CALCULATIONS FOR

Job:	**HY-VEE PICKUP CANOPY**	**JOB (17704)**
Address:	**LEE'S SUMMIT, MO.**	
	40'-0" X 45'-7" X 14'-0" & WALKWAY (8) COL CANOPY	
	(These calculations apply to the job at this address only.)	
Client:	**ARNING COMPANIES**	

STATE OF MISSOURI
STEVEN WILLIAM SCHAUB
NUMBER
E-2001006052
REGISTERED PROFESSIONAL ENGINEER

MAY 19 2021

Index to Calculations

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JOB # 0890 -21

S.E. CONSULTANTS, INC.

5800 East Thomas Road, Suite 104
Scottsdale, Arizona 85251
(480) 946-2010 Fax: (480) 946-1909

SPECIFICATIONS:

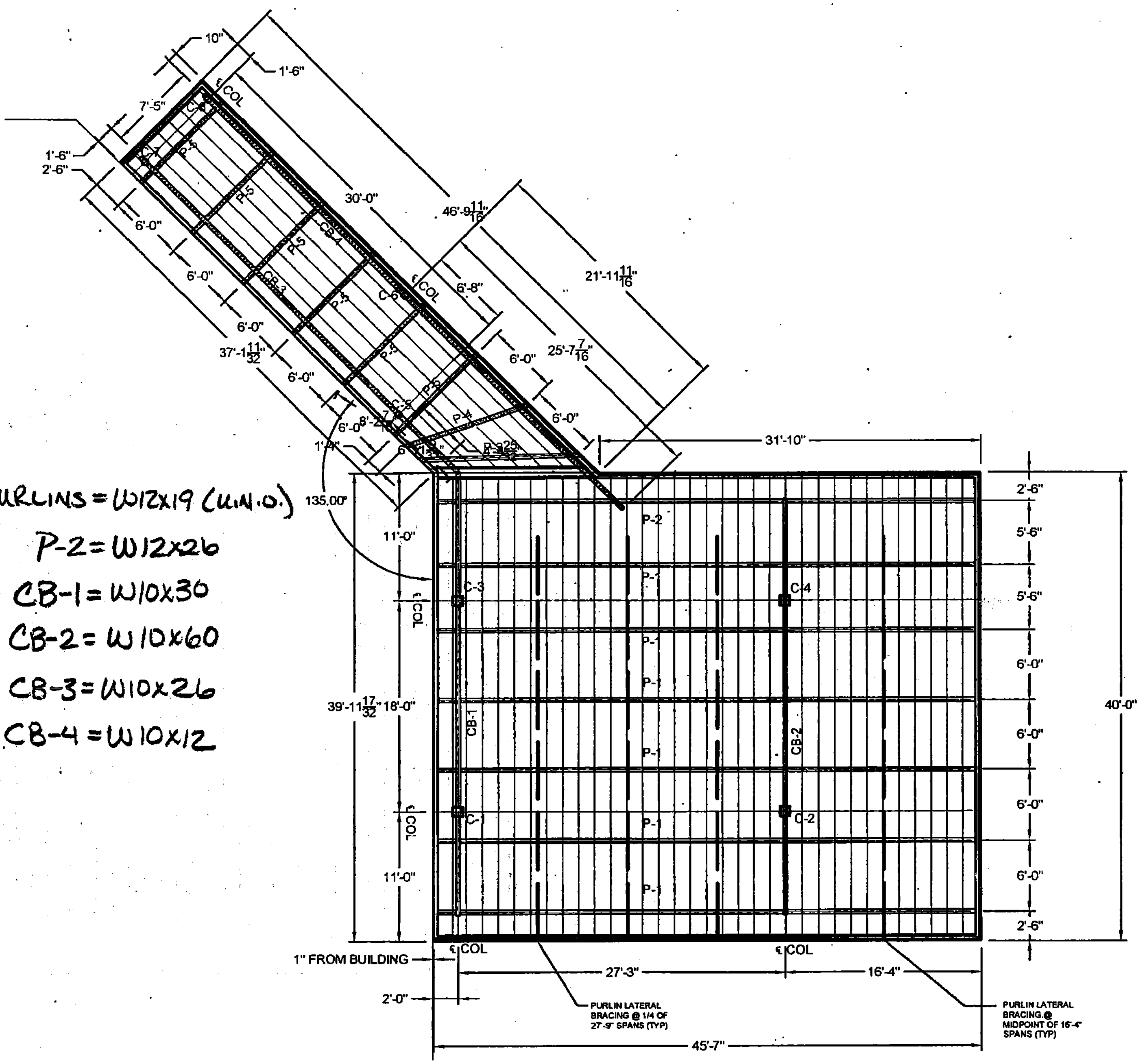
Job number: 17704
Customer: HY-VEE PICKUP #2
Location: LEE'S SUMMIT, MO.

Size Width: 40' Length: 45'-7" W/ 9'-9" x 46'-9" Walkway
Clearance: 14'- 0" Number of columns: (8)
Column spacing: Widthwise: 18'
Lengthwise: 2'-0", 27'-3", 16'-4"
Fascia: 30"
Live load: 30 psf Ground Snow: 20 psf Wind load: 37.34 psf Uplift: 13.96 psf
(SEI/ASCE 7-16 & 115 mph Exp. 'C')

SEE NEXT PAGE FOR LAYOUT

* S.E. Consultants is not responsible for deflection interference to or from other structure

ROOF PANELS: Use 3" x 16" x 20 Ga. Steel flat roof panel by Metal Works. Capacity of these panels for 30 psf (max load) is 10'-0" Max. Span & 2'-6" Max overhang and OK as shown with drifting.



PURLINS = W12x19 (U.N.O.)

