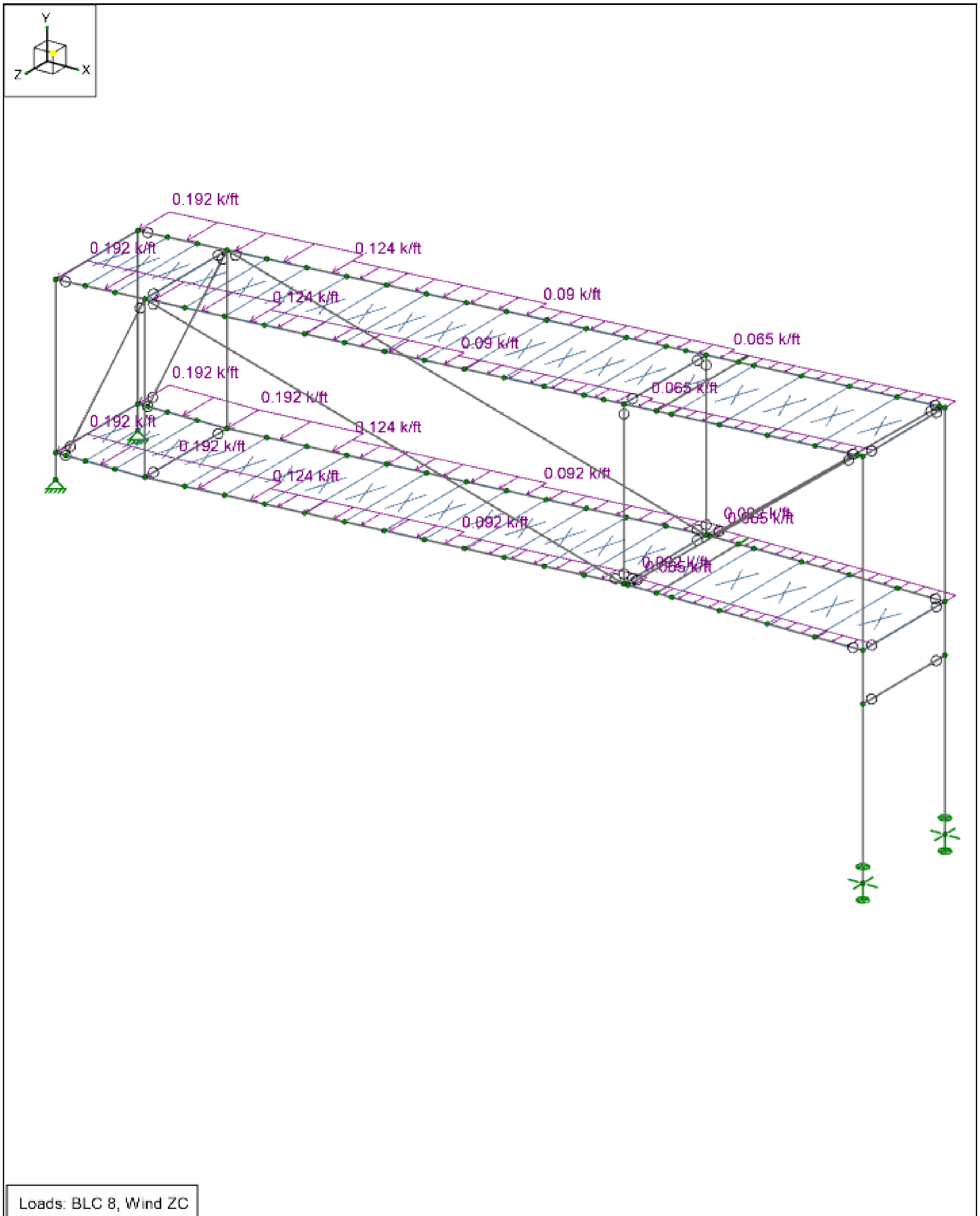
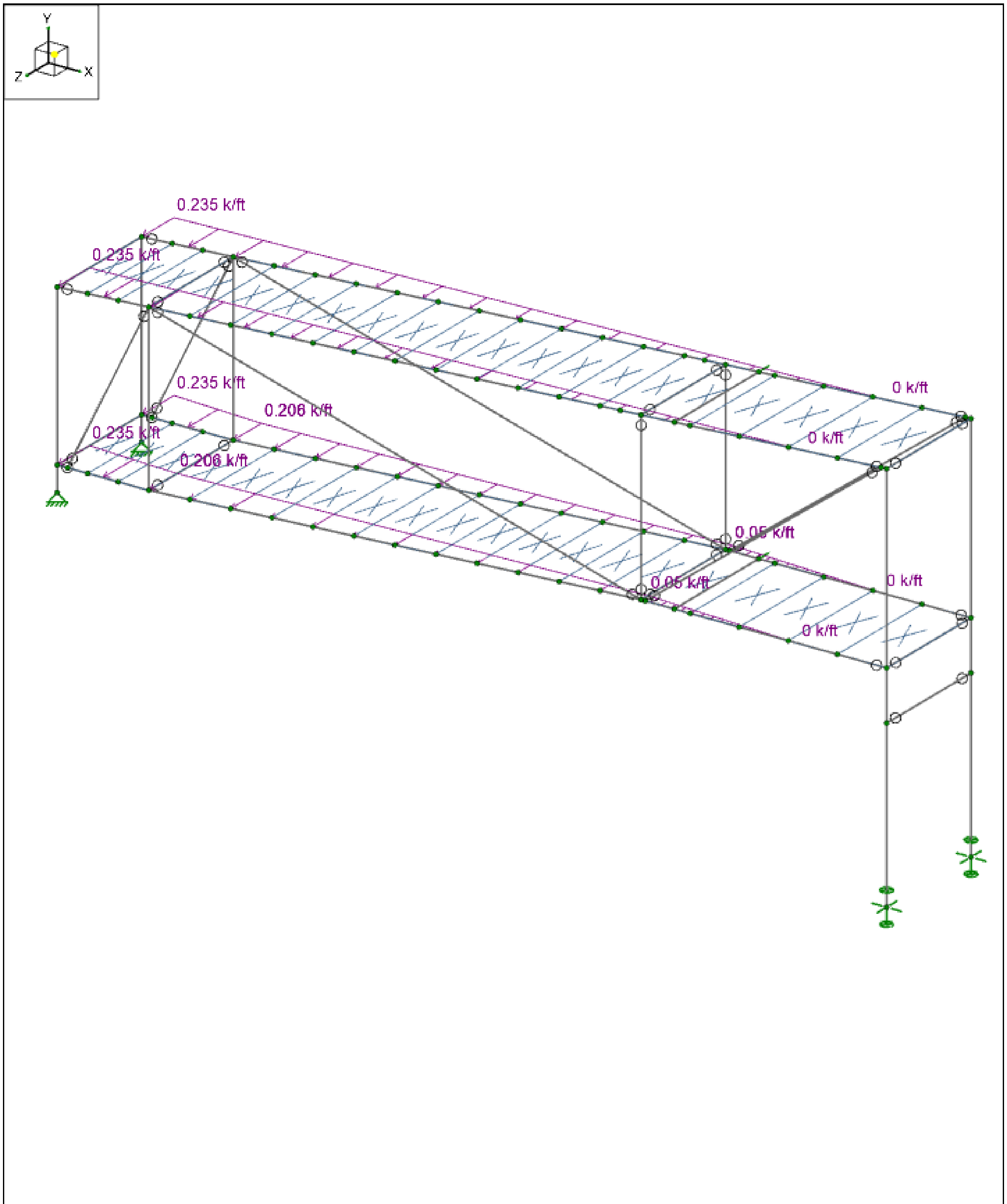
			OWTC Bridge Replacement -...



RISA	<Licensed Company>		SK-12
	jaredm		Apr 10, 2025 at 05:20 PM
			OWTC Bridge Replacement -...



	<Licensed Company>		SK-13
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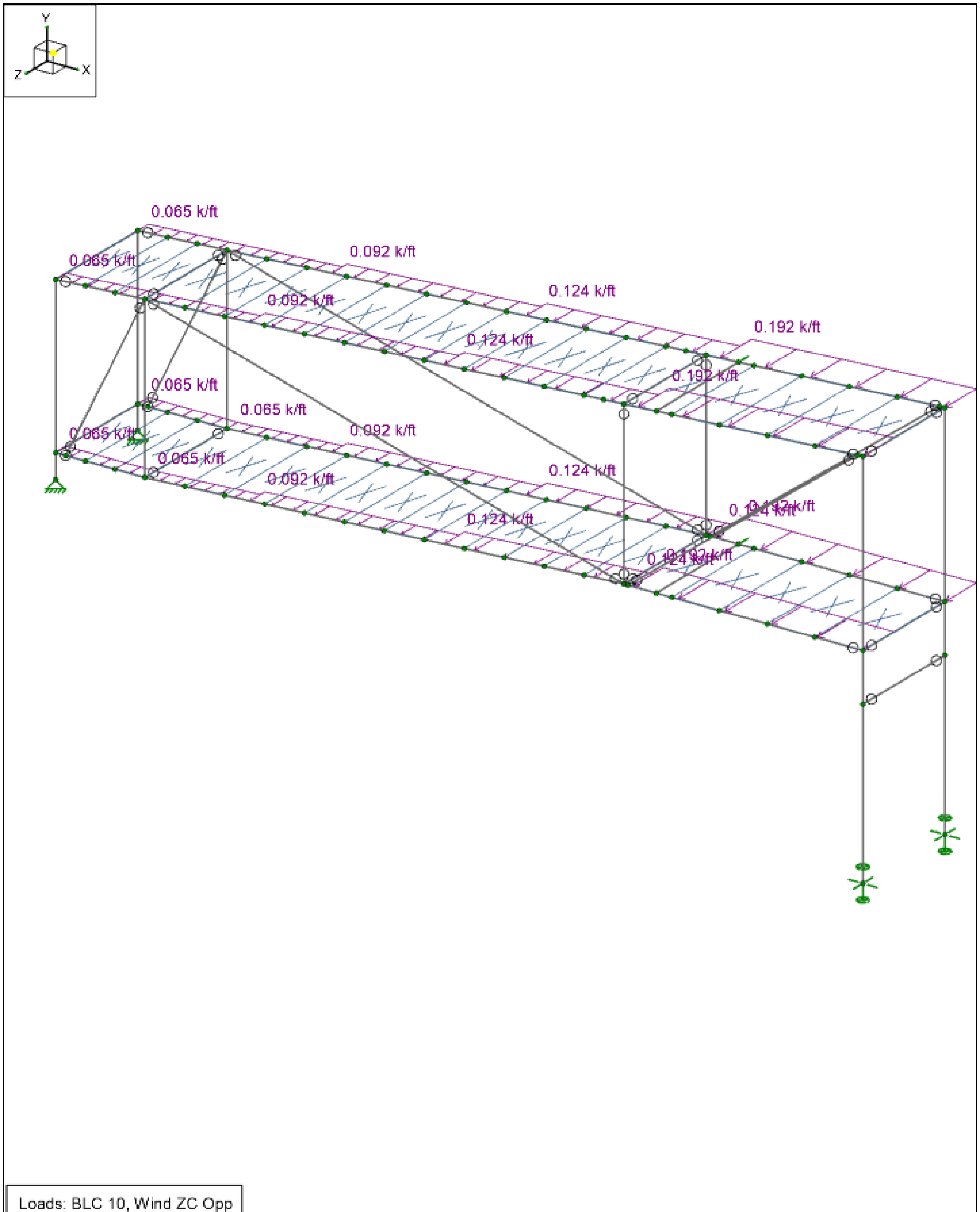
			OWTC Bridge Replacement -...



RISA	<Licensed Company>		SK-14
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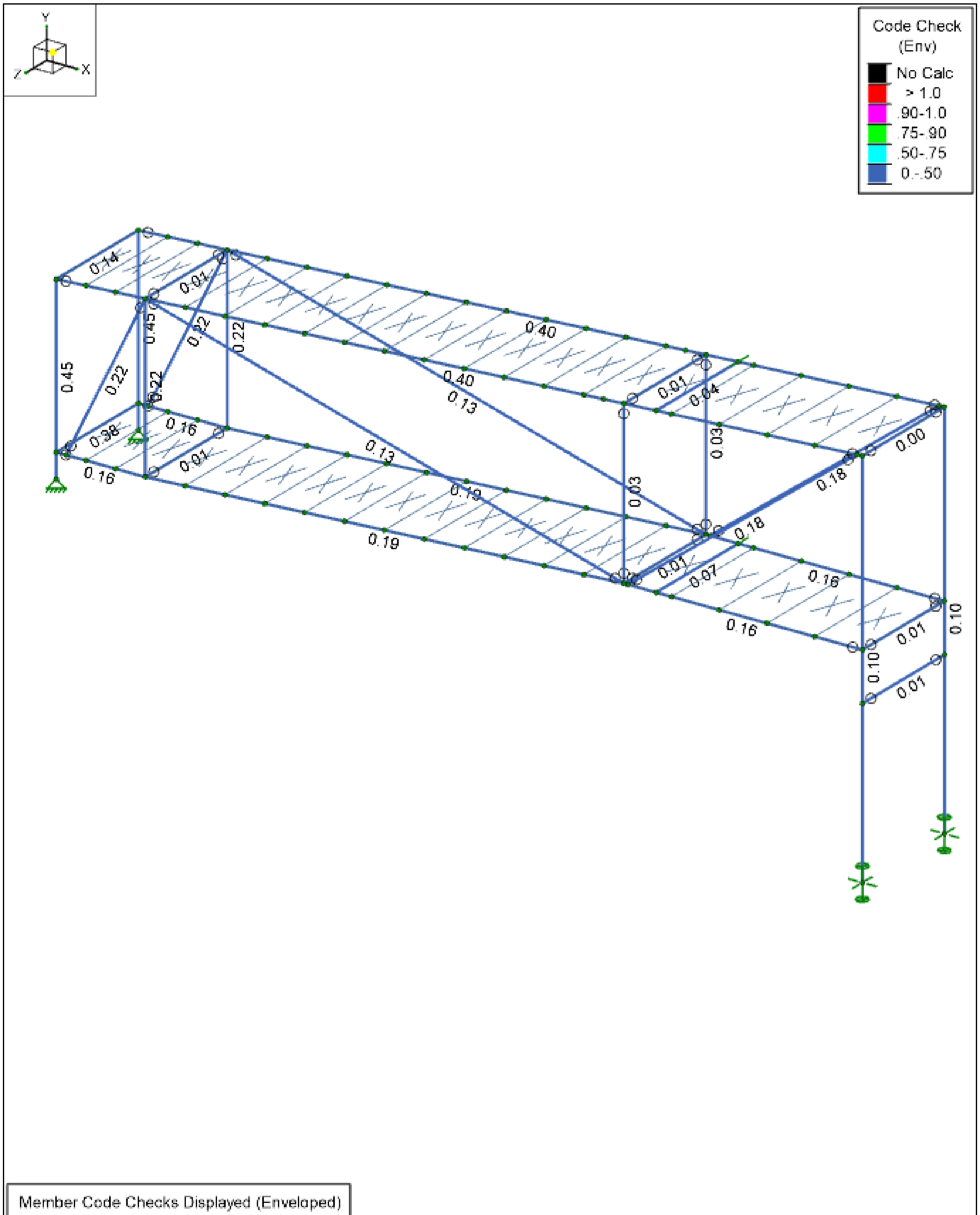
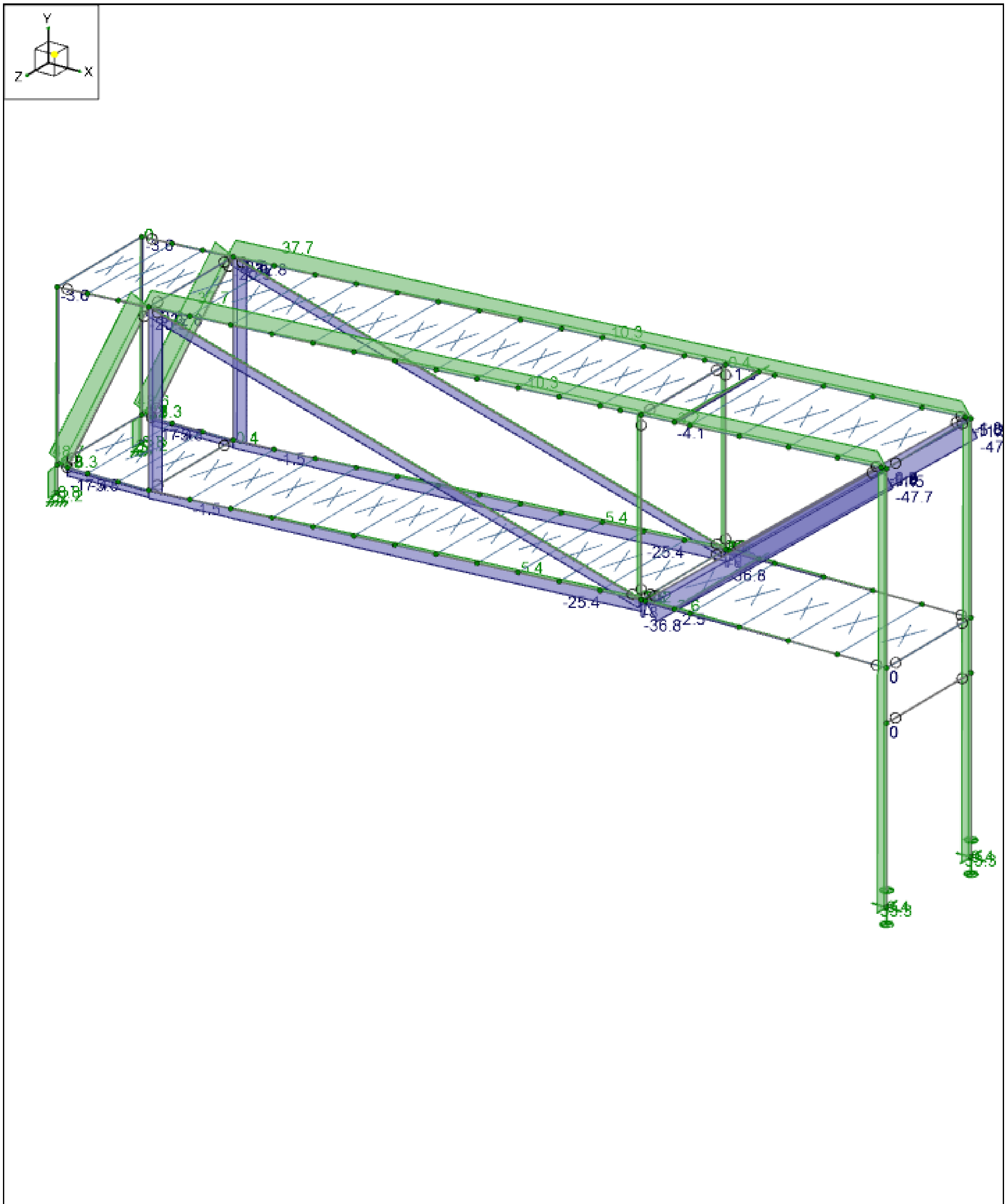
			OWTC Bridge Replacement -...

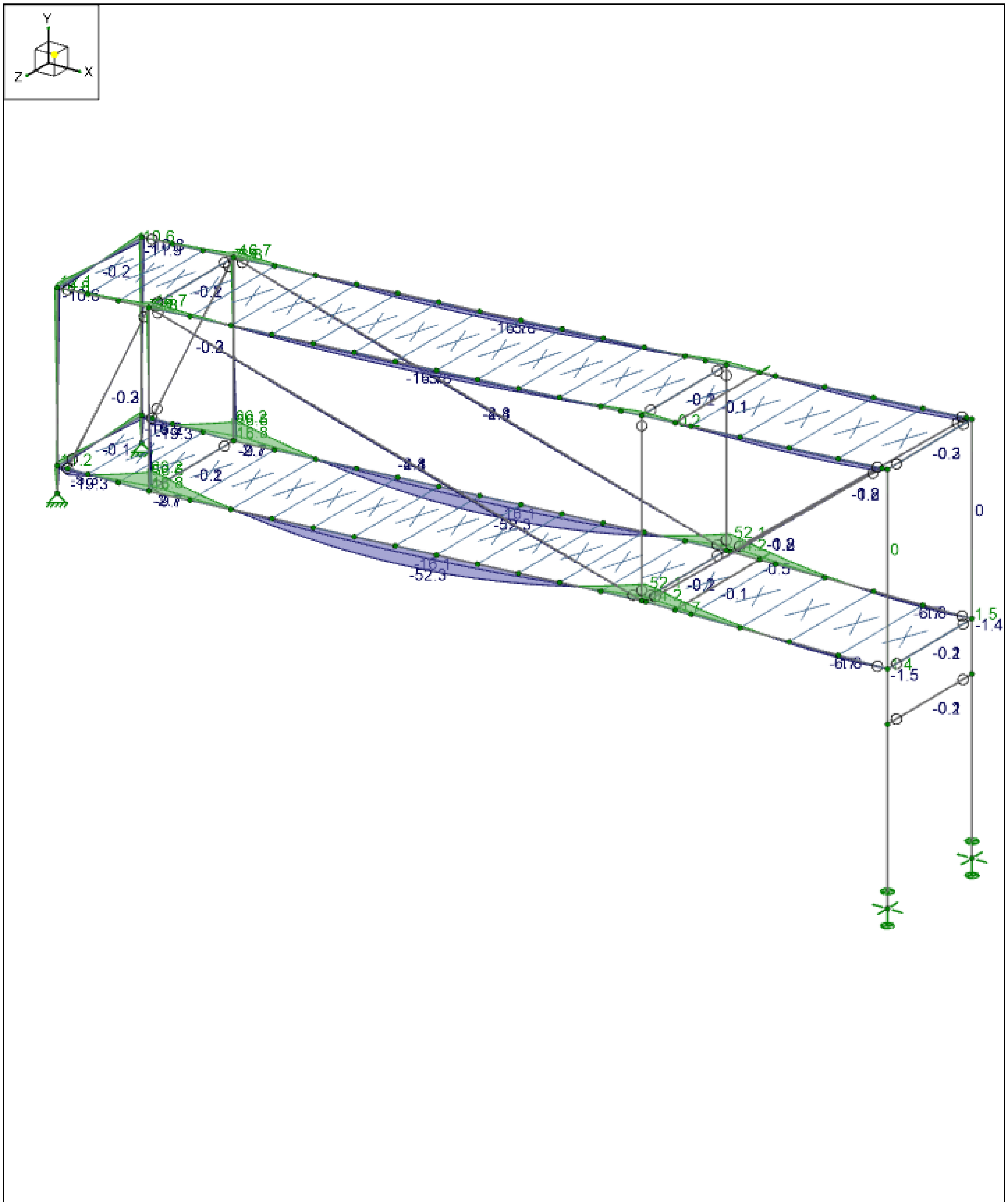


<Licensed Company>		SK-18
jaredm		Apr 10, 2025 at 05:25 PM
		OWTC Bridge Replacemen...



