FOR REFERENCE ONLY

Revised Structural Calculations

77.5'x85' CANOPY

MVE #23-1096

REVISED PER REVIEW - NOVEMBER 9, 2023

TANK CANOPY
PORTLAND INTERNATIONAL AIRPORT
5000 NE Marine Drive, Portland, Oregon

NOV 09 2023

STRUCTURAL
REGISTERED PROFESSIONAL ENGINEER
95267PE
Brad Wallace
OREGON
SEP. 10, 2019
BRAD WALLACE

EXPIRES JUN 30 2024

Canopy Supplied by:

BESTWORTH ROMMEL LLC
19818 74th Ave. NE
Arlington, WA 98223

Structural Design by:

MOUNTAIN VIEW ENGINEERING, INC.
345 No. Main, Suite A • Brigham City, Utah 84302
Phone (435) 734-9700 • Fax (435) 734-9519

#363261

37 pages of Calculations

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Job: MVE #23-1096 B-R #363261
Subject: TANK CANOPY
Date: 11/09/23
By: CRH

CANOPY DESIGN CRITERIA

CANOPY SPECIFICATIONS:

Length	85	ft	Total Height of Canopy	24	ft max.
Width	77.5	ft	Number of Column Rows	4	
Fascia Height	3	ft	Number of Columns/Row	4	
Canopy Clear Height	21	ft max.	Site Elevation	95	ft

CODE: OSSC 2022

INCLUDES 3.4 KIPS FOR SUPPORTED PIPES

Dead and Live Loads:

Total Canopy Dead Load	9	psf	Canopy Area	6587.5	ft^2
Dead Load on Purlins	5	psf	Mansard Roof Area	0	ft^2
Mansard Dead Load	0	psf	Total Canopy Dead Load	62.69	kips
Ground Snow Load	11	psf **	Max. Column Trib. Width	22	ft
Roof Snow Load	13.2	psf	Max. Column Trib. Length	24	ft
Thermal Factor	1.2		Max. Column Trib. Area	528	ft^2
Importance Factor	1.2				
Exposure Factor	1.0				
Roof Live Load	20	psf			

Live Load Reduction per OSSC 1607.13.2.1
At = 528 ft^2 F = 0
R1 = 0.67 R2 = 1.00
Reduced Live Load (Lr) 13.4 psf
(for columns & footings)

***Rain on snow surcharge must be applied as per ASCE 7-16 Section 7.10, therefore 5 psf has been added to the roof snow load for the balanced load case only. Rain on snow surcharge need not be applied to drift or unbalanced loads.*

S_{D1} based on Fv = 1.925 for Site Class D as per ASCE 7-16 Table 11.4-2.

Earthquake Design Data:

Site Class				D	(default)
Seismic Design Category				D	
S_S	0.846	g	S_{DS}	0.677	g
S_1	0.375	g	S_{D1}	0.481	g
Importance Factor				1.5	

Wind Design Data:

Basic Wind Speed (V)	110	mph
ASD Wind Speed (V_{asd})	85	mph
Exposure	C	
Risk Category	IV	
Rainfall Intensity:	1.5	in/hr

The canopy is classified as cantilevered column system detailed to conform to the requirements for Steel Ordinary Cantilever Column Systems as per ASCE 7-16 Table 12.2-1 and has been designed using the Equivalent Lateral Force Procedure as per Section 12.8.

From ASCE 7-16 Section 12.8

T = Ct Hn^x = 0.207 TL = 6 from ASCE 7-16 Fig 22-14 R = 1.25
Ct = 0.02 x = 0.75 for T≤TL Cs max = 2.787 Cs min = 0.447
Hn = 22.58 ~~for T>TL Cs max = 80.72 Cs min = 0.225 for S1>0.6~~
Ts = SD1/SDS = 0.7 *T ≤ 1.5Ts, Therefore, site specific ground motion analysis not required.*
(Eqn. 12.8-2) Cs for design = 0.81 **OK**

For SDC = D, E, or F ***Ω and ρ need not be used in the same load combinations (ASCE 12.4).***
ρ = 1.3 ***Therefore, the canopy has been designed for Ω = ρ =*** **1.3**

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ASCE 7-16 WIND FORCES (CHAPTER 27 DIRECTIONAL PROCEDURE)

Open Building with Monoslope Roof (Section 27.3.2) with fascia panels as Parapets (Section 27.3.4)

Basic Wind Speed (V) =	110	mph
Exposure (Section 26.7) =	C	
Risk Category =	IV	
Canopy Clear Height =	21	ft
Fascia Height =	3	ft
Mean Roof Height =	21.333	ft
Kd (Table 26.6-1) =	0.85	
Wind Profile Area (As) =	255	ft^2
Site Elevation =	95	ft

Gust Effect Factor (Section 26.11)

The canopy's fundamental natural frequency is greater than 1 Hz, and is therefore rigid as defined in Section 26.2. Therfore, as per Section 26.11.1, G = 0.85

Ground Elevation Factor (Section 26.9)

Ke (Table 26.9-1) = 1.00

Note: *Topographic effects need not be applied, therefore Kht = Kpt = 1.0.*

Velocity Pressure (Section 26.10.2, Table 26.11-1)

α (Table 26.11-1) =	9.5		zg (Table 26.11-1) =	900	ft
For Open Buildings:			For Parapets (Fascia Panels):		
h =	21.333	ft	p =	24	ft
Kh = 2.01 (h/zg)^2/α =	0.914		Kp = 2.01 (p/zg)^2/α =	0.937	
q_h = 0.00256 (Kh) (Kht) (Kd) (Ke) (V^2)			q_p = 0.00256 (Kp) (Kpt) (Kd) (Ke) (V^2)		
Therefore, qh =	24.0	psf	Therefore, q_p =	24.6	psf

MWFRS Horizontal Forces (Section 27.3.4)

Top of Windward Fascia	p_1 =	24	ft	Top of Leeward Fascia	p_2 =	24	ft
Bottom of Windward Fascia	z_1 =	21	ft	Bottom of Leeward Fascia	z_2 =	21.33	ft
Windward Fascia Height	=	3	ft	Leeward Fascia Height	=	2.667	ft

Parapet Wind Pressure for MWFRS **$p_p = q_p GC_{pn}$**

From Section 27.3.4, Windward GC_{pn} = 1.5 Leeward GC_{pn} = -1.0

Windward Parapet Pressure = **36.9 psf** (i.e. towards fascia)

Leeward Parapet Pressure = **-24.6 psf** (i.e. away from fascia)

Canopy Length = **85 ft**

THEREFORE: Total Horizontal Force (F) = 14980 lbs = $\boxed{\textbf{14.98 kips}}$ (on fascia)

MWFRS Vertical Forces (Figure 27.3-4)

Clear Leeward or Windward Flow will control design (obstructions always < 50%).

B = 85 ft L = 77.5 ft Θ = 0 degrees

From Figure 27.3-4, Worst Case C_N = -1.1 Therefore, Design Uplift Pressure = -22.4 psf

From Figure 27.3-4, Worst Case C_N = 1.2 Therefore, Design Down Pressure = 24.5 psf

See the Unbalanced Loads page for MWFRS unbalanced wind loads on single row canopies (if applicable).

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ASCE 7-16 WIND FORCES (CHAPTER 27 DIRECTIONAL PROCEDURE) continued

Open Building with Monoslope Roof (Section 27.3.2) with fascia panels as Parapets (Section 27.3.4)

MWFRS Vertical Forces (Figure 27.3-4) continued

MWFRS Wind, Transverse (Figure 27.3-4)

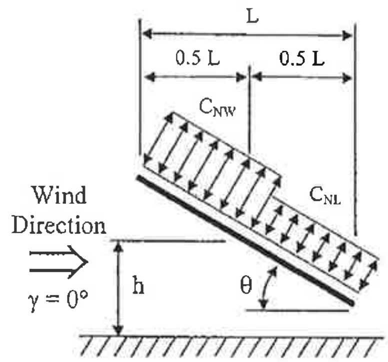
B =	85	ft	L =	77.5	ft
Θ =	0	°		0, 180	°

γ = 0 ° *Load Case A, Clear Wind Flow*

C_{NW} =	1.2	p =	24.5	psf
C_{NL} =	0.3	p =	6.1	psf

γ = 0 ° *Load Case B, Clear Wind Flow*

C_{NW} =	-1.1	p =	-22.4	psf
C_{NL} =	-0.1	p =	-2.0	psf

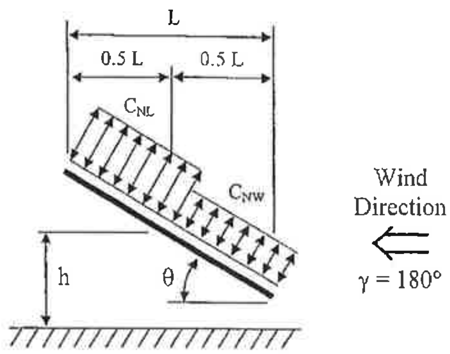


γ = 180 ° *Load Case A, Clear Wind Flow*

C_{NW} =	1.2	p =	24.5	psf
C_{NL} =	0.3	p =	6.1	psf

γ = 180 ° *Load Case B, Clear Wind Flow*

C_{NW} =	-1.1	p =	-22.4	psf
C_{NL} =	-0.1	p =	-2.0	psf



MWFRS Wind, Longitudinal (Figure 27.3-7)

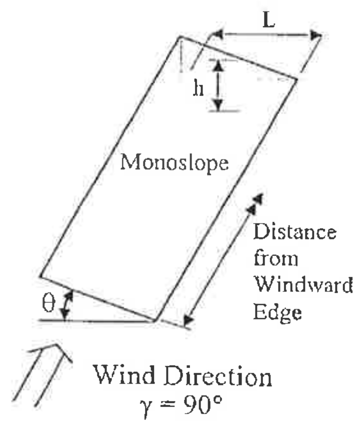
B =	85	ft	L =	77.5	ft
Θ =	0	°	γ =	90	°

Load Case A, Clear Wind Flow

For ≤ h	C_N =	-0.8	p =	-16.3	psf
For > h, ≤ 2h	C_N =	-0.6	p =	-12.2	psf
For > 2h	C_N =	-0.3	p =	-6.1	psf

Load Case B, Clear Wind Flow

For ≤ h	C_N =	0.8	p =	16.3	psf
For > h, ≤ 2h	C_N =	0.5	p =	10.2	psf
For > 2h	C_N =	0.3	p =	6.1	psf



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ASCE 7-16 WIND FORCES (CHAPTER 30 - COMPONENTS & CLADDING)

Part 5 - Open Buildings (Section 30.7)

G = 0.85 (from page 2)
q_h = 24.0 psf (from page 2)
q_p = 24.6 psf (from page 2)

h = 21.33 ft
B = 85 ft
L = 77.5 ft

Roof Component Pressure

Clear Wind Flow will control design (obstructions always < 50%).

Component Wind Pressure for Open Buildings **$p = q_h GC_N$**

Θ = 0 degrees

a = lesser of: { 10% of least horizontal dimension = 7.75 ft; 0.4h = 8.53 ft }

minimum: { 4% of least horizontal dimension = 3.10 ft; 3 ft = 3 ft }

Therefore:
a = 7.75 ft
a^2 = 60.1 sq.ft.
$4.0a^2$ = 240.3 sq.ft.

	Zone 3				Zone 2				Zone 1			
	C_N	p	C_N	p	C_N	p	C_N	p	C_N	p	C_N	p
$<a^2$	2.4	48.9	-3.3	-67.3	1.8	36.7	-1.7	-34.7	1.2	24.5	-1.1	-22.4
a^2 to $4.0a^2$	1.8	36.7	-1.7	-34.7	1.8	36.7	-1.7	-34.7	1.2	24.5	-1.1	-22.4
$> 4.0a^2$	1.2	24.5	-1.1	-22.4	1.2	24.5	-1.1	-22.4	1.2	24.5	-1.1	-22.4

Area of Single Deck Pan = 103.33 ft^2 ***>a^2 and <4.0a^2***
Max. Downward Component Pressure = **36.7** psf ULT = 48.9 plf ULT per deck pan
Max. Uplift Component Pressure = **-34.7** psf ULT = -46.2 plf ULT per deck pan

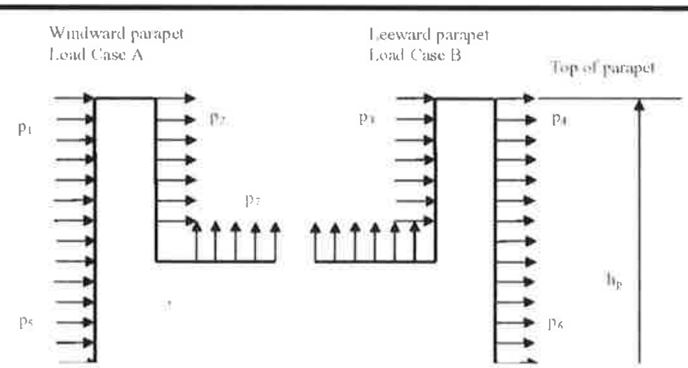
ASCE 7-16 WIND FORCES (CHAPTER 30 - COMPONENTS & CLADDING)

Part 6 - Parapets (Section 30.8)

Parapet Component Pressure

Component Wind Pressure for Parapets:
$p = q_p ((GC_p) - (GC_{pi}))$
GC_{pi} = 0 (Table 26.13-1)
Therefore, $p = q_p GC_p$

Fascia Height = 3 ft
Fascia Frame Spacing = **4** ft
Effective Wind Area = 12 ft^2



***See Fig. 30.3-2A Footnote 5 and 30.5-1 Footnote 7.*

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ASCE 7-16 WIND FORCES (CHAPTER 30 - COMPONENTS & CLADDING) continued

Part 6 - Parapets (Section 30.8)

For Fascia Height < 3 ft

A = 10 ft²			A = 20 ft²		
Zone	GC_p	GC_p	Zone	GC_p	GC_p
5	-1.4	1.0	5	-1.3	0.95
4	-1.1	1.0	4	-1.05	0.95
3	-2.8	0.3	3	-2.3	0.3
2	-1.8	0.3	2	-1.6	0.3

negative	LOWER	UPPER	ACTUAL	p (psf)
Area	10	20	12	
Z5	-1.4	-1.3	-1.38	-28.8
Z4	-1.1	-1.05	-1.09	-22.8
Z3	-2.8	-2.3	-2.70	-56.4
Z2	-1.8	-1.6	-1.76	-36.8

positive	LOWER	UPPER	ACTUAL	p (psf)
Area	10	20	12	
Z5	1.0	0.95	0.99	20.7
Z4	1.0	0.95	0.99	20.7

Load Case A
P1 = 20.7
P2 = -36.8
57.5 psf

Load Case B
P3 = 20.7
P4 = -22.8
43.5 psf

For Fascia Height ≥ 3 ft

A = 10 ft²			A = 20 ft²		
Zone	GC_p	GC_p	Zone	GC_p	GC_p
5	-1.4	1.0	5	-1.3	0.95
4	-1.1	1.0	4	-1.05	0.95
3	-1.8	1.0	3	-1.6	0.95
2	-1.8	1.0	2	-1.6	0.95

negative	LOWER	UPPER	ACTUAL	p (psf)
Area	10	20	12	
Z5	-1.4	-1.3	-1.38	-28.8
Z4	-1.1	-1.05	-1.09	-22.8
Z3	-1.8	-1.6	-1.76	-36.8
Z2	-1.8	-1.6	-1.76	-36.8

positive	LOWER	UPPER	ACTUAL	p (psf)
Area	10	20	12	
Z5	1.0	0.95	0.99	20.7
Z4	1.0	0.95	0.99	20.7

Load Case A
P1 = 20.7
P2 = -36.8
57.5 psf

Load Case B
P3 = 20.7
P4 = -22.8
43.5 psf

LOAD PATH FOR THE LATERAL FORCE RESISTING SYSTEM

Wind forces on the canopy fascia are transferred to the LFRS from the fascia brackets screwed to the deck pans, and through the deck pans to the purlins via canopy deck clips or bolts (see plans). The canopy deck pans are not fastened together (i.e. there are no side-lap screws), therefore the canopy deck does not act as a traditional diaphragm. Lateral wind and seismic forces are transferred directly to the cantilevered columns through weak axis flexure of the purlins and beams in both the transverse and longitudinal directions. Weak axis flexure and combined biaxial forces have been checked for both purlins and beams in the following pages.

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LATERAL ANALYSIS

WIND (from page 2)

qp =	24.6	psf
Total Base Shear (V) =	**14.98**	**kip**

SEISMIC

SDS = 0.677 SDC = D

R =	1.25	
Cs =	0.81	
Seismic W =	62.69	kip
Total Base Shear (V) =	**50.93**	**kip**
W at top of Column =	1.06	kip
Qe at top of Column (Ω not incl.) =	4.08	kip
Distance From Base =	22.58	ft

PURLINS QTY.: 8

Max. Purlin Trib. Width =	10.67	ft
Purlin Trib. Width # 2 =	10.42	ft
Purlin Trib. Width # 3 =	9.42	ft
Purlin Trib. Width # 4 =	8.25	ft
Left Cantilever =	6.5	ft
3 Bay(s) @	24	ft
Right Cantilever =	6.5	ft

Strong Axis Loads:

D =	0.005	ksf	S =	0.0182	ksf
Lr =	0.02	ksf	Wd =	0.0245	ksf
Ev =	0.0098	klf	Wu =	-0.0224	ksf

Weak Axis Loads:

Eh =	0.059	klf	Wh =	0.022	klf

From Pages 9-15, Use W 12 x 19

BEAMS QTY.: 4

Left Cantilever =	4	ft
3 Bay(s) @	26	ft max.
Right Cantilever =	11.5	ft
Maximum Tributary Width =	24	ft

Strong Axis Loads: Ev = 0.0042 klf

(k)	D	Lr	S	Wd	Wu	Ev
P1	2.20	5.12	4.66	6.26	-5.74	0.24
P2	2.15	5.00	4.55	6.12	-5.61	0.23
P3	1.94	4.52	4.11	5.53	-5.07	0.21
P4	1.66	3.87	3.52	4.73	-4.34	0.18

Weak Axis Loads:

Eh = 0.161 klf Wh = 0.044 klf

B1&B3 From Pages 16-19, Use W 16 x 26

B2 From Pages 20-22, Use W 16 x 31

COLUMNS QTY.: 16

Column Specification: **HSS 12x12x1/4**

t =	0.233	in	Fy =	50	ksi
A =	10.8	in2	Length =	22.58	ft
W =	39.4	plf	Z =	47.6	in3

Reduced Z for 4.75" Dia. Hole =	40.4	in3
Max. Allowable Stress Ratio =	**0.849**	
Max. D Reaction from Beam =	5.66	kip
Max. Lr Reaction from Beam =	11.69	kip
Reduced Lr Reaction =	7.83	kip
Max. S Reaction from Beam =	10.63	kip
Max. Wd Reaction from Beam =	14.29	kip
Max. Wu Reaction from Beam =	-13.1	kip
Max. Ev from Beam =	0.89	kip
Column Weight =	0.89	kip

• *Seismic Load Combinations with overstrength are required as per AISC 341-16*

Load Combinations (ASCE 7-16 2.4.5)

8. 1.0D + 0.7Ev + 0.7ΩQe

9. 1.0D + 0.525Ev + 0.525ΩQe + 0.75S

10. 0.6D - 0.7Ev + 0.7ΩQe Ω = 1.3

	D		S		$E_{vert.}$		$E_{horiz.}$	
8. =	6.55	k	0	k	0.62	k	3.71	k
9 =	6.55	k	7.976	k	0.47	k	2.79	k
10. =	3.93	k	0	k	-0.62	k	3.71	k

From Page 23, Prc < 0.15Pc as required by ASCE 7-16 12.2.5.2.