P-2 = W12x26

CB-1 = W10x30

CB-2 = W10x60

CB-3 = W10x26

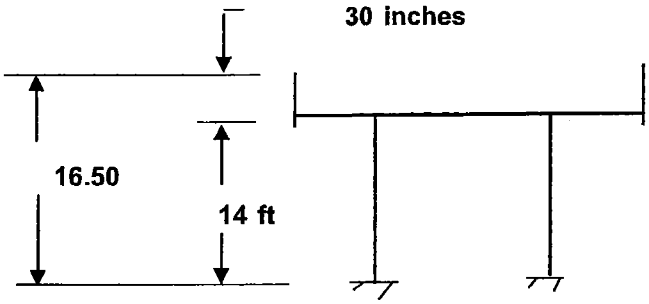
CB-4 = W10x12

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 17704 Date: 5-21 By: NDW Job No.: 089D21 SH.: 1B

S.E. CONSULTANTS, INC.

Arning Canopy Wind Load Design - ASCE 7-16



Length = **45.58 ft**
Width = **40 ft**

Building Enclosure Type: **Open**

Using method 2 - Analytical procedure for wind loads.

Velocity Pressure (section 26.10.2)

q_p = 0.00256 (Kh)(Kht)(Kd)(V^2)(.6) where,

q_p = 14.94 psf

Kh =	0.8650	table 26.10-1
Kht =	1.00	fig. 26.8-1
Kd =	0.85	table 26.6-1
V =	**115**	mph (ultimate)
V =	89.08	mph (service)

Risk Category II Exp: **C**

ASD combo W factor = 0.60 used, so

Parapets (section 27.3.4) (MWFRS)

$p_p = q_p(GC_{pn})$ where,

p_p = 37.34 psf (Min.)

GC_{pn} = 1.5 windward
-1.0 leeward

Uplift Forces (section 27.3.2) (MWFRS)

$p = q\ G_f\ C_n$ where,

p = 13.96 psf (Min.)

G_f = 0.85
C_n = 1.10

*** NEW OR EXISTING LOWER STRUCTURES MAY NEED TO BE CHECKED (BY OTHERS) AGAINST POSSIBLE DRIFTING LOADS INCURRED**

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 17704 Date: 5-21 By: AWW Job No.: 0890-21 SH.: 1C

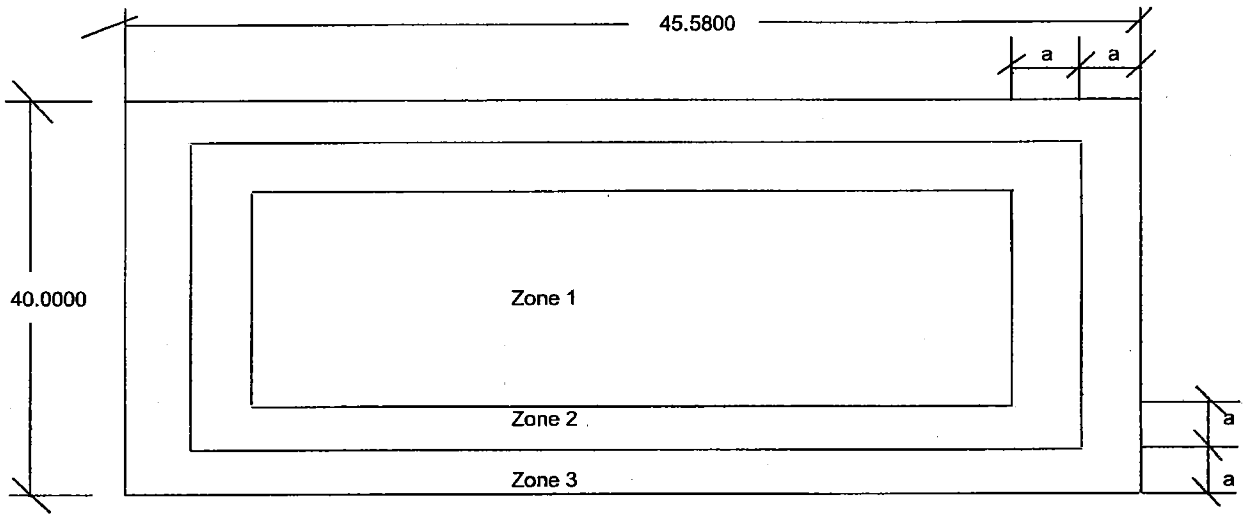
S.E. CONSULTANTS, INC.

Arning Canopy Wind Load Design - ASCE 7-16

Components and Cladding Purlin Figure 30.7-1

Min 10 % least horizontal dimension =	4.0000 ft
40% of mean roof height, (0.4h) =	6.6000 ft
4% of least horizontal dimension =	1.6000 ft
Edge zone dimension, a =	4.0000 ft
a^2 =	16.0000 ft
$4.0a^2$ =	64.0000 ft

Purlin	Trib (ft)	Span (ft)	Effective Area (ft^2)
#1	**6.0000**	**27.2500**	247.5208
#2	**5.7500**	**27.2500**	247.5208
#3	**5.5000**	**27.2500**	247.5208
#4	**6.0000**	**7.4170**	44.5020
#5	**5.5000**	**7.4170**	40.7935
#6	**0.0000**	**0.0000**	0.0000



Roof Angle	Effective Wind Area	C_N					
		Clear Wind Flow					
		Zone 3		Zone 2		Zone 1	
0	<=a^2	30.4686	-41.8943	22.8514	-21.5819	15.2343	-13.9648
	>a^2, <=$4a^2$	22.8514	-21.5819	22.8514	-21.5819	15.2343	-13.9648
	>=$4a^2$	15.2343	-13.9648	15.2343	-13.9648	15.2343	-13.9648
7.5	<=a^2	40.6248	-53.3200	30.4686	-26.6600	20.3124	-17.7733
	>a^2, <=$4a^2$	30.4686	-26.6600	30.4686	-26.6600	20.3124	-17.7733
	>=$4a^2$	20.3124	-17.7733	20.3124	-17.7733	20.3124	-17.7733

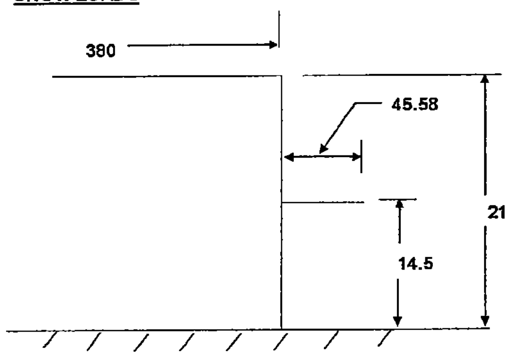
PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 17704 Date: 05-21 By: NJW Job No.: 0890-21 SH.: 1D

S.E. CONSULTANTS, INC.