Member Axial Forces (kips) (Enveloped)

RISA	<Licensed Company>		SK-16
	jaredm		Apr 10, 2025 at 05:22 PM
			OWTC Bridge Replacement -...



Member z Bending Moments (kip-ft) (Enveloped)

RISA	<Licensed Company>		SK-17
	jaredm		Apr 10, 2025 at 05:22 PM
			OWTC Bridge Replacement -...

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	Y Rot [k-ft/rad]
1	N100			Reaction	
2	N97			Reaction	
3	N17	Reaction	Reaction	Reaction	Reaction
4	N18	Reaction	Reaction	Reaction	Reaction
5	N95	Reaction	Reaction	Reaction	
6	N96	Reaction	Reaction	Reaction	

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	Yield [ksi]	Ry	Fu [ksi]	Rt
1	A992	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
2	A36 Gr.36	29000	11154	0.3	0.65	0.49	36	1.5	58	1.2
3	A572 Gr.50	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
4	A500 Gr.B RND	29000	11154	0.3	0.65	0.527	42	1.4	58	1.3
5	A500 Gr.B RECT	29000	11154	0.3	0.65	0.527	46	1.4	58	1.3
6	A500 Gr.C RND	29000	11154	0.3	0.65	0.527	46	1.4	62	1.3
7	A500 Gr.C RECT	29000	11154	0.3	0.65	0.527	50	1.4	62	1.3
8	A53 Gr.B	29000	11154	0.3	0.65	0.49	35	1.6	60	1.2
9	A1085	29000	11154	0.3	0.65	0.49	50	1.4	65	1.3
10	A913 Gr.65	29000	11154	0.3	0.65	0.49	65	1.1	80	1.1

General Materials Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	Plate Methodology
1	gen_Conc3NW	3155	1372	0.15	0.6	0.145	Isotropic
2	gen_Conc4NW	3644	1584	0.15	0.6	0.145	Isotropic
3	gen_Conc3LW	2085	906	0.15	0.6	0.11	Isotropic
4	gen_Conc4LW	2408	1047	0.15	0.6	0.11	Isotropic
5	gen_Alum	10100	4077	0.3	1.29	0.173	Isotropic
6	gen_Steel	29000	11154	0.3	0.65	0.49	Isotropic
7	Weightless Steel	29000	11154	0.3	0.65	0	Isotropic
8	Weightless Concrete	2085	906	0.15	0.6	0	Isotropic
9	gen_Plywood	1800	38	0	0.3	0.035	Isotropic
10	RIGID	1e+6		0.3	0	0	Isotropic
11	gen_Ortho				0.65	0.49	Orthotropic

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Chord	HSS16X8X8	Beam	Tube	A500 Gr.C RECT	Typical	20.9	230	679	563
2	Web	HSS8X6X5	VBrace	Tube	A500 Gr.C RECT	Typical	7.59	43.8	68.3	85.8
3	Verticals	HSS8X6X5	Column	Tube	A500 Gr.C RECT	Typical	7.59	43.8	68.3	85.8
4	Moment Frame Beams	HSS6X6X6	Column	Tube	A500 Gr.C RECT	Typical	7.58	39.5	39.5	64.6
5	Moment Frame Columns	HSS8X8X8	Column	Tube	A500 Gr.C RECT	Typical	13.5	125	125	204
6	Cross Strut	HSS6X6X4	VBrace	Tube	A500 Gr.C RECT	Typical	5.24	28.6	28.6	45.6
7	Top Beam	HSS12X6X5	VBrace	Tube	A500 Gr.C RECT	Typical	9.92	62.8	184	152
8	Middle Beams	HSS6X6X4	VBrace	Tube	A500 Gr.C RECT	Typical	5.24	28.6	28.6	45.6
9	Tall Columns	HSS8X8X6	VBrace	Tube	A500 Gr.C RECT	Typical	10.4	100	100	160
10	Drag Strut	HSS4X4X8	VBrace	Tube	A500 Gr.C RECT	Typical	6.02	11.9	11.9	21
11	Upper Moment Frame Beam	HSS12X6X6	VBrace	Tube	A500 Gr.C RECT	Typical	11.8	72.9	215	178

Member Primary Data

	Label	I Node	J Node	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N10	N12		Chord	Beam	Tube	A500 Gr.C RECT	Typical
2	M2	N9	N11		Chord	Beam	Tube	A500 Gr.C RECT	Typical
3	M3	N2	N8		Chord	Beam	Tube	A500 Gr.C RECT	Typical
4	M4	N1	N7		Chord	Beam	Tube	A500 Gr.C RECT	Typical
5	M5	N7	N5		Chord	Beam	Tube	A500 Gr.C RECT	Typical
6	M6	N5	N3		Chord	Beam	Tube	A500 Gr.C RECT	Typical
7	M7	N8	N6		Chord	Beam	Tube	A500 Gr.C RECT	Typical
8	M8	N6	N4		Chord	Beam	Tube	A500 Gr.C RECT	Typical
9	M9	N6	N13		Verticals	Column	Tube	A500 Gr.C RECT	Typical
10	M10	N5	N14		Verticals	Column	Tube	A500 Gr.C RECT	Typical
11	M11	N8	N15		Verticals	Column	Tube	A500 Gr.C RECT	Typical
12	M12	N7	N16		Verticals	Column	Tube	A500 Gr.C RECT	Typical
13	M13	N96	N10	90	Moment Frame Columns	Column	Tube	A500 Gr.C RECT	Typical
14	M14	N95	N9	90	Moment Frame Columns	Column	Tube	A500 Gr.C RECT	Typical
15	M17	N15	N6		Web	VBrace	Tube	A500 Gr.C RECT	Typical
16	M18	N103	N92		Web	VBrace	Tube	A500 Gr.C RECT	Typical
17	M19	N16	N5		Web	VBrace	Tube	A500 Gr.C RECT	Typical
18	M20	N104	N91		Web	VBrace	Tube	A500 Gr.C RECT	Typical
19	M21	N18	N12	90	Tall Columns	VBrace	Tube	A500 Gr.C RECT	Typical
20	M22	N17	N11	90	Tall Columns	VBrace	Tube	A500 Gr.C RECT	Typical
21	M23	N15	N94		Web	VBrace	Tube	A500 Gr.C RECT	Typical
22	M24	N16	N93		Web	VBrace	Tube	A500 Gr.C RECT	Typical
23	M25	N12	N11		Top Beam	VBrace	Tube	A500 Gr.C RECT	Typical
24	M26	N4	N3		Middle Beams	VBrace	Tube	A500 Gr.C RECT	Typical
25	M27	N1	N2		Moment Frame Beams	Column	Tube	A500 Gr.C RECT	Typical
26	M30	N99	N100		Drag Strut	VBrace	Tube	A500 Gr.C RECT	Typical
27	M31	N98	N97		Drag Strut	VBrace	Tube	A500 Gr.C RECT	Typical
28	M28	N9	N10		Upper Moment Frame Beam	VBrace	Tube	A500 Gr.C RECT	Typical
29	M29	N16	N15		Cross Strut	VBrace	Tube	A500 Gr.C RECT	Typical
30	M32	N14	N13		Cross Strut	VBrace	Tube	A500 Gr.C RECT	Typical
31	M33	N7	N8		Cross Strut	VBrace	Tube	A500 Gr.C RECT	Typical
32	M34	N5	N6		Cross Strut	VBrace	Tube	A500 Gr.C RECT	Typical
33	M35	N101	N102		Middle Beams	VBrace	Tube	A500 Gr.C RECT	Typical

Member Advanced Data

	Label	I Release	J Release	Col-Wall Vert Release	Physical	Deflection Ratio Options	Seismic DR
1	M1	BenPIN	BenPIN		Yes	Default	None
2	M2	BenPIN	BenPIN		Yes	Default	None
3	M3	BenPIN			Yes	Default	None
4	M4	BenPIN			Yes	Default	None
5	M5				Yes	Default	None
6	M6		BenPIN		Yes	Default	None
7	M7				Yes	Default	None
8	M8		BenPIN		Yes	Default	None
9	M9	BenPIN	BenPIN		Yes	** NA **	None
10	M10	BenPIN	BenPIN		Yes	** NA **	None
11	M11				Yes	** NA **	None
12	M12				Yes	** NA **	None
13	M13				Yes	** NA **	None
14	M14				Yes	** NA **	None
15	M17	BenPIN	BenPIN		Yes	** NA **	None
16	M18	BenPIN	BenPIN		Yes	** NA **	None
17	M19	BenPIN	BenPIN		Yes	** NA **	None
18	M20	BenPIN	BenPIN		Yes	** NA **	None

Member Advanced Data (Continued)

	Label	I Release	J Release	Col-Wall Vert Release	Physical	Deflection Ratio Options	Seismic DR
19	M21				Yes	** NA **	None
20	M22				Yes	** NA **	None
21	M23	BenPIN	BenPIN		Yes	** NA **	None
22	M24	BenPIN	BenPIN		Yes	** NA **	None
23	M25	BenPIN	BenPIN		Yes	** NA **	None
24	M26	BenPIN	BenPIN		Yes	** NA **	None
25	M27				Yes	** NA **	None
26	M30				Yes	** NA **	None
27	M31				Yes	** NA **	None
28	M28				Yes	** NA **	None
29	M29	BenPIN	BenPIN		Yes	** NA **	None
30	M32	BenPIN	BenPIN		Yes	** NA **	None
31	M33	BenPIN	BenPIN		Yes	** NA **	None
32	M34	BenPIN	BenPIN		Yes	** NA **	None
33	M35	BenPIN	BenPIN		Yes	** NA **	None

Plate Primary Data

	Label	A Node	B Node	C Node	D Node	Material	Thickness [in]
1	P4	N9	N10	N19	N20	Weightless Steel	0.05
2	P5	N20	N19	N21	N22	Weightless Steel	0.05
3	P6	N22	N21	N15	N16	Weightless Steel	0.05
4	P7	N16	N15	N23	N24	Weightless Steel	0.05
5	P8	N24	N23	N25	N26	Weightless Steel	0.05
6	P9	N26	N25	N27	N28	Weightless Steel	0.05
7	P10	N28	N27	N29	N30	Weightless Steel	0.05
8	P11	N30	N29	N31	N32	Weightless Steel	0.05
9	P12	N32	N31	N33	N34	Weightless Steel	0.05
10	P13	N34	N33	N35	N36	Weightless Steel	0.05
11	P14	N36	N35	N37	N38	Weightless Steel	0.05
12	P15	N38	N37	N39	N40	Weightless Steel	0.05
13	P16	N40	N39	N41	N42	Weightless Steel	0.05
14	P17	N42	N41	N43	N44	Weightless Steel	0.05
15	P18	N44	N43	N13	N14	Weightless Steel	0.05
16	P19	N14	N13	N45	N46	Weightless Steel	0.05
17	P20	N46	N45	N47	N48	Weightless Steel	0.05
18	P21	N48	N47	N49	N50	Weightless Steel	0.05
19	P22	N50	N49	N51	N52	Weightless Steel	0.05
20	P23	N52	N51	N12	N11	Weightless Steel	0.05
21	P24	N2	N53	N54	N1	Weightless Concrete	2
22	P25	N53	N55	N56	N54	Weightless Concrete	2
23	P26	N55	N8	N7	N56	Weightless Concrete	2
24	P27	N8	N57	N58	N7	Weightless Concrete	2
25	P28	N57	N59	N60	N58	Weightless Concrete	2
26	P29	N59	N61	N62	N60	Weightless Concrete	2
27	P30	N61	N63	N64	N62	Weightless Concrete	2
28	P31	N63	N65	N66	N64	Weightless Concrete	2
29	P32	N65	N67	N68	N66	Weightless Concrete	2
30	P33	N67	N69	N70	N68	Weightless Concrete	2
31	P34	N69	N71	N72	N70	Weightless Concrete	2
32	P35	N71	N73	N74	N72	Weightless Concrete	2
33	P36	N73	N75	N76	N74	Weightless Concrete	2
34	P37	N75	N77	N78	N76	Weightless Concrete	2
35	P38	N77	N6	N5	N78	Weightless Concrete	2
36	P39	N6	N79	N80	N5	Weightless Concrete	2
37	P40	N79	N81	N82	N80	Weightless Concrete	2

Plate Primary Data (Continued)

	Label	A Node	B Node	C Node	D Node	Material	Thickness [in]
38	P41	N81	N83	N84	N82	Weightless Concrete	2
39	P42	N83	N85	N86	N84	Weightless Concrete	2
40	P43	N85	N4	N3	N86	Weightless Concrete	2

Member Distributed Loads (BLC 1 : Dead)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M3	Y	-0.12	-0.12	0	%100
2	M4	Y	-0.12	-0.12	0	%100
3	M6	Y	-0.12	-0.12	0	%100
4	M5	Y	-0.12	-0.12	0	%100
5	M7	Y	-0.12	-0.12	0	%100
6	M8	Y	-0.12	-0.12	0	%100

Member Distributed Loads (BLC 4 : Seismic X)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	X	0.243	0.243	0	%100
2	M2	X	0.243	0.243	0	%100
3	M7	X	0.369	0.369	0	%100
4	M6	X	0.369	0.369	0	%100
5	M5	X	0.369	0.369	0	%100
6	M3	X	0.369	0.369	0	%100
7	M4	X	0.369	0.369	0	%100
8	M8	X	0.369	0.369	0	%100

Member Distributed Loads (BLC 5 : Seismic Z)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M3	Z	0.369	0.369	0	%100
2	M5	Z	0.369	0.369	0	%100
3	M4	Z	0.369	0.369	0	%100
4	M8	Z	0.369	0.369	0	%100
5	M6	Z	0.369	0.369	0	%100
6	M7	Z	0.369	0.369	0	%100
7	M1	Z	0.243	0.243	0	%100
8	M2	Z	0.243	0.243	0	%100

Member Distributed Loads (BLC 6 : Wind ZA)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Z	0.106	0.106	0	%100
2	M2	Z	0.106	0.106	0	%100
3	M8	Z	0.106	0.106	0	%100
4	M3	Z	0.106	0.106	0	%100
5	M4	Z	0.106	0.106	0	%100
6	M5	Z	0.106	0.106	0	%100
7	M6	Z	0.106	0.106	0	%100
8	M7	Z	0.106	0.106	0	%100

Member Distributed Loads (BLC 7 : Wind ZB)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Z	0	0.235	5	%100
2	M2	Z	0	0.235	5	%100
3	M5	Z	0	0.156	0	%100
4	M7	Z	0	0.156	0	%100
5	M8	Z	0.156	0.235	0	%100
6	M6	Z	0.156	0.235	0	%100

Member Distributed Loads (BLC 8 : Wind ZC)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Z	0.192	0.192	0	12
2	M2	Z	0.192	0.192	0	12
3	M1	Z	0.124	0.124	12	24
4	M2	Z	0.124	0.124	12	24
5	M1	Z	0.09	0.09	24	36
6	M2	Z	0.09	0.09	24	36
7	M1	Z	0.065	0.065	36	%100
8	M2	Z	0.065	0.065	36	%100
9	M3	Z	0.192	0.192	0	%100
10	M4	Z	0.192	0.192	0	%100
11	M7	Z	0.192	0.192	0	6.5
12	M5	Z	0.192	0.192	0	6.5
13	M5	Z	0.124	0.124	6.5	18.5
14	M7	Z	0.124	0.124	6.5	18.5
15	M5	Z	0.092	0.092	18.5	%100
16	M7	Z	0.092	0.092	18.5	%100
17	M8	Z	0.092	0.092	0	0.5
18	M6	Z	0.092	0.092	0	0.5
19	M8	Z	0.065	0.065	0.5	%100
20	M6	Z	0.065	0.065	0.5	%100

Member Distributed Loads (BLC 9 : Wind ZB Opp)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Z	0.235	0	0	45
2	M2	Z	0.235	0	0	45
3	M3	Z	0.235	0.206	0	%100
4	M4	Z	0.235	0.206	0	%100
5	M5	Z	0.206	0.05	0	%100
6	M7	Z	0.206	0.05	0	%100
7	M8	Z	0.05	0	0	9.5
8	M6	Z	0.05	0	0	9.5

Member Distributed Loads (BLC 10 : Wind ZC Opp)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Z	0.065	0.065	0	12
2	M2	Z	0.065	0.065	0	12
3	M1	Z	0.092	0.092	12	24
4	M2	Z	0.092	0.092	12	24
5	M1	Z	0.124	0.124	24	36
6	M2	Z	0.124	0.124	24	36
7	M1	Z	0.192	0.192	36	%100
8	M2	Z	0.192	0.192	36	%100

Member Distributed Loads (BLC 10 : Wind ZC Opp) (Continued)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
9	M3	Z	0.065	0.065	0	%100
10	M4	Z	0.065	0.065	0	%100
11	M5	Z	0.065	0.065	0	6.5
12	M7	Z	0.065	0.065	0	6.5
13	M5	Z	0.092	0.092	6.5	18.5
14	M7	Z	0.092	0.092	6.5	18.5
15	M5	Z	0.124	0.124	18.5	%100
16	M7	Z	0.124	0.124	18.5	%100
17	M8	Z	0.124	0.124	0	0.5
18	M6	Z	0.124	0.124	0	0.5
19	M8	Z	0.192	0.192	0.5	%100
20	M6	Z	0.192	0.192	0.5	%100

Member Distributed Loads (BLC 11 : BLC 1 Transient Area Loads)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.026	-0.026	1.11e-14	49.905
2	M2	Y	-0.026	-0.026	8.382e-15	49.905
3	M3	Y	-0.148	-0.148	2.776e-16	5.5
4	M4	Y	-0.148	-0.148	9.437e-16	5.5
5	M5	Y	-0.148	-0.148	3.553e-15	29.621
6	M7	Y	-0.148	-0.148	3.553e-15	29.621
7	M6	Y	-0.148	-0.148	1.499e-14	14.75
8	M8	Y	-0.148	-0.148	1.366e-14	14.75

Member Distributed Loads (BLC 12 : BLC 2 Transient Area Loads)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M4	Y	-0.371	-0.371	9.437e-16	5.5
2	M5	Y	-0.371	-0.371	3.553e-15	29.621
3	M7	Y	-0.371	-0.371	3.553e-15	29.621
4	M6	Y	-0.371	-0.371	1.499e-14	14.75
5	M8	Y	-0.371	-0.371	1.366e-14	14.75
6	M3	Y	-0.371	-0.371	2.776e-16	5.5

Member Distributed Loads (BLC 13 : BLC 3 Transient Area Loads)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.1	-0.1	1.332e-15	5.508
2	M2	Y	-0.1	-0.1	2.22e-15	5.508
3	M1	Y	-0.1	-0.1	5.508	35.133
4	M2	Y	-0.1	-0.1	5.508	35.133
5	M1	Y	-0.1	-0.1	35.133	49.905
6	M2	Y	-0.1	-0.1	35.133	49.905
7	M1	Y	2.45e-15	-0.271	33.882	49.905
8	M2	Y	2.358e-15	-0.271	33.882	49.905

Member Area Loads (BLC 1 : Dead)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [ksf]	B Magnitude [ksf]	C Magnitude [ksf]	D Magnitude [ksf]	Exclude Braces
1	N9	N10	N12	N11	Y	A-B	-0.007	-0.007	-0.007	-0.007	Yes
2	N1	N2	N8	N7	Y	A-B	-0.04	-0.04	-0.04	-0.04	Yes
3	N7	N8	N6	N5	Y	A-B	-0.04	-0.04	-0.04	-0.04	Yes
4	N5	N6	N4	N3	Y	A-B	-0.04	-0.04	-0.04	-0.04	Yes

Member Area Loads (BLC 2 : Live)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [ksf]	B Magnitude [ksf]	C Magnitude [ksf]	D Magnitude [ksf]	Exclude Braces
1	N1	N2	N8	N7	Y	A-B	-0.1	-0.1	-0.1	-0.1	Yes
2	N7	N8	N6	N5	Y	A-B	-0.1	-0.1	-0.1	-0.1	Yes
3	N5	N6	N4	N3	Y	A-B	-0.1	-0.1	-0.1	-0.1	Yes

Member Area Loads (BLC 3 : Snow)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [ksf]	B Magnitude [ksf]	C Magnitude [ksf]	D Magnitude [ksf]	Exclude Braces
1	N9	N10	N15	N16	Y	A-B	-0.027	-0.027	-0.027	-0.027	Yes
2	N16	N15	N13	N14	Y	A-B	-0.027	-0.027	-0.027	-0.027	Yes
3	N14	N13	N12	N11	Y	A-B	-0.027	-0.027	-0.027	-0.027	Yes
4	N89	N90	N12	N11	Y	A-B	0	0	-0.073	-0.073	Yes

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed	Area(Member)
1	Dead	None	-1	6	4
2	Live	None			3
3	Snow	None			4
4	Seismic X	None		8	
5	Seismic Z	None		8	
6	Wind ZA	None		8	
7	Wind ZB	None		6	
8	Wind ZC	None		20	
9	Wind ZB Opp	None		8	
10	Wind ZC Opp	None		20	
11	BLC 1 Transient Area Loads	None		8	
12	BLC 2 Transient Area Loads	None		6	
13	BLC 3 Transient Area Loads	None		8	

ASD COMBINATIONS WERE USED TO DESIGN THE STEEL COMPONENTS

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	D Only		Y	1	1						
2	L Only		Y	2	1						
3	D+L (Defl)		Y	1	1	2	1				
4	Ex		Y	4	1						
5	Ez		Y	5	1						
6	WzA		Y	6	1						
7	WzB		Y	7	1						
8	WzC		Y	8	1						
9	WzBopp		Y	9	1						
10	WzCopp		Y	10	1						
11			Y								
12	D	Yes	Y	1	1						
13	D+L	Yes	Y	1	1	2	1				
14	D+S	Yes	Y	1	1	3	1				
15	D+0.75L+0.75 S	Yes	Y	1	1	2	0.75	3	0.75		
16	D+0.6WzA	Yes	Y	1	1	6	0.6				
17	D-0.6WzA	Yes	Y	1	1	6	-0.6				
18	D+0.6WzB	Yes	Y	1	1	7	0.6				
19	D-0.6WzB	Yes	Y	1	1	7	-0.6				
20	D+0.6WzC	Yes	Y	1	1	8	0.6				
21	D-0.6WzC	Yes	Y	1	1	8	-0.6				
22	D+0.6WzBopp	Yes	Y	1	1	9	0.6				