From Page 24, the Column is OK.

• *An increase in allowable stress of 1.2 is allowed for members designed using overstrength as per ASCE 7-16 2.4.5.*

Max. Allowable Stress Ratio = **1.02**

Note: Full roof live loads are used for the design of the purlins and beams and reduced live loads are used for the design of all other members.

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Job: MVE #23-1096 B-R #363261
Date: 10/31/23
Subject: TANK CANOPY
By: CRH

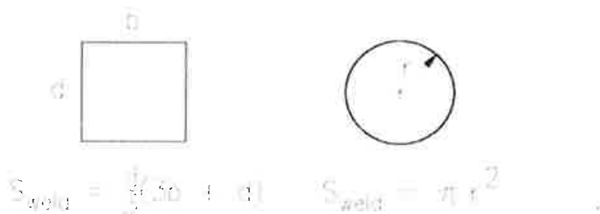
COLUMN BASE PLATE

Fy =	36	ksi
Wind Shear at Base =	1.06	kip
Seismic Shear (ΩQe) =	5.31	kip
Wind Moment at Base =	23.88	kft
Seismic Moment =	119.84	kft

**See Column section for axial loads.

From Page 25, Use 1.25"x22"x22" Base Plate

COLUMN BASE PLATE WELD



Sweld =	192	in^2
Max. Moment at Base =	119.8	kft
Max. Moment at Base =	1438	kin
Weld Strength Required =	1438 / 192	kin / in^2
Weld Strength Req'd (M only) =	7.49	k/in
Shear at Base (for max. M) =	5.31	kips
Weld Length =	48	in^2
Total Weld Strength Required =	7.6	k/in
Base Plate Thickness =	1.25	in

Min. Weld Size (per AISC Table J2.4) = **1/8**

Use 6 /16 in fillet weld all around column

G.F. = 8.352 k/in > 7.6 k/in **OK**

Check Weld Base Metal Per AISC J.4

Weld	Rn = FwAw	ϕ =	0.75
Base	Rn = Fbm Abm	t =	0.233

Fw	= 0.6Fexx	=	42	ksi
Fbm	= Fy	=	50	ksi

Aw	0.265	in2	ϕRn =	8.353	k/in	**OK**
Abm	0.233	in2	ϕRn =	8.719	k/in	**OK**

ANCHOR RODS (ASTM F1554)

No. Rods per Connection =	4	
Mean Anchor Rod Spacing =	16	in
Number of Rods in Tension =	2	in

LRFD FACTORED LOADS

(see Column Base Plate calculations)

	P (kips)	V (kips)	M (kft)	
1.	32.01	0.53	11.94	
2.	27.46	1.06	23.88	
3.	10.87	5.31	119.84	
4.	20.18	1.06	23.88	
5.	5.00	5.31	119.84	CONTROLS

ANCHOR DESIGN LOADS (Factored)

Nu (on 2 anchors) =	**89.88**	kip
Vu (on 4 anchors) =	**5.31**	kip

See Anchor Rod Design on Page 26.

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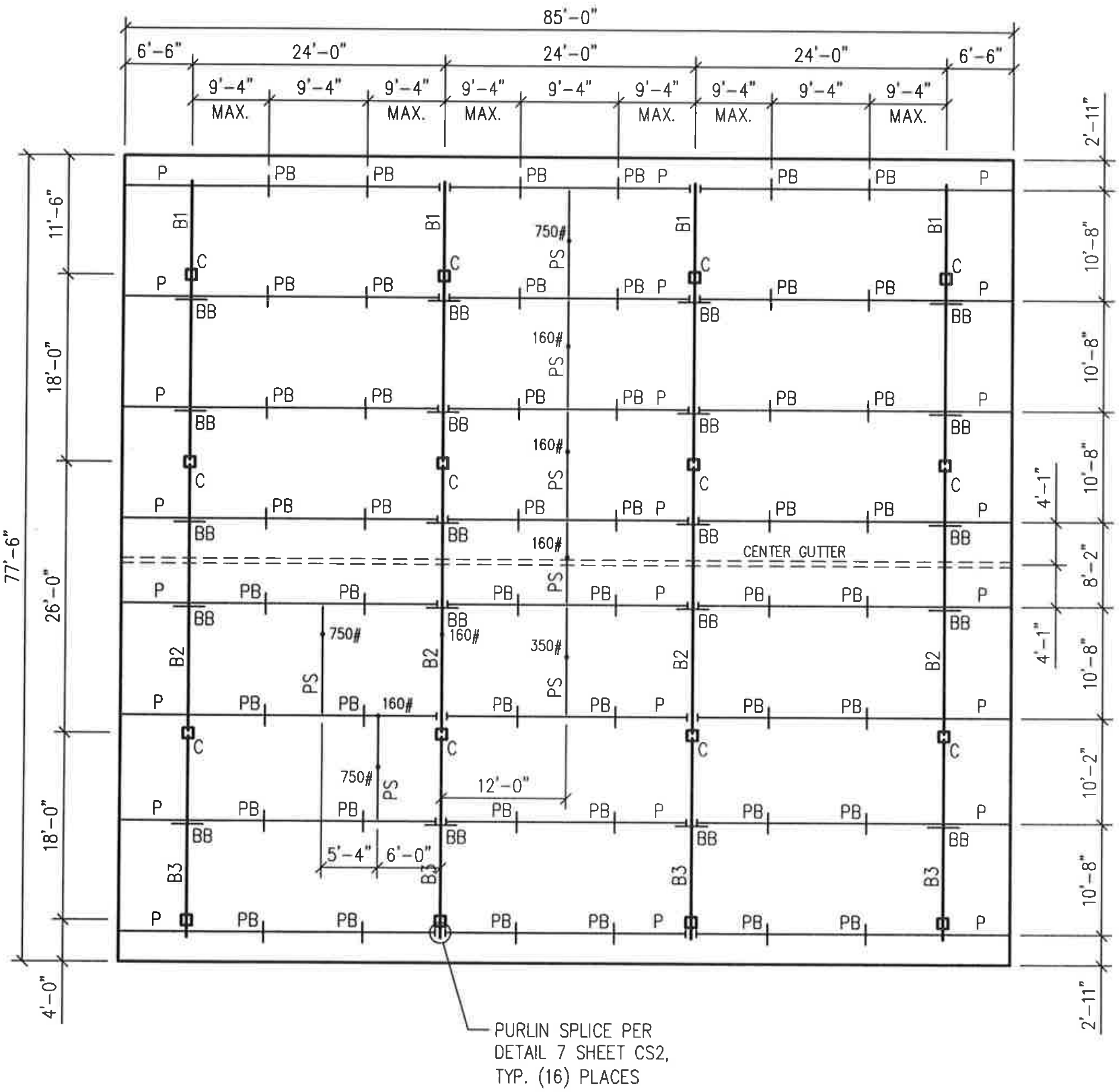
Job #23-1096 BESTWORTH ROMMEL
Subject PDX TANK CANOPY
PORTLAND, OREGON

Date 10-27-23
By C.R.H.



KEY

B1, B3 = W16x26 BEAM
B2 = W16x31 BEAM
P = W12x19 PURLIN
PS = C4x5.4 PIPE SUPPORT
C = HSS 12"x12"x$\frac{1}{4}$" COLUMN
PB = $1\frac{1}{2}$"x$1\frac{1}{2}$"x16 GA. PURLIN BRACE
BB = 2"x2"x$\frac{3}{16}$" BEAM BRACE

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DESCRIPTION: Middle Purlins (Strong Axis)

CODE REFERENCES

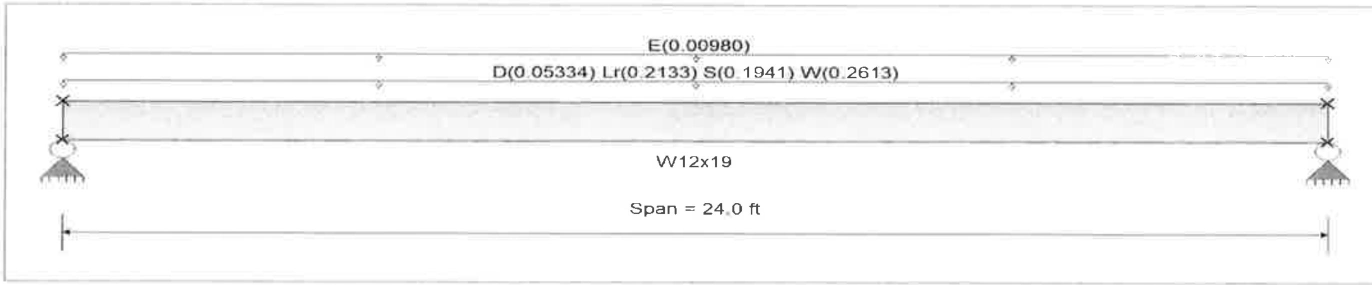
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Spacing, First Brace at ft and spaced at 9.333 ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0050, Lr = 0.020, S = 0.01820, W = 0.02450 ksf, Tributary Width = 10.667 ft

Uniform Load on ALL spans : E = 0.00980 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.740** : 1	Maximum Shear Stress Ratio =	**0.073** : 1
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	25.196 k-ft	Va : Applied	4.199 k
Mn / Omega : Allowable	34.068 k-ft	Vn/Omega : Allowable	57.340 k
Load Combination	+D+0.750Lr+0.750L+0.450W+H	Load Combination	+D+0.750Lr+0.750L+0.450W+H
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.696 in	Ratio =	414	>=180	Span: 1 : +D+0.750Lr+0.750L+0.450W+H
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.136	3.136
Max Upward from Load Cases	3.136	3.136
D Only	0.868	0.868
Lr Only	2.560	2.560
S Only	2.330	2.330
W Only	3.136	3.136
E Only	0.118	0.118
H Only		

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

10

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Middle Purlins (Strong Axis) with Pipe Load

CODE REFERENCES

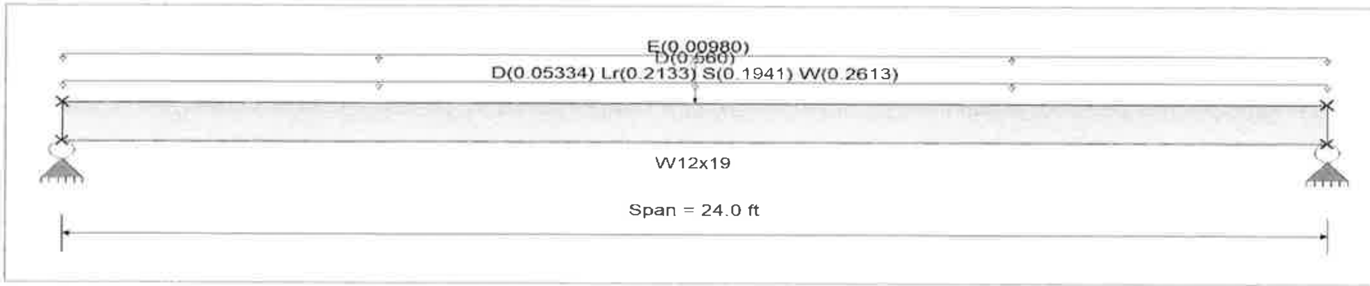
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Spacing, First Brace at ft and spaced at 9.333 ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0050, Lr = 0.020, S = 0.01820, W = 0.02450 ksf, Tributary Width = 10.667 ft

Uniform Load on ALL spans : E = 0.00980 k/ft

Load(s) for Span Number 1
Point Load : D = 0.560 k @ 12.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.827** : 1	Maximum Shear Stress Ratio =	**0.078** : 1
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	28.556 k-ft	Va : Applied	4.479 k
Mn / Omega : Allowable	34.525 k-ft	Vn/Omega : Allowable	57.340 k
Load Combination	+D+0.750Lr+0.750L+0.450W+H	Load Combination	+D+0.750Lr+0.750L+0.450W+H
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.770 in	Ratio =	374	>=180	Span: 1 : +D+0.750Lr+0.750L+0.450W+H
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.136	3.136
Max Upward from Load Cases	3.136	3.136
D Only	1.148	1.148
Lr Only	2.560	2.560
S Only	2.330	2.330
W Only	3.136	3.136
E Only	0.118	0.118
H Only		

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

11

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

DESCRIPTION: Middle Purlins (Weak Axis)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		

Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.0220, E = 0.0590 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.400** : 1	Maximum Shear Stress Ratio =	**0.009** : 1
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	2.974 k-ft	Va : Applied	0.4956 k
Mn / Omega : Allowable	7.435 k-ft	Vn/Omega : Allowable	56.140 k
Load Combination	E Only * 0.70	Load Combination	E Only * 0.70
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	2.840 in	Ratio =	101	>=60.0	Span: 1 : E Only * 0.70
Max Upward Total Deflection	0 in	Ratio =	0	<60.0	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.708	0.708
Max Upward from Load Cases	0.708	0.708
W Only	0.264	0.264
E Only	0.708	0.708

FOR REFERENCE ONLY

MOUNTAIN VIEW ENGINEERING, INC.
345 No. Main, Suite A • Brigham City, Utah 84302
Phone (435) 734-9700 • Fax (435) 734-9519

MIDDLE PURLIN LOAD COMBINATIONS FOR BIAXIAL BENDING (ASD)

Stress Ratio	Strong Axis	Weak Axis	
Dead Load (D)	0.241		
Roof Live Load (Lr)	0.451		
Floor Live Load (L)			
Snow Load (S)	0.410		
Wind Load (0.6W)	0.331	**0.128**	*These ratios already have 0.6 factor applied.*
Seismic Load (0.7E)	0.014	**0.400**	*These ratios already have 0.7 factor applied.*

ASCE 7-16 LOAD COMBINATIONS (ASD)

1. D
2. D + L
3. D + (Lr or S)
4. D + 0.75L + 0.75(Lr or S)
5. D + 0.6W
6. D + 0.75L+ 0.75(0.6W) + 0.75(Lr or S)
7. 0.6D + 0.6W
8. D + 0.7Ev + 0.7Eh
9. D + 0.75L+ 0.75(0.7E) + 0.75S
10. 0.6D - 0.7Ev + 0.7Eh

	STRONG	WEAK	COMBINED	
1.	0.241	0.000	0.241	
2.	0.241	0.000	0.241	
3.	0.692	0.000	0.692	
4.	0.579	0.000	0.579	
5.	0.572	0.128	0.700	
6.	0.828	0.096	0.924	**CONTROLS <1.0 OK**
7.	0.476	0.128	0.604	
8.	0.255	0.400	0.655	
9.	0.559	0.300	0.859	
10.	0.131	0.400	0.531	

ASCE 7-16 CH. 2.4.5 SEISMIC LOAD COMBINATIONS WITH OVERSTRENGTH (ASD)

8. 1.0D + 0.7Ev + 0.7ΩQe   Ω = 1.3
9 1.0D + 0.525Ev + 0.525ΩQe + 0.75S
10. 0.6D - 0.7Ev + 0.7ΩQe

	STRONG	WEAK	COMBINED	
8.	0.251	0.520	0.771	
9	0.556	0.273	0.829	**CONTROLS <1.2 OK**
10.	0.131	0.520	0.651	

• *An increase in allowable stress of 1.2 is allowed for members designed using overstrength as per ASCE 7-16 2.4.5.*

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

13

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: End Purlins (Strong Axis)

CODE REFERENCES

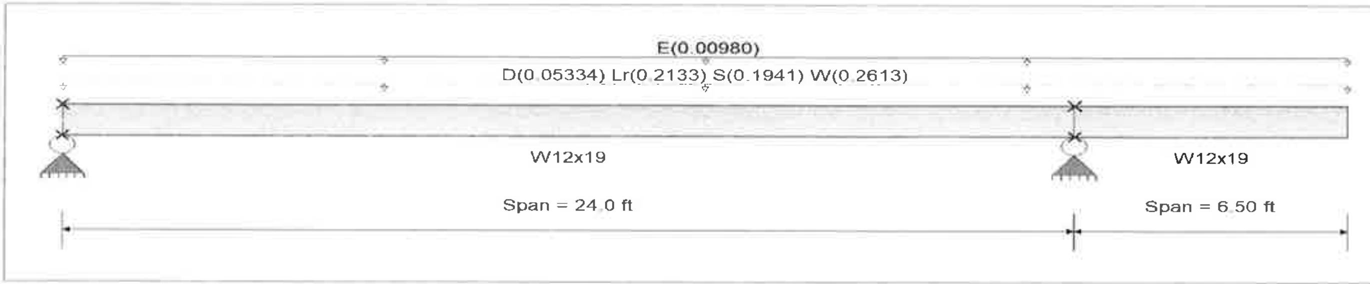
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Spacing, First Brace at ft and spaced at 9.333 ft
Span # 2, Defined Brace Spacing, First Brace at ft and spaced at 1.333 ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0050, Lr = 0.020, S = 0.01820, W = 0.02450 ksf, Tributary Width = 10.667 ft

Uniform Load on ALL spans : E = 0.00980 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.671** : 1	Maximum Shear Stress Ratio =	**0.079** : 1
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	23.230 k-ft	Va : Applied	4.507 k
Mn / Omega : Allowable	34.640 k-ft	Vn/Omega : Allowable	57.340 k
Load Combi0.750Lr+0.750L+0.450W+H, LL Comb Run (L*)		Load Combi0.750Lr+0.750L+0.450W+H, LL Comb Run (LL)	
		Location of maximum on span	24.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.631 in	Ratio =	456	>=180	Span: 2 : +D+0.750Lr+0.750L+0.450W+H, LL Comb Run
Max Upward Total Deflection	-0.486 in	Ratio =	321	>=180	Span: 2 : +D+0.750Lr+0.750L+0.450W+H, LL Comb Run

Vertical Reactions

Support notation : Far left is #   Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	2.906	5.065	
Max Upward from Load Cases	2.906	5.065	
Max Downward from all Load Conditions (Resis	-0.188		
Max Downward from Load Cases (Resisting Up	-0.188		
D Only	0.804	1.402	
Lr Only, LL Comb Run (*L)	-0.188	1.574	
Lr Only, LL Comb Run (L*)	2.560	2.560	
Lr Only, LL Comb Run (LL)	2.372	4.135	
S Only	2.159	3.762	
W Only	2.906	5.065	
E Only	0.109	0.190	
H Only			

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

14

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: End Purlins (Strong Axis) with Pipe Load

CODE REFERENCES

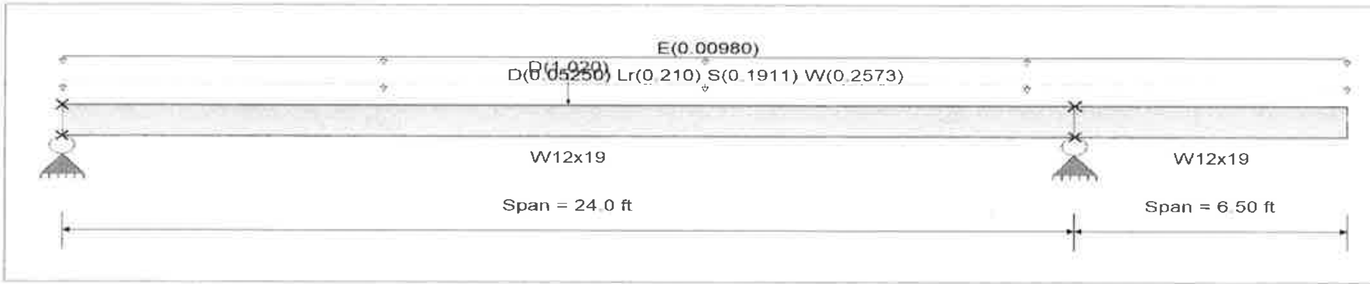
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Spacing, First Brace at ft and spaced at 9.333 ft
Span # 2, Defined Brace Spacing, First Brace at ft and spaced at 1.333 ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0050, Lr = 0.020, S = 0.01820, W = 0.02450 ksf, Tributary Width = 10.50 ft