SNOW DRIFTING

SNOW LOADS MAIN CANOPY



Pg	=	20.00 psf
Ce	=	1.00
Ct	=	1.20
Is	=	1.00
Pf	=	0.7 * Ce * Ct * I * Pg
	=	16.80 psf
		30.00 psf (min)
hc	=	6.24 ft

Note: All Dimension above are in Feet.

DRIFT HEIGHT

Hd = 0.43 * (lu)^(1/3) * (Pg + 10)^(1/4) - 1.5

= 5.789 ft or max. drift height = 5.789 ft
= 96.1 psf

Is drift Leeward Drifting = Y

If drift is Leeward, lu = 380 ft. If drift is windward, lu = 45.58 ft

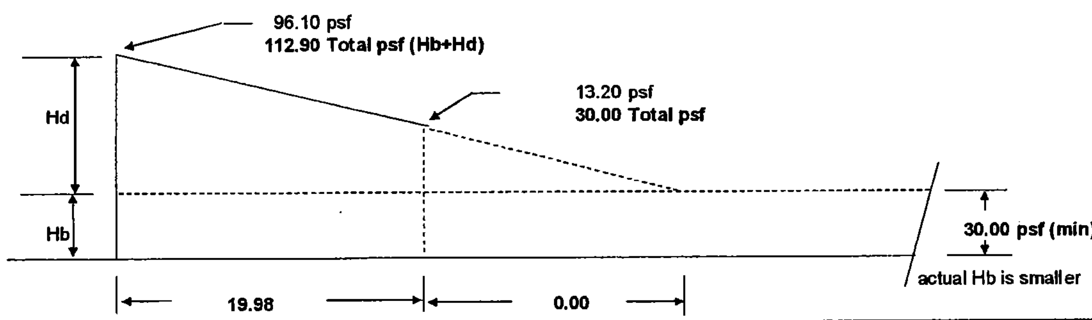
DRIFT WIDTH

δ = 0.13 * Pg + 14. < 30.0 pcf = 16.6 pcf

Hb = Pf / δ = 1.012 ft @ 16.8 psf

Wd = Hc>Hd then 4(Hd), or Hc<Hd then 4((Hd)^2)/Hc = 23.16 ft
Span only to 19.98 ft

DRIFT SNOW LOAD



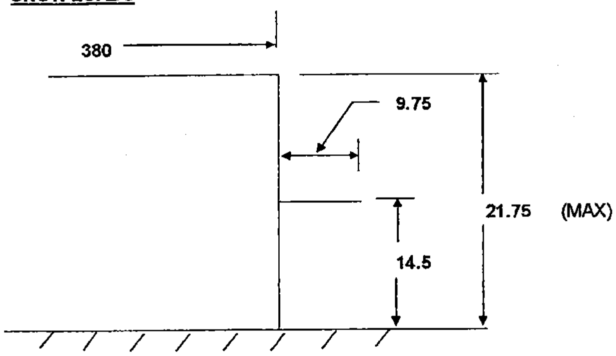
PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 17704 Date: 5-21 By: WW Job No.: 0890-21 SH.: 1E

S.E. CONSULTANTS, INC.

SNOW DRIFTING

SNOW LOADS **WALKWAY**



Pg	=	20.00 psf
Ce	=	1.00
Ct	=	1.20
Is	=	1.00
Pf	=	0.7 * Ce * Ct * I * Pg
	=	16.80 psf
		30.00 psf (min)
hc	=	6.24 ft

Note: All Dimension above are in Feet.

DRIFT HEIGHT

Hd = 0.43 * (lu)^(1/3) * (Pg + 10)^(1/4) - 1.5

= 5.789 ft or max. drift height = 5.789 ft

= 96.1 psf

Is drift Leeward Drifting = Y

If drift is Leeward, lu = 380 ft. If drift is windward, lu = 9.75 ft

DRIFT WIDTH

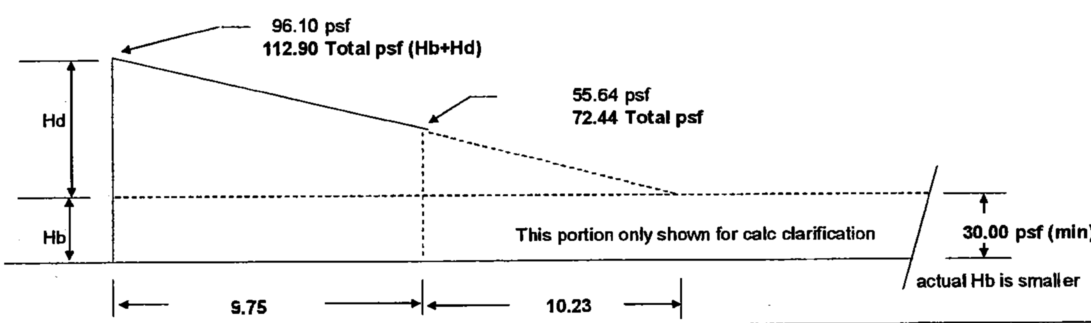
δ = 0.13 * Pg + 14. < 30.0 pcf = 16.6 pcf

Hb = Pf / δ = 1.012 ft @ 16.8 psf

Wd = Hc>Hd then 4(Hd), or Hc<Hd then 4((Hd)^2)/Hc = 23.16 ft

Span only to 9.75 ft

DRIFT SNOW LOAD



PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 17704 Date: 5-21 By: NJW Job No.: 0890-21 SH.: 1F

S.E. CONSULTANTS, INC.