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
23	D-0.6WzBopp	Yes	Y	1	1	9	-0.6				
24	D+0.6WzCopp	Yes	Y	1	1	10	0.6				
25	D-0.6WzCopp	Yes	Y	1	1	10	-0.6				
26	D+0.75L+0.75(0.6WzA)+0.75S	Yes	Y	1	1	2	0.75	6	0.45	3	0.75
27	D+0.75L-0.75(0.6WzA)+0.75S	Yes	Y	1	1	2	0.75	6	-0.45	3	0.75
28	D+0.75L+0.75(0.6WzB)+0.75S	Yes	Y	1	1	2	0.75	7	0.45	3	0.75
29	D+0.75L-0.75(0.6WzB)+0.75S	Yes	Y	1	1	2	0.75	7	-0.45	3	0.75
30	D+0.75L+0.75(0.6WzC)+0.75S	Yes	Y	1	1	2	0.75	8	0.45	3	0.75
31	D+0.75L-0.75(0.6WzC)+0.75S	Yes	Y	1	1	2	0.75	8	-0.45	3	0.75
32	D+0.75L+0.75(0.6WzBopp)+0.75S	Yes	Y	1	1	2	0.75	9	0.45	3	0.75
33	D+0.75L-0.75(0.6WzBopp)+0.75S	Yes	Y	1	1	2	0.75	9	-0.45	3	0.75
34	D+0.75L+0.75(0.6WzCopp)+0.75S	Yes	Y	1	1	2	0.75	10	0.45	3	0.75
35	D+0.75L-0.75(0.6WzCopp)+0.75S	Yes	Y	1	1	2	0.75	10	-0.45	3	0.75
36	0.6D+0.6WzA	Yes	Y	1	0.6	6	0.6				
37	0.6D-0.6WzA	Yes	Y	1	0.6	6	-0.6				
38	0.6D+0.6WzB	Yes	Y	1	0.6	7	0.6				
39	0.6D-0.6WzB	Yes	Y	1	0.6	7	-0.6				
40	0.6D+0.6WzC	Yes	Y	1	0.6	8	0.6				
41	0.6D-0.6WzC	Yes	Y	1	0.6	8	-0.6				
42	0.6D+0.6WzBopp	Yes	Y	1	0.6	9	0.6				
43	0.6D-0.6WzBopp	Yes	Y	1	0.6	9	-0.6				
44	0.6D+0.6WzCopp	Yes	Y	1	0.6	10	0.6				
45	0.6D-0.6WzCopp	Yes	Y	1	0.6	10	-0.6				
46	D+0.7Ev+0.7Ex (rho included typ)	Yes	Y	1	1.157	4	0.91				
47	D+0.7Ev-0.7Ex	Yes	Y	1	1.157	4	-0.91				
48	D+0.7Ev+0.7Ez	Yes	Y	1	1.157	5	0.91				
49	D+0.7Ev-0.7Ez	Yes	Y	1	1.157	5	-0.91				
50	D+0.525Ev+0.525Ex+0.75L+0.75S	Yes	Y	1	1.118	4	0.682	2	0.75	3	0.75
51	D+0.525Ev-0.525Ex+0.75L+0.75S	Yes	Y	1	1.118	4	-0.682	2	0.75	3	0.75
52	D+0.525Ev+0.525Ez+0.75L+0.75S	Yes	Y	1	1.118	5	0.682	2	0.75	3	0.75
53	D+0.525Ev-0.525Ez+0.75L+0.75S	Yes	Y	1	1.118	5	-0.682	2	0.75	3	0.75
54	0.6D-0.7Ev+0.7Ex	Yes	Y	1	0.443	4	0.91				
55	0.6D-0.7Ev-0.7Ex	Yes	Y	1	0.443	4	-0.91				
56	0.6D-0.7Ev+0.7Ez	Yes	Y	1	0.443	5	0.91				
57	0.6D-0.7Ev-0.7Ez	Yes	Y	1	0.443	5	-0.91				
58											
59	1.4D		Y	1	1.4						
60	1.2D+1.6L+0.5S		Y	1	1.2	2	1.6	3	0.5		
61	1.2D+1.6S+L		Y	1	1.2	3	1.6	2	1		
62	1.2D+WzA+L+0.5S		Y	1	1.2	6	1	2	1	3	0.5
63	1.2D-WzA+L+0.5S		Y	1	1.2	6	-1	2	1	3	0.5
64	1.2D+WzB+L+0.5S		Y	1	1.2	7	1	2	1	3	0.5
65	1.2D-WzB+L+0.5S		Y	1	1.2	7	-1	2	1	3	0.5
66	1.2D+WzC+L+0.5S		Y	1	1.2	8	1	2	1	3	0.5
67	1.2D-WzC+L+0.5S		Y	1	1.2	8	-1	2	1	3	0.5
68	1.2D+WzBopp+L+0.5S		Y	1	1.2	9	1	2	1	3	0.5
69	1.2D-WzBopp+L+0.5S		Y	1	1.2	9	-1	2	1	3	0.5
70	1.2D+WzCopp+L+0.5S		Y	1	1.2	10	1	2	1	3	0.5
71	1.2D-WzCopp+L+0.5S		Y	1	1.2	10	-1	2	1	3	0.5
72	1.2D+WzA+L+0.5S		Y	1	1.2	6	1	2	1	3	0.5
73	1.2D-WzA+L+0.5S		Y	1	1.2	6	-1	2	1	3	0.5
74	1.2D+WzB+L+0.5S		Y	1	1.2	7	1	2	1	3	0.5
75	1.2D-WzB+L+0.5S		Y	1	1.2	7	-1	2	1	3	0.5
76	1.2D+WzC+L+0.5S		Y	1	1.2	8	1	2	1	3	0.5
77	1.2D-WzC+L+0.5S		Y	1	1.2	8	-1	2	1	3	0.5

THESE COMBINATIONS WERE USED TO DESIGN THE CONCRETE FOUNDATIONS.

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
78	1.2D+WzBopp+L+0.5S		Y	1	1.2	9	1	2	1	3	0.5
79	1.2D-WzBopp+L+0.5S		Y	1	1.2	9	-1	2	1	3	0.5
80	1.2D+WzCopp+L+0.5S		Y	1	1.2	10	1	2	1	3	0.5
81	1.2D-WzCopp+L+0.5S		Y	1	1.2	10	-1	2	1	3	0.5
82	0.9D+WzA		Y	1	0.9	6	1				
83	0.9D-WzA		Y	1	0.9	6	-1				
84	0.9D+WzB		Y	1	0.9	7	1				
85	0.9D-WzB		Y	1	0.9	7	-1				
86	0.9D+WzC		Y	1	0.9	8	1				
87	0.9D-WzC		Y	1	0.9	8	-1				
88	0.9D+WzBopp		Y	1	0.9	9	1				
89	0.9D-WzBopp		Y	1	0.9	9	-1				
90	0.9D+WzCopp		Y	1	0.9	10	1				
91	0.9D-WzCopp		Y	1	0.9	10	-1				
92	1.2D+Ev+Ex+L+0.2S (rho included typ)		Y	1	1.424	4	1.3	2	1	3	0.2
93	1.2D+Ev-Ex+L+0.2S		Y	1	1.424	4	-1.3	2	1	3	0.2
94	1.2D+Ev+Ez+L+0.2S		Y	1	1.424	5	1.3	2	1	3	0.2
95	1.2D+Ev-Ez+L+0.2S		Y	1	1.424	5	-1.3	2	1	3	0.2
96	0.9-Ev+Ex		Y	1	0.676	4	1.3				
97	0.9-Ev-Ex		Y	1	0.676	4	-1.3				
98	0.9-Ev+Ez		Y	1	0.676	5	1.3				
99	0.9-Ev-Ez		Y	1	0.676	5	-1.3				

Envelope AISC 15TH (360-16): ASD Member Steel Code Checks

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
1	M1	HSS16X8X8	0.399	5.718	53	0.104	49.905	y	52	96.397	625.749	150.332	264.471	1.845	H1-1a
2	M2	HSS16X8X8	0.399	5.718	52	0.104	49.905	y	53	96.397	625.749	150.332	264.471	1.844	H1-1a
3	M3	HSS16X8X8	0.162	5.5	53	0.122	0	y	53	607.898	625.749	150.332	264.471	2.294	H1-1b
4	M4	HSS16X8X8	0.162	5.5	52	0.122	0	y	52	607.898	625.749	150.332	264.471	2.294	H1-1b
5	M5	HSS16X8X8	0.187	0	13	0.056	0	y	52	270.283	625.749	150.332	264.471	1.584	H1-1b
6	M6	HSS16X8X8	0.159	0.154	13	0.093	2.151	z	48	508.159	625.749	150.332	264.471	3	H1-1b
7	M7	HSS16X8X8	0.187	0	13	0.056	0	y	53	270.283	625.749	150.332	264.471	1.584	H1-1b
8	M8	HSS16X8X8	0.159	0.154	13	0.08	0.154	y	52	508.159	625.749	150.332	264.471	3	H1-1b
9	M9	HSS8X6X5	0.035	0	14	0.009	10.044	y	48	189.042	227.246	42.166	51.397	1	H1-1b*
10	M10	HSS8X6X5	0.035	0	14	0.01	10.044	y	49	189.042	227.246	42.166	51.397	1	H1-1b*
11	M11	HSS8X6X5	0.218	9.961	53	0.026	9.961	y	53	189.616	227.246	42.166	51.397	2.239	H1-1b
12	M12	HSS8X6X5	0.218	9.961	52	0.026	9.961	y	52	189.616	227.246	42.166	51.397	2.239	H1-1b
13	M13	HSS8X8X8	0.448	1.512	47	0.253	1.396	z	47	350.759	404.192	93.563	93.563	2.217	H1-1b
14	M14	HSS8X8X8	0.448	1.512	47	0.253	1.396	z	47	350.759	404.192	93.563	93.563	2.254	H1-1b
15	M17	HSS8X6X5	0.132	30.769	56	0.008	30.769	y	48	48.29	227.246	42.166	51.397	1.136	H1-1b*
16	M18	HSS8X6X5	0.179	17.824	50	0.008	17.824	y	49	127.284	227.246	42.166	51.397	1.136	H1-1b*
17	M19	HSS8X6X5	0.132	30.769	57	0.008	30.769	y	49	48.29	227.246	42.166	51.397	1.136	H1-1b*
18	M20	HSS8X6X5	0.179	17.824	50	0.009	17.824	y	48	127.284	227.246	42.166	51.397	1.136	H1-1b*
19	M21	HSS8X8X6	0.101	0	50	0.005	23.833	y	49	279.082	311.377	73.353	73.353	1.667	H1-1b*
20	M22	HSS8X8X6	0.101	0	50	0.005	23.833	y	48	279.082	311.377	73.353	73.353	1.667	H1-1b*
21	M23	HSS8X6X5	0.222	6.238	51	0.006	11.09	y	48	181.57	227.246	42.166	51.397	1.136	H1-1a
22	M24	HSS8X6X5	0.222	6.238	51	0.006	11.09	y	49	181.57	227.246	42.166	51.397	1.136	H1-1a
23	M25	HSS12X6X5	0.004	3.708	50	0.007	7.417	y	49	262.224	297.006	49.211	95.06	1.136	H1-1b
24	M26	HSS6X6X4	0.005	3.708	47	0.006	7.417	y	49	141.091	156.886	27.944	27.944	1.136	H1-1b
25	M27	HSS6X6X6	0.377	0	49	0.069	0	y	49	203.074	226.946	39.421	39.421	2.276	H1-1b
26	M30	HSS4X4X8	0.043	7.417	57	0.003	0	y	48	134.464	180.24	19.212	19.212	2.277	H1-1b*
27	M31	HSS4X4X8	0.068	7.417	49	0.007	0	y	48	134.464	180.24	19.212	19.212	2.298	H1-1b
28	M28	HSS12X6X6	0.136	7.417	48	0.038	0	y	49	321.678	353.293	63.579	111.776	2.258	H1-1b
29	M29	HSS6X6X4	0.007	3.708	50	0.005	7.417	y	49	141.091	156.886	27.944	27.944	1.136	H1-1b
30	M32	HSS6X6X4	0.01	7.417	48	0.002	7.417	y	48	141.091	156.886	27.944	27.944	1.136	H1-1b*

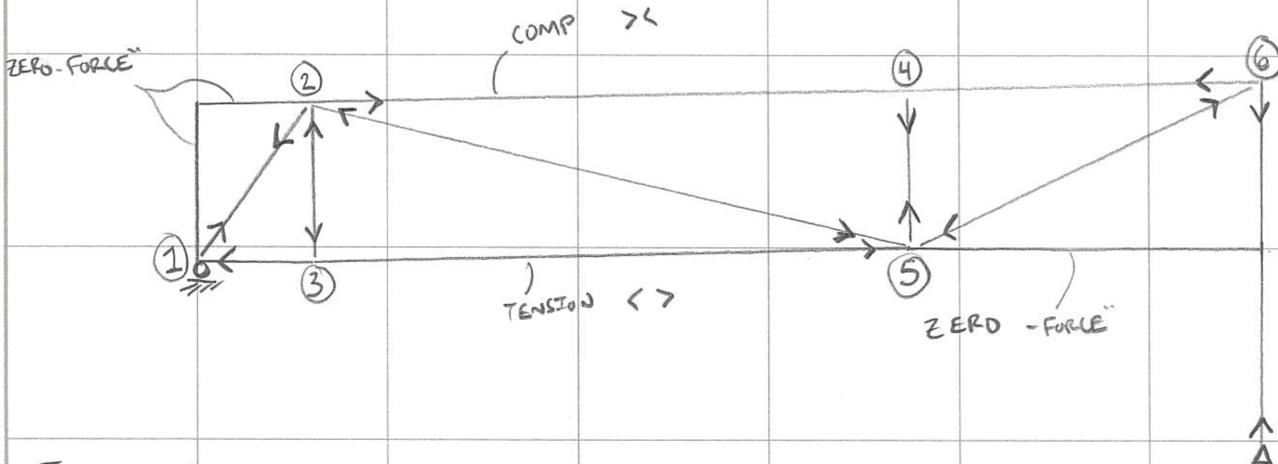
Envelope AISC 15TH (360-16): ASD Member Steel Code Checks (Continued)

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
31	M33	HSS6X6X4	0.008	3.708	46	0.006	7.417	y	49	141.091	156.886	27.944	27.944	1.136	H1-1b
32	M34	HSS6X6X4	0.009	3.708	49	0.003	7.417	y	49	141.091	156.886	27.944	27.944	1.136	H1-1b
33	M35	HSS6X6X4	0.006	3.708	48	0.005	7.417	y	49	141.091	156.886	27.944	27.944	1.136	H1-1b

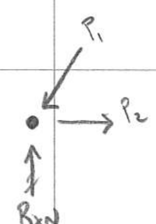
HSS CONNECTIONS:

AN ARTICLE BY DR. JEFFREY PACKER THAT WAS PUBLISHED BY THE STEEL TUBE INSTITUTE (STI) PROVIDES A METHODOLOGY FOR DESIGNING KT-CONNECTIONS. THIS BRIDGE WILL HAVE TWO KT-CONNECTIONS. ONE NEAR THE SIDEWALK AND ONE NEAR THE EXISTING BUILDING.

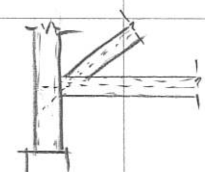
TRUSS FORCE DIAGRAM: FOR GRAVITY LOADING



JOINT 1:



3.4k SHEAR @ BEAM TO COLUMN.



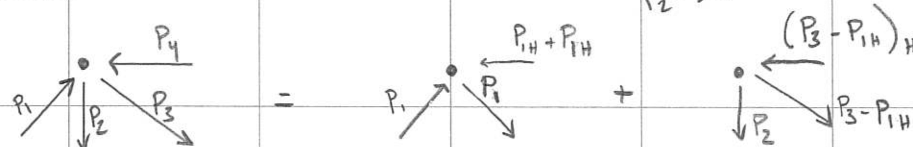
$P_1 = 50$ k (COMP) LRFD TYP.
$P_2 = 46$ k (TENS)

TRY (2) 6" LONG 1/4" FILLET WELDS TO TAKE FULL SHEAR CONSERVATIVELY

$6 \times 2 \times 4 \times 1.392$ k/in $= 67$k $\therefore$ ok

JOINT 2:

$P_1 = 50^k$ (COMP) $P_3 = 27^k$ (TENS)
$P_2 = 33^k$ (TENS) $P_4 = 48^k$ (COMP)



KT-CONNECTION.

K-CONNECTION BRANCHES 1 & 3

K-CONNECTION. BRANCHES 2 & 3

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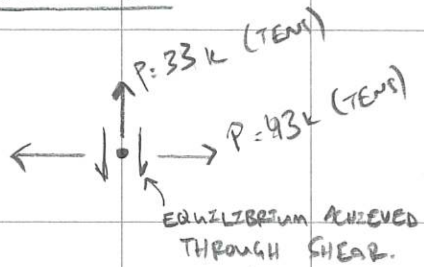
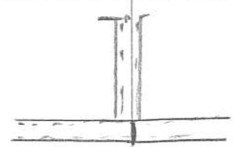
Project Number: 23358.A
Project: OWTC BRIDGE
Prepared By: JWM
Sheet Number:
Date: 1/23/2025

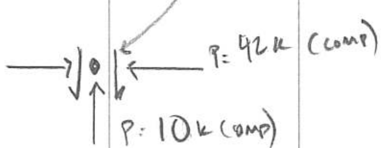
HSS CONNECTIONS - CONT'D:

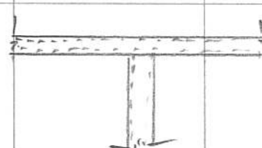
JOINT 3:



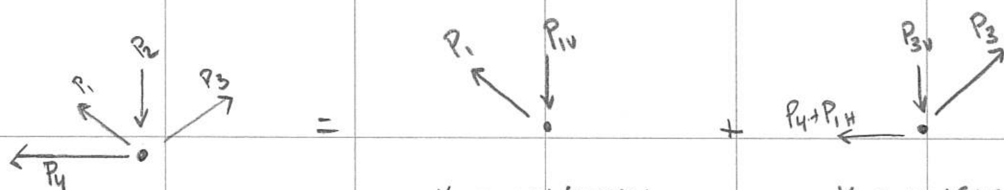


JOINT 4:





JOINT 5:



P_1 = 27K (TENS)
P_2 = 10k (COMP)
P_3 = 52k (TENS)
P_4 = 34k (TENS)

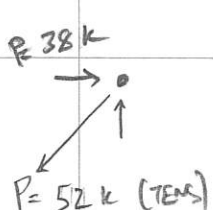
KT-CONNECTION

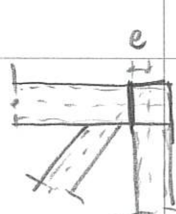
K-CONNECTION BRANCHES 1 & 2

K-CONNECTION BRANCHES 2 & 3

ALSO CHECK TOTAL UTILIZATION OF BRANCH 2.

JOINT 6:





TYP e=0

SEE SPREADSHEET RESULTS FOR JOINTS 1, 3, 4, AND 6.

SEE HAND CALCULATION FOR JOINTS 2 AND 5.

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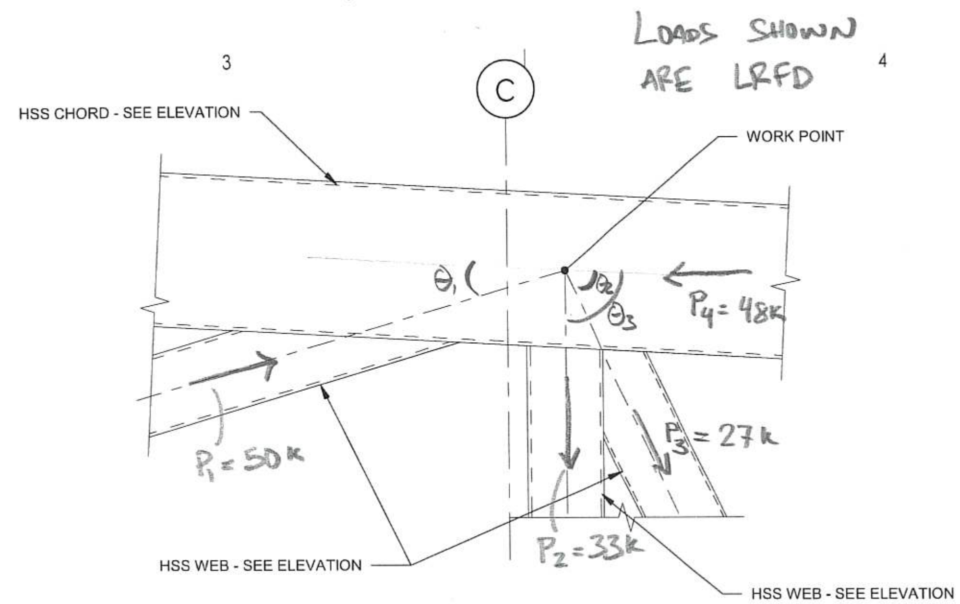
Project Number	23358. A	Sheet Number	
Project	OWTC BRIDGE		
Prepared By	JWM	Date	3/6/2025

KT CONNECTIONS:

JOINT #2:

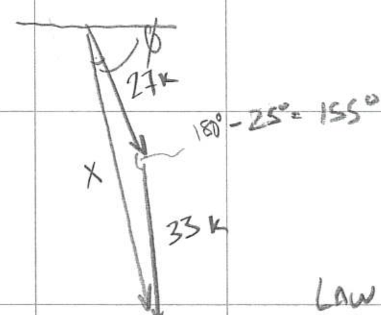


$\theta_1 = 19.3°$

$\theta_2 = 62°$

$\theta_3 = 87°$

TREAT P_2 & P_3 AS RESULTANT Y-CONNECTION TO CHORD. THEN USE SPREADSHEET TO CHECK GAPPED K-CONNECTION.