Uniform Load on ALL spans : E = 0.00980 k/ft

Load(s) for Span Number 1
Point Load : D = 1.020 k @ 12.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.821** : 1	Maximum Shear Stress Ratio =	**0.086** : 1
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	28.965 k-ft	Va : Applied	4.951 k
Mn / Omega : Allowable	35.295 k-ft	Vn/Omega : Allowable	57.340 k
Load Combination +D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*)		Load Combination +D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LL)	
		Location of maximum on span	24.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.757 in	Ratio =	380	>=180	Span: 2 : +D+0.750Lr+0.750L+0.450W+H, LL Comb Run
Max Upward Total Deflection	-0.588 in	Ratio =	265	>=180	Span: 2 : +D+0.750Lr+0.750L+0.450W+H, LL Comb Run

Vertical Reactions

Support notation : Far left is #   Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	2.861	4.986	
Max Upward from Load Cases	2.861	4.986	
Max Downward from all Load Conditions (Resis	-0.185		
Max Downward from Load Cases (Resisting Up	-0.185		
D Only	1.305	1.896	
Lr Only, LL Comb Run (*L)	-0.185	1.550	
Lr Only, LL Comb Run (L*)	2.520	2.520	
Lr Only, LL Comb Run (LL)	2.335	4.070	
S Only	2.125	3.704	
W Only	2.861	4.986	
E Only	0.109	0.190	
H Only			

FOR REFERENCE ONLY

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

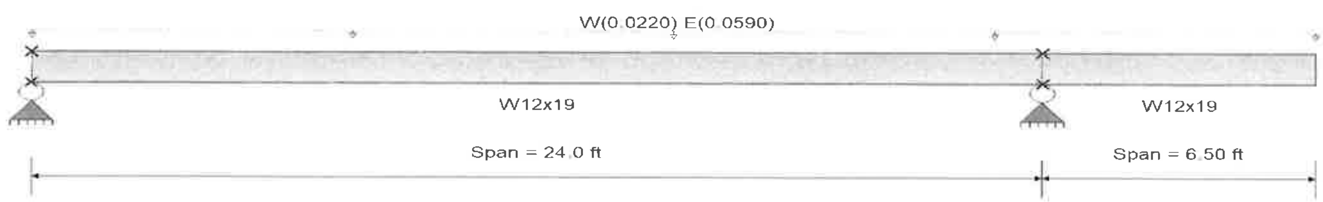
DESCRIPTION: End Purlins (Weak Axis)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.0220, E = 0.0590 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.343** : 1	Maximum Shear Stress Ratio =	**0.009** : 1
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	2.553 k-ft	Va : Applied	0.5320 k
Mn / Omega : Allowable	7.435 k-ft	Vn/Omega : Allowable	56.140 k
Load Combination	E Only * 0.70	Load Combination	E Only * 0.70
		Location of maximum on span	24.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	2.348 in	Ratio =	123	>=60.0	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-1.586 in	Ratio =	98	>=60.0	Span: 2 : E Only * 0.70

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.656	1.143	
Max Upward from Load Cases	0.656	1.143	
W Only	0.245	0.426	
E Only	0.656	1.143	

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

16

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Beam 1 (Strong Axis)

CODE REFERENCES

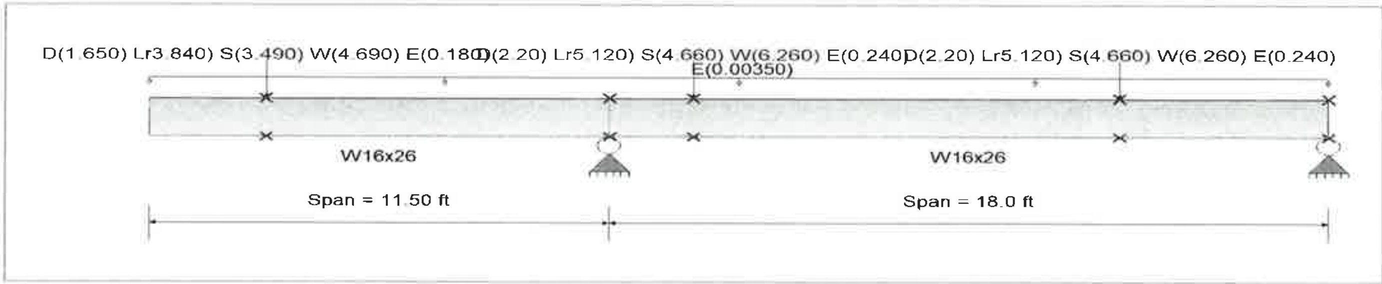
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Locations, First Brace at 2.917 ft, Second Brace at ft, Third Brace at ft
Span # 2, Defined Brace Locations, First Brace at 2.083 ft, Second Brace at 12.750 ft, Third Brace at ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : E = 0.00350 k/ft

Load(s) for Span Number 1
Point Load : D = 1.650, Lr = 3.840, S = 3.490, W = 4.690, E = 0.180 k @ 2.917 ft

Load(s) for Span Number 2
Point Load : D = 2.20, Lr = 5.120, S = 4.660, W = 6.260, E = 0.240 k @ 2.083 ft
Point Load : D = 2.20, Lr = 5.120, S = 4.660, W = 6.260, E = 0.240 k @ 12.750 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.711** : 1	Maximum Shear Stress Ratio =	**0.197** : 1
Section used for this span	**W16x26**	Section used for this span	**W16x26**
Ma : Applied	58.724 k-ft	Va : Applied	13.913 k
Mn / Omega : Allowable	82.587 k-ft	Vn/Omega : Allowable	70.509 k
Load Combination +D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*)		Load Combination +D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LL)	
		Location of maximum on span	11.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.935 in	Ratio =	295	>=180	Span: 2 : +D+0.750Lr+0.750L+0.450W+H, LL Comb Run
Max Upward Total Deflection	-0.140 in	Ratio =	1544	>=180	Span: 2 : +D+Lr+H, LL Comb Run (L*)

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions		14.288	4.219
Max Upward from Load Cases		14.288	4.219
Max Downward from all Load Conditions (Resis			-1.831
Max Downward from Load Cases (Resisting Up			-1.831
D Only		5.656	1.165
Lr Only, LL Comb Run (*L)		6.021	4.219
Lr Only, LL Comb Run (L*)		5.671	-1.831
Lr Only, LL Comb Run (LL)		11.692	2.388
S Only		10.634	2.176
W Only		14.288	2.922
E Only		0.633	0.131

WORST CASE AXIAL LOAD TO COLUMNS

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

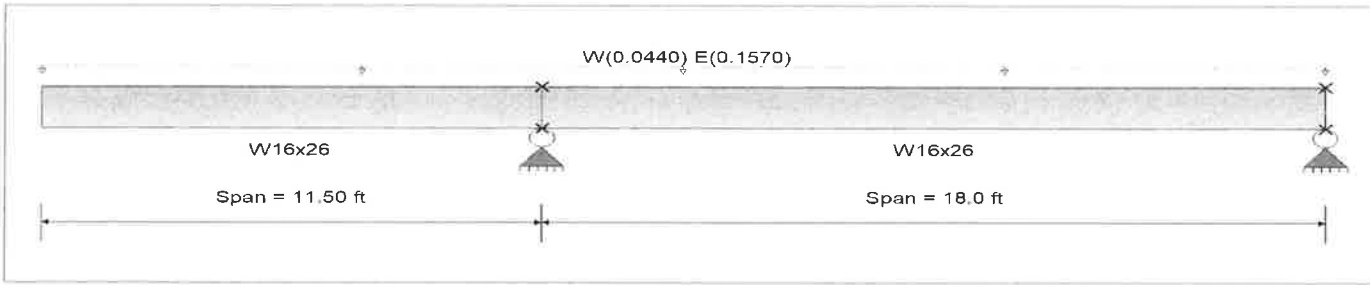
DESCRIPTION: Beam 1 (Weak Axis)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.0440, E = 0.1570 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.532** : 1	Maximum Shear Stress Ratio =	**0.020** : 1
Section used for this span	**W16x26**	Section used for this span	**W16x26**
Ma : Applied	7.267 k-ft	Va : Applied	1.393 k
Mn / Omega : Allowable	13.673 k-ft	Vn/Omega : Allowable	68.174 k
Load Combination	E Only * 0.70	Load Combination	E Only * 0.70
		Location of maximum on span	11.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in Ratio =	0	<60.0	n/a
Max Upward Transient Deflection	0 in Ratio =	0	<60.0	n/a
Max Downward Total Deflection	2.704 in Ratio =	102	>=60.0	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.153 in Ratio =	1413	>=60.0	Span: 2 : E Only * 0.70

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions		3.795	0.836
Max Upward from Load Cases		3.795	0.836
W Only		1.064	0.234
E Only		3.795	0.836

FOR REFERENCE ONLY

MOUNTAIN VIEW ENGINEERING, INC.
345 No. Main, Suite A • Brigham City, Utah 84302
Phone (435) 734-9700 • Fax (435) 734-9519

BEAM 1 LOAD COMBINATIONS FOR BIAXIAL BENDING (ASD)

Stress Ratio	Strong Axis	Weak Axis	
Dead Load (D)	0.192		
Roof Live Load (Lr)	0.399		
Floor Live Load (L)			
Snow Load (S)	0.363		
Wind Load (0.6W)	0.293	**0.128**	*These ratios already have 0.6 factor applied.*
Seismic Load (0.7E)	0.015	**0.532**	*These ratios already have 0.7 factor applied.*

ASCE 7-16 LOAD COMBINATIONS (ASD)

1. D
2. D + L
3. D + (Lr or S)
4. D + 0.75L + 0.75(Lr or S)
5. D + 0.6W
6. D + 0.75L+ 0.75(0.6W) + 0.75(Lr or S)
7. 0.6D + 0.6W
8. D + 0.7Ev + 0.7Eh
9. D + 0.75L+ 0.75(0.7E) + 0.75S
10. 0.6D - 0.7Ev + 0.7Eh

	STRONG	WEAK	COMBINED	
1.	0.192	0.000	0.192	
2.	0.192	0.000	0.192	
3.	0.591	0.000	0.591	
4.	0.491	0.000	0.491	
5.	0.485	0.128	0.613	
6.	0.711	0.096	0.807	
7.	0.408	0.128	0.536	
8.	0.207	0.532	0.739	
9.	0.476	0.399	0.875	**CONTROLS <1.0 OK**
10.	0.100	0.532	0.632	

ASCE 7-16 CH. 2.4.5 SEISMIC LOAD COMBINATIONS WITH OVERSTRENGTH (ASD)

8. 1.0D + 0.7Ev + 0.7ΩQe   Ω = 1.3
9 1.0D + 0.525Ev + 0.525ΩQe + 0.75S
10. 0.6D - 0.7Ev + 0.7ΩQe

	STRONG	WEAK	COMBINED	
8.	0.203	0.692	0.894	**CONTROLS <1.2 OK**
9	0.472	0.363	0.835	
10.	0.100	0.692	0.792	

• *An increase in allowable stress of 1.2 is allowed for members designed using overstrength as per ASCE 7-16 2.4.5.*

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

19

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Beam 3 (Strong Axis)

CODE REFERENCES

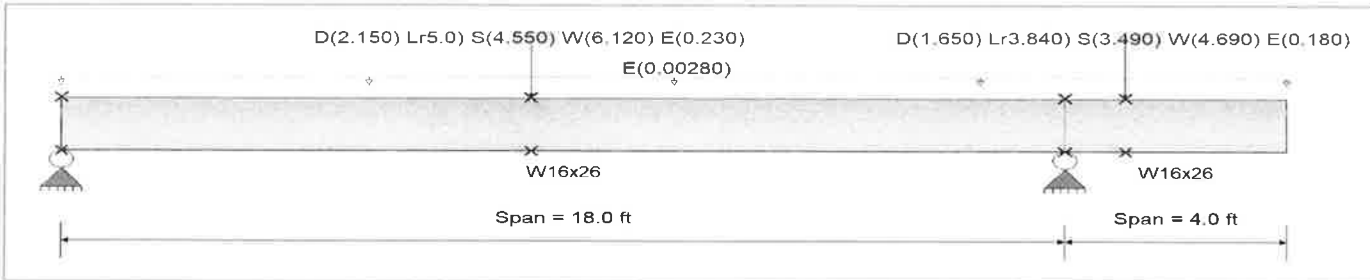
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Locations, First Brace at 8.417 ft, Second Brace at 0.0 ft, Third Brace at 0.0 ft
Span # 2, Defined Brace Locations, First Brace at 1.083 ft, Second Brace at ft, Third Brace at ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : E = 0.00280 k/ft

Load(s) for Span Number 1
Point Load : D = 2.150, Lr = 5.0, S = 4.550, W = 6.120, E = 0.230 k @ 8.417 ft

Load(s) for Span Number 2
Point Load : D = 1.650, Lr = 3.840, S = 3.490, W = 4.690, E = 0.180 k @ 1.083 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.343** : 1	Maximum Shear Stress Ratio =	**0.096** : 1
Section used for this span	**W16x26**	Section used for this span	**W16x26**
Ma : Applied	37.797 k-ft	Va : Applied	6.745 k
Mn / Omega : Allowable	110.279 k-ft	Vn/Omega : Allowable	70.509 k
Load Combination +D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*)		Load Combination +D+0.750Lr+0.750L+0.450W+H, LL Comb Run (*L)	
		Location of maximum on span	18.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.198 in	Ratio =	1090	>=180	Span: 2 : +D+0.750Lr+0.750L+0.450W+H, LL Comb Run
Max Upward Total Deflection	-0.118 in	Ratio =	813	>=180	Span: 2 : +D+0.750Lr+0.750L+0.450W+H, LL Comb Run

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	2.976	7.834	
Max Upward from Load Cases	2.976	7.834	
Max Downward from all Load Conditions (Resis	-0.231		
Max Downward from Load Cases (Resisting Up	-0.231		
D Only	1.268	3.104	
Lr Only, LL Comb Run (*L)	-0.231	4.071	
Lr Only, LL Comb Run (L*)	2.662	2.338	
Lr Only, LL Comb Run (LL)	2.431	6.409	
S Only	2.212	5.828	
W Only	2.976	7.834	
E Only	0.136	0.336	
H Only			

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Beam 2 (Strong Axis)

CODE REFERENCES

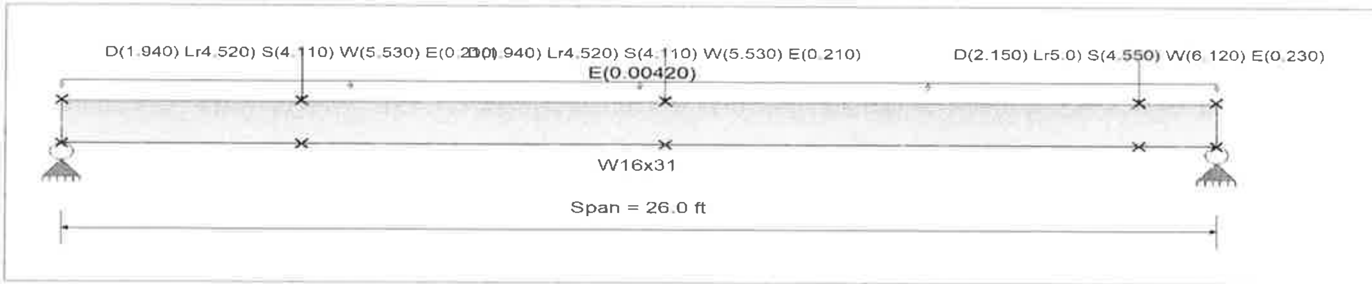
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Locations, First Brace at 5.417 ft, Second Brace at 13.583 ft, Third Brace at 24.250 ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : E = 0.00420 k/ft

Load(s) for Span Number 1
Point Load : D = 1.940, Lr = 4.520, S = 4.110, W = 5.530, E = 0.210 k @ 5.417 ft

Point Load : D = 1.940, Lr = 4.520, S = 4.110, W = 5.530, E = 0.210 k @ 13.583 ft

Point Load : D = 2.150, Lr = 5.0, S = 4.550, W = 6.120, E = 0.230 k @ 24.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.673** : 1	Maximum Shear Stress Ratio =	**0.162** : 1
Section used for this span	**W16x31**	Section used for this span	**W16x31**
Ma : Applied	81.303 k-ft	Va : Applied	14.188 k
Mn / Omega : Allowable	120.865 k-ft	Vn/Omega : Allowable	87.450 k
Load Combination	+D+0.750Lr+0.750L+0.450W+H	Load Combination	+D+0.750Lr+0.750L+0.450W+H
		Location of maximum on span	26.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.856 in Ratio =	365	>=180	Span: 1 : +D+0.750Lr+0.750L+0.450W+H
Max Upward Total Deflection	0 in Ratio =	0	<180	n/a

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	7.431	9.749
Max Upward from Load Cases	7.431	9.749
D Only	3.010	3.826
Lr Only	6.073	7.967
S Only	5.523	7.247
W Only	7.431	9.749
E Only	0.337	0.423
H Only		

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

21

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

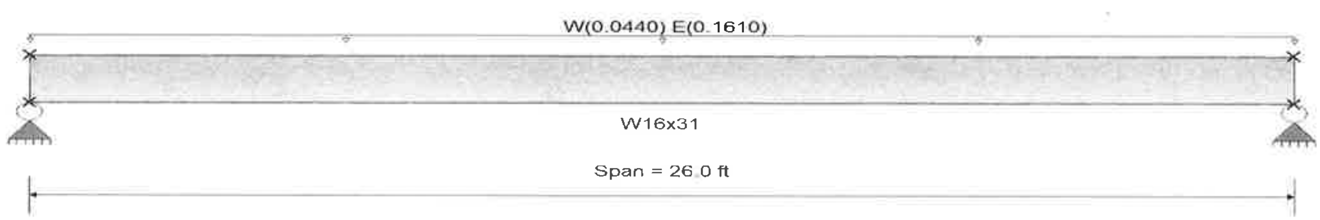
DESCRIPTION: Beam 2 (Weak Axis)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.0440, E = 0.1610 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.543** : 1	Maximum Shear Stress Ratio =	**0.015** : 1
Section used for this span	**W16x31**	Section used for this span	**W16x31**
Ma : Applied	9.523 k-ft	Va : Applied	1.465 k
Mn / Omega : Allowable	17.540 k-ft	Vn/Omega : Allowable	97.328 k
Load Combination	E Only * 0.70	Load Combination	E Only * 0.70
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<60.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<60.0	n/a
Max Downward Total Deflection	3.237 in	Ratio =	96	>=60.0	Span: 1 : E Only * 0.70
Max Upward Total Deflection	0 in	Ratio =	0	<60.0	n/a