DESCRIPTION: PURLIN Main Trib = 5.25' 0890-21 (17704) r2

CODE REFERENCES

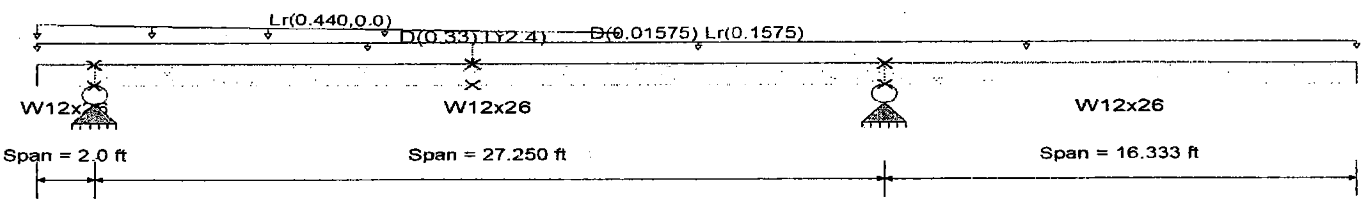
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

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 ft
Span # 2, Defined Brace Locations, First Brace at ft, Second Brace at 13.080 ft, Third Brace at ft
Span # 3, Defined Brace Spacing, First Brace at ft and spaced 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 : D = 0.0030, Lr = 0.030 ksf, Tributary Width = 5.250 ft

Varying Uniform Load : Lr= 0.440->0.0 k/ft, Extent = 0.0 -->> 20.0 ft

Load(s) for Span Number 2
Point Load : D = 0.330, Lr = 2.40 k @ 13.080 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.528 : 1**	Maximum Shear Stress Ratio =	**0.107 : 1**
Section used for this span	**W12x26**	Section used for this span	**W12x26**
Ma : Applied	26.577 k-ft	Va : Applied	5.984 k
Mn / Omega : Allowable	50.288 k-ft	Vn/Omega : Allowable	56.120 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	0.000 ft	Location of maximum on span	2.000 ft
Span # where maximum occurs	Span # 3	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.577 in	Ratio =	566 >=180.
Max Upward Transient Deflection	-0.146 in	Ratio =	329 >=180.
Max Downward Total Deflection	0.631 in	Ratio =	518 >=150.
Max Upward Total Deflection	-0.160 in	Ratio =	300 >=150.

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+Lr+H													
Dsgn. L = 2.00 ft	1	0.013	0.107		-1.25	1.25	155.00	92.81		1.00	5.98	84.18	56.12
Dsgn. L = 13.08 ft	2	0.411	0.107	34.31	-1.25	34.31	139.44	83.50	1.29	1.00	5.98	84.18	56.12
Dsgn. L = 14.17 ft	2	0.370	0.102	34.31	-26.58	34.31	155.00	92.81	1.98	1.00	5.74	84.18	56.12
Dsgn. L = 16.33 ft	3	0.528	0.058		-26.58	26.58	83.98	50.29		1.00	3.25	84.18	56.12

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+Lr+H	-0.1602	0.000
+D+Lr+H	2	0.6310	12.353		0.0000	0.000
	3	0.0000	12.353	Lr Only	-0.2166	16.333

2A

DESCRIPTION: PURLIN Main Trib = 5.25' 0890-21 (17704) r2

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		6.596	7.383	
Overall MINimum		0.623	1.610	
D Only		0.623	1.610	
Lr Only		6.596	7.383	

Steel Beam

DESCRIPTION: PURLIN Main Trib = 6' 0890-21 (17704) r2

CODE REFERENCES

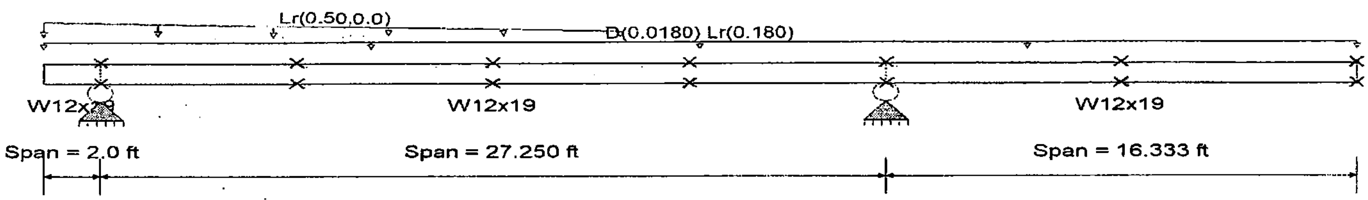
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

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 ft
Span # 2, Braced @ 1/4 Points
Span # 3, Braced @ Mid Span



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 = 0.0030, Lr = 0.030 ksf, Tributary Width = 6.0 ft

Varying Uniform Load : Lr= 0.50->0.0 k/ft, Extent = 0.0 -->> 20.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.740 : 1**	Maximum Shear Stress Ratio =	**0.089 : 1**
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	28.944 k-ft	Va : Applied	5.104 k
Mn / Omega : Allowable	39.127 k-ft	Vn/Omega : Allowable	57.340 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	0.000 ft	Location of maximum on span	2.000 ft
Span # where maximum occurs	Span # 3	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.508 in	Ratio =	643 >=180.
Max Upward Transient Deflection	-0.143 in	Ratio =	336 >=180.
Max Downward Total Deflection	0.809 in	Ratio =	485 >=150.
Max Upward Total Deflection	-0.150 in	Ratio =	319 >=150.