LAW OF COSINES

$$X = \sqrt{(27k)^2 + (33k)^2 - 2 \times 27k \times 33k \times \cos 155°} = 58.6k$$

$$33k = \sqrt{(58.6k)^2 + (27k)^2 - 2 \times 58.6k \times 27k \times \cos \phi}$$

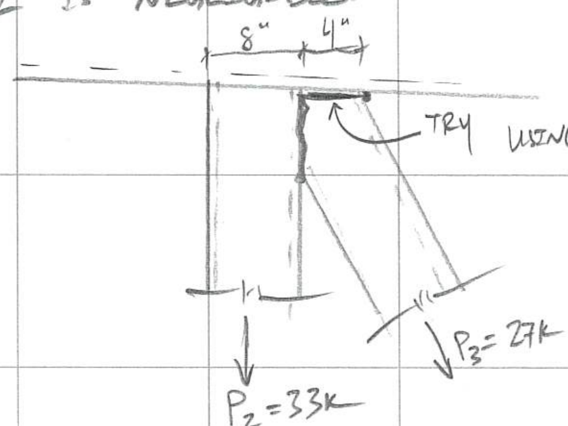
SOLVE FOR $\phi \Rightarrow \phi = 13.7°$

TOTAL ANGLE $= 62° + 13.7° = 76.3°$

KT CONNECTIONS - CONT'D:

JOINT #2 - CONT'D:

CHECK LENGTH OF WELD FROM BRANCH 3 TO CHORD ASSUMING THE MATCHED WIDTH CONTRIBUTION CONNECTION TO BRANCH 2 IS NEGLIGIBLE.



$l_{REQ'D}$ OF 5/16" WELD

$$\frac{27k}{5 \times 1.392} = 3.9"$$

THEREFORE, EVEN WITHOUT HEEL OR WELD TO WEB 2, FULL LOAD CAN BE TRANSFERRED.

CONSERVATIVELY, LOOK AT GAPPED K-CONNECTION WITH RESULTANT ACTING AS AN HSS 8x6 EVEN THOUGH RESULTANT PROFILE IS LARGER.
SEE SPREADSHEET OUTPUT.

SINCE THE 30° LIMIT OF APPLICABILITY IS EXCEEDED, ENSURE CAPACITY IS ACHIEVED WITH SIDEWALL WELDS ONLY.

$$l_{REQ'D} = \frac{50k}{5 \times 1.392 k/in} = 7.2" = 3.6" \text{ EACH SIDE.}$$

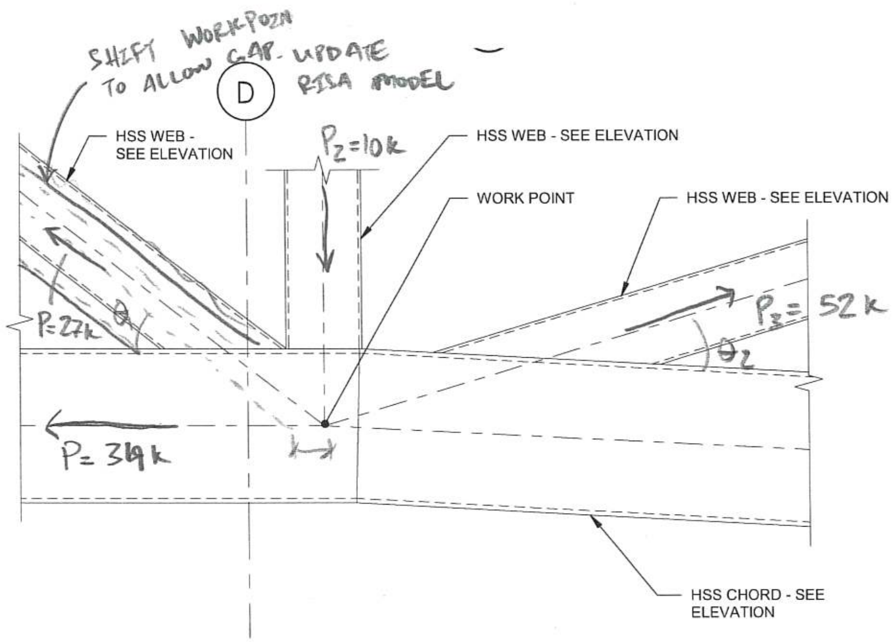
∴ OK BY INSPECTION.

ARW ENGINEERS structural consultants

Project Number 23358.A
Sheet Number
Project OWTC BRIDGE
Prepared By JWM
Date 3/6/2025

KT CONNECTIONS:

JOINT 5



$\theta_1 = 37°$ $\quad P_{1V} = 27k \times \sin 37° = 16.3k$

$\theta_2 = 19.4°$ $\quad P_{3V} = 17.3k$

SINCE $P_2 << P_1$ AND P_3, EXAMINE P_1 AND P_3 AS A Y-CONNECTIONS THEN RUN P_2 INDEPENDENTLY.

SEE SPREADSHEET RESULTS.

HSS K-Gapped Connection

Version Date: Dec. 28, 2016 | Author: Arek Higgs | Reviewed By: Troy M. Dye | 3/6/2025

ENGINEERS

JOB TITLE: HSS - K gapped connection | JOB #:
DESCRIPTION: Joint 2 - Resultant Branches | ENGINEER:

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C Fy= 50 ksi
B:	8.0	in.	Member Orientation- A
H:	16.0	in.	
t:	0.465	in.	
Compression Force:	-48.0	kip, Compression	
Web Member j:	HSS8X6X5/16	Material-	Grade C Fy= 50 ksi
$ø_j$	19.3	deg	0.34 rad
B_j:	6.0	in.	Member Orientation- A
H_j:	8.0	in.	
t_j:	0.291	in.	
$H_{j\ Projection}$:	24.20	in.	
Member Force P_u:	-50.0	kip, Compression	
Web Member i:	HSS8X6X5/16	Material-	Grade C Fy= 50 ksi
$ø_i$	76.3	deg	1.33 rad
B_i:	6.0	in.	Member Orientation- A
H_i:	8.0	in.	
t_i:	0.291	in.	
$H_{i\ Projection}$:	8.23	in.	
Member Force P_u:	58.6	kip, Tension	

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

Limits of Applicability (Table K2.2A):

Eccentricity:	$-0.55 \le e \le 0.25$
Branch ø:	$\ge 30°$
Chord Wall Slenderness:	$B/t \le 35$ & $H/t \le 35$
Branch Wall Slenderness:	
	Tension B_b/t_b & $H_b/t_b \le 35$
	Compression Member Bb/tb & Hb/tb $\le 1.25\sqrt{(E/fy)}$
Wdith Ratio:	Bb/B & Hb/B $\ge 0.1+\gamma/50$
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_b/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_b/t_{bj} \le 1.0$
Chors Slenderness Ratio:	$\ge 0.1+\gamma/50$
	$\beta_{eff} \ge 0.35$
Gap Ratio	$\zeta \ge 0.5(1-\beta_{eff})$
Gap	$g \ge t_b+t_{bj}$
Branch Size:	smaller $B_b \ge 0.63$ larger B_b

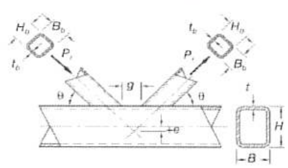
(Note: Max Gap is controled by e/H limits, if gap is large, treat as (2) Y- Connections)

Eccentricity:
e = 0 in (see table K2.2A for possitive and negative values)

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK
Branch j:		
B_{bj}/t_{bj} =	20.6	OK
H_{bj}/t_{bj} =	27.5	OK
B_{bj}/B =	0.75	OK
H_{bj}/B =	1.00	OK
H_{bj}/B_{bj} =	1.3	OK
Branch i:		
B_b/t_{bi} =	20.6	OK
H_b/t_{bi} =	27.5	OK
B_b/B =	0.75	OK
H_b/B =	1.00	OK
H_b/B_{bi} =	1.3	OK
both Branches:		
B_b/B_{bj} =	1.0	OK
t_b/t_{bj} =	1.0	OK
$ø_j$ =	19.3	NG
$ø_i$ =	76.3	OK
$ø_{ij}$ =	84.4	OK
Gap =	8.58	OK
(Gap Ratio) ζ =	1.07	OK
β_{jeff} =	0.88	OK
Branch Size =	Small B > 0.63 Large B; Therefore OK	
e=	0.00	OK

SEE CONSERVATIVE WELD CALC. BY HAND. EVEN WITH ONLY SIDEWALLS THERE IS MORE THAN ENOUGH CAPACITY



Member Capacity:

Chord Wall Plastification, For All β

$Q_{f,j}$ =	1.00		
U_j =	0.05		
β_{jeff} =	0.875		
γ =	8.60		
Φ =	0.90		
$P_{n,j}$ =	822.7	Kips	EQ-(K2-14)
$ΦP_{n,j}$ =	**740.4**	**Kips**	
	Therefore OK		
$Q_{f,i}$ =	1.00		
U_i =	0.05		
β_{eff} =	0.875		
γ =	8.60		
$P_{n,i}$ =	279.9	Kips	EQ-(K2-14)
$ΦP_{n,i}$ =	**251.9**	**Kips**	
	Therefore OK		

Shear Yielding (Punching), when $B_b < B-2t$

Is Branch Square? NO Therefore Required
Is $B_{bj} < B-2t$? Yes, Therefore Required
Is $B_{bi} < B-2t$? Yes, Therefore Required

η_j =	4.03		
β_j =	0.75		
β_{eop-j} =	0.44		
Φ =	0.95		
$P_{n,j}$ =	3124.7	Kips	EQ-(K2-15)
$ΦP_{n,j}$ =	**2968.5**	**Kips**	
	Therefore OK		
η_i =	1.37		
β_i =	0.75		
β_{eop-i} =	0.44		
$P_{n,i}$ =	451.5	Kips	EQ-(K2-15)
$ΦP_{n,i}$ =	**196.8**	**Kips**	
	Therefore OK		

Shear of Chord Sidewalls *(in Gaped Region)*

Is Chord Square? NO Therefore Required

Φ =	0.90		
k_v =	5.0		
C_v =	1.0		
V_n =	446.4	Kips	EQ-(G2-1)
$ΦV_n$ =	**401.8**	**Kips**	
	Therefore OK		

Local Yielding of Branch(es) Due to Uneven Load Distribution

Is Branch Square? NO Therefore Required
Is B/t ≥ 15? Yes, Therefore Required

$b_{eoi,j}$ =	5.6		
Φ =	0.95		
$P_{n,j}$ =	384.24818	Kips	EQ-(K2-16)
$ΦP_{n,i}$ =	**365.0**	**Kips**	
	Therefore OK		
$b_{eoi,i}$ =	5.6		
Pn,j =	384.24818	Kips	EQ-(K2-16)
ΦPn,j =	**2141.3**	**Kips**	
	Therefore OK		

Weld Design:

$l_{ej,50°}$ =	57.60	in	EQ-(K4-8)
$l_{ej,60°}$ =	51.95	in	EQ-(K4-9)
l_{ej} =	57.60	in	
$F_{nw,j}$ =	70	ksi	
Φ =	0.75	Fillet	
$t_{wj\ min}$ =	1/16	in	
t_{wj} =	$\boxed{3/16}$	in	
$l_{ei,50°}$ =	27.05		EQ-(K4-8)
$l_{ei,60°}$ =	21.40		EQ-(K4-9)
l_{ei} =	21.40	in	
$F_{nw,i}$ =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	2/16	in	
t_{wi} =	$\boxed{3/16}$	in	

ENGINEERS

HSS T- & Y- Connections 3/6/2025

Version Date: September 25, 2017 Author: Arek Higgs Reviewed By: Troy M. Dye

JOB TITLE: HSS -T- & Y- connections JOB #: 23358.A

DESCRIPTION: Joint 1 ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	16.0	in.		
t:	0.465	in.		
Compression Force:	46.0	kip, Tension		
Branch:	HSS8X6X5/16	Material-	Grade C Fy=	50 ksi
$ø_b$:	64.6	deg	1.13	rad
B_b:	6.0	in.	Member Orientation-	A
H_b:	8.0	in.		
t_b:	0.291	in.		
$H_{b\ Pojection}$:	8.85	in.		
Member Force P_u:	-50.0	kip, Compression		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	B/t ≤ 35 & H/t ≤ 35
Branch Wall Slenderness:	
	Tension B_b/t_b & $H_b/t_b \le 35$
	Compression Member Bb/tb & Hb/tb ≤ 1.25√(E/fy)
Wdith Ratio:	Bb/B & Hb/B ≥ 0.25
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_b/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_b/t_{bj} \le 1.0$

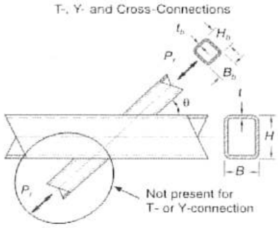
Limit Checks: (see limits table)

Chord:

B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK

Branch:

B_b/t_b =	20.6	OK
H_b/t_b =	27.5	OK
B_b/B =	0.8	OK
H_b/B =	1.0	OK
H_b/B_b =	1.3	OK



Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	0.75	Applies	
η =	1.11		
Q_f =	1.00		
U=	0.044		
Φ =	1.00		
P_n =	201.7	kip	EQ-(K2-7)
$ΦP_n$ =	**201.7**	**kip**	**OK**

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	0.75	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.436		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	0.75	NA	
Branch in Compression:	YES		
l_b =	8.85	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-9)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	0.75	NA	
b_{eoi} =	5.57	in.	
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-12)
$ΦP_n$ =	**NA**	**kip**	

Weld Design:

l_e =	28.9	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	1/16	in	
t_{wi} =	**3/16**	**in**	

HSS T- & Y- Connections 3/6/2025

Version Date: September 25, 2017 Author: Arek Higgs Reviewed By: Troy M. Dye

JOB TITLE: HSS -T- & Y- connections JOB #: 23358.A

DESCRIPTION: Joint 1 ENGINEER: JWM

ENGINEERS

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS8X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	8.0	in.		
t:	0.465	in.		
Compression Force:	-35.0	kip, Compression		
Branch:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
$\varnothing_b$:	90.0	deg	1.57	rad
B_b:	8.0	in.	Member Orientation-	A
H_b:	16.0	in.		
t_b:	0.465	in.		
$H_{b\ Pojection}$:	16.00	in.		
Member Force P_u:	46.0	kip, Tension		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

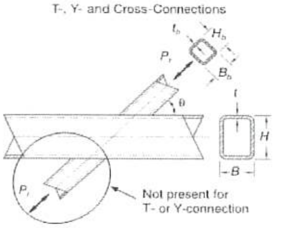
Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	$B/t \le 35$ & $H/t \le 35$
Branch Wall Slenderness:	
Tension	B_b/t_b & $H_b/t_b \le 35$
Compression Member	Bb/tb & $Hb/tb \le 1.25\sqrt{(E/fy)}$
Wdith Ratio:	Bb/B & $Hb/B \ge 0.25$
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_b/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_b/t_{bj} \le 1.0$

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	17.2	OK
H/B =	1.0	OK
Branch:		
B_b/t_b =	17.2	OK
H_b/t_b =	34.4	OK
B_b/B =	1.0	OK
H_b/B =	2.0	OK
H_b/B_b =	2.0	OK

T-, Y- and Cross-Connections



Weld Design:

l_e =	41.3	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	1/16	in	
t_{wi} =	$\boxed{3/16}$	in	

Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	1.00	NA	
η =	2.00		
Q_f =	1.00		
U=	0.052		
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-7)
ΦP_n =	**NA**	**kip**	

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	1.00	NA	
B/t =	17.20		
γ =	8.60		
β_{eop} =	0.581		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
ΦP_n =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	1.00	Applies	
Branch in Compression:	NO		
l_b =	16.00	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	906.2	kip	EQ-(K2-9)
ΦP_n =	**906.2**	**kip**	**OK**

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
ΦP_n =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	1.00	Applies	
b_{eoi} =	4.65	in.	
Φ =	0.95		
P_n =	917.0	kip	EQ-(K2-12)
ΦP_n =	**871.1**	**kip**	**OK**

ARW ENGINEERS

HSS T- & Y- Connections — 3/6/2025

Version Date: September 25, 2017 — Author: Arek Higgs — Reviewed By: Troy M. Dye

JOB TITLE: HSS -T- & Y- connections — JOB #: 23358.A

DESCRIPTION: Joint 3 — ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	16.0	in.		
t:	0.465	in.		
Compression Force:	43.0	kip, Tension		
Branch:	HSS8X6X5/16	Material-	Grade C Fy=	50 ksi
$ø_b$:	90.0	deg	1.57	rad
B_b:	6.0	in.	Member Orientation-	A
H_b:	8.0	in.		
t_b:	0.291	in.		
$H_{b\ Pojection}$:	8.00	in.		
Member Force P_u:	33.0	kip, Tension		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

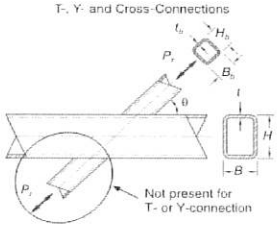
Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	B/t ≤ 35 & H/t ≤ 35
Branch Wall Slenderness:	
	Tension B_b/t_b & $H_b/t_b \le 35$
	Compression Member Bb/tb & Hb/tb ≤ 1.25√(E/fy)
Wdith Ratio:	Bb/B & Hb/B ≥ 0.25
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_{bi}/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_{bi}/t_{bj} \le 1.0$

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK
Branch:		
B_b/t_b =	20.6	OK
H_b/t_b =	27.5	OK
B_b/B =	0.8	OK
H_b/B =	1.0	OK
H_b/B_b =	1.3	OK

T-, Y- and Cross-Connections



Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	0.75	Applies	
η =	1.00		
Q_f =	1.00		
U=	0.041		
Φ =	1.00		
P_n =	173.0	kip	EQ-(K2-7)
$ΦP_n$ =	**173.0**	**kip**	**OK**

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	0.75	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.436		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	0.75	NA	
Branch in Compression:	NO		
l_b =	8.00	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-9)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	0.75	NA	
b_{eoi} =	5.57	in.	
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-12)
$ΦP_n$ =	**NA**	**kip**	

Weld Design:

l_e =	27.1	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	1/16	in	
t_{wi} =	**3/16**	in	

ENGINEERS

HSS T- & Y- Connections

Version Date: September 25, 2017 — Author: Arek Higgs — Reviewed By: Troy M. Dye — 3/6/2025

JOB TITLE: HSS -T- & Y- connections — JOB #: 23358.A

DESCRIPTION: Joint 4 — ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C	Fy=	50 ksi
B:	8.0	in.	Member Orientation-		A
H:	16.0	in.			
t:	0.465	in.			
Compression Force:	-42.0	kip, Compression			
Branch:	HSS8X6X5/16	Material-	Grade C	Fy=	50 ksi
$ø_b$:	90.0	deg		1.57	rad
B_b:	6.0	in.	Member Orientation-		A
H_b:	8.0	in.			
t_b:	0.291	in.			
$H_{b\ Pojection}$:	8.00	in.			
Member Force P_u:	-10.0	kip, Compression			

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	$B/t \le 35$ & $H/t \le 35$
Branch Wall Slenderness:	
	Tension B_b/t_b & $H_b/t_b \le 35$
	Compression Member Bb/tb & Hb/tb ≤ 1.25√(E/fy)
Wdith Ratio:	Bb/B & Hb/B ≥ 0.25
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_b/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_b/t_{bj} \le 1.0$

Limit Checks: (see limits table)

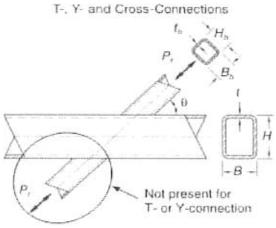
Chord:

B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK

Branch:

B_b/t_b =	20.6	OK
H_b/t_b =	27.5	OK
B_b/B =	0.8	OK
H_b/B =	1.0	OK
H_b/B_b =	1.3	OK

T-, Y- and Cross-Connections



Weld Design:

l_e =	27.1	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	0	in	
t_{wi} =	3/16	in	

Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	0.75	Applies	
η =	1.00		
Q_f =	1.00		
U=	0.040		
Φ =	1.00		
P_n =	173.0	kip	EQ-(K2-7)
$ΦP_n$ =	**173.0**	**kip**	**OK**

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	0.75	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.436		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	0.75	NA	
Branch in Compression:	YES		
l_b =	8.00	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-9)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	0.75	NA	
b_{eoi} =	5.57	in.	
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-12)
$ΦP_n$ =	**NA**	**kip**	

ARW ENGINEERS

HSS T- & Y- Connections

Version Date: September 25, 2017 Author: Arek Higgs Reviewed By: Troy M. Dye

3/6/2025

JOB TITLE: HSS -T- & Y- connections JOB #: 23358.A

DESCRIPTION: Joint 6 ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	16.0	in.		
t:	0.465	in.		
Compression Force:	-38.0	kip, Compression		
Branch:	HSS8X6X5/16	Material-	Grade C Fy=	50 ksi
$ø_b$:	34.0	deg	0.59	rad
B_b:	6.0	in.	Member Orientation-	A
H_b:	8.0	in.		
t_b:	0.291	in.		
$H_{b\ Pojection}$:	14.31	in.		
Member Force P_u:	52.0	kip, Tension		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

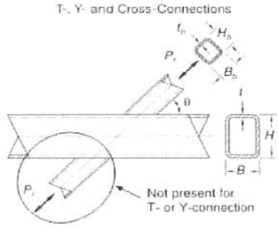
Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	$B/t \le 35$ & $H/t \le 35$
Branch Wall Slenderness:	
	Tension B_b/t_b & $H_b/t_b \le 35$
	Compression Member Bb/tb & $Hb/tb \le 1.25\sqrt{(E/fy)}$
Wdith Ratio:	Bb/B & $Hb/B \ge 0.25$
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_{bi}/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_{bi}/t_{bj} \le 1.0$

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK
Branch:		
B_b/t_b =	20.6	OK
H_b/t_b =	27.5	OK
B_b/B =	0.8	OK
H_b/B =	1.0	OK
H_b/B_b =	1.3	OK

T-, Y- and Cross-Connections



Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	0.75	Applies	
η =	1.79		
Q_f =	1.00		
U=	0.036		
Φ =	1.00		
P_n =	431.3	kip	EQ-(K2-7)
$ΦP_n$ =	**431.3**	**kip**	**OK**

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	0.75	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.436		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	0.75	NA	
Branch in Compression:	NO		
l_b =	14.31	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-9)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	0.75	NA	
b_{eoi} =	5.57	in.	
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-12)
$ΦP_n$ =	**NA**	**kip**	

Weld Design:

l_e =	39.8	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	1/16	in	
t_{wi} =	**3/16**	**in**	

ENGINEERS

HSS T- & Y- Connections — 3/6/2025

Version Date: September 25, 2017 — Author: Arek Higgs — Reviewed By: Troy M. Dye

JOB TITLE: HSS -T- & Y- connections — JOB #: 23358.A

DESCRIPTION: Joint 6 — ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS8X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	8.0	in.		
t:	0.465	in.		
Compression Force:	-30.0	kip, Compression		
Branch:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
$ø_b$:	90.0	deg	1.57	rad
B_b:	8.0	in.	Member Orientation-	A
H_b:	16.0	in.		
t_b:	0.465	in.		
$H_{b\ Pojection}$:	16.00	in.		
Member Force P_u:	-38.0	kip, Compression		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

Limits of Applicability (Table K2.2A):

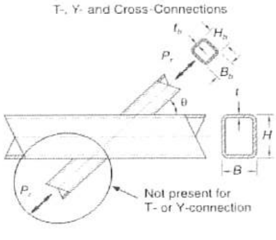
Branch ø:	≥ 30°
Chord Wall Slenderness:	B/t ≤ 35 & H/t ≤ 35
Branch Wall Slenderness:	
Tension	B_b/t_b & $H_b/t_b \le 35$
Compression Member	Bb/tb & Hb/tb ≤ 1.25√(E/fy)
Wdith Ratio:	Bb/B & Hb/B ≥ 0.25
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_b/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_b/t_{bj} \le 1.0$

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	17.2	OK
H/B =	1.0	OK
Branch:		
B_b/t_b =	17.2	OK
H_b/t_b =	34.4	NG
B_b/B =	1.0	OK
H_b/B =	2.0	OK
H_b/B_b =	2.0	OK

LIMIT = 30. HOWEVER, THE CHORD IS ONLY UTILIZED TO 40% ∴ ENGINEERING JUDGEMENT IS USED. THIS IS ALSO CALLING THE CHORD OF THE TRUSS THE "BRANCH" WHERE IT CONNECTS TO THE COLUMN.