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.093	2.093
Max Upward from Load Combinations		
Max Upward from Load Cases	2.093	2.093
Max Downward from all Load Conditions (Resis		
Max Downward from Load Combinations (Resi:		
Max Downward from Load Cases (Resisting Uр		
W Only	0.572	0.572
E Only	2.093	2.093

FOR REFERENCE ONLY
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Phone (435) 734-9700 • Fax (435) 734-9519

BEAM 2 LOAD COMBINATIONS FOR BIAXIAL BENDING (ASD)

Stress Ratio	Strong Axis	Weak Axis	
Dead Load (D)	0.184		
Roof Live Load (Lr)	0.376		
Floor Live Load (L)			
Snow Load (S)	0.342		
Wind Load (0.6W)	0.276	**0.127**	*These ratios already have 0.6 factor applied.*
Seismic Load (0.7E)	0.014	**0.543**	*These ratios already have 0.7 factor applied.*

ASCE 7-16 LOAD COMBINATIONS (ASD)

1. D
2. D + L
3. D + (Lr or S)
4. D + 0.75L + 0.75(Lr or S)
5. D + 0.6W
6. D + 0.75L+ 0.75(0.6W) + 0.75(Lr or S)
7. 0.6D + 0.6W
8. D + 0.7Ev + 0.7Eh
9. D + 0.75L+ 0.75(0.7E) + 0.75S
10. 0.6D - 0.7Ev + 0.7Eh

	STRONG	WEAK	COMBINED	
1.	0.184	0.000	0.184	
2.	0.184	0.000	0.184	
3.	0.560	0.000	0.560	
4.	0.466	0.000	0.466	
5.	0.460	0.127	0.587	
6.	0.673	0.095	0.768	
7.	0.386	0.127	0.513	
8.	0.198	0.543	0.741	
9.	0.451	0.407	0.858	**CONTROLS <1.0 OK**
10.	0.096	0.543	0.639	

ASCE 7-16 CH. 2.4.5 SEISMIC LOAD COMBINATIONS WITH OVERSTRENGTH (ASD)

8. 1.0D + 0.7Ev + 0.7ΩQe — Ω = 1.3
9 1.0D + 0.525Ev + 0.525ΩQe + 0.75S
10. 0.6D - 0.7Ev + 0.7ΩQe

	STRONG	WEAK	COMBINED	
8.	0.194	0.706	0.900	**CONTROLS <1.2 OK**
9	0.448	0.371	0.818	
10.	0.096	0.706	0.802	

• *An increase in allowable stress of 1.2 is allowed for members designed using overstrength as per ASCE 7-16 2.4.5.*

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

23

Steel Column

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Columns (Verify Axial Stress <15%, all loads input as DL since L.C. are already done on page 6)

Code References

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

General Information

Steel Section Name : **HSS12x12x1/4**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 22.583 ft
Top & Bottom Fixity Top Free, Bottom Fixed
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 22.583 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 22.583 ft, K = 2.1

Applied Loads

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

Column self weight included : 890.45 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 22.583 ft, D = 14.990 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.1381** : 1
Load Combination D Only
Location of max.above base 0.0 ft
At maximum location values are . . .
Pa : Axial 15.880 k
Pn / Omega : Allowabl 115.004 k
Ma-x : Applied 0.0 k-ft
Mn-x / Omega : Allowable 86.793 k-ft
Ma-y : Applied 0.0 k-ft
Mn-y / Omega : Allowable 86.793 k-ft

PASS Maximum Shear Stress Rati **0.0** : 1
Load Combination 0.0
Location of max.above base 0.0 ft
At maximum location values are . . .
Va : Applied 0.0 k
Vn / Omega : Allowable 0.0 k

Maximum Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum Load Deflections . . .
Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.138	PASS	0.00 ft	1.00	1.00	118.81	118.81	0.000	PASS	0.00 ft

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

24

Steel Column

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Columns

Code References

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

General Information

Steel Section Name : **HSS12x12x1/4**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 22.583 ft
Top & Bottom Fixity Top Free, Bottom Fixed
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 22.583 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 22.583 ft, K = 2.1

Applied Loads

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

Column self weight included : 890.45 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 22.583 ft, Xecc = 1.0 in, Yecc = -1.0 in, D = 5.660, LR = 7.830, S = 10.630, W = 14.290, E = 0.890 k
BENDING LOADS . . .
Lat. Point Load at 22.583 ft creating Mx-x, W = 1.060, E = 5.310 k

DESIGN SUMMARY

← 1.02 OK ✓

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9983** : 1	
Load Combination	+D+0.70E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	7.173	k
Pn / Omega : Allowabl	115.004	k
Ma-x : Applied	-83.417	k-ft
Mn-x / Omega : Allowable	86.793	k-ft
Ma-y : Applied	-0.5236	k-ft
Mn-y / Omega : Allowable	86.793	k-ft
PASS Maximum Shear Stress Rati	**0.03929** : 1	
Load Combination	+D+0.70E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	3.717	k
Vn / Omega : Allowable	94.604	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	5.310 k

Maximum Load Deflections . . .

Along Y-Y 3.391 in at 22.583 ft above base
for load combination : +0.60D+0.70E
Along X-X 0.1017 in at 22.583 ft above base
for load combination : +D+0.750S+0.450W

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.039	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.000	PASS	0.00 ft
+D+Lr	0.088	PASS	20.01 ft	1.67	1.00	118.81	118.81	0.000	PASS	0.00 ft
+D+S	0.106	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.000	PASS	0.00 ft
+D+0.750Lr	0.076	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.000	PASS	0.00 ft
+D+0.750S	0.089	PASS	1.36 ft	1.67	1.00	118.81	118.81	0.000	PASS	0.00 ft
+D+0.60W	0.231	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.007	PASS	0.00 ft
+D+0.750Lr+0.450W	0.206	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.005	PASS	0.00 ft
+D+0.750S+0.450W	0.215	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.005	PASS	0.00 ft
+0.60D+0.60W	0.220	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.007	PASS	0.00 ft
+D+0.70E	0.998	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.039	PASS	0.00 ft
+D+0.750S+0.5250E	0.791	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.029	PASS	0.00 ft
+0.60D+0.70E	0.987	PASS	0.00 ft	1.67	1.00	118.81	118.81	0.039	PASS	0.00 ft

FOR REFERENCE ONLY

MOUNTAIN VIEW ENGINEERING, INC.
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Phone (435) 734-9700 • Fax (435) 734-9519

BASEPLATE CALCULATIONS

***Overstrength included in E.*

COLUMN BASE REACTIONS	*Axial (P, kips)*	*Shear (V, kips)*	*Moment (M, kip*ft)*
Dead Load (D)	6.55	0.00	0.00
Roof Live Load (Lr)	7.83	0.00	0.00
Snow Load (S)	10.63	0.00	0.00
Wind Load (W, ult)	14.29	1.06	23.88
Seismic Load (Ev, Emh, ult)	0.89	5.31	119.84

ASCE 7-16 LOAD COMBINATIONS (LRFD)		***P (kips)***	***V (kips)***	***M (kip*ft)***	
3. 1.2D + 1.6(Lr or S) + 0.5W	3.	32.01	0.53	11.94	
4. 1.2D + 1.0W + 0.5(Lr or S)	4.	27.46	1.06	23.88	
5. 0.9D + 1.0W	6.	20.18	1.06	23.88	
6. 1.2D + 1.0Ev + 1.0Emh + 0.2S	5.	10.87	5.31	119.84	
7. 0.9D - 1.0Ev + 1.0Emh	7.	5.00	5.31	119.84	**CONTROLS**

CONTROLLING LOADS

P = 5.00 kips
V = 5.31 kips
M = 119.84 kipft

Total Number of Anchor Rods = **4**
Number of Anchor Rods in Tension = 2
Number of Anchor Rods in Shear = 4
For Baseplate, Fy = 36 ksi
For F1554 Anchors, Futa = 58 ksi

SDC = D
f'c = 3500 psi

Baseplate Dimensions: N = 22 in B = 22 in

Anchor Rows: 2 anchors @ d = 16 in
0 anchors @ d = 0 in
0 anchors @ d = 0 in

d' = 16 in

Distance of Anchors from Plate Edge = 3 in

Tension on Each Anchor = 44.94 kips (LRFD)
Shear on Each Anchor = 1.33 kips (LRFD)

From AISC 360: Φ_B = 0.9

Column Outisde Dimension = 12 in
Side Bending Moment Arm = 2 in
Side Bending Moment in Plate = 179.76 kip*in
Bending Plane (W_{plate}) = 22 in

Round or Square? square
Corner Bending Moment Arm = 2.83 in
Corner Bending Moment in Plate = 127.11 kip*in
Bending Plane (W_{plate}) = 14.142 in

Plate Thickness Required = $\left(\dfrac{4 * M}{Fy * W_{plate} * \Phi_B}\right)^{0.5}$ = **1.053 inches** **Use 1.25 inch thick plate**

THEREFORE, 1.25 in. x 22 in. x 22 in. BASEPLATE IS OK

FOR REFERENCE ONLY

Mmax @ Column Base = **119.84** k*ft (LRFD)
Vmax @ Column Base = **5.31** kips (LRFD)

λ_a = 1.0 for cast-in-place anchors in normal weight concrete (see ACI 318-19 Section 19.2.4)

ANCHOR ROD GROUP CHECK (per ACI 318-19 Chapter 17, headed anchors)

Tensile Force on Anchors (N_U) =	89.88	kips
Shear Force on Anchors (V_U) =	5.31	kips
Seismic Design Category =	D	
Number of Anchors (n) =	4	
Number of Anchors in Tension (n) =	2	
Anchor Diameter (d_a) = **1.5** in A_{SE} =	1.410	in^2
Anchor Spacing Perpendicular to Load (s_1) =	16	in
Anchor Spacing Parallel to Load (s_2) =	16	in

Steel Strength of Anchors in Tension (17.6.1)

$N_{sa} = nA_{SE}F_{uta}$ = 163.56 kips
ϕ = 0.75 ϕN_{sa} = 122.67 kips

Steel Strength of Anchors in Shear (17.7.1)

Built-up grout pads used? YES *(0.8 factor applied below)*
$V_S = 0.6nA_{SE}F_{uta}$ = 157.02 kips
ϕ = 0.65 ϕV_S = 102.06 kips

ANCHOR ROD GROUP CAPACITY

ALLOW. TENSION (ΦN_N) = 122.67 kips > 89.9 kips **OK**
ALLOW. SHEAR (ΦV_N) = 102.06 kips > 5.31 kips **OK**

CHECK SHEAR/TENSION INTERACTION

$N_{UA} > 0.2\Phi N_N$? YES
$V_{UA} > 0.2\Phi V_N$? NO

UNITY CHECK NOT NECESSARY

$$\frac{N_{UA}}{\Phi N_N} + \frac{V_{UA}}{\Phi V_N} = 0.78 < 1.20 \text{ OK}$$

PLEASE NOTE: MOUNTAIN VIEW ENGINEERING HAS PROVIDED A CHECK OF THE STEEL STRENGTH OF THE ANCHOR RODS ONLY. FOUNDATION DESIGN IS BY OTHERS SO DESIGN OF THE ANCHOR RODS TO RESIST CONCRETE BREAKOUT, PULLOUT, AND SIDE FACE BLOWOUT ARE THE RESPONSIBILITY OF THE FOUNDATION DESIGNER. FOUNDATION DESIGNER SHALL DETERMINE ANCHOR ROD EMBEDMENT DEPTH REQUIRED IN THEIR FOUNDATION DESIGN.

FOR REFERENCE ONLY

PURLIN/BEAM CONNECTION DESIGN (ASD)

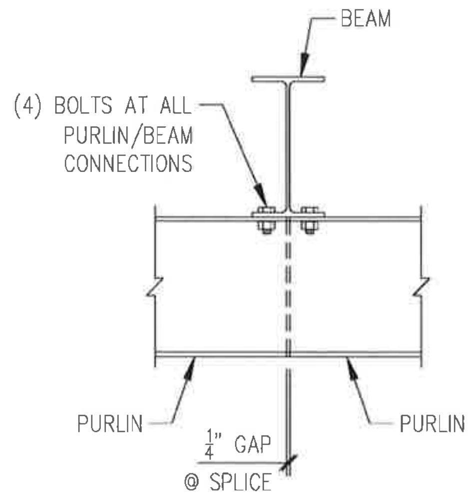
MAX. PURLIN REACTIONS

Occurs @ Splice? **NO**

P1 (D) =	2.20	k
P1 (Lr) =	5.12	k
P1 (S) =	4.66	k
P1 (Wd) =	6.26	k
P1 (Wu) =	-5.74	k
P1 (Ev) =	0.24	k
V1 (W) =	0.53	k
V1 (Eh) =	1.41	k

Load Combos:		P			V	
D+Lr	=	7.3	k	=	0.0	k
D+S	=	6.9	k	=	0.0	k
D+0.75(Lr or S)+0.45W	=	8.9	k	=	0.2	k
D+0.6W	=	6.0	k	=	0.3	k
D+0.7E	=	2.4	k	=	1.0	k

Applied Factor of Safety = **1.25**
Max. Gravity Design Load = **11.1** **k (ASD)**
Max. Shear Design Load = **1.2** **k (ASD)**



BOLT DESIGN

Try **(4)** **3/4" ø A325**
d_b = 0.75 in

G.F. = **19.9** kips tension
G.F. = **11.9** kips shear

Tension / Bolt = **2.77** **k (ASD)** Unity Check: 0.139 **OK**
Shear / Bolt = **0.31** **k (ASD)** Unity Check: 0.026 **OK**
0.165 **OK**

USE (4) 3/4" ø A325 BOLTS

PRYING ACTION

Beam: **W16X26**

T = 2.77 * 2 bolts = 6 kips

F_U = **65** ksi b_f = 5.50 in
g = 2.75 in t_f = 0.345 in
t_w = 0.250 in d = 15.70 in

b = 1.25 in (web to CL bolt)
b' = 0.875 in (moment arm)
p = 4.50 in (eff. flange width)

$$t\ min. = \left(\frac{6.66\ T\ b'}{p\ Fu}\right)^{0.5}$$

= **0.332 in** < **0.345 in** **OK**

FLANGE THICKNESSES ARE SUFFICIENT TO ELIMINATE PRYING ACTION, NO OTHER CHECKS NEEDED.

Purlin: **W12X19**

T = 2.77 * 2 bolts = 6 kips

F_U = **65** ksi b_f = 4.01 in
g = 2.25 in t_f = 0.350 in
t_w = 0.235 in d = 12.20 in

b = 1.0075 in (web to CL bolt)
b' = 0.6325 in (moment arm)
p = 5.50 in (eff. flange width)

$$t\ min. = \left(\frac{6.66\ T\ b'}{p\ Fu}\right)^{0.5}$$

= **0.255 in** < **0.350 in** **OK**

FLANGE THICKNESSES ARE SUFFICIENT TO ELIMINATE PRYING ACTION, NO OTHER CHECKS NEEDED.

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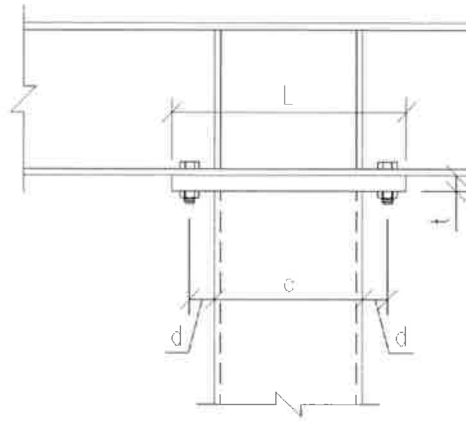
345 No. Main, Suite A • Brigham City, Utah 84302
Phone (435) 734-9700 • Fax (435) 734-9519

COLUMN CAP PLATE DESIGN

Wind Uplift @ Top of Column =	-7.86	k (ASD)
Wind Shear @ Top of Column =	0.63	k (ASD)
Seismic Shear @ Top of Column =	3.71	k (ASD)
Applied Factor of Safety =	1.5	



DEAD LOADS HAVE BEEN NEGLECTED (CONSERVATIVE)

WELD DESIGN

Column Type = **square**
Column Width (c) = **12** in

L weld = 48 in

Weld Strength Required = $\frac{11.79 \text{ k}}{48 \text{ in}} + \frac{0.95 \text{ k}}{48 \text{ in}} = \mathbf{0.27}$ **k/in**

USE 3 /16" E-70XX FILLET WELD ALL AROUND
G.F. = 2.78 k/in

BOLT DESIGN

Distance From Column (d) = **2** in
Bolt Gage = **16** in

Try **(4) 3/4" ø A325** G.F. = **19.9** kips tension
d_b = 0.75 in G.F. = **11.9** kips shear

Tension / Bolt =	**2.95** **k (ASD)**	Unity Check:	0.148	**OK**
Shear / Bolt =	**1.39** **k (ASD)**	Unity Check:	0.117	**OK**
			0.265	**OK**

USE (4) 3/4" ø A325 BOLTS

PLATE DESIGN (FLEXURE FROM BOLT TENSION)

Plate Width (w) = **14** in Fy = 36
Moment in Plate = **11.8** **k*in**

t required = $\left(\frac{(6.68)(11.8 \text{ k*in})}{(36 \text{ ksi})(14 \text{ in})} \right)^{0.5} = \mathbf{0.395}$ **in**

USE 0.75" x 14" x 20" PLATE

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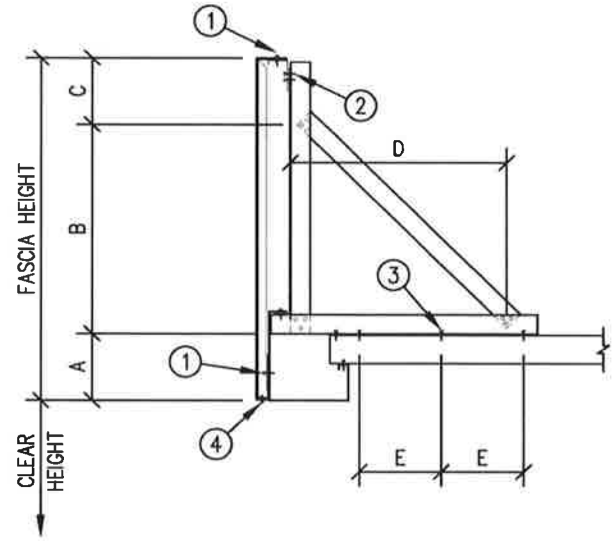
345 No. Main, Suite A • Brigham City, Utah 84302
Phone (435) 734-9700 • Fax (435) 734-9519

FASCIA FRAME DESIGN

DIMENSIONS

Clear Height	=	**21**	ft	Edge Distance	=	**2.917**	ft
Fascia Height	=	**36**	in	Gutter Width	=	**8**	in
Frame Spacing	=	**48**	in o.c.				