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+Lr+H													
Dsgn. L = 2.00 ft	1	0.023	0.089		-1.40	1.40	102.92	61.63		1.00	5.10	86.01	57.34
Dsgn. L = 6.72 ft	2	0.310	0.089	19.11	-1.40	19.11	102.92	61.63	1.48	1.00	5.10	86.01	57.34
Dsgn. L = 6.90 ft	2	0.453	0.029	20.57	16.77	20.57	75.92	45.46	1.02	1.00	1.66	86.01	57.34
Dsgn. L = 6.72 ft	2	0.272	0.059	16.77	-0.57	16.77	102.92	61.63	1.53	1.00	3.36	86.01	57.34
Dsgn. L = 6.90 ft	2	0.470	0.085	-0.00	-28.94	28.94	102.92	61.63	1.73	1.00	4.86	86.01	57.34
Dsgn. L = 8.17 ft	3	0.740	0.062	-0.00	-28.94	28.94	65.34	39.13		1.00	3.54	86.01	57.34
Dsgn. L = 8.17 ft	3	0.185	0.031		-7.24	7.24	65.34	39.13		1.00	1.77	86.01	57.34

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+Lr+H	-0.1502	0.000
+D+Lr+H	2	0.5280	11.263		0.0000	0.000
+D+Lr+H	3	0.8090	16.333	Lr Only	-0.0056	0.762

2C

DESCRIPTION: PURLIN Main Trib = 6' 0890-21 (17704) r2

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		6.088	7.117	
Overall MINimum		0.400	1.287	
D Only		0.400	1.287	
Lr Only		6.088	7.117	

Steel Beam

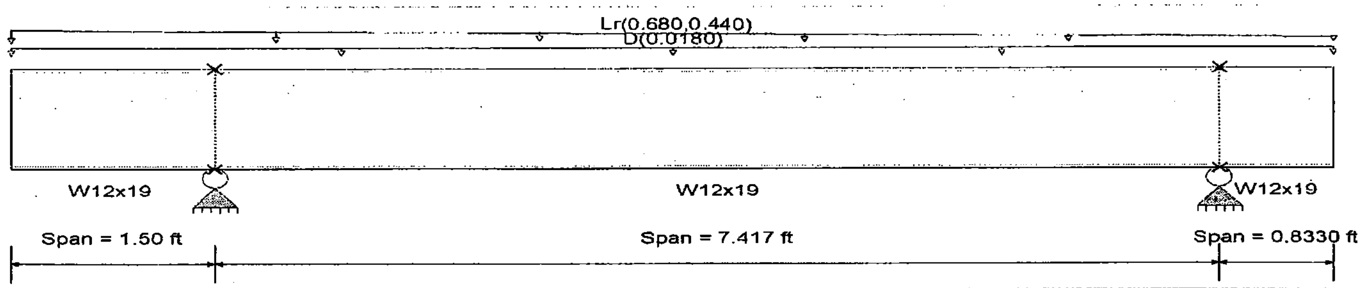
DESCRIPTION: PURLIN Walkway Trib = 6' 0890-21 (17704) r2

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, 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 :	Major Axis Bending		



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 = 0.0030 ksf, Tributary Width = 6.0 ft

Varying Uniform Load : Lr= 0.680->0.440 k/ft, Extent = 0.0 -->> 9.750 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.072 : 1**	Maximum Shear Stress Ratio =	**0.042 : 1**
Section used for this span	**W12x19**	Section used for this span	**W12x19**
Ma : Applied	3.569 k-ft	Va : Applied	2.381 k
Mn / Omega : Allowable	49.231 k-ft	Vn/Omega : Allowable	57.340 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	3.758ft	Location of maximum on span	1.500 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.009 in	Ratio =	10,321 >=180.
Max Upward Transient Deflection	-0.003 in	Ratio =	6,599 >=180.
Max Downward Total Deflection	0.009 in	Ratio =	9662 >=150.
Max Upward Total Deflection	-0.003 in	Ratio =	6177 >=150.

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+Lr+H													
Dsgn. L = 1.50 ft	1	0.013	0.042		-0.79	0.79	102.92	61.63	1.00	1.00	2.38	86.01	57.34
Dsgn. L = 7.42 ft	2	0.072	0.042	3.57	-0.79	3.57	82.22	49.23	1.16	1.00	2.38	86.01	57.34
Dsgn. L = 0.83 ft	3	0.003	0.007		-0.17	0.17	102.92	61.63	1.00	1.00	0.41	86.01	57.34

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+Lr+H	-0.0053	0.000
+D+Lr+H	2	0.0092	3.758		0.0000	0.000
	3	0.0000	3.758	+D+Lr+H	-0.0032	0.833

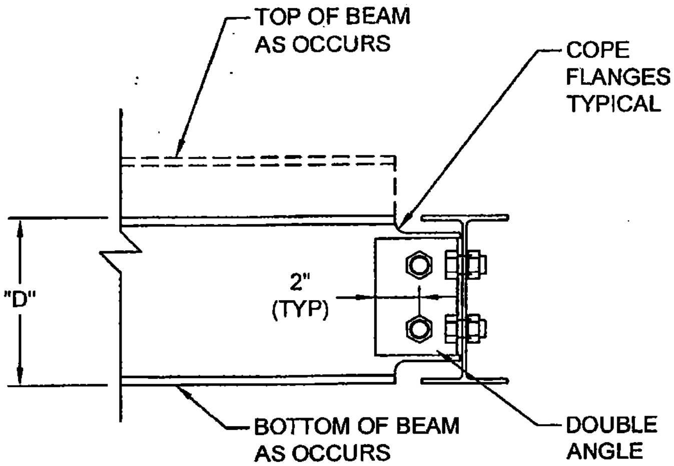
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		3.232	2.228	
Overall MINimum		0.197	0.164	
D Only		0.197	0.164	
Lr Only		3.232	2.228	

* BEAM SIZES AND ORIENTATION VARIES - SEE PLAN
* USE ∠4"X4"X¼" WITH 1-1/4" VERTICAL EDGE DISTANCE AND 3" BOLT SPACING
* "D" IS THE SMALLER OF THE CONNECTING BEAM SIZES

C-1/C-3 SIMILAR USING BENT ANGLE



BEAM TO BEAM

FLUSH FRAME CONNECTION SCHEDULE		
BEAM DEPTH "D"	NUMBER OF 3/4"Ø BOLTS (A325)	CONNECTION CAPACITY MIN. (kips)
W8 & W10	2	11
W12 & W14	3	21
W16	4	36
W18	5	55
W21 & W24	6	81
W27	7	116
W30	8	132
W33	9	148
W36	10	164

FLUSH FRAME BEAM CONNECTION

N.T.S.