T-, Y- and Cross-Connections



Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	1.00	NA	
η =	2.00		
Q_f =	1.00		
U=	0.044		
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-7)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	1.00	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.581		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	1.00	Applies	
Branch in Compression:	YES		
l_b =	16.00	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	906.2	kip	EQ-(K2-9)
$ΦP_n$ =	**906.2**	**kip**	**OK**

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	3444.1	kip	EQ-(K2-10)
$ΦP_n$ =	**2583.0**	**kip**	**OK**

Limit State: Local Yielding of Branches, When β > 0.85

β =	1.00	Applies	
b_{eoi} =	4.65	in.	
Φ =	0.95		
P_n =	917.0	kip	EQ-(K2-12)
$ΦP_n$ =	**871.1**	**kip**	**OK**

Weld Design:

l_e =	41.3	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	1/16	in	
t_{wi} =	3/16	in	

ARW ENGINEERS

HSS T- & Y- Connections

Version Date: September 25, 2017 | Author: Arek Higgs | Reviewed By: Troy M. Dye | 3/6/2025

JOB TITLE: HSS -T- & Y- connections | JOB #: 23358.A

DESCRIPTION: Joint 5 - Branch 1 | ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	16.0	in.		
t:	0.465	in.		
Compression Force:	39.0	kip, Tension		
Branch:	HSS8X6X5/16	Material-	Grade C Fy=	50 ksi
$ø_b$:	37.0	deg	0.65	rad
B_b:	6.0	in.	Member Orientation-	A
H_b:	8.0	in.		
t_b:	0.291	in.		
$H_{b\ Pojection}$:	13.29	in.		
Member Force P_u:	27.0	kip, Tension		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

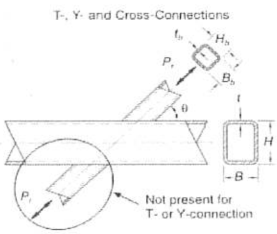
(NOTE: All connections per this sheet shall have no moment in webs)

Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	$B/t \leq 35$ & $H/t \leq 35$
Branch Wall Slenderness:	
	Tension B_b/t_b & $H_b/t_b \leq 35$
	Compression Member Bb/tb & Hb/tb ≤ 1.25√(E/fy)
Wdith Ratio:	Bb/B & Hb/B ≥ 0.25
Aspect Ratio:	$0.5 \leq H_b/B_b \leq 2$
	$0.5 \leq H/B \leq 2$
Overlap Ratio:	$25\% \leq O_v \leq 100\%$
Branch Width Ratio:	$B_b/B_{bj} \geq 0.75$
Branch Thickness Ratio:	$t_b/t_{bj} \leq 1.0$

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK
Branch:		
B_b/t_b =	20.6	OK
H_b/t_b =	27.5	OK
B_b/B =	0.8	OK
H_b/B =	1.0	OK
H_b/B_b =	1.3	OK



Weld Design:

l_e =	37.7	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	1/16	in	
t_{wi} =	$\boxed{3/16}$	in	

Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	0.75	Applies	
η =	1.66		
Q_f =	1.00		
U=	0.037		
Φ =	1.00		
P_n =	382.5	kip	EQ-(K2-7)
$ΦP_n$ =	**382.5**	**kip**	**OK**

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	0.75	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.436		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	0.75	NA	
Branch in Compression:	NO		
l_b =	13.29	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-9)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	0.75	NA	
b_{eoi} =	5.57	in.	
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-12)
$ΦP_n$ =	**NA**	**kip**	

HSS T- & Y- Connections 3/6/2025

Version Date: September 25, 2017 Author: Arek Higgs Reviewed By: Troy M. Dye

ENGINEERS

JOB TITLE: HSS -T- & Y- connections JOB #: 23358.A

DESCRIPTION: Joint 5 - Branch 1 ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	16.0	in.		
t:	0.465	in.		
Compression Force:	39.0	kip, Tension		
Branch:	HSS8X6X5/16	Material-	Grade C Fy=	50 ksi
$ø_b$:	90.0	deg	1.57	rad
B_b:	6.0	in.	Member Orientation-	A
H_b:	8.0	in.		
t_b:	0.291	in.		
$H_{b\ Pojection}$:	8.00	in.		
Member Force P_u:	-10.0	kip, Compression		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

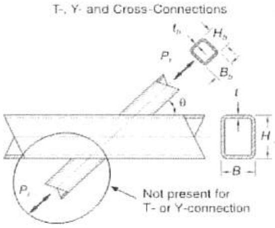
Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	B/t ≤ 35 & H/t ≤ 35
Branch Wall Slenderness:	
	Tension B_b/t_b & $H_b/t_b \le 35$
	Compression Member Bb/tb & Hb/tb ≤ 1.25√(E/fy)
Wdith Ratio:	Bb/B & Hb/B ≥ 0.25
Aspect Ratio:	$0.5 \le H_b/B_b \le 2$
	$0.5 \le H/B \le 2$
Overlap Ratio:	$25\% \le O_v \le 100\%$
Branch Width Ratio:	$B_b/B_{bj} \ge 0.75$
Branch Thickness Ratio:	$t_b/t_{bj} \le 1.0$

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK
Branch:		
B_b/t_b =	20.6	OK
H_b/t_b =	27.5	OK
B_b/B =	0.8	OK
H_b/B =	1.0	OK
H_b/B_b =	1.3	OK

T-, Y- and Cross-Connections



Weld Design:

l_e =	27.1	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	0	in	
t_{wi} =	**3/16**	**in**	

Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	0.75	Applies	
η =	1.00		
Q_f =	1.00		
U=	0.037		
Φ =	1.00		
P_n =	173.0	kip	EQ-(K2-7)
$ΦP_n$ =	**173.0**	**kip**	**OK**

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	0.75	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.436		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	0.75	NA	
Branch in Compression:	YES		
l_b =	8.00	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-9)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	0.75	NA	
b_{eoi} =	5.57	in.	
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-12)
$ΦP_n$ =	**NA**	**kip**	

ENGINEERS

HSS T- & Y- Connections

Version Date: September 25, 2017 Author: Arek Higgs Reviewed By: Troy M. Dye

3/6/2025

JOB TITLE: HSS -T- & Y- connections JOB #: 23358.A

DESCRIPTION: Joint 5 - Branch 1 ENGINEER: JWM

(Note: Negative Loads indicate Compression Forces)

Geometry:

Chord Member:	HSS16X8X1/2	Material-	Grade C Fy=	50 ksi
B:	8.0	in.	Member Orientation-	A
H:	16.0	in.		
t:	0.465	in.		
Compression Force:	39.0	kip, Tension		
Branch:	HSS8X6X5/16	Material-	Grade C Fy=	50 ksi
$ø_b$:	19.4	deg		0.34 rad
B_b:	6.0	in.	Member Orientation-	A
H_b:	8.0	in.		
t_b:	0.291	in.		
$H_{b\ Pojection}$:	24.08	in.		
Member Force P_u:	52.0	kip, Tension		

Rectangulare Member Orientation A - B=smaller side, Orientation B - B=wider side

(NOTE: All connections per this sheet shall have no moment in webs)

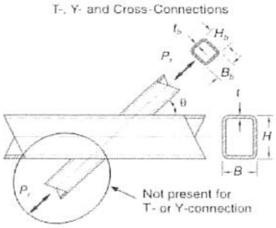
Limits of Applicability (Table K2.2A):

Branch ø:	≥ 30°
Chord Wall Slenderness:	B/t ≤ 35 & H/t ≤ 35
Branch Wall Slenderness:	
	Tension B_b/t_b & H_b/t_b ≤ 35
	Compression Member Bb/tb & Hb/tb ≤ 1.25√(E/fy)
Wdith Ratio:	Bb/B & Hb/B ≥ 0.25
Aspect Ratio:	0.5 ≤ H_b/B_b ≤ 2
	0.5 ≤ H/B ≤ 2
Overlap Ratio:	25% ≤ O_v ≤ 100%
Branch Width Ratio:	B_b/B_{bj} ≥ 0.75
Branch Thickness Ratio:	t_b/t_{bj} ≤ 1.0

Limit Checks: (see limits table)

Chord:		
B/t =	17.2	OK
H/t =	34.4	OK
H/B =	2.0	OK
Branch:		
B_b/t_b =	20.6	OK
H_b/t_b =	27.5	OK
B_b/B =	0.8	OK
H_b/B =	1.0	OK
H_b/B_b =	1.3	OK

T-, Y- and Cross-Connections



Member Capacity:

Limit State: Chord Wall Plastification, When β ≤ 0.85

β =	0.75	Applies	
η =	3.01		
Q_f =	1.00		
U=	0.037		
Φ =	1.00		
P_n =	1044.3	kip	EQ-(K2-7)
$ΦP_n$ =	**1044.3**	**kip**	OK

Limit State: Shear Yielding (punching), When 0.85 < β ≤ 1-1/γ or B/t < 10

β =	0.75	NA	
B/t =	17.20		
γ =	8.60		
$β_{eop}$ =	0.436		
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-8)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Chord Sidewalls, When β = 1.0

β =	0.75	NA	
Branch in Compression:	NO		
l_b =	24.08	in.	
k =	0.698	in. (k is typically taken as 1.5t, radious of corner)	
Φ =	1.00		
P_n =	NA	kip	EQ-(K2-9)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Crippling of Chord Sidewalls, When β = 1.0 & Branch in Compression

Φ =	0.75		
P_n =	NA	kip	EQ-(K2-10)
$ΦP_n$ =	**NA**	**kip**	

Limit State: Local Yielding of Branches, When β > 0.85

β =	0.75	NA	
b_{eoi} =	5.57	in.	
Φ =	0.95		
P_n =	NA	kip	EQ-(K2-12)
$ΦP_n$ =	**NA**	**kip**	

Weld Design:

l_e =	59.3	in. Effective Weld Length	EQ-(K4-5)
F_{nw} =	70	ksi	
Φ =	0.75	Fillet	
$t_{wi\ min}$ =	1/16	in	
t_{wi} =	$\boxed{3/16}$	in	

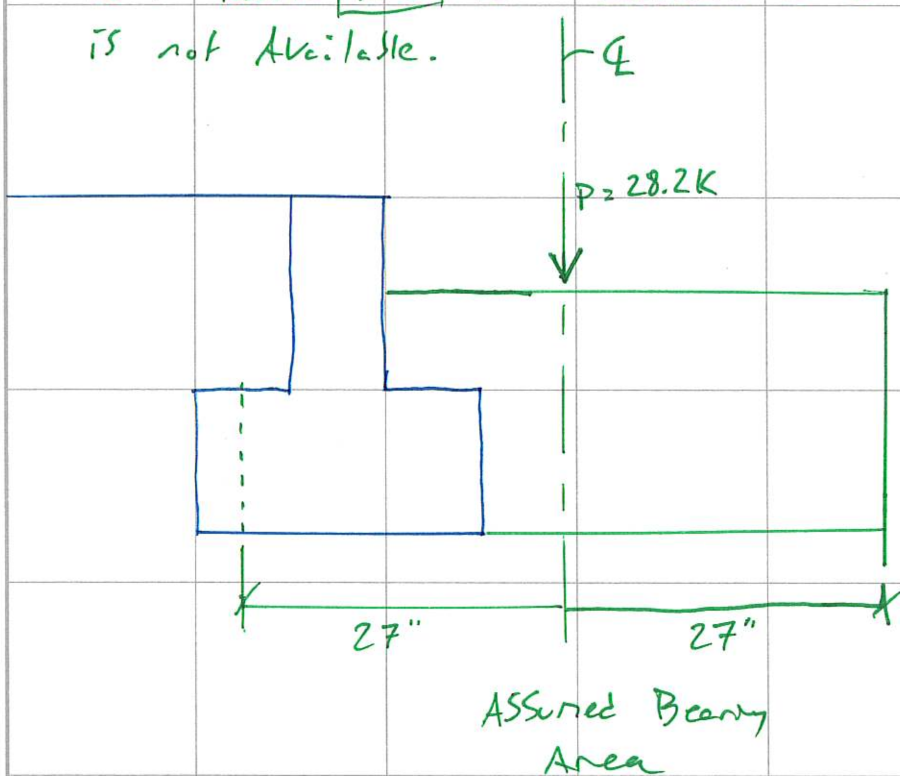
Footings @ West Side

Max ASD reaction = $\boxed{28.2K}$

Existing 5' footing to have load removed from Column

Existing Drawings show 3000 psf Allowable Bearing Capacity.

We'll Assume $\boxed{1500}$ to be Conservative as a new Soil report is not Available.



ARW ENGINEERS structural consultants ARW Engineers 1594 W Park Circle Ogden, UT 84404	Project			Job Ref.	
	Section			Sheet no./rev. 1	
	Calc. by S	Date 3/4/2025	Chk'd by	Date	App'd by	Date

FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.09

Summary results

Overall design status PASS

Overall design utilisation 0.938

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	65.4			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.548	1.65	0.938	Pass

increased for 12" of concrete

Pad footing details

Length of footing L_x = **4.5** ft

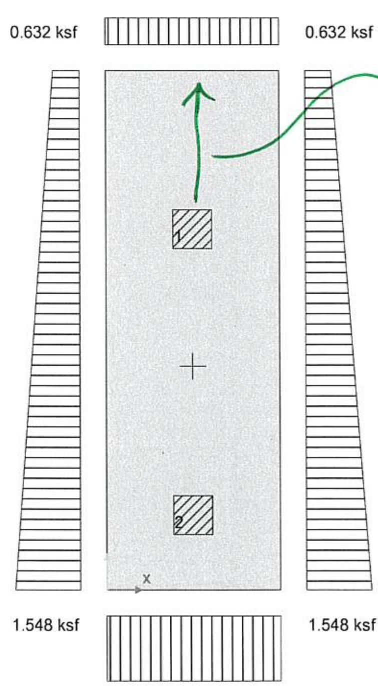
Width of footing L_y = **13.33** ft

Footing area $A = L_x \times L_y$ = **59.985** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **0** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Extended to North Side of 5' footing

Column no.1 details

Length of column l_{x1} = **12.00** in

Width of column l_{y1} = **12.00** in

position in x-axis x_1 = **27.00** in

position in y-axis y_1 = **111.00** in

Column no.2 details

Length of column l_{x2} = **12.00** in

Width of column l_{y2} = **12.00** in

position in x-axis x_2 = **27.00** in

position in y-axis y_2 = **23.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **1.65** ksf

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Column no.1 loads

Dead load in z F_{Dz1} = **28.2** kips

Column no.2 loads

Dead load in z F_{Dz2} = **28.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.938)

Combination 1 results: 1.0D

Forces on footing

Force in z-axis $F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})$ = **65.4** kips

Moments on footing

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2)$ = **147.1** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2)$ = **374.9** kip_ft

Uplift verification

Vertical force F_{dz} = **65.398** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **-11.194** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.548** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.632** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.548** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.632}$ ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{0.632}$ ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{1.548}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} = \mathbf{1.65}$ ksf

$q_{max} / q_{allow} = \mathbf{0.938}$

PASS - Allowable bearing capacity exceeds design base pressure

DIAPHRAGM CHECKS:

CHECK ROOF DECK SHEAR ATTACHMENT @ ENDS:

V @ ROOF = 24.3 k.

12.3 k @ EACH END

$$V_E = \frac{0.7 \times 12.3k \times \overset{\rho}{1.3}}{6.75'} = 1.66 \text{ klf} \quad (ASD)$$

TRY 18 GA. W/ PUNCHLOK @ 12" O.C. OK FOR 1,900 plf ∴ OK

FLOOR DIAPHRAGM:

$$V = \frac{1.3 \times 36.8k}{6.75' \times 2} = 3.5 \text{ klf} \quad (LRFD)$$

2" CONCRETE (LW) WITH #3 @ 10" O.C.
DIAPHRAGM CAPACITY = ϕV_n = 7.4 k/ft.

DIAPHRAGM TRANSFER @ PERIMETER HSS
USE 3/4" Ø HSA'S @ 12" O.C.
Q_n = 17.1 k FROM TABLE 3-21 LIGHTWEIGHT 3ksi PARALLEL TO FLUTES.

$\phi Q_n = 0.65 \times 17.1k = 11.2$ k/ANCHOR

$\phi V_n = 11.2 \text{ k/ft} > 3.5 \text{ klf}$ ∴ OK

Concrete Diaphragm Shear Capacity

ARW ENGINEERS

Version Date: October 28, 2016 | Author: Scott Porter | Reviewed by: Troy Dye

3/6/25 3:27 PM

Code: IBC 2018 using ACI 318-14 with seismic provisions

JOB TITLE: OWTC Bridge | JOB #: 23358.A

Description: Slab on Deck | Designed by: JWM

Concrete Properties

Deck thickness (min. 2" composite or 2-1/2" non-composite per 18.12.6.1) — $t =$ 2 in

Area of concrete section resisting shear — $A_{cv} =$ 24 in^2/ft

Concrete compressive strength — $f'_c =$ 3000 psi

Lightweight concrete factor — $\lambda =$ 0.75

Shear Reinforcement Properties

Rebar yield strength — $f_y =$ 60000 psi

Concrete Strength

Concrete shear strength per foot (Eq. 12.5.3.3) — $V_c =$ **1972 lb/ft**

Rebar Shear Strength (Eqn 12.5.3.3) — $\Phi V_c =$ **1479 lb/ft**

V_s (lb/ft)

Slab rebar spacing (in)

bar #	A_{vs}/bar	4	6	8	9	10	12	16	18
3	0.11	19800	13200	9900	8800	7920	6600	4950	4400
4	0.2	36000	24000	18000	16000	14400	12000	9000	8000
5	0.31	55800	37200	27900	24800	22320	18600	13950	12400
6	0.44	79200	52800	39600	35200	31680	26400	19800	17600
7	0.6	108000	72000	54000	48000	43200	36000	27000	24000
8	0.79	142200	94800	71100	63200	56880	47400	35550	31600

Minimum Steel Requirements

Table 9.6.3.3 — $A_v \geq$ 0.02 in^2/ft

24.4.3 Minimum shrinkage and temperature reinforcing — $A_v \geq$ 0.0432 in^2/ft

Maximum shear strength of structural diaphragm for SDC D, E or F

18.12.9.1 $V_{n\ MAX}$ with ($\rho_t = \rho_l$)

$V_{n\ MAX}$ (lb/ft)

Slab rebar spacing (in)

bar #	A_{vs}/bar	4	6	8	9	10	12	16	18
3	0.11	21772	15172	11872	10772	9891.8	8571.8	6921.8	6371.8
4	0.2	37972	25972	19972	17972	16372	13972	10972	9971.8
5	0.31	57772	39172	29872	26772	24292	20572	15922	14372
6	0.44	81172	54772	41572	37172	33652	28372	21772	19572
7	0.6	109972	73972	55972	49972	45172	37972	28972	25972
8	0.79	144172	96772	73072	65172	58852	49372	37522	33572

18.12.9.2 $V_{n\ MAX}$ — $V_{n\ MAX}$ **10516 lb/ft**

Diaphragm Shear Capacity

$\phi_V =$ 0.75

$\phi_V V_n$ (lb/ft)

Slab rebar spacing (in)

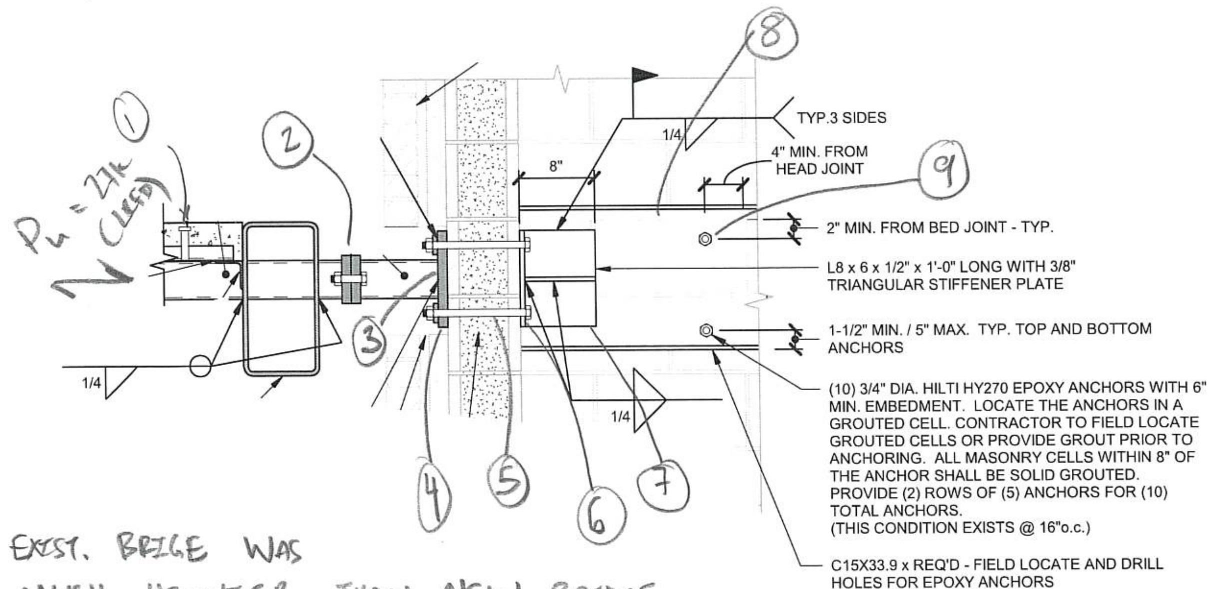
bar #	A_{vs}/bar	4	6	8	9	10	12	16	18
3	0.11	7887.2	7887.2	7887.2	7887.2	7419	--	--	--
4	0.2	7887.2	7887.2	7887.2	7887.2	7887.2	--	--	--
5	0.31	7887.2	7887.2	7887.2	7887.2	7887.2	--	--	--
6	0.44	7887.2	7887.2	7887.2	7887.2	7887.2	--	--	--
7	0.6	7887.2	7887.2	7887.2	7887.2	7887.2	--	--	--
8	0.79	7887.2	7887.2	7887.2	7887.2	7887.2	--	--	--

s_{max} 10 in

ARW ENGINEERS
structural consultants

Project Number 23358.A
Sheet Number
Project OWTC BRIDGE
Prepared By JWM
Date 3/6/2025

DRAG @ EXIST. BLDG:



EXIST. BRIGE WAS MUCH HEAVIER THAN NEW BRIDGE, THEREFORE, 10% RULE FOR LATERAL PER IEBC IS VALID. LOAD TO WALL IS REDUCED

1) HSA DIAPHRAGM TRANSFER. SEE DIAPHRAGM CALC ✓

2) TUBE WELD, PLATE BENDING / PRYING CHECK, AND BOLTS IN TENSION. ALSO CHECK STRUT IN COMP.