A	=	**7**	in	D	=	**22**	in
B	=	**22**	in	E	=	**12**	in @ sides
C	=	**7**	in	E	=	**16**	in @ ends



FORCES

Fascia Weight (incl. frames) = **4** psf = **12.0** plf

Wind Force (from page 5) = **57.5** psf ULT = **34.5** psf ASD

Due to the fascia's light weight, wind force will always control over seismic. = **138.0** plf ASD

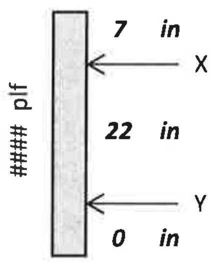
FRAME MEMBERS

Try: **1"x2"x1" 18 GA. Z** ASTM A653 SS Fy = 33 ksi

A = 0.173 in2 r = 0.4033 in2

S = 0.1088 in2

BENDING ON MEMBER XY



M_C = 0.282 kin

M_B = -0.280 kin

M_A = 0.000 kin

Mmax = **0.282** kin

From CFS:

M allow = 1.850 k*in **OK**

COMPRESSION ON MEMBER XZ

Length of XZ = 31.1 in

Rxn @ X = 339 lbs

Rxn @ Y = 207 lbs

P = **479** lbs

From CFS:

P allow = **1425** lb **OK**

SCREW CAPACITIES

#8 TEK G.F. = 177 LB Shear (Lapped 18 GA.)

#1/4 TEK G.F. = 230 LB Pullout (2 layers of 20 GA. base)

#1/4 TEK G.F. = 205 LB Shear (Lapped 20 GA.)

#1/4 TEK G.F. = 306 LB Shear (Lapped 18 GA.)

#1/4 TEK G.F. = 153 LB Pullout (18 GA.)

CHECK SCREWS

1	#1/4-14 TEK 1 @ 12 IN O.C.	Trib. Area =	1.5	sq.ft.	Shear F =	51.7 lb <	306 lb	**OK**
2	(2) #1/4 TEK @ EACH FRAME	Trib. Area =	6	sq.ft.	Tensile F =	207 lb <	306 lb	**OK**
	(Check Fasica Weight)	Trib. Area =	6	sq.ft.	Shear F =	24 lb <	306 lb	**OK**
3	3 #1/4 TEK @ EACH FRAME	Trib. Area =	12	sq.ft.	Shear F =	414 lb <	615 lb	**OK**
	(Check Tension from Member XY wind from right)				Tensile F =	339 lb <	690 lb	**OK**
4	#8 TEK @ 12 IN O.C.	Trib. Area =	1.5	sq.ft.	Shear F =	52 lb <	177 lb	**OK**

NOTE: *Fascia frame connections at X, Y, and Z are pinned, therefore, there will be no moment on the connection of the frames to the deck pans (screw mark #3) Forces will transfer to screws as direct tension and shear. There is no tension due to moment.*

FOR REFERENCE ONLY

PURLIN BRACE DESIGN (ASD)

Nodal Bracing as per AISC 360-16 Appendix 6, 6.3.1.b

Mmax. in Purlin (ASD) = **28.56** kft
Applied Factor of Safety = **1**
Mmax. For Design (M_r)= **342.72** kin

REQUIRED STRENGTH $P_{br} = \dfrac{0.02\, M_r\, C_d}{h_0} =$ 0.58 kips

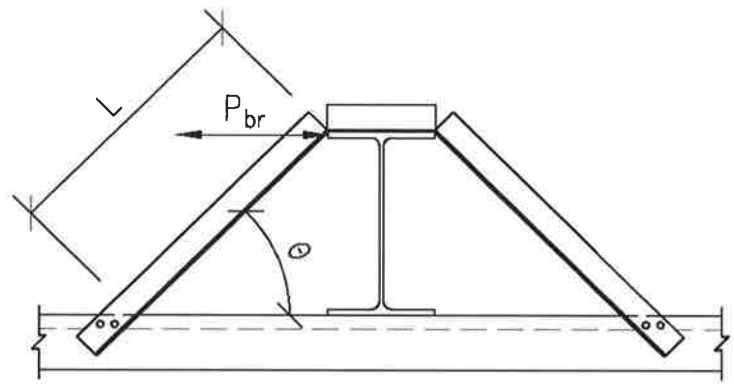
Tension only braces? **YES**

REQUIRED STIFFNESS $B_{br} = \Omega \left(\dfrac{10\, M_r\, C_d}{L_b\, h_0} \right) =$ 5.2 k/in

Purlin: **W12X19** Ω = 2.0

F_U	=	**65**	ksi	b_f	=	4.01	in
g	=	2.25	in	t_f	=	0.350	in
t_w	=	0.235	in	d	=	12.20	in

h_0 = 11.850 in (CL to CL flange)
C_d = 1.0 (single curvature)
L_b = **9.333** ft (unbraced length)



Brace: **L1.5x1.5x16 GA.** Ω = 2.0

F_Y	=	33	ksi	θ	=	45	degrees
E	=	29000	ksi	L	=	17.253	in
t	=	0.064	in	r_Y	=	0.4742	ksi
d	=	1.5	in				
A	=	0.187	in^2	A	=	0.095	in^2 (reduced for coped angle)

ACTUAL STIFFNESS $B_b = \cos^2\theta \left(\dfrac{A\,E}{L_b} \right) =$ 13.3 k/in > 5.2 k/in **STIFFNESS OK**

$P_{AXIAL} = \left(\dfrac{17.253\ \text{in}}{12.2\ \text{in}} \right)($ 0.58 kips $)$ = **0.82** kips

TENSILE STRENGTH $P_n = \dfrac{F_Y\, A}{\Omega} =$ 1.6 kips > 0.82 kips **STRENGTH OK**

CONNECTION FORCES: Shear Force = 578 lbs Tensile Force = 578 lbs

ATTACHEMENT:

PURLIN	3	#12 TEK 2	G.F. =	430	LB Shear (16 GA. min. base)	Shear F =	578	lb <	1290	lb	**OK**
SCREWS:	3	#12 TEK 2	G.F. =	515	LB Tension (3/16")	Tensile F =	578	lb <	1545	lb	**OK**
						Combined:	0.823	<	1.0		**OK**
DECK	3	#12 TEK 2	G.F. =	400	LB Shear (2 layers of 20 GA. base)	Shear F =	578	lb <	1200	lb	**OK**
SCREWS:	3	#12 TEK 2	G.F. =	400	LB Shear (2 layers of 20 GA. base)	Tensile F =	578	lb <	1200	lb	**OK**
						Combined:	0.964	<	1.0		**OK**

FOR REFERENCE ONLY

BEAM BRACE DESIGN (ASD)

Nodal Bracing as per AISC 360-16 Appendix 6, 6.3.1.b

Mmax. in Beam (ASD) = **81.3** kft
Applied Factor of Safety = **1.5**
Mmax. For Design (M_r)= **1463.4** kin

REQUIRED STRENGTH $P_{br} = \dfrac{0.02\, M_r\, C_d}{h_0} = 1.89$ kips

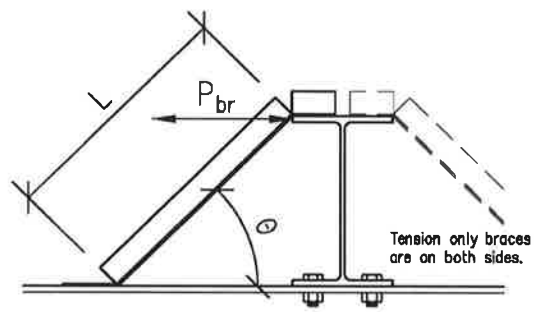
Tension only braces? **YES**

REQUIRED STIFFNESS $B_{br} = \Omega \left(\dfrac{10\, M_r\, C_d}{L_b\, h_0} \right) = 19.7$ k/in

Beam: **W16X31** $\Omega = 2.0$

F_U	=	**65**	ksi	b_f	=	5.53	in
g	=	2.75	in	t_f	=	0.440	in
t_w	=	0.275	in	d	=	15.90	in

h_0 = 15.460 in (CL to CL flange)
C_d = 1.0 (single curvature)
L_b = 8 ft (unbraced length)



Brace: **L2X2X3/16** $\Omega = 2.0$

F_Y	=	36	ksi	θ	=	45	degrees
E	=	29000	ksi	L	=	22.486	in
t	=	0.188	in	r_Y	=	0.612	ksi
d	=	2	in				
A	=	0.722	in^2	A	=	0.381	in^2 (reduced for coped angle)

ACTUAL STIFFNESS $B_b = \cos^2\theta \left(\dfrac{A\,E}{L_b} \right) = 60.2$ k/in > 19.7 k/in **STIFFNESS OK**

$P_{AXIAL} = \left(\dfrac{22.486 \text{ in}}{15.9 \text{ in}} \right)(1.89 \text{ kips}) =$ **2.68** kips

TENSILE STRENGTH $P_n = \dfrac{F_Y\, A}{\Omega} = 6.9$ kips > 2.68 kips **STRENGTH OK**

COMPRESSIVE STRENGTH
KL/r = 99.6 OK Fe >= 0.44 Fy Therefore, Fcr = Fy * 0.658^(Fy/Fe)
Fe = 28.88 ksi Therefore, Fa = 12.8 ksi
$P_n = \dfrac{F_{cr}\, A_g}{\Omega} = 4.62$ kips > 2.68 kips **STRENGTH OK**

CONNECTION FORCES: Shear Force = 1893 lbs Tensile Force = 1893 lbs

ATTACHEMENT:

BOLTS:									
1	3/4"Ø A307 G.F. =	5300	LB Shear	Shear F =	1893	lb <	5300	lb	**OK**
1	3/4"Ø A307 G.F. =	9940	LB Tension	Tensile F =	1893	lb <	9940	lb	**OK**
				Combined:	0.548	<	1.0		**OK**

FOR REFERENCE ONLY

Mountain View Engineering, Inc.
345 N. Main St. Ste. A
Brigham City, UT 84302

Project Title:
Engineer: CRH
Project ID:
Project Descr: Structural Design of Canopy

Steel Beam

Project File: 23-1096 ibc 2021 double row 9-7-2023.ec6

LIC# : KW-06014791, Build:20.23.08.30 MOUNTAIN VIEW ENGINEERING, INC. (c) ENERCALC INC 1983-2023

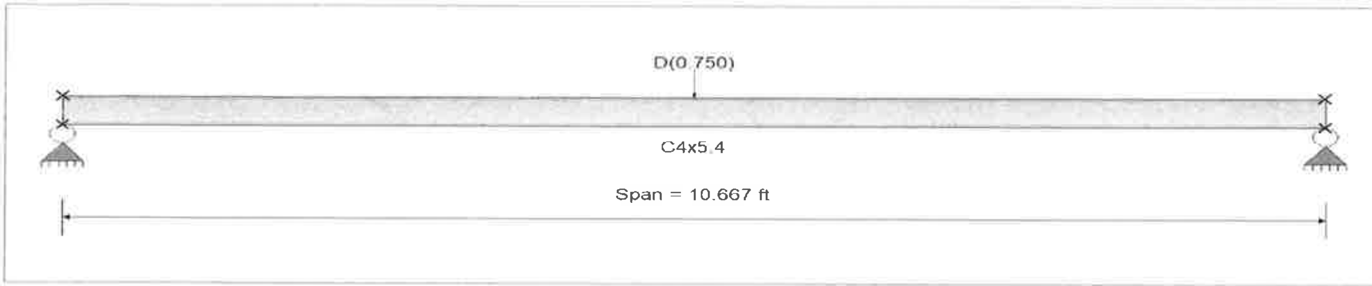
DESCRIPTION: Pipe Supports

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load(s) for Span Number 1
Point Load : D = 0.750 k @ 5.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.657** : 1	Maximum Shear Stress Ratio =	**0.042** : 1
Section used for this span	**C4x5.4**	Section used for this span	**C4x5.4**
Ma : Applied	2.077 k-ft	Va : Applied	0.4038 k
Mn / Omega : Allowable	3.159 k-ft	Vn/Omega : Allowable	9.520 k
Load Combination	+D+H	Load Combination	+D+H
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<240.0	n/a
Max Downward Total Deflection	0.309 in	Ratio =	414	>=180	Span: 1 : +D+H
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.404	0.404
Max Upward from Load Cases	0.404	0.404
D Only	0.404	0.404
H Only		

CFS Version 12.0.2
Section: Deck Pans.cfss
Deck Pans

Christopher R. Handy
Mountain View Engineering, Inc.

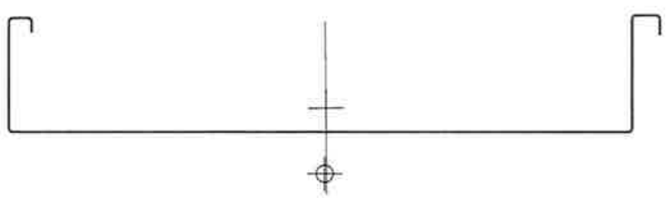
Rev. Date: 1/14/2019 9:16:13 AM
By: Christopher R. Handy
Printed: 4/21/2020 1:45:58 PM

Section Inputs

Material: A653 SS Grade 40
No cold work of forming strength increase.
No inelastic reserve strength increase.
Modulus of Elasticity, E 29500 ksi
Yield Strength, Fy 40 ksi
Tensile Strength, Fu 55 ksi
Torsion Constant Override, J 0 in⁴
Warping Constant Override, Cw 0 in⁶

Part 1, Thickness 0.0346 in (20 Gage)
Placement of Part from Origin:
X to center of gravity 0 in
Y to center of gravity 0 in
Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	0.375	90.000	0.076400	Single	0.000	0.000	0.188
2	0.625	180.000	0.076400	Single	0.000	0.000	0.313
3	2.938	270.000	0.076400	Single	0.000	0.000	1.469
4	16.000	0.000	0.076400	Single	0.000	0.000	8.000
5	3.000	90.000	0.076400	Single	0.000	0.000	1.500
6	0.750	0.000	0.076400	Single	0.000	0.000	0.375
7	0.500	270.000	0.076400	Single	0.000	0.000	0.250



CFS Version 12.0.2
Section: Deck Pans.cfss
Deck Pans

Christopher R. Handy
Mountain View Engineering, Inc.

Rev. Date: 1/14/2019 9:16:13 AM
By: Christopher R. Handy
Printed: 4/21/2020 1:46:54 PM

Full Section Properties

Area	0.82136 in²	Wt.	0.0027926 k/ft	Width	23.739 in
Ix	0.844 in⁴	rx	1.014 in	Ixy	0.275 in⁴
Sx(t)	0.3544 in³	y(t)	2.382 in	α	-89.439 deg
Sx(b)	1.3658 in³	y(b)	0.618 in		
Zx	0.4970 in³	Height	3.000 in		
Iy	28.989 in⁴	ry	5.941 in	x_o	-0.035 in
Sy(l)	3.5594 in³	x(l)	8.144 in	y_o	-1.681 in
Sy(r)	3.3823 in³	x(r)	8.571 in	jx	-0.056 in
Zy	4.3582 in³	Width	16.715 in	jy	10.891 in
I_1	28.992 in⁴	r_1	5.941 in	Cw	37.831 in⁶
I_2	0.841 in⁴	r_2	1.012 in	J	0.0003278 in⁴
Ic	29.833 in⁴	rc	6.027 in		
I_o	32.154 in⁴	r_o	6.257 in		

Fully Braced Strength - AISI S100-16/S1-18, US, ASD

Material Type: A653 SS Grade 40, Fy=40 ksi

Axial		Positive Bending		Positive Bending	
Pao	4.991 k	Maxo	0.5531 k-ft	Mayo	3.1288 k-ft
Ae	0.24957 in²	Ixe	0.802 in⁴	Iye	20.225 in⁴
Ta	19.673 k	Sxe(t)	0.3319 in³	Sye(l)	3.4040 in³
		Sxe(b)	1.3718 in³	Sye(r)	1.8773 in³
Shear					
Vay	2.593 k	Negative Bending		Negative Bending	
Vax	0.717 k	Maxo	0.5492 k-ft	Mayo	3.0362 k-ft
		Ixe	0.418 in⁴	Iye	19.228 in⁴
Torsion		Sxe(t)	0.2752 in³	Sye(l)	1.8217 in³
Ba	53.901 k-in²	Sxe(b)	0.2817 in³	Sye(r)	3.1213 in³

FOR REFERENCE ONLY

33

FOR REFERENCE ONLY

Rev. Date: 10/27/2023 10:51:59 AM
By: CHRIS
Printed: 10/27/2023 2:50:00 PM

Analysis Inputs

General
Member Orientation: Horizontal
Calculate global buckling using specification equations
Do not include torsion in member checks

Members

	Section File	Revision Date and Time
1	Deck Pans.cfss	1/14/2019 9:16:13 AM

	Material	Area (in²)	Length (ft)	Weight (k)
1	A653 SS Grade 40	0.82136	38.500	0.10752
	Total		38.500	0.10752

	Start Loc. (ft)	End Loc. (ft)	Braced Flange	R	kϕ (k)	Lm (ft)	ex (in)	ey (in)
1	0.000	38.500	None	0.0000	0.0000	20.000	0.000	0.000

Supports

	Type	Location (ft)	Bearing (in)	Fastened	K
1	XYT	2.917	1.00	Yes	1.0000
2	XYT	13.583	1.00	Yes	1.0000
3	XYT	23.750	1.00	Yes	1.0000
4	XYT	34.417	1.00	Yes	1.0000

Loading: Dead Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	90.000	0.000	38.500	-0.006665	-0.006665 k/ft
2	Concentrated	90.000	0.100		-0.020000 k, Width=1 in	
3	Concentrated	90.000	38.400		-0.020000 k, Width=1 in	

Loading: Roof Live Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	90.000	0.000	38.500	-0.026660	-0.026660 k/ft

Rev. Date: 10/27/2023 10:51:59 AM
By: CHRIS
Printed: 10/27/2023 2:50:00 PM

Loading: Snow Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	90.000	0.000	38.500	-0.024300	-0.024300 k/ft

Loading: Wind Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	270.000	0.000	38.500	0.048900	0.048900 k/ft

Load Combination: D
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000

Load Combination: D+Lr
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Roof Live Load	1.000

Load Combination: D+S
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Snow Load	1.000

Load Combination: D+0.6W
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Wind Load	0.600

Load Combination: D+0.75(0.6W+L+Lr)
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Live Load	0.750
4	Product Load	0.750
5	Roof Live Load	0.750
6	Wind Load	0.450

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```
Load Combination: D+0.75(0.6W+L+S)
Specification:    AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No
   Loading                    Factor
 1 Beam Self Weight            1.000
 2 Dead Load                   1.000
 3 Live Load                   0.750
 4 Product Load                0.750
 5 Snow Load                   0.750
 6 Wind Load                   0.450

Load Combination: 0.6D+0.6W
Specification:    AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No
   Loading                    Factor
 1 Beam Self Weight            0.600
 2 Dead Load                   0.600
 3 Wind Load                   0.600
```

Member Check - AISI S100-16/S1-18, US, ASD

```
Load Combination: D+0.75(0.6W+L+Lr)
Design Parameters at 13.583 ft, Left side:
Lx      10.666 ft    Ly      10.666 ft    Lt      10.666 ft
Kx      1.0000       Ky      1.0000       Kt      1.0000

Section: Deck Pans.cfss
Material Type: A653 SS Grade 40, Fy=40 ksi
Cbx     1.2324       Cby     1.0000       ex      0.0000 in
Cmx     1.0000       Cmy     1.0000       ey      0.0000 in
Braced Flange: None  kϕ           0 k
Red. Factor, R: 0    Lm      20.000 ft

Loads        P        Mx        Vy        My        Vx
            (k)     (k-in)     (k)      (k-in)     (k)
Total     0.0000    -6.250   -0.2974    0.000    0.0000
Applied   0.0000    -6.250   -0.2974    0.000    0.0000
Strength  2.9304     6.591    2.5934   29.243    0.6701