DESCRIPTION: CB-1 Main 0890-21 (17704) r2

CODE REFERENCES

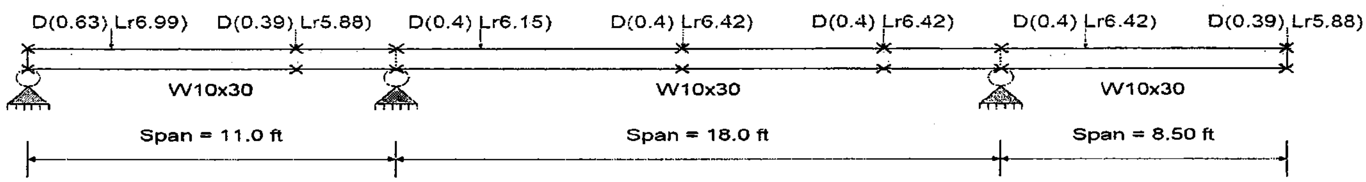
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

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 2.50 ft, Second Brace at 8.0 ft, Third Brace at ft
Span # 2, Defined Brace Locations, First Brace at 2.50 ft, Second Brace at 8.50 ft, Third Brace at 14.50 ft
Span # 3, Defined Brace Locations, First Brace at 2.50 ft, Second Brace at 8.50 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
Load(s) for Span Number 1
Point Load : D = 0.630, Lr = 6.990 k @ 2.50 ft

Point Load : D = 0.390, Lr = 5.880 k @ 8.0 ft

Load(s) for Span Number 2
Point Load : D = 0.40, Lr = 6.150 k @ 2.50 ft

Point Load : D = 0.40, Lr = 6.420 k @ 8.50 ft

Point Load : D = 0.40, Lr = 6.420 k @ 14.50 ft

Load(s) for Span Number 3
Point Load : D = 0.40, Lr = 6.420 k @ 2.50 ft

Point Load : D = 0.390, Lr = 5.880 k @ 8.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.782** : 1	Maximum Shear Stress Ratio =	**0.212** : 1
Section used for this span	**W10x30**	Section used for this span	**W10x30**
Ma : Applied	71.429 k-ft	Va : Applied	13.345 k
Mn / Omega : Allowable	91.317 k-ft	Vn/Omega : Allowable	63.0 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	18.000 ft	Location of maximum on span	18.000 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.981 in	Ratio =	207 >=180.
Max Upward Transient Deflection	-0.098 in	Ratio =	2,195 >=180.
Max Downward Total Deflection	1.057 in	Ratio =	193 >=150.
Max Upward Total Deflection	-0.105 in	Ratio =	2055 >=150.

Maximum Forces & Stresses for Load Combinations

DESCRIPTION: CB-1 Main 0890-21 (17704) r2

Load Combination Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+Lr+H													
Dsgn. L = 2.49 ft	1	0.170	0.099	15.54		15.54	152.50	91.32	1.68	1.00	6.27	94.50	63.00
Dsgn. L = 5.50 ft	1	0.170	0.098	15.54	7.29	15.54	152.50	91.32	1.26	1.00	6.19	94.50	63.00
Dsgn. L = 3.01 ft	1	0.180	0.126	7.29	-16.44	16.44	152.50	91.32	2.12	1.00	7.95	94.50	63.00
Dsgn. L = 2.40 ft	2	0.180	0.124	2.14	-16.44	16.44	152.50	91.32	1.83	1.00	7.78	94.50	63.00
Dsgn. L = 6.00 ft	2	0.101	0.122	9.21	2.14	9.21	152.50	91.32	1.37	1.00	7.71	94.50	63.00
Dsgn. L = 6.00 ft	2	0.281	0.096	9.21	-25.69	25.69	152.50	91.32	2.16	1.00	6.02	94.50	63.00
Dsgn. L = 3.60 ft	2	0.782	0.212	-0.00	-71.43	71.43	152.50	91.32	1.35	1.00	13.35	94.50	63.00
Dsgn. L = 2.49 ft	3	0.782	0.212	-0.00	-71.43	71.43	152.50	91.32		1.00	13.35	94.50	63.00
Dsgn. L = 6.01 ft	3	0.436	0.211		-38.25	38.25	146.51	87.73		1.00	13.27	94.50	63.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	0.0521	4.840		0.0000	0.000
	2	0.0000	4.840	+D+Lr+H	-0.1051	14.040
+D+Lr+H	3	1.0572	8.500		0.0000	14.040

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	5.654	14.307	24.200	
Overall MINimum	0.615	1.426	2.094	
D Only	0.615	1.426	2.094	
Lr Only	5.654	14.307	24.200	

DESCRIPTION: CB-2 Main 0890-21 (17704) r2

CODE REFERENCES

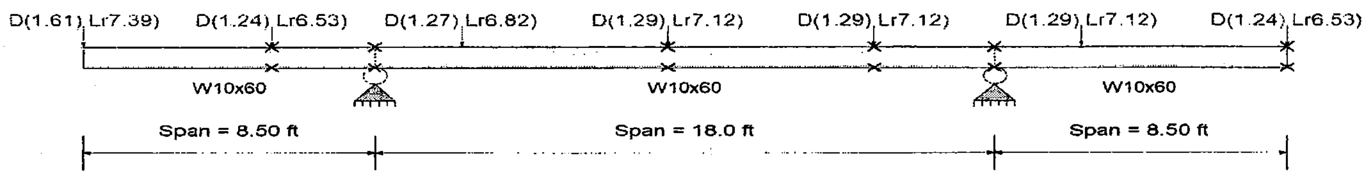
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

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 ft, Second Brace at 5.50 ft, Third Brace at ft
Span # 2, Defined Brace Locations, First Brace at 2.50 ft, Second Brace at 8.50 ft, Third Brace at 14.50 ft
Span # 3, Defined Brace Locations, First Brace at 2.50 ft, Second Brace at 8.50 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
Load(s) for Span Number 1
Point Load : D = 1.610, Lr = 7.390 k @ 0.0 ft

Point Load : D = 1.240, Lr = 6.530 k @ 5.50 ft

Load(s) for Span Number 2
Point Load : D = 1.270, Lr = 6.820 k @ 2.50 ft

Point Load : D = 1.290, Lr = 7.120 k @ 8.50 ft

Point Load : D = 1.290, Lr = 7.120 k @ 14.50 ft

Load(s) for Span Number 3
Point Load : D = 1.290, Lr = 7.120 k @ 2.50 ft

Point Load : D = 1.240, Lr = 6.530 k @ 8.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.548 : 1**	Maximum Shear Stress Ratio =	**0.202 : 1**
Section used for this span	**W10x60**	Section used for this span	**W10x60**
Ma : Applied	101.978 k-ft	Va : Applied	17.280 k
Mn / Omega : Allowable	186.128 k-ft	Vn/Omega : Allowable	85.680 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	8.500ft	Location of maximum on span	8.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.919 in	Ratio =	221 >=180.
Max Upward Transient Deflection	-0.262 in	Ratio =	823 >=180.
Max Downward Total Deflection	1.134 in	Ratio =	180 >=150.
Max Upward Total Deflection	-0.321 in	Ratio =	672 >=150.