3) WELD TO PLATE

4) PRYING / PLATE BENDING

5) BOLTS IN TENSION.

6) ANGLE BENDING / PRYING.

7) ANGLE WELD TO CHANNEL

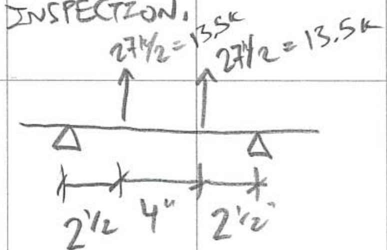
8) CHANNEL IN COMPRESSION.

9) EPOXY ANCHORS TO WALL

DRAG @ EXIST BLDG - CONT'D:

(2) WELD REQ'D $= \dfrac{27k}{1.392 \times 4} = 4.8"$ OF 1/4" WELD.

FOR 1/4" WELD ALL AROUND HSS4x4 TUBE, OK BY INSPECTION.



$M_u = 13.5k \times 2\tfrac{1}{2}" = 33.75$ k-in

TRY 1" GR. 50 PLATE

$\phi M_n = 0.90 \times 50\text{ ksi} \times \dfrac{5" \times (1")^2}{4} = 56.3$ k-in.

> 33.8 k-in ∴ OK

REQ'D THICKNESS TO ELIMINATE PRYING:

$$t_{np} = \sqrt{\frac{4 \times 13.5k \times \left(2\tfrac{1}{2}" - \tfrac{3}{4}"/2\right)}{0.9 \times 5" \times 65\text{ ksi}}} = 0.63"$$

USE A 1" PLATE.

HSS 4x4x1/2 HAS 265 k OF COMP. CAPACITY FOR A 2'-0" UNBRACED LENGTH. ∴ OK

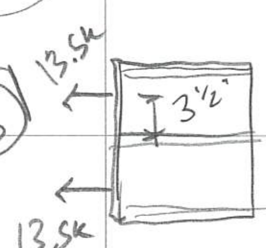
(3) WELD OK. SEE CHECK (2)

(4) USE 1" PLATE. SINCE MOMENT ARM IS LESS THAN CHECK (2), THIS IS ACCEPTABLE.

(5) (2) 3/4" BOLTS A325 $\phi R_n = 2 \times 29.8k = 59.6k$

FROM TABLE 7-2 OF AISC 360-16.

(6)



$$t_{np} = \sqrt{\frac{4 \times 13.5k \times \left(3\tfrac{1}{2}" - 0.75"/2\right)}{0.9 \times 6" \times 65\text{ ksi}}} = 0.69"$$ USE 3/4" ANGLE.

ARW ENGINEERS
structural consultants

Project Number 23358.A | Sheet Number
Project OWTC BRIDGE
Prepared By JWM | Date 3/6/2025

DRAG @ EXIST. BLDG - CONT'D:

(7) REQ'D WELD LENGTH = $\frac{27k}{4 \times 1.392}$ = 4.8".

WELD IS **OK**

(8) C 15x33.9 CHANNEL BRACED LENGTH = 2'-0"
HAS 0.36 STRESS RATIO.

(9) EPOXY ANCHORS TO WALL. SINGLE ANCHOR
IS GOOD FOR ϕV_n = 2.7 k

$27k / 2.7k$ = 10 ANCHORS.

3/4" DIA HILTI HIT HY EPOXY ANCHORS W/ 6" EMBEDMENT

Steel Column

Project File: 23358.A - OWTC Bridge Replacement Project.ec6

LIC# : KW-06015770, Build:20.24.05.02 ARW ENGINEERS

DESCRIPTION: Channel Strut

Code References

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **C15x33.9**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 2 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 2 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 2 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 67.80 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 2.0 ft, Xecc = 1.70 in, Yecc = 2.0 in, E = 27.0 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.3612**	: 1
Load Combination	+1.20D+E	
Location of max.above base	2.0	ft
At maximum location values are . . .		
Pu	27.081	k
0.9 * Pn	323.688	k
Mu-x	-4.50	k-ft
0.9 * Mn-x :	137.160	k-ft
Mu-y	-3.825	k-ft
0.9 * Mn-y :	13.349	k-ft
PASS Maximum Shear Stress Rati	**0.02671**	: 1
Load Combination	+1.20D+E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	1.913	k
Vn * Phi : Allowable	71.604	k

Maximum Load Reactions . .

Top along X-X	1.913 k
Bottom along X-X	1.913 k
Top along Y-Y	2.250 k
Bottom along Y-Y	2.250 k

Maximum Load Deflections . . .

Along Y-Y -0.000220 in at 1.168ft above base
for load combination :E Only

Along X-X -0.007310 in at 1.168ft above base
for load combination :E Only

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.000	PASS	0.00 ft	1.66	1.66	4.28	26.64	0.000	PASS	0.00 ft
+1.20D	0.000	PASS	0.00 ft	1.66	1.66	4.28	26.64	0.000	PASS	0.00 ft
+1.20D+E	0.361	PASS	2.00 ft	1.66	1.66	4.28	26.64	0.027	PASS	0.00 ft
+0.90D	0.000	PASS	0.00 ft	1.66	1.66	4.28	26.64	0.000	PASS	0.00 ft
+0.90D+E	0.361	PASS	2.00 ft	1.66	1.66	4.28	26.64	0.027	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	0.068								
+D+0.70E	18.968	1.339	1.339	-1.575	1.575				
+D+0.5250E	14.243	1.004	1.004	-1.181	1.181				
+0.60D	0.041								
+0.60D+0.70E	18.941	1.339	1.339	-1.575	1.575				
E Only	27.000	1.913	1.913	-2.250	2.250				

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	27.000	1.913	1.913	-2.250	2.250		-4.500		-3.825
"	Minimum	0.041								

Steel Column

DESCRIPTION: Channel Strut

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Reaction, X-X Axis Base	Maximum	27.000	1.913	1.913	-2.250	2.250		-4.500		-3.825
"	Minimum	0.068								
Reaction, Y-Y Axis Base	Maximum	0.068								
"	Minimum	27.000	1.913	1.913	-2.250	2.250		-4.500		-3.825
Reaction, X-X Axis Top	Maximum	27.000	1.913	1.913	-2.250	2.250		-4.500		-3.825
"	Minimum	0.068								
Reaction, Y-Y Axis Top	Maximum	27.000	1.913	1.913	-2.250	2.250		-4.500		-3.825
"	Minimum	0.068								
Moment, X-X Axis Base	Maximum	0.068								
"	Minimum	0.068								
Moment, Y-Y Axis Base	Maximum	0.068								
"	Minimum	0.068								
Moment, X-X Axis Top	Maximum	0.068								
"	Minimum	27.000	1.913	1.913	-2.250	2.250		-4.500		-3.825
Moment, Y-Y Axis Top	Maximum	0.068								
"	Minimum	27.000	1.913	1.913	-2.250	2.250		-4.500		-3.825

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	-0.0051 in	1.168 ft	-0.000 in	1.168 ft
+D+0.5250E	-0.0038 in	1.168 ft	-0.000 in	1.168 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	-0.0051 in	1.168 ft	-0.000 in	1.168 ft
E Only	-0.0073 in	1.168 ft	-0.000 in	1.168 ft

Steel Section Properties : C15x33.9

Depth	=	15.000 in	I xx	=	315.00 in^4	J	=	1.010 in^4
Web Thick	=	0.400 in	S xx	=	42.00 in^3	Cw	=	358.00 in^6
Flange Width	=	3.400 in	R xx	=	5.610 in	Ro	=	5.940 in
Flange Thick	=	0.650 in	Zx	=	50.800 in^3	H	=	0.920 in
Area	=	10.000 in^2	I yy	=	8.070 in^4			
Weight	=	33.900 plf	S yy	=	3.090 in^3	Wno	=	15.100 in^2
Kdesign	=	1.440 in	R yy	=	0.901 in	Sw	=	10.400 in^4
			Zy	=	6.190 in^3	Qf	=	14.000 in^3
rts	=	1.130 in				Qw	=	25.200 in^3
Ycg	=	0.000 in				Wn2	=	7.860
Xcg	=	0.788 in				Sw2	=	7.550
Xp	=	0.332 in				Sw3	=	3.810
Eo	=	0.896 in						

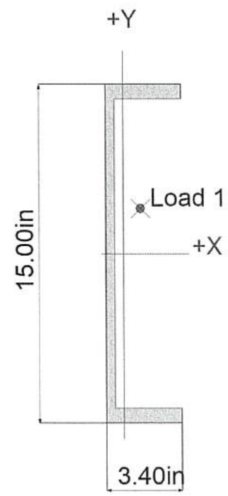
Steel Column

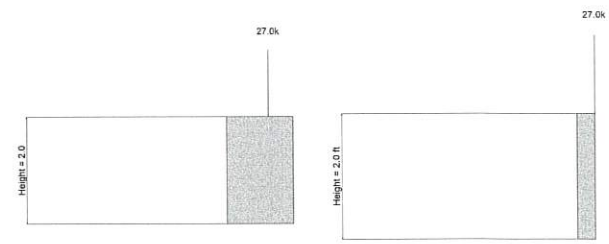
Project File: 23358.A - OWTC Bridge Replacement Project.ec6

LIC# : KW-06015770, Build:20.24.05.02 ARW ENGINEERS

DESCRIPTION: Channel Strut

Sketches





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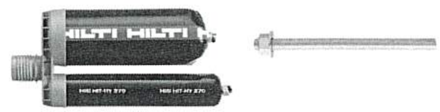
Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Single Anchor Capacity - OWTC Wall	Date:	3/7/2025
Fastening point:			

Specifier's comments:

1 Input data

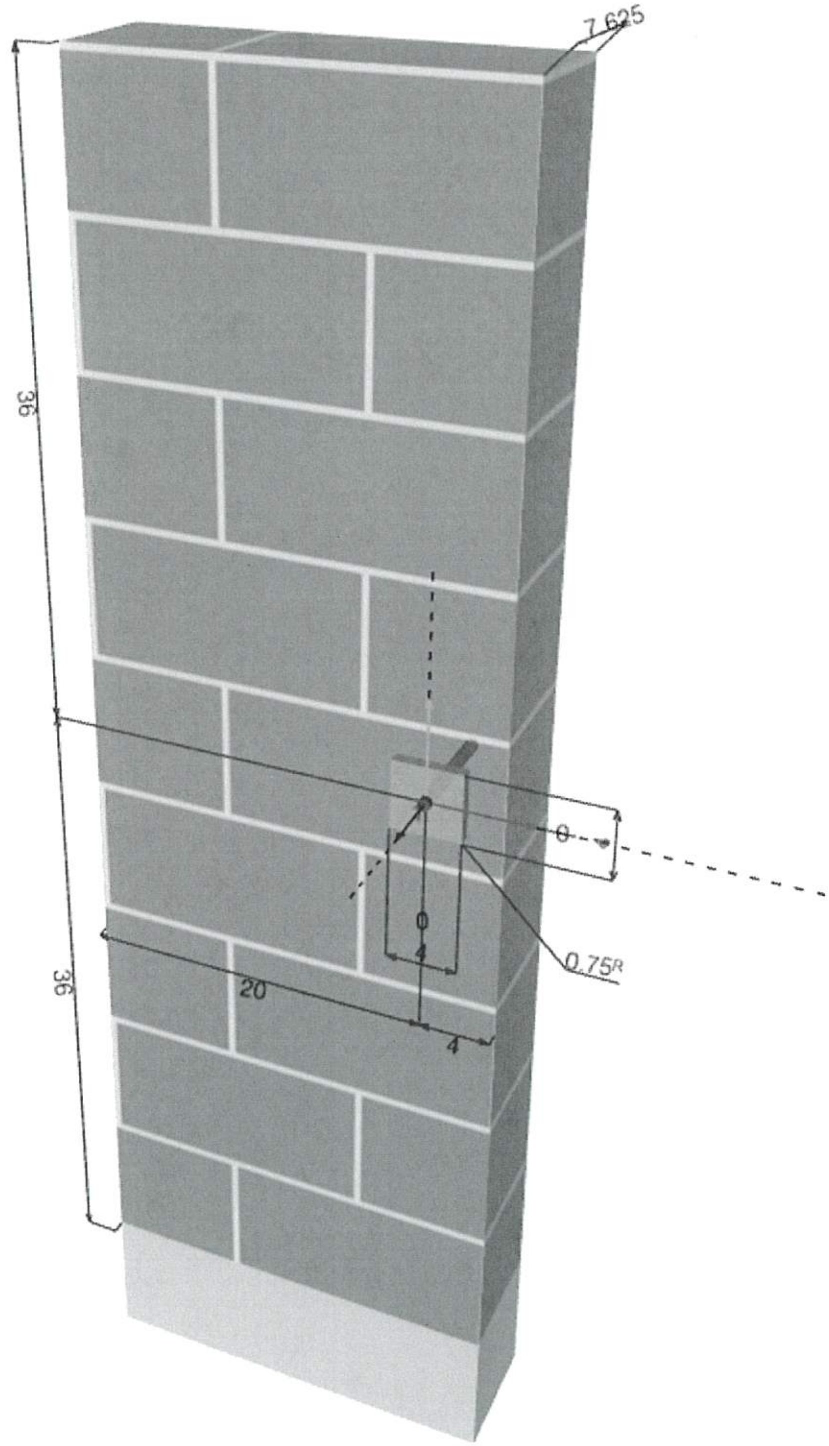


Anchor type and diameter:	**HY 270 + threaded rod 5.8 3/4**
Item number:	2081383 HAS 5.8 3/4"x8" (element) / 2194247 HIT-HY 270 (adhesive)
Specification text:	Hilti ∅ 3/4 HY 270 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4143 , Hammer drilled installation per MPII,
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	5.8
Evaluation Service Report:	ESR-4143
Issued I Valid:	11/1/2023 | 1/1/2025
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-19
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.750 in.
Anchor plate[R] :	l_x x l_y x t = 4.000 in. x 4.000 in. x 0.750 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	uncracked Grout-filled CMU, f=1500, f = 1,500 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Solid Head Joint: no; open ended unit: no
Temp. short/long:	32 °F / 32 °F
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Face installation, Drill hole: Hammer drilled, Installation condition: Dry
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

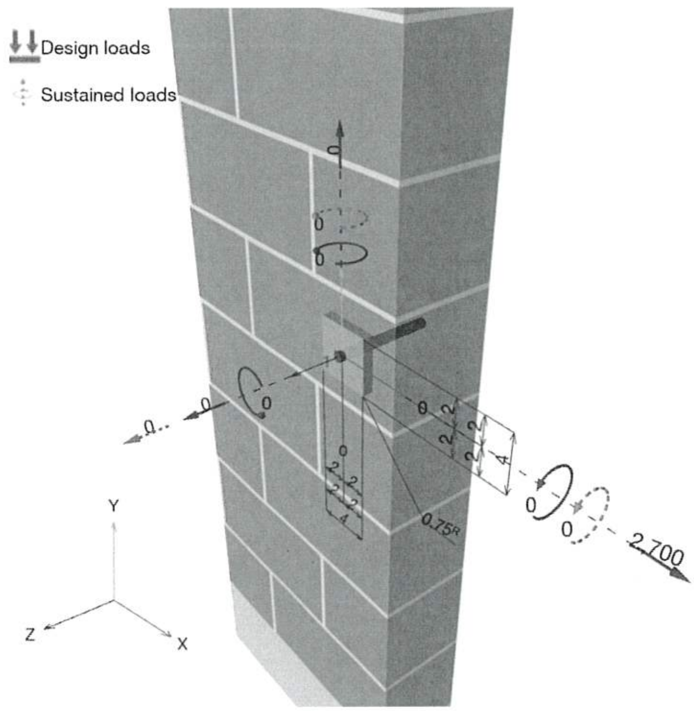
Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Single Anchor Capacity - OWTC Wall	Date:	3/7/2025
Fastening point:			

Geometry [in.]



Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Single Anchor Capacity - OWTC Wall	Date:	3/7/2025
Fastening point:			

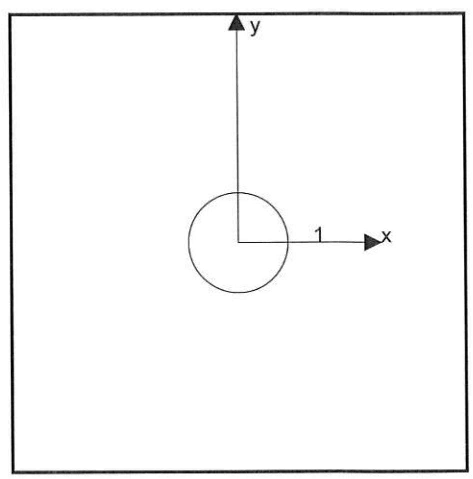
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0$; $V_x = 2{,}700$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$; $N_{sus} = 0$; $M_{x,sus} = 0$; $M_{y,sus} = 0$;	yes	99

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	2,700	2,700	0

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 0 [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Shear load

	Load [lb]	Capacity [lb]	Utilization β_V [%]	Status
Steel	2,700	6,111	45	OK
Pryout bond	2,700	3,881	70	OK
Masonry crushing strength	2,700	4,141	66	OK
Masonry breakout in direction x+	2,700	2,732	99	OK

3.1 Steel strength

V_{sa} = ESR value — refer to ICC-ES ESR-4143

$\phi\ V_{sa} \geq V_{ua}$ — AC58 Table 3.2 + ACI 318-19 Table 17.5.2

$V_{sa,seismic}$ [lb]	ϕ	$\phi_{seismic}$	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
10,184	0.600	1.000	6,111	2,700

3.2 Pryout strength (Bond strength controls)

$V_{mpg} = \min(k_{cp} \cdot N_{mag}, k_{cp} \cdot N_{mbg})$

$\phi\ V_{mpg} \geq V_{ua}$ — AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ — ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ — ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e_N}{c_{Na}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{uncr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ — ACI 318-19 Eq. (17.6.5.2.1)

Variables

k_{cp}	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]
2	0.750	6.000	4.000	200

α_{sat}	α_{top}	$\alpha_{V,seis}$	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a
1.000	1.000	0.980	0.000	0.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$	$\psi_{ec1,Na}$	$\psi_{ec2,Na}$
3.184	40.55	40.55	1.000	1.000	1.000

$\psi_{cp,Na}$	$N_{ba,m}$ [lb]
1.000	2,772

Results

V_{mpg} [lb]	ϕ	$\phi_{seismic}$	$\phi\ V_{mpg}$ [lb]	V_{ua} [lb]
5,544	0.700	1.000	3,881	2,700

3.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot \left(f'_m \cdot A_{se,V}\right)^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,v}$ [in.^{2}]
1,500	0.33

Results

V_{mc} [lb]	ϕ	$\phi_{seismic}$	$\phi\ V_{mc}$ [lb]	V_{ua} [lb]
8,282	0.500	1.000	4,141	2,700

3.4 Masonry breakout strength x+

$V_{mb} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \psi_{ed,V,m}\ \psi_{m,V}\ \psi_{h,V,m}\ \psi_{parallel,V}\ V_{b,m}$ ACI 318-19 Eq. (17.7.2.1a)

$\phi\ V_{mb} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Vm} see ACI 318-19 , Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vm0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V,m} = \left(\frac{1}{1 + \frac{2e'_V}{3c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V,m} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V,m} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_{b,m} = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_m}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
6.000	0.750	4.000	36.000	72.00	72.00	1,500

Calculations

$\psi_{ed,V,m}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V,m}$	$\psi_{m,V}$	$\psi_{h,V,m}$
1.000	1.000	0.000	1.000	1.400	1.000

Results

$V_{b,m}$ [lb]	ϕ	$\phi_{seismic}$	$\phi\ V_{mbg}$ [lb]	V_{ua} [lb]
2,788	0.700	1.000	2,732	2,700

COLUMN ANCHORAGE:

MAX SHEAR (PARALLEL TO BRIDGE SPAN):

$V_u = 39.6\ k$ (INCLUDES $\rho = 1.3$)

MAX SHEAR (PERPENDICULAR TO THE BRIDGE SPAN):

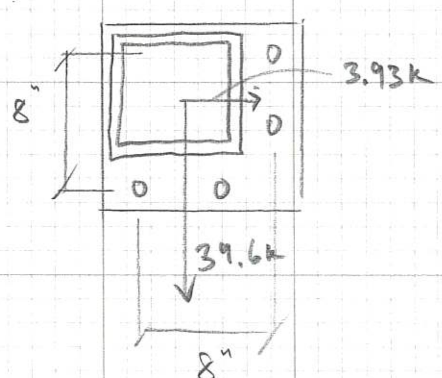
$V_u = 13.1\ k$

USE ORTHOGONAL EFFECTS:

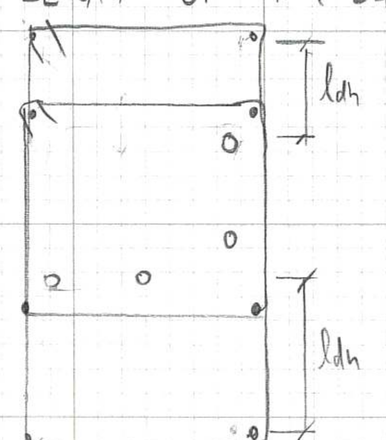
$V_{ux} = 39.6\ k$

$V_{uy} = 0.3 \times 13.1\ k = 3.93\ k$

CONFINE THE ANCHORS FOR SHEAR.