Interaction Equations
Eq. H1.2-1   (P, Mx, My)  0.000 + 0.948 + 0.000 = 0.948 <= 1.0
Eq. H2-1        (Mx, Vy)    Sqrt(0.899 + 0.013)= 0.955 <= 1.0
Eq. H2-1        (My, Vx)    Sqrt(0.000 + 0.000)= 0.000 <= 1.0
```


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Maximum Shears, Moments, and Deflections

Envelope of All Combinations, Y Direction

Location (ft)	Shear(l) (k)	Shear(r) (k)	Reaction (k)
2.917	-0.17010	0.25140	0.42150
13.583	-0.29745	0.26935	0.56679
23.750	-0.25382	0.26820	0.52202
34.417	-0.28070	0.23010	0.51080

Location (ft)	Moment (k-in)	Location (ft)	Deflection (in)
2.917	-3.3032	0.000	0.09790
7.803	4.0663	7.926	-0.21017
13.583	-6.2499	14.169	0.00500
18.817	2.2092	18.848	-0.06539
23.750	-5.3029	28.968	-0.13302
28.962	3.0842	32.544	0.01132
34.417	-6.1030	38.500	-0.15201

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Analysis Inputs

General
- Member Orientation: Horizontal
- Calculate global buckling using specification equations
- Do not include torsion in member checks

Members

	Section File	Revision Date and Time
1	Deck Pans.cfss	1/14/2019 9:16:13 AM

	Material	Area (in²)	Length (ft)	Weight (k)
1	A653 SS Grade 40	0.82136	39.000	0.10891
	Total		39.000	0.10891

	Start Loc. (ft)	End Loc. (ft)	Braced Flange	R	kϕ (k)	Lm (ft)	ex (in)	ey (in)
1	0.000	39.000	None	0.0000	0.0000	20.000	0.000	0.000

Supports

	Type	Location (ft)	Bearing (in)	Fastened	K
1	XYT	2.917	1.00	Yes	1.0000
2	XYT	13.583	1.00	Yes	1.0000
3	XYT	24.250	1.00	Yes	1.0000
4	XYT	34.917	1.00	Yes	1.0000

Loading: Dead Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	90.000	0.000	39.000	-0.006665	-0.006665 k/ft
2	Concentrated	90.000	0.100		-0.020000 k, Width=1 in	
3	Concentrated	90.000	38.650		-0.020000 k, Width=1 in	

Loading: Roof Live Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	90.000	0.000	39.000	-0.026660	-0.026660 k/ft

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Loading: Snow Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	90.000	0.000	39.000	-0.024300	-0.024300 k/ft

Loading: Wind Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1	Distributed	270.000	0.000	39.000	0.048900	0.048900 k/ft

Load Combination: D
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000

Load Combination: D+Lr
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Roof Live Load	1.000

Load Combination: D+S
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Snow Load	1.000

Load Combination: D+0.6W
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Wind Load	0.600

Load Combination: D+0.75(0.6W+L+Lr)
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Live Load	0.750
4	Product Load	0.750
5	Roof Live Load	0.750
6	Wind Load	0.450

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Load Combination: D+0.75(0.6W+L+S)
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

Loading	Factor
1 Beam Self Weight	1.000
2 Dead Load	1.000
3 Live Load	0.750
4 Product Load	0.750
5 Snow Load	0.750
6 Wind Load	0.450

Load Combination: 0.6D+0.6W
Specification: AISI S100-16/S1-18, US, ASD
Inflection Point Bracing: No

Loading	Factor
1 Beam Self Weight	0.600
2 Dead Load	0.600
3 Wind Load	0.600

Member Check - AISI S100-16/S1-18, US, ASD

Load Combination: D+0.75(0.6W+Lr)
Design Parameters at 13.583 ft, Left side:

Lx	10.666 ft	Ly	10.666 ft	Lt	10.666 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000

Section: Deck Pans.cfss
Material Type: A653 SS Grade 40, Fy=40 ksi

Cbx	1.2472	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		kφ	0 k		
Red. Factor, R: 0		Lm	20.000 ft		

Loads:	P (k)	Mx (k-in)	Vy (k)	My (k-in)	Vx (k)
Total	0.0000	-6.547	-0.2998	0.000	0.0000
Applied	0.0000	-6.547	-0.2998	0.000	0.0000
Strength	2.9304	6.591	2.5934	29.243	0.6701

Interaction Equations