Maximum Forces & Stresses for Load Combinations

Steel Beam

DESCRIPTION: CB-2 Main 0890-21 (17704) r2

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+Lr+H													
Dsgn. L = 5.50 ft	1	0.271	0.109		-50.38	50.38	310.83	186.13		1.00	9.33	128.52	85.68
Dsgn. L = 3.00 ft	1	0.548	0.202	-0.00	-101.98	101.98	310.83	186.13		1.00	17.28	128.52	85.68
Dsgn. L = 2.40 ft	2	0.548	0.167	-0.00	-101.98	101.98	310.83	186.13	1.16	1.00	14.29	128.52	85.68
Dsgn. L = 6.00 ft	2	0.365	0.165	-0.00	-67.86	67.86	310.83	186.13	1.28	1.00	14.14	128.52	85.68
Dsgn. L = 6.00 ft	2	0.260	0.066	-0.00	-48.34	48.34	310.83	186.13	1.18	1.00	5.69	128.52	85.68
Dsgn. L = 3.60 ft	2	0.479	0.195	-0.00	-89.24	89.24	310.83	186.13	1.23	1.00	16.69	128.52	85.68
Dsgn. L = 2.49 ft	3	0.479	0.195	-0.00	-89.24	89.24	310.83	186.13		1.00	16.69	128.52	85.68
Dsgn. L = 6.01 ft	3	0.257	0.193		-47.81	47.81	310.83	186.13		1.00	16.54	128.52	85.68

Summary of Moment Values: Mmax + through Rm; Summary of Shear Values: Va Max through Vnx/Omega.

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	1.1337	0.000		0.0000	0.000
	2	0.0000	0.000	+D+Lr+H	-0.3215	8.760
+D+Lr+H	3	1.0198	8.500		0.0000	8.760

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		25.441	23.189	
Overall MINimum		6.128	5.203	
D Only		6.128	5.203	
Lr Only		25.441	23.189	

DESCRIPTION: CB-3 Walkway 0890-21 (17704) r2

CODE REFERENCES

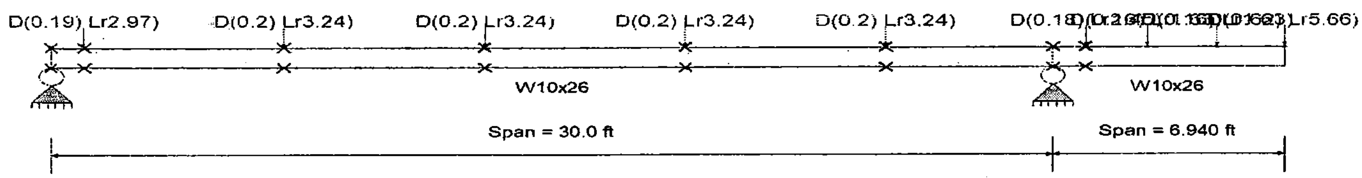
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined as a set spacing over all spans	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

First Brace starts at 1.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 6.0 ft



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 = 0.190, Lr = 2.970 k @ 1.0 ft

Point Load : D = 0.20, Lr = 3.240 k @ 7.0 ft

Point Load : D = 0.20, Lr = 3.240 k @ 13.0 ft

Point Load : D = 0.20, Lr = 3.240 k @ 19.0 ft

Point Load : D = 0.20, Lr = 3.240 k @ 25.0 ft

Load(s) for Span Number 2
Point Load : D = 0.180, Lr = 2.450 k @ 1.0 ft

Point Load : D = 0.160, Lr = 1.630 k @ 2.833 ft

Point Load : D = 0.160, Lr = 1.630 k @ 4.917 ft

Point Load : D = 0.620, Lr = 5.660 k @ 6.940 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.777 : 1**	Maximum Shear Stress Ratio =	**0.237 : 1**
Section used for this span	**W10x26**	Section used for this span	**W10x26**
Ma : Applied	60.712 k-ft	Va : Applied	12.670 k
Mn / Omega : Allowable	78.094 k-ft	Vn/Omega : Allowable	53.560 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	30.000 ft	Location of maximum on span	30.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.108 in	Ratio =	324 >=180.
Max Upward Transient Deflection	-0.014 in	Ratio =	11,830 >=180.
Max Downward Total Deflection	1.223 in	Ratio =	294 >=150.
Max Upward Total Deflection	-0.014 in	Ratio =	12027 >=150.

DESCRIPTION: CB-3 Walkway 0890-21 (17704) r2

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+Lr+H													
Dsgn. L = 0.96 ft	1	0.096	0.146	7.52		7.52	130.42	78.09	1.67	1.00	7.84	80.34	53.56
Dsgn. L = 6.00 ft	1	0.450	0.146	35.12	7.52	35.12	130.42	78.09	1.46	1.00	7.82	80.34	53.56
Dsgn. L = 6.00 ft	1	0.527	0.084	41.16	35.12	41.16	130.42	78.09	1.06	1.00	4.50	80.34	53.56
Dsgn. L = 6.00 ft	1	0.527	0.050	41.16	25.62	41.16	130.42	78.09	1.17	1.00	2.69	80.34	53.56
Dsgn. L = 6.00 ft	1	0.328	0.117	25.62	-11.50	25.62	130.42	78.09	2.15	1.00	6.29	80.34	53.56
Dsgn. L = 5.04 ft	1	0.777	0.237	-0.00	-60.71	60.71	130.42	78.09	1.48	1.00	12.67	80.34	53.56
Dsgn. L = 1.00 ft	2	0.777	0.237	-0.00	-60.71	60.71	130.42	78.09	1.00	1.00	12.67	80.34	53.56
Dsgn. L = 5.94 ft	2	0.643	0.236		-48.06	48.06	124.91	74.80	1.00	1.00	12.64	80.34	53.56