HOOK LENGTH OF #4 BAR IN 4,500 psi CONCRETE = 9"



(3) #4 CONFINING BARS

$= 0.9 \times 60\ ksi \times 0.6\ in^2 \times 2$

$= 65k > 40\ k \therefore$ OK

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Apr 1, 2025	Date:	4/1/2025
Fastening point:			

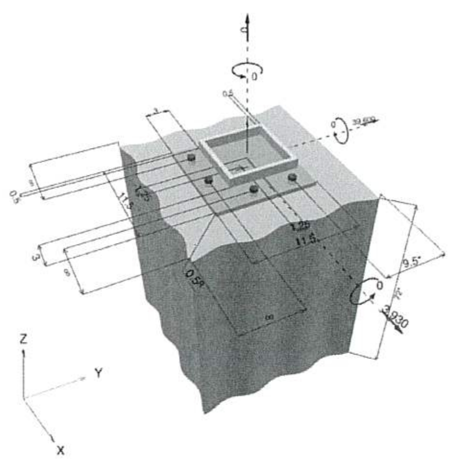
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 105 7/8**
Item number:	not available
Specification text:	∅ 7/8 in Hex Head ASTM F 1554 GR. 105 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-14 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-14 Fig. R.17.5.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 11.500 in. x 11.500 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS8X8X.500; (L x W x T) = 8.000 in. x 8.000 in. x 0.500 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 72.000 in.
Reinforcement:	tension: condition B, shear: condition A; anchor reinforcement: shear edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d)) Shear load: yes (17.2.3.5.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Apr 1, 2025	Date:	4/1/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 3,930; V_y = 39,600; M_x = 0; M_y = 0; M_z = 0;	yes	76

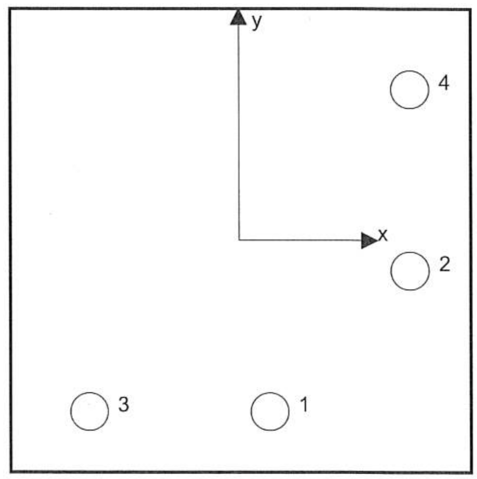
2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	11,092	-2,821	10,727
2	0	6,359	1,809	6,097
3	0	16,917	-2,821	16,680
4	0	9,870	7,762	6,097

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]



Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity Φ N_n [lb]	Utilization $\beta_N = N_{ua}/\Phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Apr 1, 2025	Date:	4/1/2025
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	16,917	22,522	76	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	39,795	126,147	32	OK
Concrete edge failure in direction **[1]	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

[1] Shear Anchor Reinforcement has been selected!

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.46	125,001

Calculations

V_{sa} [lb]
34,650

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
34,650	0.650	22,522	16,917

4.2 Pryout Strength

$$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b \right]$$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}} \right) \leq 1.0$$ ACI 318-14 Eq. (17.4.2.4)

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$$ ACI 318-14 Eq. (17.4.2.5b)

$$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = 16\, \lambda_a \sqrt{f_c'}\, h_{ef}^{5/3}$ ACI 318-14 Eq. (17.4.2.2b)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	18.000	2.857	0.284	9.500

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,746.75	2,916.00	0.904	0.990	0.806	1.000	132,693

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
180,210	0.700	1.000	1.000	126,147	39,795

6 Installation data

Profile: Square HSS (AISC), HSS8X8X.500; (L x W x T) = 8.000 in. x 8.000 in. x 0.500 in.

Hole diameter in the fixture: d_f = 0.938 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 105 7/8

Item number: not available

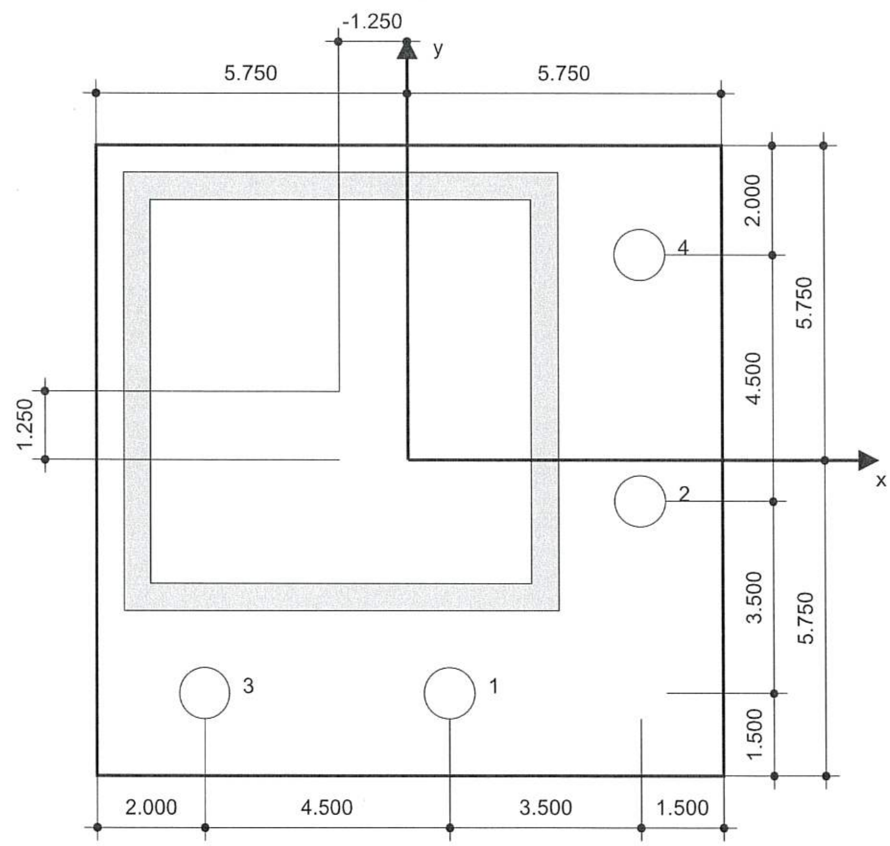
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.052 in.

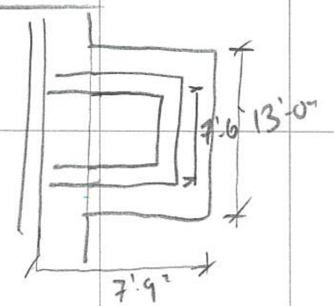
∅ 7/8 in Hex Head ASTM F 1554 GR. 105 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII

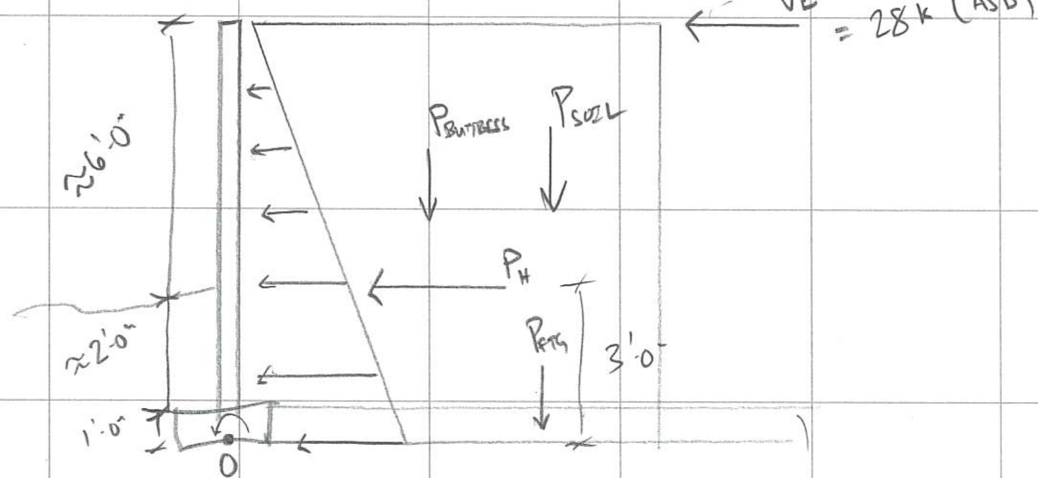


Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.750	-4.250	-	-	-	17.500
2	4.250	-0.750	-	-	-	14.000
3	-3.750	-4.250	-	-	-	17.500
4	4.250	3.750	-	-	-	9.500

BUTTRESS WALL:





$P_D = 13.4\ k$

$P_{SOIL} = 115\ pcf \times 8'\text{-}0" \times \left(\frac{13'\text{-}0"}{2}\right) \times 7.75' = 46.3\ k$

$P_{BUTTRESS} = (150\ pcf - 115\ pcf)\ 1'\text{-}0" \times 6'\text{-}0" \times 8'\text{-}0" = 1.7\ k$

$P_{FTG} = 150\ pcf \times 7.75' \times 13'/2 \times 1'\text{-}0" = 7.6\ k$

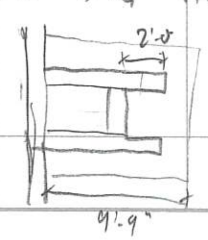
USE 40 psf ACTIVE PRESSURE:

$P_H = \left[\frac{1}{2} \times 40\ psf \times 9'\text{-}0" \times 9'\text{-}0"\right] \times \left[7'\text{-}6"/2 + 9'\text{-}0"\right] = 20.7\ k$

M_R ABOUT POINT O $= 7.6k \times \frac{7.75'}{2} + 1.7k \times \frac{6'\text{-}0"}{2} + 46.3k \times 7.75'/2$
$+ 13.4k \times 6'\text{-}0"$
$= 294.4\ k\text{-}ft$

$M_{OT} = 20.7k \times 3'\text{-}0" + 28k \times 9'\text{-}0" = 314\ k\text{-}ft$

EXTEND FTG AND BUTTRESS 2'-0" EXTRA



$P_{SOIL} = 58.3\ k$
$P_{BUTTRESS} = 2.24\ k$
$P_{FTG} = 9.5\ k$

$M_R = 420.9\ k\text{-}ft$
$F.O.S = 1.34 > 1.1 \therefore$ OK

ARW ENGINEERS structural consultants

Project Number 23358.A | Sheet Number
Project OWTC BRIDGE REPLACEMENT
Prepared By JWM | Date 3/28/2025

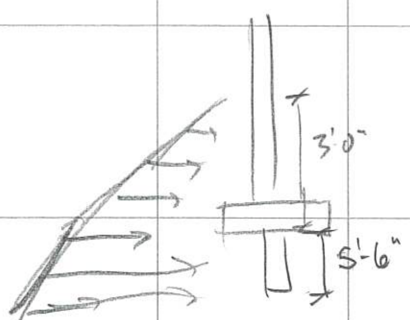
BUTTRESS WALL - CONT'D :

CHECK SLIDING :

$V_{TOTAL} = 20.7k + 28k = 49k$

$FRICTION = 0.3(58.3k + 2.24k + 9.5k + 13.4k)$
$= 25.0k << 49k$ ∴ ADD KEY

TRY 300 psf/ft PASSIVE W/5'-6" KEY :



$P_R = 1/2 \times (300\,pcf \times 7'6" \times 7'-6" \times (\frac{13'-0"}{2})) = 54.8k$

$F.O.S. = \frac{54.8k}{49k} = 1.11 > 1.1$ ∴ OK

REQ'D THICKNESS : TRY 12" THICK

$\phi V_c = 2\sqrt{3{,}000} \times 9" \times 12" = 11.8\,k/ft$

$\phi V_{c\,TOTAL} = (\frac{13'}{2}) \times 11.7\,k/ft = 76.1k$ ∴ 12" OK

BENDING FORCE

$M_u = 900\,psf \times 4'-6" \times (\frac{4'-6"}{2})$
$+ 1/2 \times (2{,}250\,pcf - 900\,pcf) \times 4'-6" \times (\frac{4'-6"}{3} \cdot 2)$
$= 18.2\,k\text{-}ft/ft$

TRY #5 @ 12" o.c.

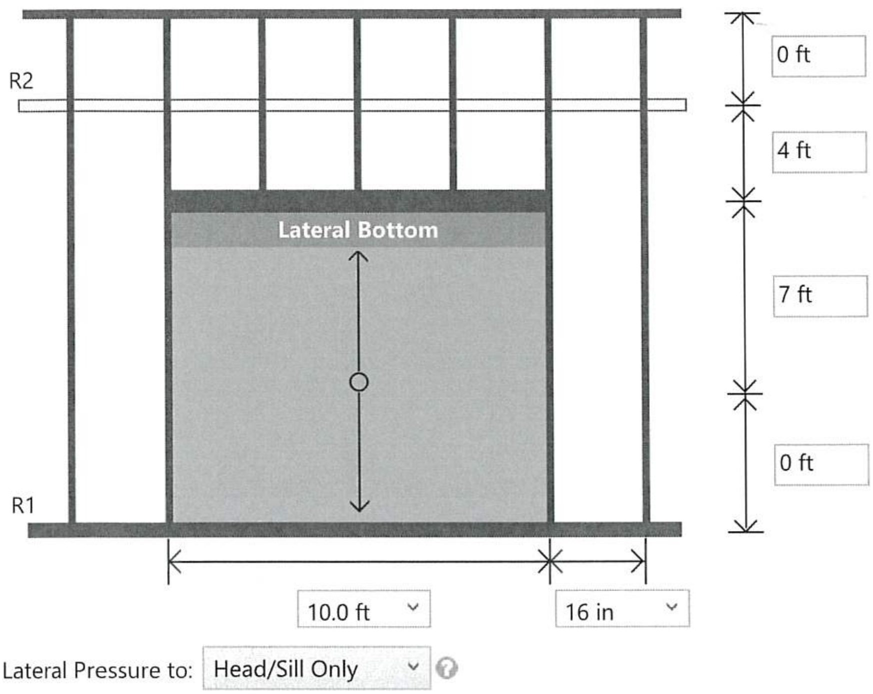
$\phi M_n = 0.9 \times 0.31\,in^2 \times 60\,ksi \times (10" - \frac{0.31\,in^2 \times 60\,ksi}{2 \times 0.85 \times 3\,ksi \times 12"})$
$= 12.2\,k\text{-}ft < 25.2\,k\text{-}ft/ft$

TRY #6 @ 8" o.c => $A_s = 0.66\,in^2$

$\phi M_n = 28.0\,k\text{-}ft > 25.2\,k\text{-}ft/ft$ ∴ #6 @ 8" o.c WORKS FOR SHEAR KEY.

Project Name: Metal Studs
Model: 10' Opening
Code: 2012 NASPEC [AISI S100-2012]

Date: 04/08/2025
Simpson Strong-Tie® CFS Designer™ 5.0.0.1



Design Loads

Wall Lateral Pressure :	**21.12 psf**
Parapet Lateral Pressure :	
RO Lateral Pressure :	**Head/Sill Only**
Lateral element force multiplier	
Strength :	**1.0**
Deflection :	**0.7**
Header:	**Box (lateral top, bottom)**
Gravity Load at Header:	**10 psf**

Back-to-Back Member L/6 Interconnection Spacing per AISI S100 D1.1

Member	Span	Cantilever
Jamb Studs	22.0 in	0.0 in

See AISI S100 D1.1 for Add'nl Requirements

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Wall Studs	400S162-43(33), Single@16 in o/c	60 in	60 in	60 in	0	None	N/A
Jamb Studs	400S162-54(50), Back-To-Back	60 in	60 in	60 in	0	None	12 in
Vertical Header	600S162-54(50), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	400T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	400S162-43(33), Single@16 in o/c	0.0	N/A	425.9	154.9	154.9	154.9
Jamb Studs	400S162-54(50), Back-To-Back	200.0	84	1675.5	447.0	658.2	447.0
Vertical Header	600S162-54(50), Boxed	N/A	N/A	500.0	200.0	N/A	200.0
Lat. Top Head	400T125-54(50), Single	N/A	N/A	528.0	211.2	N/A	211.2
Lat. Bottom Head	600T125-54(50), Single	N/A	N/A	924.0	369.6	N/A	369.6

Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	A + M Interaction	V + M Interaction	Web Stiffners	Design OK
Wall Studs	400S162-43(33), Single@16 in o/c	L/535	L/0	0.689	0.09	NA	Yes
Jamb Studs	400S162-54(50), Back-To-Back	L/410	L/0	0.724	0.69	NA	Yes
Vertical Header	600S162-54(50), Boxed	L/2250	NA	0.10	0.10	No	Yes
Lat. Top Head	400T125-54(50), Single	L/452	NA	0.59	0.59	No	Yes
Lat. Bottom Head	600T125-54(50), Single	L/681	NA	0.63	0.14	No	Yes

Simpson Strong-Tie® Connectors @ Studs

Connector	Anchor

Project Name: Metal Studs

Model: 10' Opening

Date: 04/08/2025

Code: 2012 NASPEC [AISI S100-2012]

Simpson Strong-Tie® CFS Designer™ 5.0.0.1

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Interaction	Interaction
R2	154.88	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	25.39 %	13.89 %
R1	154.88	146.67	FCB43.5 Min(4#12-14) & (2) #12-24 SST X or XL to A36 Steel	28.98 %	37.36 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	447.04	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	58.82 %	40.09 %
R1	658.24	320.00	(2) FCB43.5 Min(4#12-14) & (2) #12-24 SST X or XL to A36 Steel	44.49 %	55.12 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	60	N/A	0.0	No Soln	No Soln	No Soln	No Soln	No Soln	No Soln

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	60	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.