```
Eq. H1.2-1   (P, Mx, My)  0.000 + 0.993 + 0.000 = 0.993 <= 1.0
Eq. H2-1        (Mx, Vy)     Sqrt(0.987 + 0.013)= 1.000 <= 1.0
Eq. H2-1        (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 1.0
```


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Maximum Shears, Moments, and Deflections

Envelope of All Combinations, Y Direction

Location (ft)	Shear(l) (k)	Shear(r) (k)	Reaction (k)
2.917	-0.17010	0.24908	0.41918
13.583	-0.29977	0.28158	0.58134
24.250	-0.26732	0.27126	0.53859
34.917	-0.27764	0.23010	0.50774

Location (ft)	Moment (k-in)	Location (ft)	Deflection (in)
2.917	-3.3032	0.000	0.08898
7.757	3.9307	7.880	-0.19826
13.583	-6.5472	13.929	0.00201
19.055	2.6974	19.067	-0.10115
24.250	-5.6351	29.522	-0.12174
29.522	2.9448	33.022	0.01105
34.917	-6.0430	39.000	-0.15803

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STRUCTURAL NOTES:

A. GENERAL

1. The contractor shall verify all conditions and dimensions at the site.
2. Observation visits to the site by the design engineer shall neither be construed as inspection nor approval of construction.
3. During and after construction, builder and/or owner shall keep loads on the structure within limits of design loads.
4. Typical details shall apply where specific details are not shown.
5. These drawing are for use in construction of a single canopy (U.N.O.) with materials provided and fabricated by BESTWORTH ROMMEL LLC only. Use by another canopy manufacturer will invalidate MVE's certification.

B. 2022 OREGON STRUCTURAL SPECIALTY CODE DESIGN CRITERIA

1. Roof Live Load . . . . 20 psf (reducible)
2. Canopy Dead Load . . . . 9 psf
3. Ground Snow Load P_g . . . . 11 psf
   Flat-roof Snow Load P_f . . . . 13.2 psf
   Rain-on-Snow Surcharge . . . . 5 psf
   Exposure Factor C_e 1.0
   Importance Factor I_s 1.2
   Thermal Factor C_t 1.2
4. Rainfall Intensity . . . . 1.5 inches per hour
5. Basic Design Wind Speed V . . . . 110 mph
   Allowable Stress Design Wind Speed V_{asd} . . 85 mph
   Risk Category IV
   Exposure C
   Component & Cladding . . . . See ASCE 7-16 Chapter 30
6. Earthquake Design Data
   Importance Factor I_e 1.5
   S_s . . . . 0.846g S_{DS} 0.677g
   S_1 . . . . 0.375g S_{D1} . . . . 0.481g (Fv=1.925)
   Site Class . . . . D (default)
   Seismic Design Category . . . . D
   Seismic Force Resisting System . . . . G.2. Steel ordinary cantilever column systems
   Design Base Shear . . . . 50.93 kips
   Seismic Response Coefficient C_s 0.81
   Response Modification Factor R 1.25

C. FOUNDATION – BY OTHERS

D. CONCRETE AND REINFORCEMENT – BY OTHERS

1. Concrete Foundations – f'c=5000 p.s.i. min. 1
2. Grout under steel base plates shall be non-shrink grout complying with ASTM C1107, f'c=5000 p.s.i., and installed per manufacturer's recommendations. Special inspection required.

E. STRUCTURAL STEEL

1. Structural steel shall be fabricated and erected in accordance with the latest edition of the following:
   a. AISC 360 "Specifications for the Design Fabrication and Erection of Structural Steel Buildings."
   b. AISC 303 "Code of Standard Practice" excluding the following sections: 4.4, 4.4.1, 4.4.2, and 7.15.
   c. AISI S100 "North American Specification for the Design of Cold-formed Steel Structural Members."
2. Structural steel shall conform to ASTM A36 for channels, plates, and angles, ASTM A500 grade C for HSS (46 k.s.i. min. pipes and 50 k.s.i. min. for squares and rectangles), and ASTM A992 grade 50 for all beams and purlins.
3. All welding shall be performed by certified welders in accordance with the American Welding Society using E-70xx electrodes.
4. On all bolted connections, use ASTM A307 bolts unless otherwise specified (no special inspection required).
5. Metal roof deck shall be 20 ga. ASTM A653 grade 40. All other cold form steel members shall be ASTM A653 grade 33.
6. All exposed structural steel shall be primed with rust-inhibitive steel primer paint. All welds and other exposed areas shall be primed with prime coat after fabrication. Sheet metal components are either galvanized or factory pre-finished baked-on enamel.

F. SPECIAL INSPECTION REQUIREMENTS

1. Special inspection shall be provided by owner according to Chapter 17 of the OSSC. The special inspector shall observe the work for conformance with the contract documents. The special inspector shall send reports to the owner, building official, architect, engineer and contractor. All discrepancies shall be brought to the attention of the contractor for correction. The special inspector shall submit a final signed report stating that the special inspection work was, to the best of his/her knowledge, in conformance with the plans specifications and applicable workmanship provision of the OSSC.
2. Continuous special inspection means the full-time observation of work requiring special inspection by an approved special inspector who is present in the area where the work is being performed. Periodic special inspection means the part-time or intermittent observation of work requiring special inspection by an approved special inspector who is present in the area where the work has been or is being performed and at the completion of the work.
3. The following systems are subject to the special inspection requirements noted herein:
   a. The Seismic-Force-Resisting System including:
      i. Columns iv. Purlins
      ii. Column Base Connection v. Beams
      iii. Footings
4. Special inspection is required for the following work. All inspections to be continuous unless noted otherwise:
   a. Welding – See Table 1 at right for inspection requirements. Special inspection is not required where welding is performed in an approved fabricator shop as per OSSC 1705.2.
   b. Bolting – See Table 2 for inspection requirements.
   c. Cold Formed Steel Deck (OSSC 1705.2.2) – N.A..
   d. Other Steel Elements – See Table 3 for inspection requirements.
   e. Concrete Construction (OSSC 1705.3)
      i. Special inspection for concrete foundations will be determined by the foundation designer (not MVE).
      ii. Special inspection of non shrink grout required as per Table 5.
      iii. Special inspection of anchor rods required as per Table 5.
   f. Masonry Construction (OSSC 1705.4) – N.A.
   g. Wood Construction (OSSC 1705.5) – N.A.
   h. Soils (OSSC 1705.6) – See Table 4 for inspection requirements.
   j. Driven Deep Foundations (OSSC 1705.7) – N.A.
   k. Cast-in-place Deep Foundations (OSSC 1705.8) – N.A.
   m. Helical Pile Foundations (OSSC 1705.9) – N.A.
   n. Special Inspections for Wind Resistance (OSSC 1705.11) – N.A. (V_{asd} < 110 mph)
   p. Special Inspections for Seismic Resistance (OSSC 1705.12) – See Tables, 1, 2, and 3 for inspection requirements.
5. Special inspector must be qualified as required by OSSC 1704.2.1 and approved by the local building department.
6. Structural Observations: None Required

TABLE 1 – INSPECTION TASKS FOR WELDING

VERIFICATION AND INSPECTION TASK	CONTINUOUS	PERIODIC	DETAILED INSTRUCTIONS
1. PRIOR TO WELDING (TABLE N5.4-1, AISC 360-16):			
a. Verify welder qualification records and continuity records	X	–	
b. WPS available	X	–	
c. Manufacturer certifications for welding consumables available	X	–	
d. Material identification	–	X	Verify type and grade of material.
e. Welder identification system	–	X	A system shall be maintained by which a welder who has welded a joint or member can be identified.
f. Fit-up of groove welds	–	X	Verify joint preparation, dimensions, cleanliness, tacking, and backing.
g. Fit-up of CJP groove welds of HSS T-, Y-, and K-joints without backing	X	–	Verify joint preparation, dimensions, cleanliness, and tacking.
h. Access holes	–	X	Verify configuration and finish.
j. Fit-up of fillet welds	–	X	Verify alignment, gaps at root, cleanliness of steel surfaces, and tack weld quality and location.
k. Check welding equipment	–	X	
2. DURING WELDING (TABLE N5.4-2, AISC 360-16):			
a. Control and handling of welding consumables	–	X	Verify packaging and exposure control.
b. Cracked tack welds	–	X	Verify welding does not occur over cracked tack welds.
c. Environmental conditions	–	X	Verify wind speed is within limits as well as precipitation and temperature.
d. WPS followed	–	X	Verify items such as settings on welding equipment, travel speed, welding materials, shielding gas type/flow rate, preheat applied, interpass temperature maintained, and proper position.
e. Welding techniques	–	X	Verify interpass and final cleaning, each pass is within profile limitations, and quality of each pass.
f. Steel headed stud anchors	X	–	Verify placement and installation.
3. AFTER WELDING (TABLE N5.4-3, AISC 360-16):			
a. Welds cleaned	–	X	Verify that welds have been properly cleaned.
b. Size, length, and location of welds	X	–	
c. Welds meet visual acceptance criteria	X	–	
d. Arc strikes	X	–	
e. k-area	X	–	
f. Backing & weld tabs removed	X	–	
g. Repair activities	X	–	
h. Document acceptance or rejection of welded joint/member	X	–	
4. NONDESTRUCTIVE TESTING (SECTION N5.5, AISC 360-16, SECTION J6.2 AISC 314-16):			
a. CJP welds (Risk Cat. II)	–	X	Ultrasonic testing shall be performed on 10% of CJP groove welds in butt, T-, and corner joints subject to transversely applied tension loading in materials 5/16 inch thick or greater. Testing rate must be increased to 100% if > 5% of welds tested have unacceptable defects.

TABLE 2 – INSPECTION TASKS FOR BOLTING

VERIFICATION AND INSPECTION TASK	CONTINUOUS	PERIODIC	DETAILED INSTRUCTIONS
1. PRIOR TO BOLTING (TABLE N5.6-1, AISC 360-16): Not required, only snug-tight joints are specified [per Section N5.6(a) of AISC 360-16].			
2. DURING BOLTING (TABLE N5.6-2, AISC 360-16): Not required, only snug-tight joints are specified [per Section N5.6(a) of AISC 360-16].			
3. AFTER BOLTING (TABLE N5.6-3, AISC 360-16):			
a. Document acceptance or rejection of bolted connections	X	–	

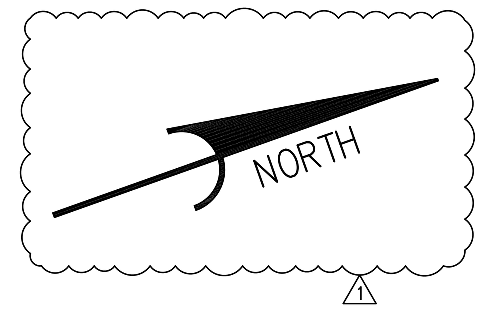
TABLE 3 – INSPECTION TASKS FOR OTHER STEEL ELEMENTS

TYPE	CONTINUOUS	PERIODIC	DETAILED INSTRUCTIONS
4. OTHER STEEL INSPECTIONS (SECTION N5.8, AISC 360-16; Tables J8-1 & J10-1, AISC 341-16):			
a. Structural steel details	–	X	All fabricated steel or steel frames shall be inspected to verify compliance with the details in the construction documents, such as braces, stiffeners, member locations, and proper application of joint details at each connection.
b. Anchor rods and other embedments supporting structural steel	–	X	Shall be on the premises during the placement of anchor rods and other embedments supporting structural steel for compliance with construction documents. Verify the diameter, grade, type, and length of the anchor rod or embedded item, and the extent or depth of embedment prior to placement of concrete.

TABLE 4 – REQUIRED SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION (OSSC TABLE 1705.3)

TYPE	CONTINUOUS	PERIODIC	DETAILED INSTRUCTIONS
1. Inspect reinforcement and verify placement.	–	X	Verify prior to placing concrete that reinforcing is of specified type, grade and size; that it is free of oil, dirt and rust; that it is located and spaced properly; that hooks, bends, ties, stirrups and supplemental reinforcement are placed correctly; that lap lengths, stagger and offsets are provided; and that all mechanical connections are installed per the manufacturer's instructions and/or evaluation report. ACI 318: Ch. 20, 25.2, 25.3, 26.6.1-26.6.3, OSSC 1908.4
2. Welding of reinforcing steel (AWS D1.4, ACI 318: 26.6.4) – N.A.			
3. Anchors & embeds cast in concrete	–	X	Verify anchor rods and embed plates are of specified type, grade, and size. Verify anchor rod placement and embedment. All anchor rods and embed plates shall be tied in place and secured prior to placing concrete, no "wet-setting" allowed. ACI 318: 17.8.2
4. Anchors post-installed in hardened concrete members			
a. Adhesive anchors installed horizontally or vertically to resist sustained tension loads.	X	–	All post-installed anchors shall be specially inspected as required by the approved ICC-ES report. ACI 318: 17.8.2.4.
b. Mechanincal anchors and adhesive anchors not defined in 4.a.	–	X	All post-installed anchors shall be specially inspected as required by the approved ICC-ES report. ACI 318: 17.8.2.
5. Verify use of required design mix.	–	X	Verify that all mixes used comply with the approved construction documents; ACI 318: Ch. 19, 26.4.3, 26.4.44; and OSSC 1904.1, 1904.2, 1908.2, 1908.3.
6. Prior to placement, fabricate specimens for strength tests, perform slump & air content test, and determine concrete temperature.	X	–	ASTM C172, ASTM C31, ACI 318: 26.4, 26.12, OSSC 1908.10
7. Inspect concrete placement for proper application techniques.	X	–	ACI 318: 26.5, OSSC 1908.6, 1908.7, 1908.8

NOTE: THE BASE OF THE COLUMN, BASE PLATE, AND ANCHOR RODS SHALL BE COATED IN GALVANIZING PAINT 2.0-3.0 MILS THICK, INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS, IN ORDER TO PROTECT THE CONNECTION FROM CORROSION. 1



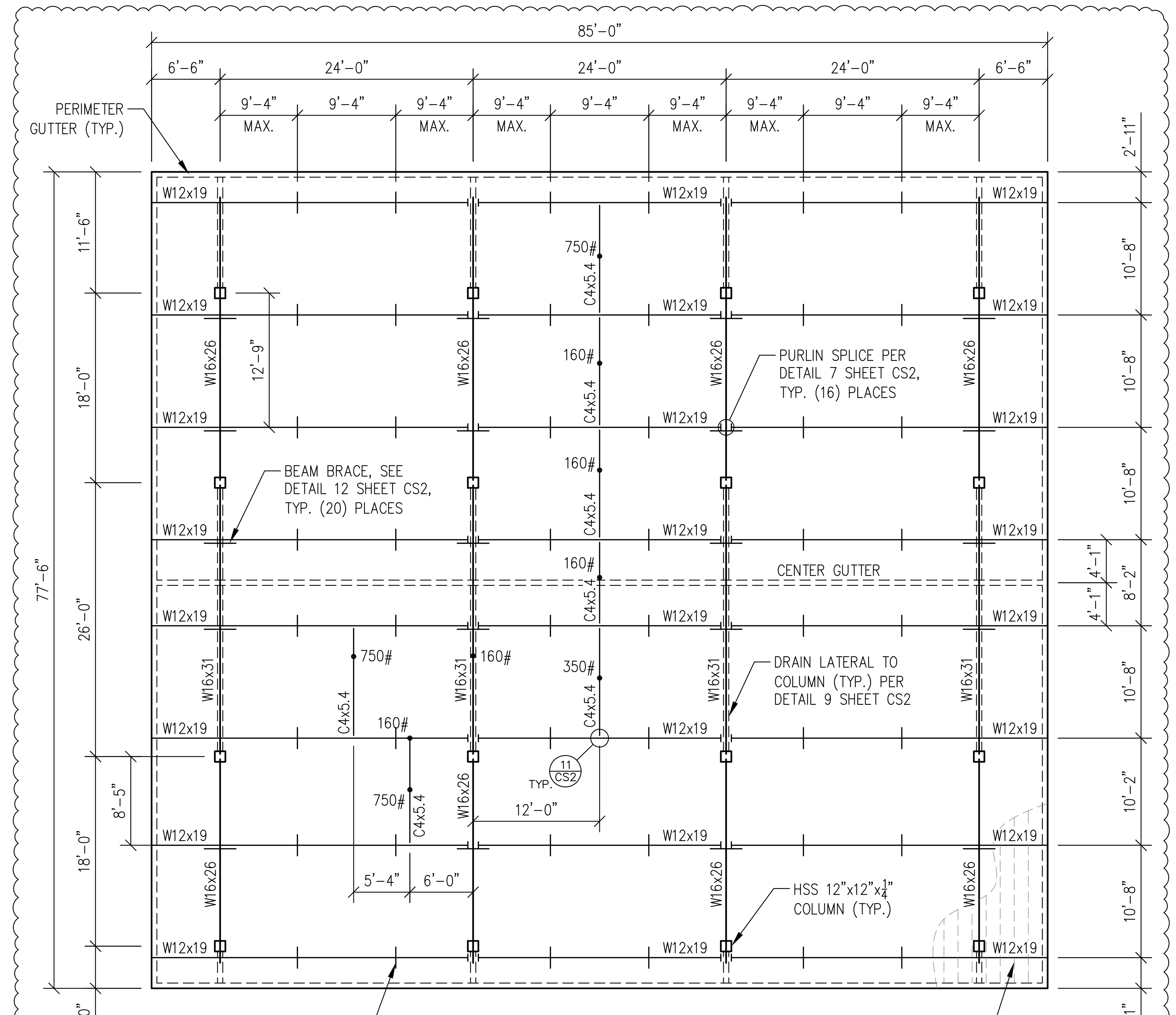
• ???# INDICATES CONCENTRATED LOAD FROM PIPE SUPPORTS. SUM OF ALL LOADS IS 3400 LBS.

NOTE: THE CANOPY FASCIA IS REYNOBOND ALUMINUM COMPOSITE MATERIAL (ACM) AND HAS BEEN TESTED IN ACCORDANCE WITH ASTM E84 AND D1929:

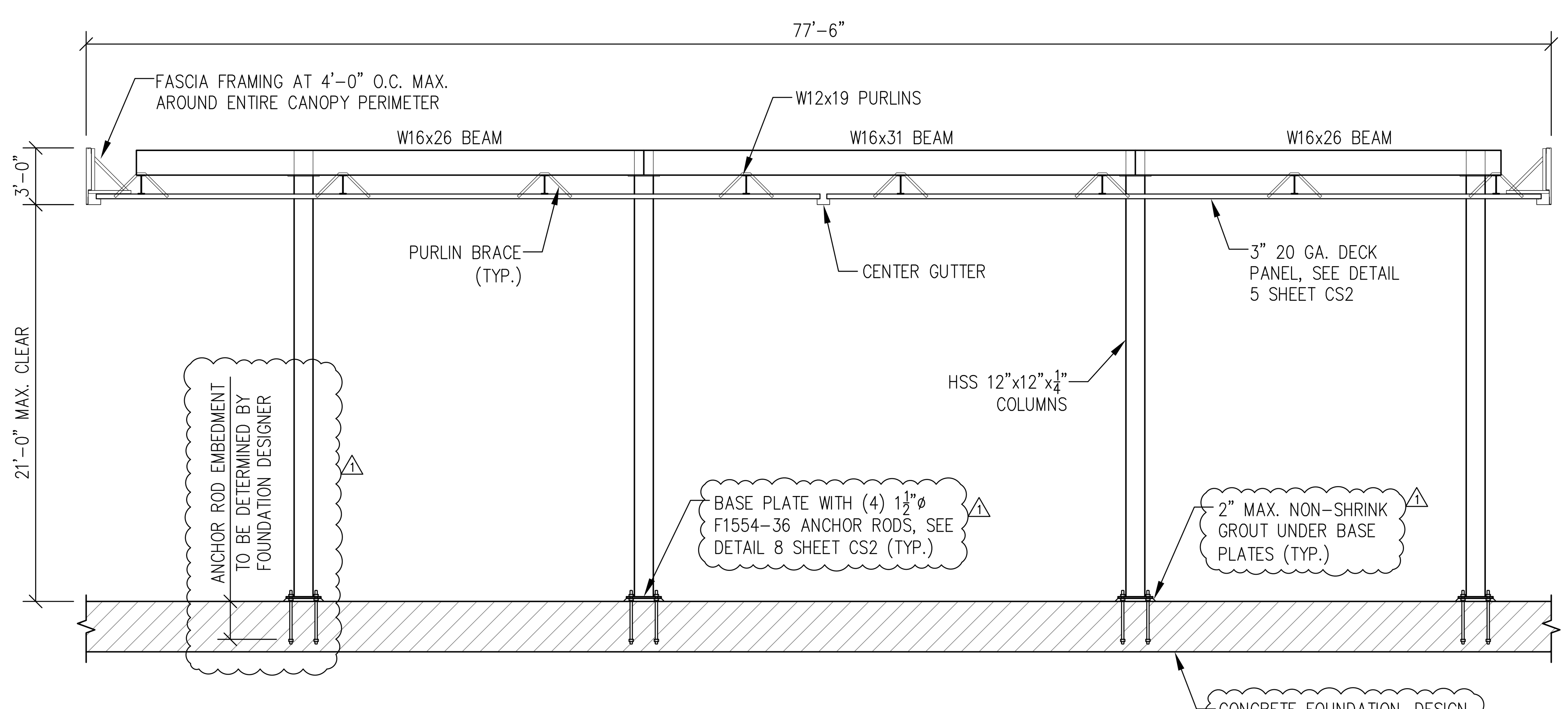
Thickness:	3 mm
Flame Spread Index:	0
Smoke Developed Index:	50
Minimum Self-Ignition Temp.:	650°

SEISMIC FORCE RESISTING SYSTEM (SFRS) INFO

A. Refer to AISC 341-16 Section A4 and ASCE 7-16.
B. Designation of the SFRS: G.2. Steel ordinary cantilever column systems (ASCE 7-16 Table 12.2-1) See AISC 341-16 Section E5 (OCCS).
C. R = 1.25, Cs = 0.81
D. Analysis Procedure = Equivalent Lateral Force
E. Members and Connections that are part of the SFRS:
   a. Columns d. Purlins
   b. Column Base Connection e. Beams
   c. Footings
F. Protected Zones – N.A.
G. See details and notes for connection configurations, material specifications, and sizes.
H. Weld filler materials for welds part of the SFRS:
   a. Yield Strength = 58 ksi min.
   b. Tensile Strength = 70 ksi min.
   c. Elongation = 22% min.
   d. CVN Toughness = 20 ft-lb min. @ 0° F
   See details and notes for other welding requirements.
J. Demand Critical Welds – N.A.
K. Lowest Anticipated Service Temperature – 0° F



FRAMING PLAN
SCALE: 1/8"=1'-0"



SECTION/ELEVATION
SCALE: 3/16"=1'-0"

PROJECT: TANK CANOPY PORTLAND INTERNATIONAL AIRPORT
LOCATON: 5000 NE MARINE DRIVE PORTLAND, OREGON
SUPPLIER: BESTWORTH ROMMEL LLC

MOUNTAIN VIEW ENGINEERING, INC.
Structural Engineering Consulting Design
345 North Main Street Ste. A, Brigham City, Utah 84302 (435) 734-9700 Fax (435) 734-9519

PLAN ISSUE DATES

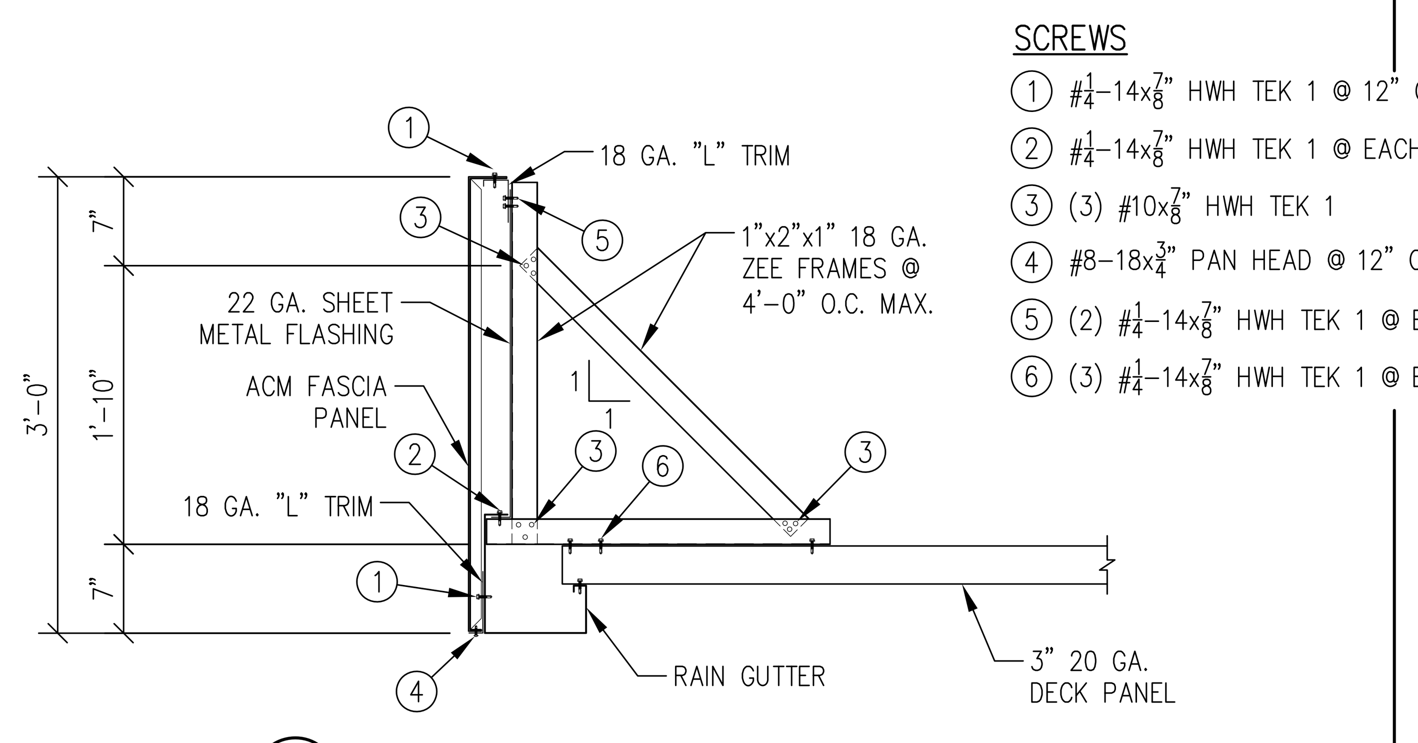
#	DATE:	BY:	DESCRIPTION:
	10-31-23	C.R.H.	FOR PERMIT
1	11-9-23	C.R.H.	REVISED PER REVIEW

NOV 09 2023
STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 95267PE OREGON SEP. 10, 2019 BRAD WALLACE
Brad Wallace
EXPIRES JUN 30 2024

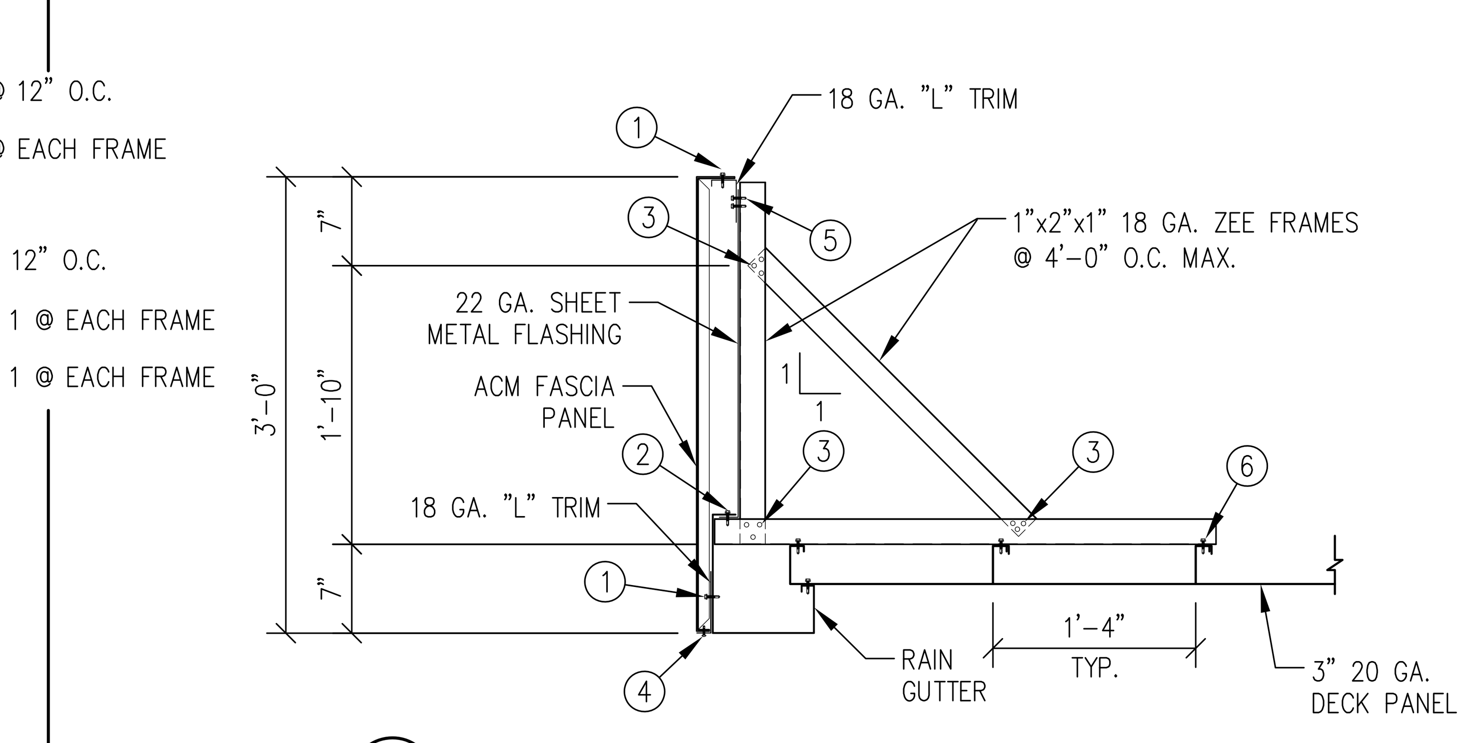
SHEET NUMBER:
CS1

DRAWN BY:	C.R.H.
ENGINEER:	C. HANDY
B-R JOB NUMBER:	363261
MVE JOB NUMBER:	23-1096

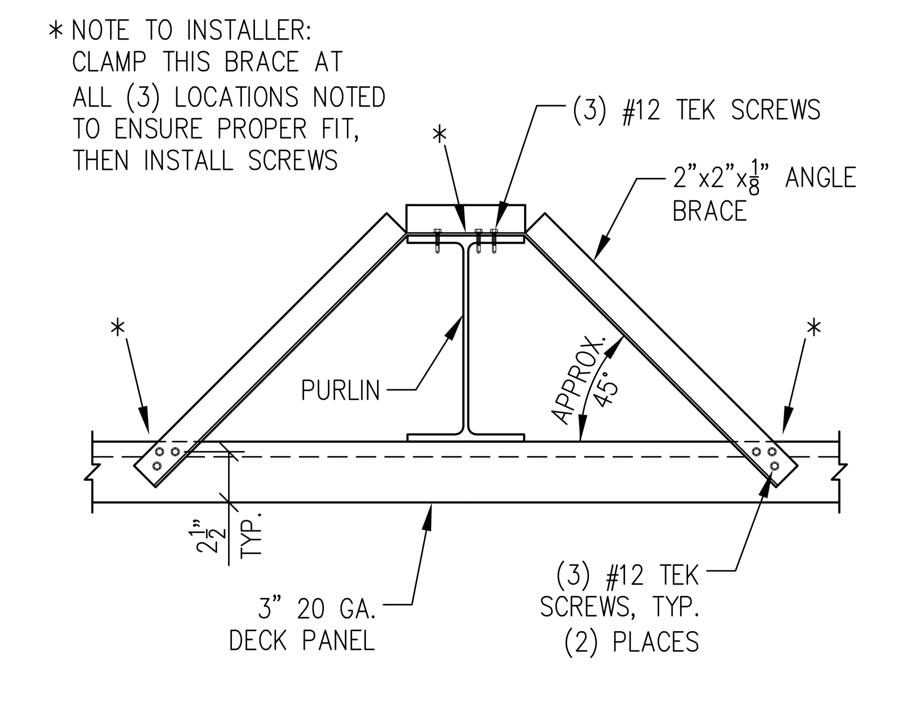
FOR REFERENCE ONLY



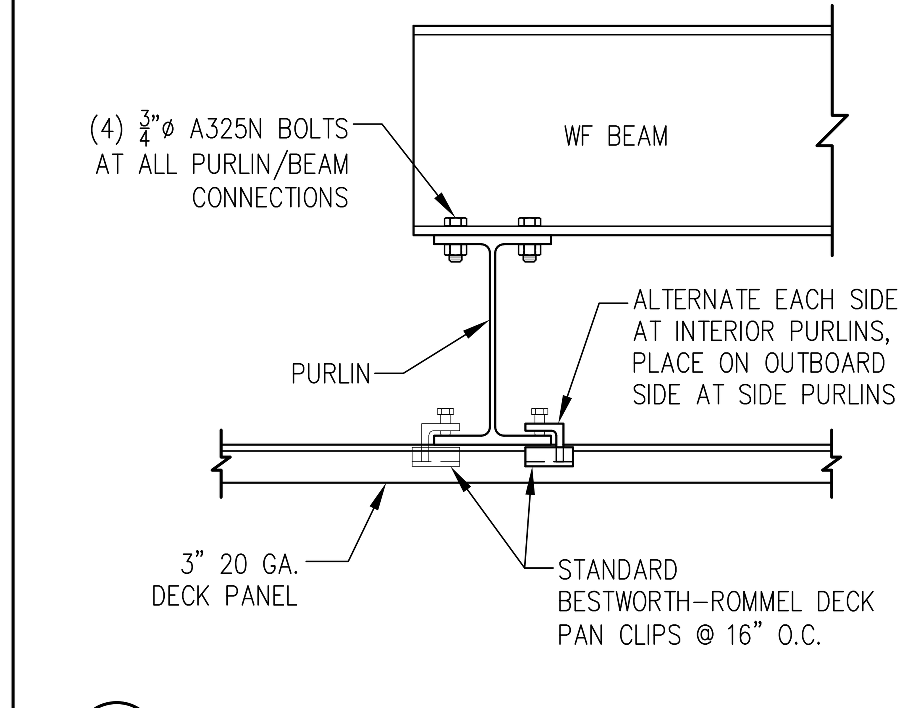
1 TYPICAL SIDE FASCIA DETAIL
1"=1'-0"



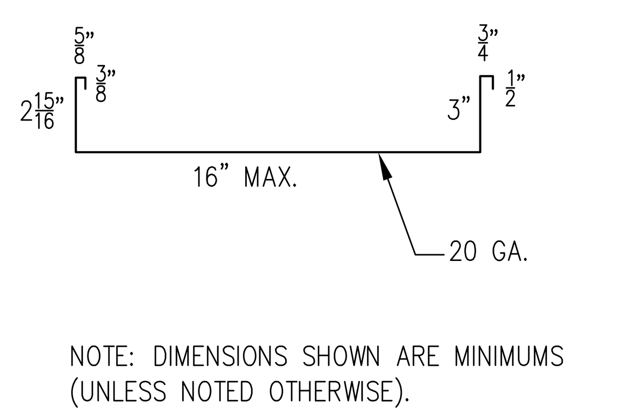
2 TYPICAL END FASCIA DETAIL
1"=1'-0"



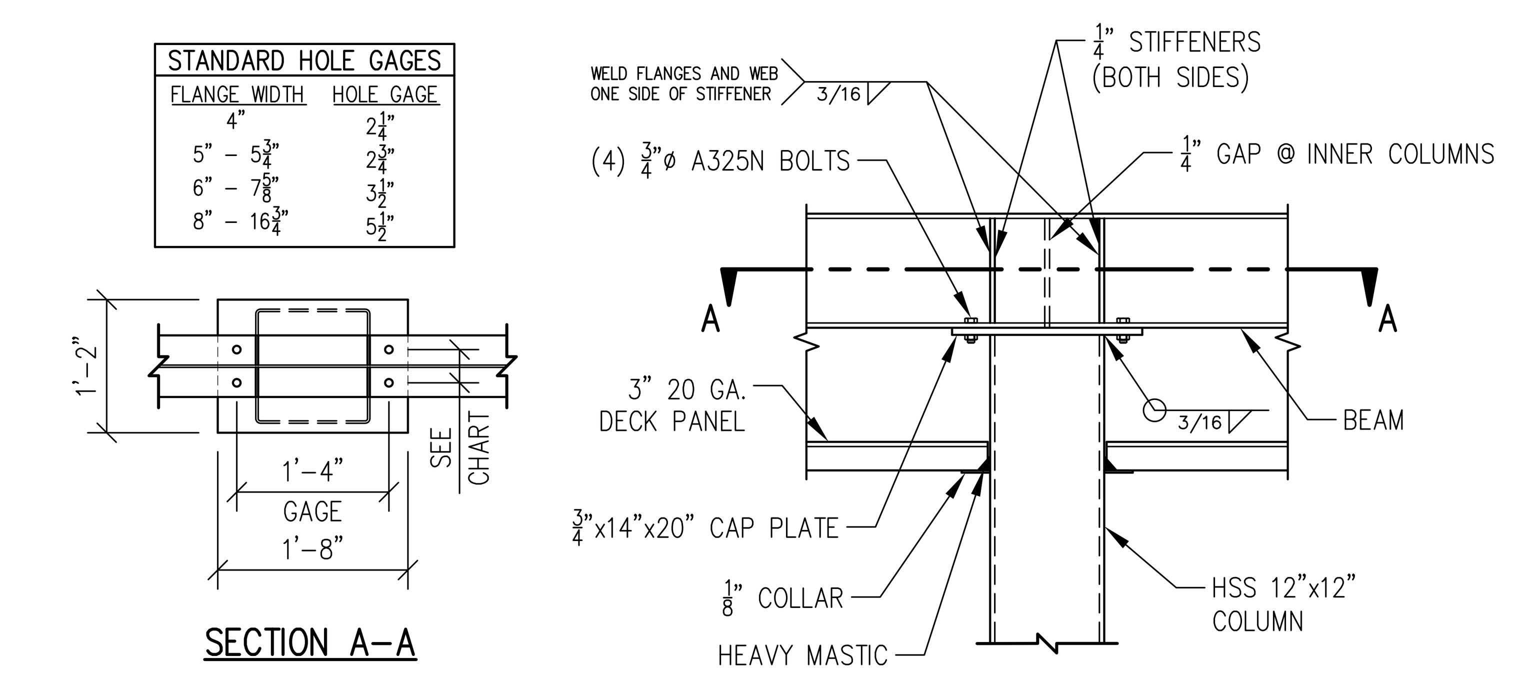
3 PURLIN BRACING DETAIL
N.T.S.



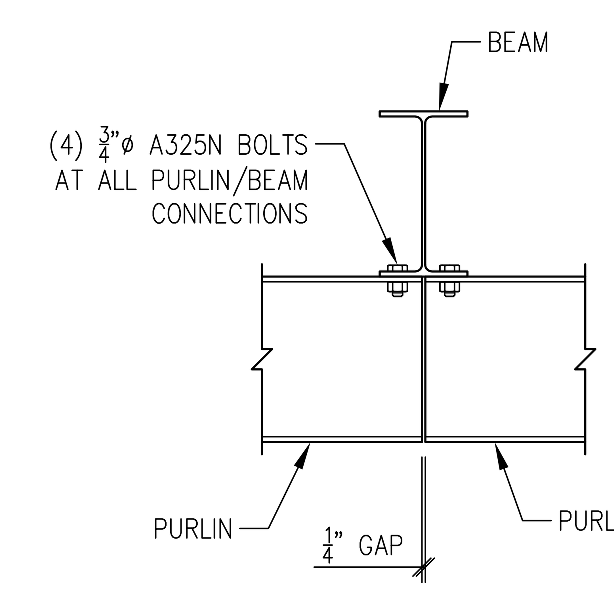
4 BEAM/PURLIN/DECK CONNECTION DETAIL
N.T.S.



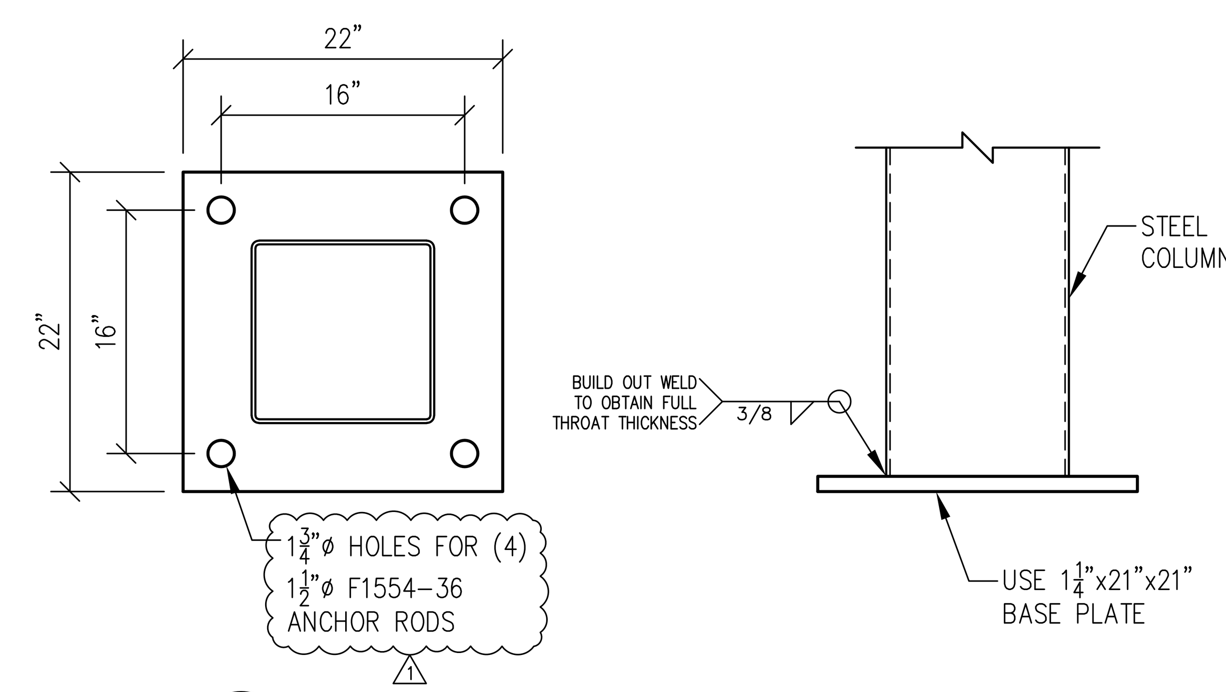
5 DECK PAN DETAIL
N.T.S.



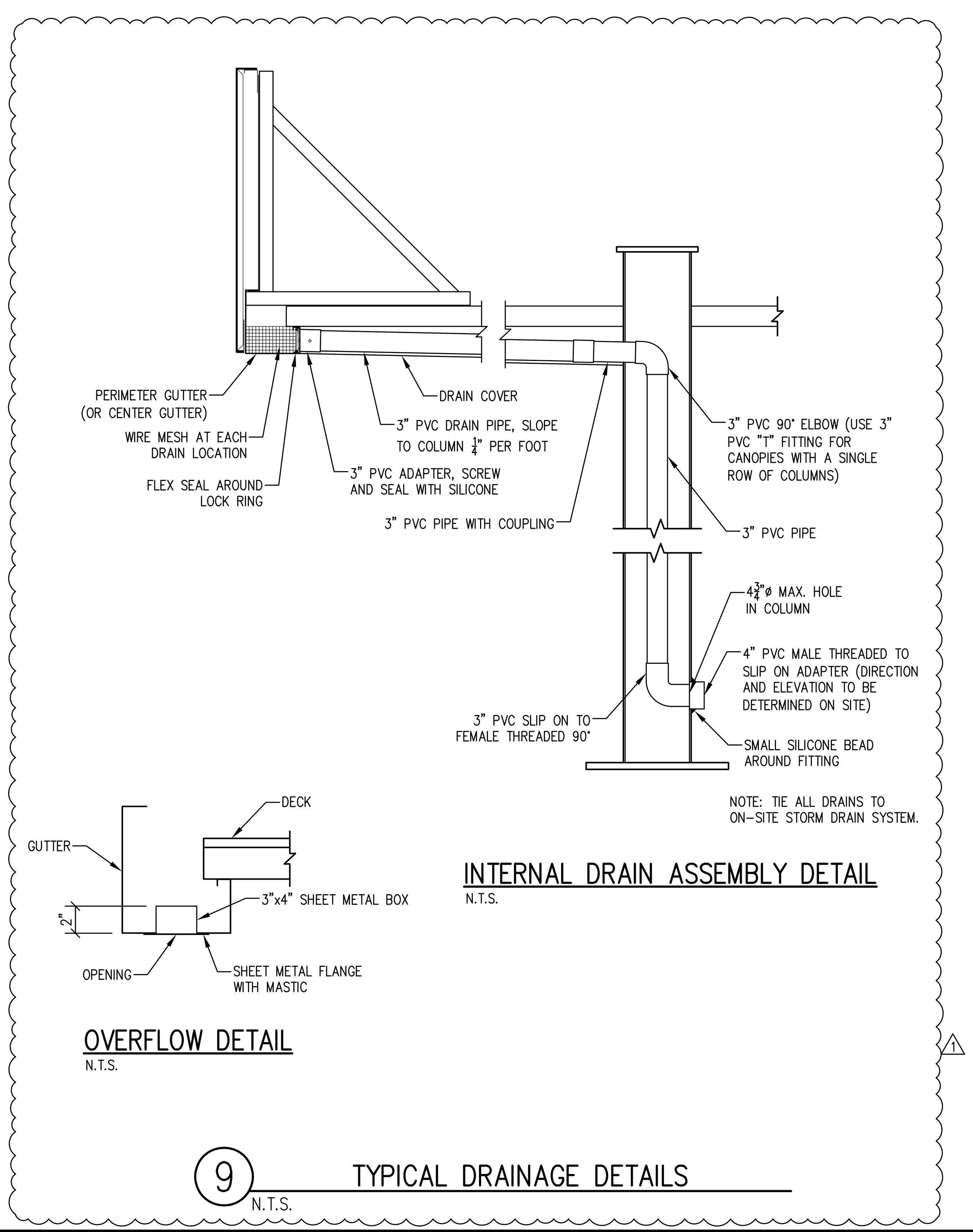
6 COLUMN/BEAM CONNECTION DETAIL
3/4"=1'-0"



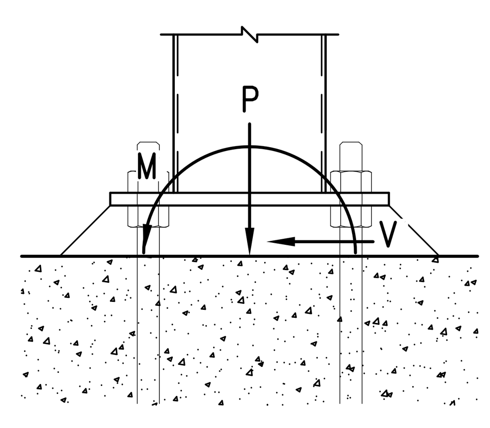
7 PURLIN SPLICE DETAIL
N.T.S.



8 BASE PLATE DETAIL
N.T.S.



9 TYPICAL DRAINAGE DETAILS
N.T.S.



REACTION DIAGRAM

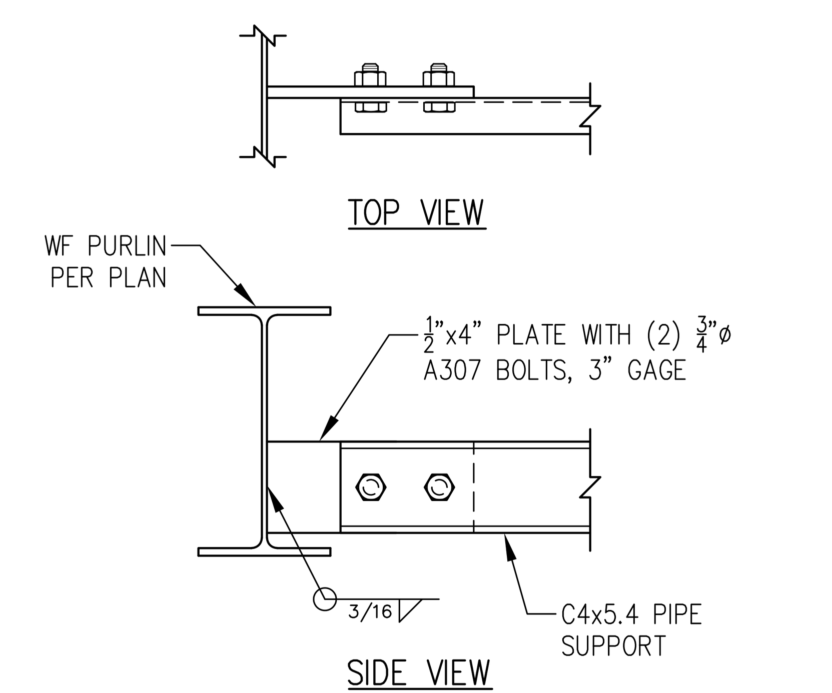
MAXIMUM REACTIONS:

NOTE: THE WORST CASE COLUMN REACTIONS ARE SHOWN BELOW. ALL COLUMNS ARE FIXED-BASE. ALL FORCES ARE UNFACTORED. WIND AND SEISMIC FORCES ARE AT ULTIMATE LEVEL. Ω IS INCLUDED IN ALL SEISMIC FORCES. DESIGN FOUNDATIONS TO RESIST LATERAL LOADS AND MOMENTS APPLIED IN ANY DIRECTION.

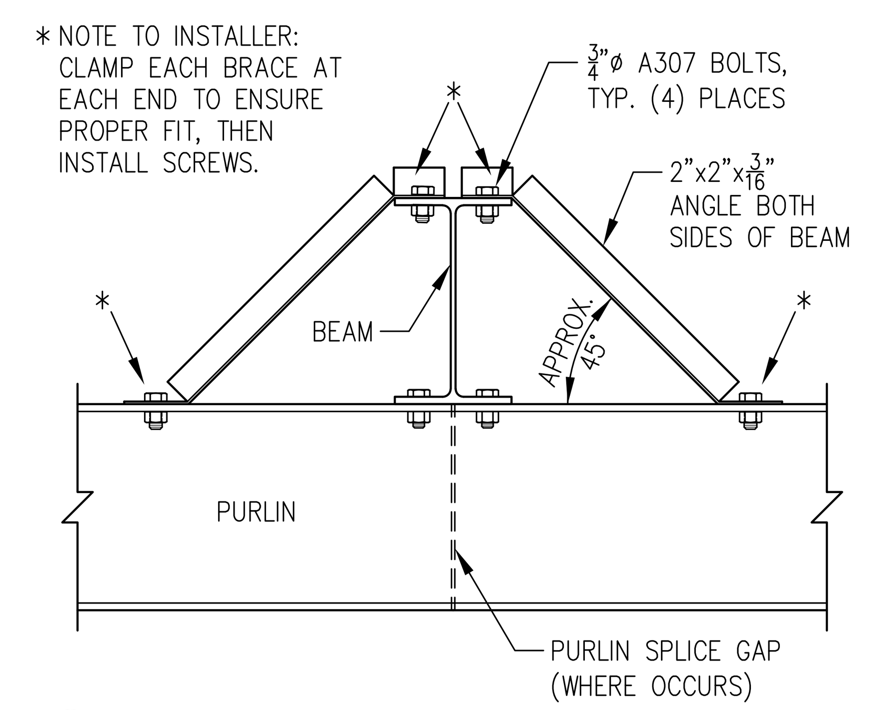
		FDNT
P =	6.55 K DEAD	
	11.69 K ROOF LIVE (7.83 K REDUCED)	
	10.63 K SNOW	
	14.29 K WIND DOWNWARD	
	-13.1 K WIND UPLIFT	
V =	±1.06 K WIND	
	±5.31 K SEISMIC	+/- 4.08 K SEISMIC
M =	±23.88 K-FT WIND	
	±119.84 K-FT SEISMIC	+/- 92.18 K-FT SEISMIC

OMEGA UTILIZED = 1.3 PER MOUNTAIN VIEW ENGINEERING CALCULATIONS

10 TYPICAL REACTIONS
N.T.S.



11 PIPE SUPPORT CONNECTION DETAIL
N.T.S.



12 BEAM BRACING DETAIL
N.T.S.

PROJECT: TANK CANOPY
PORTLAND INTERNATIONAL AIRPORT
LOCATION: 5000 NE MARINE DRIVE
PORTLAND, OREGON
SUPPLIER: BESTWORTH ROMMEL LLC

MOUNTAIN VIEW ENGINEERING, INC.
Structural Engineering · Consulting · Design
345 North Main Street Ste. A, Brigham City, Utah 84302 (435) 734-9700 Fax (435) 734-9519

PLAN ISSUE DATES

DATE:	BY:	DESCRIPTION:
10-31-23	C.R.H.	FOR PERMIT
11-9-23	C.R.H.	REVISED PER REVIEW

NOV 09 2023
REGISTERED PROFESSIONAL STRUCTURAL ENGINEER
95267PE
OREGON
SEP. 10, 2019
BRAD WALLACE
EXPIRES JUN 30 2024

SHEET NUMBER:
CS2

DRAWN BY:	C.R.H.
ENGINEER:	C. HANDY
B-R JOB NUMBER:	363261
MVE JOB NUMBER:	23-1096