SSMA

Curtain Wall Limiting Heights - Single Span

Section	Fy (ksi)	Spacing (in) oc	5 psf			15 psf			20 psf			25 psf			30 psf			35 psf			40 psf			50 psf		
			L/120	L/240	L/360	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600
400S137-43	33	12	27' 4"	21' 8"	18' 11"	16' 11"	14' 9"	12' 6"	15' 4"	13' 5"	11' 4"	13' 9"	12' 6"	10' 6"	12' 7"	11' 9"	9' 11"	11' 7"	11' 2"	9' 5"	10' 10"	10' 8"	9' 0"	9' 9"	9' 9"	8' 4"
		16	24' 10"	19' 8"	17' 2"	15' 4"	13' 5"	11' 4"	13' 4"	12' 2"	10' 4"	11' 11"	11' 4"	9' 7"	10' 10"	10' 8"	9' 0"	10' 1"	10' 1"	8' 6"	9' 5"	9' 5"	8' 2"	8' 5"e	8' 5"e	7' 7"
		24	21' 8"	17' 2"	15' 0"	12' 7"	11' 9"	9' 11"	10' 10"	10' 8"	9' 0"	9' 9"	9' 9"	8' 4"	8' 11"	8' 11"	7' 10"	8' 3"e	8' 3"e	7' 6"	7' 8"e	7' 8"e	7' 2"e	6' 11"e	6' 11"e	6' 7"e
400S162-43	33	12	28' 7"	22' 8"	19' 10"	17' 9"	15' 6"	13' 1"	16' 1"	14' 1"	11' 10"	14' 10"	13' 1"	11' 0"	13' 6"	12' 3"	10' 4"	12' 6"	11' 8"	9' 10"	11' 9"	11' 2"	9' 5"	10' 6"	10' 4"	8' 9"
		16	26' 0"	20' 7"	18' 0"	16' 1"	14' 1"	11' 10"	14' 4"	12' 9"	10' 9"	12' 10"	11' 10"	10' 0"	11' 9"	11' 2"	9' 5"	10' 10"	10' 7"	8' 11"	10' 2"	10' 2"	8' 7"	9' 1"e	9' 1"e	7' 11"
		24	22' 8"	18' 0"	15' 9"	13' 6"	12' 3"	10' 4"	11' 9"	11' 2"	9' 5"	10' 6"	10' 4"	8' 9"	9' 7"e	9' 7"e	8' 3"	8' 10"e	8' 10"e	7' 10"	8' 3"e	8' 3"e	7' 6"e	7' 5"e	7' 5"e	6' 11"e
400S200-43	33	12	30' 2"	23' 11"	20' 11"	18' 8"	16' 4"	13' 9"	17' 0"	14' 10"	12' 6"	15' 9"	13' 9"	11' 7"	14' 6"	13' 0"	10' 11"	13' 5"	12' 4"	10' 5"	12' 7"	11' 9"	9' 11"	11' 3"e	10' 11"	9' 3"
		16	27' 5"	21' 9"	19' 0"	17' 0"	14' 10"	12' 6"	15' 4"	13' 6"	11' 4"	13' 9"	12' 6"	10' 7"	12' 7"	11' 9"	9' 11"	11' 7"	11' 2"	9' 5"	10' 10"e	10' 8"e	9' 0"	9' 9"e	9' 9"e	8' 5"e
		24	23' 11"	19' 0"	16' 7"	14' 6"	13' 0"	10' 11"	12' 7"	11' 9"	9' 11"	11' 3"e	10' 11"	9' 3"	10' 3"e	10' 3"e	8' 8"	9' 6"e	9' 6"e	8' 3"e	8' 10"e	8' 10"e	7' 11"e	7' 11"e	7' 11"e	7' 4"e
400S137-54	50	12	29' 3"	23' 2"	20' 3"	18' 1"	15' 10"	13' 4"	16' 6"	14' 5"	12' 2"	15' 3"	13' 4"	11' 3"	14' 5"	12' 7"	10' 7"	13' 8"	11' 11"	10' 1"	13' 1"	11' 5"	9' 8"	12' 2"	10' 7"	8' 11"
		16	26' 7"	21' 1"	18' 5"	16' 6"	14' 5"	12' 2"	15' 0"	13' 1"	11' 0"	13' 11"	12' 2"	10' 3"	13' 1"	11' 5"	9' 8"	12' 5"	10' 10"	9' 2"	11' 10"	10' 4"	8' 9"	11' 0"	9' 8"	8' 1"
		24	23' 2"	18' 5"	16' 1"	14' 5"	12' 7"	10' 7"	13' 1"	11' 5"	9' 8"	12' 2"	10' 7"	8' 11"	11' 5"	10' 0"	8' 5"	10' 10"	9' 6"	8' 0"	10' 4"	9' 1"	7' 8"	9' 3"	8' 5"	7' 1"
400S162-54	50	12	30' 8"	24' 4"	21' 3"	19' 0"	16' 7"	14' 0"	17' 3"	15' 1"	12' 9"	16' 0"	14' 0"	11' 10"	15' 1"	13' 2"	11' 1"	14' 4"	12' 6"	10' 7"	13' 8"	12' 0"	10' 1"	12' 9"	11' 1"	9' 4"
		16	27' 10"	22' 1"	19' 4"	17' 3"	15' 1"	12' 9"	15' 8"	13' 8"	11' 7"	14' 7"	12' 9"	10' 9"	13' 8"	12' 0"	10' 1"	13' 0"	11' 4"	9' 7"	12' 5"	10' 10"	9' 2"	11' 7"	10' 1"	8' 6"
		24	24' 4"	19' 4"	16' 10"	15' 1"	13' 2"	11' 1"	13' 8"	12' 0"	10' 1"	12' 9"	11' 1"	9' 4"	12' 0"	10' 5"	8' 10"	11' 4"	9' 11"	8' 5"	10' 10"	9' 6"	8' 0"	10' 0"	8' 10"	7' 5"
400S200-54	50	12	32' 4"	25' 8"	22' 5"	20' 1"	17' 6"	14' 9"	18' 3"	15' 11"	13' 5"	16' 11"	14' 9"	12' 6"	15' 11"	13' 11"	11' 9"	15' 1"	13' 3"	11' 2"	14' 6"	12' 8"	10' 8"	13' 5"	11' 9"	9' 11"
		16	29' 5"	23' 4"	20' 5"	18' 3"	15' 11"	13' 5"	16' 7"	14' 6"	12' 2"	15' 4"	13' 5"	11' 4"	14' 6"	12' 8"	10' 8"	13' 9"	12' 0"	10' 1"	13' 2"	11' 6"	9' 8"	12' 2"	10' 8"	9' 0"
		24	25' 8"	20' 5"	17' 10"	15' 11"	13' 11"	11' 9"	14' 6"	12' 8"	10' 8"	13' 5"	11' 9"	9' 11"	12' 8"	11' 0"	9' 4"	12' 0"	10' 6"	8' 10"	11' 6"	10' 0"	8' 5"	10' 6"	9' 4"	7' 10"
400S137-68	50	12	31' 3"	24' 10"	21' 8"	19' 4"	16' 11"	14' 3"	17' 7"	15' 5"	13' 0"	16' 4"	14' 3"	12' 0"	15' 5"	13' 5"	11' 4"	14' 7"	12' 9"	10' 9"	14' 0"	12' 2"	10' 4"	13' 0"	11' 4"	9' 7"
		16	28' 5"	22' 7"	19' 8"	17' 7"	15' 5"	13' 0"	16' 0"	14' 0"	11' 9"	14' 10"	13' 0"	10' 11"	14' 0"	12' 2"	10' 4"	13' 3"	11' 7"	9' 9"	12' 8"	11' 1"	9' 4"	11' 9"	10' 4"	8' 8"
		24	24' 10"	19' 8"	17' 2"	15' 5"	13' 5"	11' 4"	14' 0"	12' 2"	10' 4"	13' 0"	11' 4"	9' 7"	12' 2"	10' 8"	9' 0"	11' 7"	10' 2"	8' 7"	11' 1"	9' 8"	8' 2"	10' 4"	9' 0"	7' 7"
400S162-68	50	12	32' 10"	26' 0"	22' 9"	20' 4"	17' 9"	15' 0"	18' 6"	16' 2"	13' 7"	17' 2"	15' 0"	12' 8"	16' 2"	14' 1"	11' 11"	15' 4"	13' 5"	11' 4"	14' 8"	12' 10"	10' 10"	13' 7"	11' 11"	10' 0"
		16	29' 10"	23' 8"	20' 8"	18' 6"	16' 2"	13' 7"	16' 9"	14' 8"	12' 4"	15' 7"	13' 7"	11' 6"	14' 8"	12' 10"	10' 10"	13' 11"	12' 2"	10' 3"	13' 4"	11' 8"	9' 10"	12' 4"	10' 10"	9' 1"
		24	26' 0"	20' 8"	18' 1"	16' 2"	14' 1"	11' 11"	14' 8"	12' 10"	10' 10"	13' 7"	11' 11"	10' 0"	12' 10"	11' 2"	9' 5"	12' 2"	10' 8"	9' 0"	11' 8"	10' 2"	8' 7"	10' 10"	9' 5"	8' 0"
400S200-68	50	12	34' 8"	27' 6"	24' 0"	21' 6"	18' 9"	15' 10"	19' 6"	17' 1"	14' 5"	18' 1"	15' 10"	13' 4"	17' 1"	14' 11"	12' 7"	16' 2"	14' 2"	11' 11"	15' 6"	13' 6"	11' 5"	14' 5"	12' 7"	10' 7"
		16	31' 6"	25' 0"	21' 10"	19' 6"	17' 1"	14' 5"	17' 9"	15' 6"	13' 1"	16' 6"	14' 5"	12' 2"	15' 6"	13' 6"	11' 5"	14' 9"	12' 10"	10' 10"	14' 1"	12' 4"	10' 4"	13' 1"	11' 5"	9' 8"
		24	27' 6"	21' 10"	19' 1"	17' 1"	14' 11"	12' 7"	15' 6"	13' 6"	11' 5"	14' 5"	12' 7"	10' 7"	13' 6"	11' 10"	10' 0"	12' 10"	11' 3"	9' 6"	12' 4"	10' 9"	9' 1"	11' 5"	10' 0"	8' 5"
550S162-33	33	12	33' 8"	26' 9"	23' 4"	19' 7"	18' 3"	15' 5"	16' 11"e	16' 7"e	14' 0"	15' 2"e	15' 2"e	13' 0"e	13' 10"e	13' 10"e	12' 3"e	12' 10"e	12' 10"e	11' 7"e	12' 0"e	12' 0"e	11' 1"e	10' 9"e	10' 9"e	10' 4"e
		16	29' 4"	24' 4"	21' 3"	16' 11"e	16' 7"e	14' 0"	14' 8"e	14' 8"e	12' 8"e	13' 2"e	13' 2"e	11' 9"e	12' 0"e	12' 0"e	11' 1"e	11' 1"e	11' 1"e	10' 6"e	10' 5"e	10' 5"e	10' 1"e	9' 3"e	9' 3"e	9' 3"e
		24	24' 0"	21' 3"	18' 6"	13' 10"e	13' 10"e	12' 3"e	12' 0"e	12' 0"e	11' 1"e	10' 9"e	10' 9"e	10' 4"e	9' 9"e	9' 9"e	9' 8"e	9' 1"e	9' 1"e	9' 1"e	8' 6"e	8' 6"e	8' 6"e	7' 7"e	7' 7"e	7' 7"e
550S162-43	33	12	36' 8"	29' 1"	25' 5"	22' 9"	19' 10"	16' 9"	20' 8"	18' 1"	15' 3"	18' 9"	16' 9"	14' 2"	17' 1"	15' 9"	13' 4"	15' 10"e	15' 0"	12' 8"	14' 10"e	14' 4"e	12' 1"	13' 3"e	13' 3"e	11' 3"e
		16	33' 4"	26' 5"	23' 1"	20' 8"	18' 1"	15' 3"	18' 1"	16' 5"	13' 10"	16' 3"e	15' 3"	12' 10"	14' 10"e	14' 4"e	12' 1"	13' 8"e	13' 7"e	11' 6"e	12' 10"e	12' 10"e	11' 0"e	11' 6"e	11' 6"e	10' 2"e
		24	29' 1"	23' 1"	20' 2"	17' 1"	15' 9"	13' 4"	14' 10"e	14' 4"e	12' 1"	13' 3"e	13' 3"e	11' 3"e	12' 1"e	12' 1"e	10' 7"e	11' 2"e	11' 2"e	10' 0"e	10' 6"e	10' 6"e	9' 7"e	9' 4"e	9' 4"e	8' 11"e
550S162-54	50	12	39' 4"	31' 3"	27' 3"	24' 5"	21' 4"	18' 0"	22' 2"	19' 4"	16' 4"	20' 7"	18' 0"	15' 2"	19' 4"	16' 11"	14' 3"	18' 5"	16' 1"	13' 7"	17' 7"	15' 4"	13' 0"	16' 4"	14' 3"	12' 0"
		16	35' 9"	28' 5"	24' 9"	22' 2"	19' 4"	16' 4"	20' 2"	17' 7"	14' 10"	18' 8"	16' 4"	13' 9"	17' 7"	15' 4"	13' 0"	16' 8"	14' 7"	12' 4"	16' 0"	14' 0"	11' 9"	14' 10"	13' 0"	10' 11"
		24	31' 3"	24' 9"	21' 8"	19' 4"	16' 11"	14' 3"	17' 7"	15' 4"	13' 0"	16' 4"	14' 3"	12' 0"	15' 4"	13' 5"	11' 4"	14' 7"	12' 9"	10' 9"	14' 0"	12' 2"	10' 3"	12' 6"e	11' 4"	9' 7"
550S162-68	50	12	42' 2"	33' 6"	29' 3"	26' 2"	22' 10"	19' 3"	23' 9"	20' 9"	17' 6"	22' 1"	19' 3"	16' 3"	20' 9"	18' 2"	15' 3"	19' 9"	17' 3"	14' 6"	18' 10"	16' 6"	13' 11"	17' 6"	15' 3"	12' 11"
		16	38' 4"	30' 5"	26' 7"	23' 9"	20' 9"	17' 6"	21' 7"	18' 10"	15' 11"	20' 0"	17' 6"	14' 9"	18' 10"	16' 6"	13' 11"	17' 11"	15' 8"	13' 2"	17' 2"	15' 0"	12' 7"	15' 11"	13' 11"	11' 9"
		24	33' 6"	26' 7"	23' 3"	20' 9"	18' 2"	15' 3"	18' 10"	16' 6"	13' 11"	17' 6"	15' 3"	12' 11"	16' 6"	14' 5"	12' 2"	15' 8"	13' 8"	11' 6"	15' 0"	13' 1"	11' 0"	13' 11"	12' 2"	10' 3"
600S137-33	33	12	33' 0"	27' 6"	24' 0"	19' 1"	18' 9"	15' 10"	16' 6"e	16' 6"e	14' 4"	14' 9"e	14' 9"e	13' 4"e	13' 6"e	13' 6"e	12' 7"e	12' 6"e	12' 6"e	11' 11"e	11' 8"e	11' 8"e	11' 5"e	10' 5"e	10' 5"e	10' 5"e
		16	28' 7"	24' 11"	21' 10"	16' 6"e	16' 6"e	14' 4"	14' 4"e	14' 4"e	13' 1"e	12' 10"e	12' 10"e	12' 1"e	11' 8"e	11' 8"e	11' 5"e	10' 10"e	10' 10"e	10' 10"e	10' 1"e	10' 1"e	10' 1"e	9' 1"e	9' 1"e	9' 1"e
		24	23' 4"	21' 9"	19' 1"	13' 6"e	13' 6"e	12' 7"e	11' 8"e	11' 8"e	11' 5"e	10' 5"e	10' 5"e	10' 5"e	9' 6"e	9' 6"e	9' 6"e	8' 10"e	8' 10"e	8' 10"e	8' 3"e	8' 3"e	8' 3"e	7' 5"e	7' 5"e	7' 5"e
600S162-33	33	12	35' 6"	28' 8"	25' 0"	20' 6"e	19' 6"	16' 6"	17' 9"e	17' 9"e	15' 0"	15' 11"e	15' 11"e	13' 11"e	14' 6"e	14' 6"e	13' 1"e	13' 5"e	13' 5"e	12' 5"e	12' 7"e	12' 7"e	11' 11"e	11' 3"e	11' 3"e	11' 0"e
		16	30' 9"	26' 0"	22' 9"	17' 9"e	17' 9"e	15' 0"	15' 5"e	15' 5"e	13' 7"e	13' 9"e	13' 9"e	12' 8"e	12' 7"e	12' 7"e	11' 11"e	11' 8"e	11' 8"e	11' 3"e	10' 11"e	10' 11"e	10' 10"e	9' 9"e	9' 9"e	9' 9"e
		24	25' 1"	22' 9"	19' 10"	14' 6"e	14' 6"e	13' 1"e	12' 7"e	12' 7"e	11' 11"e	11' 3"e	11' 3"e	11' 0"e	10' 3"e	10' 3"e	10' 3"e	9' 6"e	9' 6"e	9' 6"e	8' 11"e	8' 11"e	8' 11"e	7' 11"e	7' 11"e	7' 11"e
600S200-33	33	12	37' 11"	30' 1"	26' 3"	21' 10"e	20' 6"e	17' 4"	18' 11"e	18' 8"e	15' 9"e	16' 11"e	16' 11"e	14' 7"e	15' 6"e	15' 6"e	13' 9"e	14' 4"e	14' 4"e	13' 1"e	13' 5"e	13' 5"e	12' 6"e	12' 0"e	12' 0"e	11' 7"e
		16	32' 10"	27' 4"	23' 10"	18' 11"e	18' 8"e	15' 9"e	16' 5"e	16' 5"e	14' 3"e	14' 8"e	14' 8"e	13' 3"e	13' 5"e	13' 5"e	12' 6"e	12' 5"e	12' 5"e	11' 10"e	11' 7"e	11' 7"e	11' 4"e	10' 4"e	10' 4"e	10' 4"e
		24	26' 9"	23' 10"	20' 10"	15' 6"e	15' 6"e	13' 9"e	13' 5"e	13' 5"e	12' 6"e	12' 0"e	12' 0"e	11' 7"e	10' 11"e	10' 11"e	10' 11"e	10' 1"e	10' 1"e	10' 1"e	9' 6"e	9' 6"e	9' 6"e	8' 6"e	8' 6"e	8' 6"e
600S137-43	33	12	37' 8"	29' 11"	26' 2"	22' 11"	20' 5"	17' 3"	19' 10"	18' 6"	15' 8"	17' 9"	17' 3"	14' 6"	16' 2"	16' 2"	13' 8"	15' 0"e	15' 0"e	13' 0"	14' 0"e	14' 0"e	12' 5"	12' 7"e	12' 7"e	11' 6"e
		16	34' 3"	27' 2"	23' 9"	19' 10"	18' 6"	15' 8"	17' 2"	16' 10"	14' 2"	15' 5"	15' 5"	13' 2"	14' 0"	14' 0"e	12' 5"	13' 0"e	13' 0"e	11' 9"e	12' 2"e	12' 2"e	11' 3"e	10' 10"e	10' 10"e	10' 6"e
		24	28' 1"	23' 9"	20' 9"	16' 2"	16' 2"	13' 8"	14' 0"e	14' 0"e	12' 5"	12' 7"e	12' 7"e	11' 6"e	11' 6"e	11' 6"e	10' 10"e	10' 7"e	10' 7"e	10' 4"e	9' 11"e	9' 11"e	9' 10"e	8' 11"e	8' 11"e	8' 11"e
600S162-43	33	12	39' 4"	31' 2"	27' 3"	24' 4"	21' 3"	17' 11"	21' 11"	19' 4"	16' 4"	19' 8"	17' 11"	15' 2"	17' 11"e	16' 11"	14' 3"	16' 7"e	16' 1"e	13' 6"	15' 6"e	15' 4"e	12' 11"	13' 11"e	13' 11"e	12' 0"e
		16	35' 9"	28' 4"	24' 9"	21' 11"	19' 4"	16' 4"	19' 0"	17' 7"	14' 10"	17' 0"e	16' 4"e	13' 9"	15' 6"e	15' 4"e	12' 11"	14' 4"e	14' 4"e	12' 4"e	13' 5"e	13' 5"e	11' 9"e	12' 0"e	12' 0"e	10' 11"e
		24	31' 1"	24' 9"	21' 8"	17' 11"e	16' 11"	14' 3"	15' 6"e	15' 4"e	12' 11"	13' 11"e	13' 11"e	12' 0"e	12' 8"e	12' 8"e	11' 4"e	11' 9"e	11' 9"e	10' 9"e	11' 0"e	11' 0"e	10' 3"e	9' 10"e	9' 10"e	9' 6"e
600S200-43	33	12	41' 3"	32' 9"	28' 7"	25' 7"	22' 4"	18' 10"	22' 8"	20' 4"	17' 2"	20' 3"	18' 10"	15' 11"	18' 6"e	17' 9"e	15' 0"	17' 1"e	16' 10"e	14' 3"	16' 0"e	16' 0"e	13' 7"e	14' 4"e	14' 4"e	12' 7"e
		16	37' 6"	29' 9"	26' 0"	22' 8"	20' 4"	17' 2"	19' 7"e	18' 5"	15' 7"	17' 7"e	17' 2"e	14' 5"	16' 0"e	16' 0"e	13' 7"e	14' 10"e	14' 10"e	12' 11"e	13' 10"e	13' 10"e	12' 4"e	12' 5"e	12' 5"e	11' 6"e
		24	32' 0"	26' 0"	22' 9"	18' 6"e	17' 9"e	15' 0"	16' 0"e	16' 0"e	13' 7"e	14' 4"e	14' 4"e	12' 7"e	13' 1"e	13' 1"e	11' 11"e	12' 1"e	12' 1"e	11' 3"e	11' 4"e	11' 4"e	10' 9"e	10' 2"e	10' 2"e	10' 0"e
600S137-54	50	12	40' 5"	32' 1"	28' 0"	25' 1"	21' 11"	18' 5"	22' 9"	19' 11"	16' 9"	21' 2"	18' 5"	15' 7"	19' 11"	17' 4"	14' 8"	18' 11"	16' 6"	13' 11"	18' 1"	15' 9"	13' 4"	16' 9"	14' 8"	12' 4"
		16	36' 9"	29' 2"	25' 6"	22' 9"	19' 11"	16' 9"	20' 8"	18' 1"	15' 3"	19' 2"	16' 9"	14' 2"	18' 1"	15' 9"	13' 4"	17' 2"	15' 0"	12' 8"	16' 4"	14' 4"	12' 1"	14' 7"	13' 4"	11' 3"
		24	32' 1"	25' 6"	22' 3"	19' 11"	17' 4"	14' 8"	18' 1"	15' 9"	13' 4"	16' 9"	14' 8"	12' 4"	15' 4"	13' 9"	11' 8"	14' 3"	13' 1"	11' 1"	13' 4"	12' 6"	10' 7"	11' 11"	11' 8"	9' 10"
600S162-54	50	12	42' 2"	33' 6"	29' 3"	26' 2"	22' 10"	19' 3"	23' 9"	20' 9"	17' 6"	22' 1"	19' 3"	16' 3"	20' 9"	18' 1"	15' 3"	19' 8"	17' 3"	14' 6"	18' 10"	16' 6"	13' 11"	17' 6"	15' 3"	12' 11"
		16	38' 4"	30' 5"	26' 7"	23' 9"	20' 9"	17' 6"	21' 7"	18' 10"	15' 11"	20' 0"	17' 6"	14' 9"	18' 10"	16' 6"	13' 11"	17' 11"	15' 8"	13' 2"	17' 2"	15' 0"	12' 7"	15' 11"	13' 11"	11' 9"
		24	33' 6"	26' 7"	23' 3"	20' 9"	18' 1"	15' 3"	18' 10"	16' 6"	13' 11"	17' 6"	15' 3"	12' 11"	16' 6"	14' 5"	12' 2"	15' 8"	13' 8"	11' 6"	14' 8"	13' 1"	11' 0"	13' 2"e	12' 2"e	10' 3"
600S200-54	50	12	44' 4"	35' 2"	30' 9"	27' 6"	24' 0"	20' 3"	24' 11"	21' 10"	18' 5"	23' 2"	20' 3"	17' 1"	21' 10"	19' 1"	16' 1"	20' 9"	18' 1"	15' 3"	19' 10"	17' 4"	14' 7"	18' 5"	16' 1"	13' 7"
		16	40' 3"	32' 0"	27' 11"	24' 11"	21' 10"	18' 5"	22' 8"	19' 10"	16' 8"	21' 1"	18' 5"	15' 6"	19' 10"	17' 4"	14' 7"	18' 10"	16' 5"	13' 10"	18' 0"	15' 9"	13' 3"	16' 7"	14' 7"	12' 4"
		24	35' 2"	27' 11"	24' 5"	21' 10"	19' 1"	16' 1"	19' 10"	17' 4"	14' 7"	18' 5"	16' 1"	13' 7"	17' 4"	15' 1"	12' 9"	16' 2"	14' 4"	12' 1"	15' 1"e	13' 9"	11' 7"	13' 6"e	12' 9"e	10' 9"
600S137-68	50	12	43' 4"	34' 4"	30' 0"	26' 10"	23' 5"	19' 9"	24' 5"	21' 4"	18' 0"	22' 8"	19' 9"	16' 8"	21' 4"	18' 7"	15' 8"	20' 3"	17' 8"	14' 11"	19' 4"	16' 11"	14' 3"	18' 0"	15' 8"	13' 3"
		16	39' 4"	31' 3"	27' 3"	24' 5"	21' 4"	18' 0"	22' 2"	19' 4"	16' 4"	20' 7"	18' 0"	15' 2"	19' 4"	16' 11"	14' 3"	18' 5"	16' 1"	13' 7"	17' 7"	15' 4"	12' 11"	16' 4"	14' 3"	12' 0"
		24	34' 4"	27' 3"	23' 10"	21' 4"	18' 7"	15' 8"	19' 4"	16' 11"	14' 3"	18' 0"	15' 8"	13' 3"	16' 11"	14' 9"	12' 5"	16' 1"	14' 0"	11' 10"	15' 4"	13' 5"	11' 4"	13' 11"	12' 5"	10' 6"
600S162-68	50	12	45' 3"	35' 11"	31' 4"	28' 0"	24' 6"	20' 8"	25' 6"	22' 3"	18' 9"	23' 8"	20' 8"	17' 5"	22' 3"	19' 5"	16' 5"	21' 2"	18' 5"	15' 7"	20' 2"	17' 8"	14' 11"	18' 9"	16' 5"	13' 10"
		16	41' 1"	32' 7"	28' 6"	25' 6"	22' 3"	18' 9"	23' 2"	20' 2"	17' 1"	21' 6"	18' 9"	15' 10"	20' 2"	17' 8"	14' 11"	19' 2"	16' 9"	14' 2"	18' 4"	16' 0"	13' 6"	17' 1"	14' 11"	12' 7"
		24	35' 11"	28' 6"	24' 11"	22' 3"	19' 5"	16' 5"	20' 2"	17' 8"	14' 11"	18' 9"	16' 5"	13' 10"	17' 8"	15' 5"	13' 0"	16' 9"	14' 8"	12' 4"	16' 0"	14' 0"	11' 10"	14' 11"	13' 0"	11' 0"

"e" web stiffeners required at ends.

See Table Notes on page 25.