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	1.2233	13.080		0.0000	0.000
+D+Lr+H	2	0.1868	6.940	Lr Only	-0.0141	1.221

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	7.107	20.193	
Overall MINimum	0.735	2.335	
D Only	0.735	2.335	
Lr Only	7.107	20.193	

DESCRIPTION: CB-4 Walkway 0890-21 (17704) r2

CODE REFERENCES

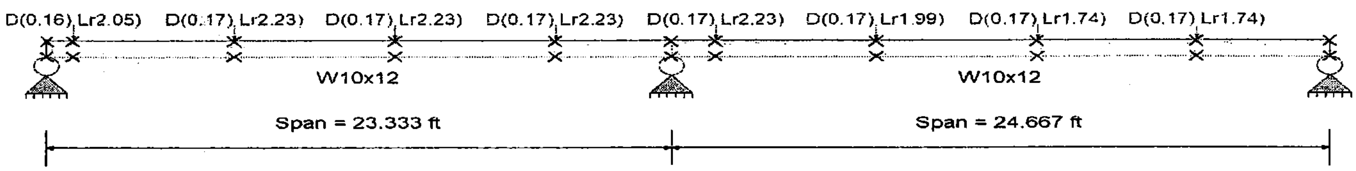
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined as a set spacing over all spans	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

First Brace starts at 1.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 6.0 ft



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 = 0.160, Lr = 2.050 k @ 1.0 ft
Point Load : D = 0.170, Lr = 2.230 k @ 7.0 ft
Point Load : D = 0.170, Lr = 2.230 k @ 13.0 ft
Point Load : D = 0.170, Lr = 2.230 k @ 19.0 ft

Load(s) for Span Number 2
Point Load : D = 0.170, Lr = 2.230 k @ 1.667 ft
Point Load : D = 0.170, Lr = 1.990 k @ 7.667 ft
Point Load : D = 0.170, Lr = 1.740 k @ 13.667 ft
Point Load : D = 0.170, Lr = 1.740 k @ 19.667 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.883** : 1	Maximum Shear Stress Ratio =	**0.166** : 1
Section used for this span	**W10x12**	Section used for this span	**W10x12**
Ma : Applied	27.565 k-ft	Va : Applied	6.231 k
Mn / Omega : Allowable	31.207 k-ft	Vn/Omega : Allowable	37.506 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	23.333ft	Location of maximum on span	23.333 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.681 in	Ratio =	410 >=180.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <180.0
Max Downward Total Deflection	0.747 in	Ratio =	375 >=150.
Max Upward Total Deflection	-0.000 in	Ratio =	557932 >=150.

Maximum Forces & Stresses for Load Combinations

DESCRIPTION: CB-4 Walkway 0890-21 (17704) r2

Load Combination Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+Lr+H													
Dsgn. L = 0.93 ft	1	0.127	0.114	3.97		3.97	52.12	31.21	1.74	1.00	4.26	56.26	37.51
Dsgn. L = 6.07 ft	1	0.522	0.113	16.28	3.97	16.28	52.12	31.21	1.43	1.00	4.25	56.26	37.51
Dsgn. L = 5.97 ft	1	0.636	0.052	16.28	13.49	16.28	42.78	25.62	1.07	1.00	1.97	56.26	37.51
Dsgn. L = 5.97 ft	1	0.432	0.079	13.49	-4.00	13.49	52.12	31.21	2.06	1.00	2.97	56.26	37.51
Dsgn. L = 4.39 ft	1	0.883	0.166	-0.00	-27.57	27.57	52.12	31.21	1.51	1.00	6.23	56.26	37.51
Dsgn. L = 1.58 ft	2	0.883	0.166	-0.00	-27.57	27.57	52.12	31.21	1.17	1.00	6.23	56.26	37.51
Dsgn. L = 6.02 ft	2	0.569	0.166	5.19	-17.74	17.74	52.12	31.21	2.06	1.00	6.21	56.26	37.51
Dsgn. L = 6.02 ft	2	0.469	0.100	14.63	5.19	14.63	52.12	31.21	1.34	1.00	3.74	56.26	37.51
Dsgn. L = 6.02 ft	2	0.576	0.040	14.69	12.09	14.69	42.59	25.50	1.07	1.00	1.51	56.26	37.51
Dsgn. L = 5.03 ft	2	0.387	0.065	12.09		12.09	52.12	31.21	1.63	1.00	2.45	56.26	37.51

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	0.7474	9.987		0.0000	0.000
+D+Lr+H	2	0.7370	14.307	Lr Only	-0.0008	0.296

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	3.868	10.450	2.121
Overall MINimum	0.394	1.208	0.324
D Only	0.394	1.208	0.324
Lr Only	3.868	10.450	2.121

S.E. Consultants
5800 E. Thomas Rd
Scottsdale, Az

Column Loads

COLUMNS C-1 THRU C-4

Gravity Loads

Dead Load	=	**10.00 psf**
Live Load	=	**36.60 psf**
Column Clear Height	=	**14.00 feet**
Tributary Width to Column	=	**34.78 feet**
Maximum Reaction to Column	=	**32.50 kips**
Effective Tributary to Column R/(LL+DL	=	697.42 ft^2
Total Dead Load to Column	=	**6.97 kips**

Wind Loads (Service)

Wind Uplift Pressure (Wp)	=	**13.96 psf**
Total Wind Uplift = Wp * Te	=	9.74 kips
Total Wind Uplift to Column	=	2.77 kips

Column Check

Use Column		Fy =	46 ksi
HSS10"x10"x1/4"		Fa =	24.9 ksi
	Depth =	10 in.	
	A =	8.96 in.^2	
	S =	28.3 in.^3	
	r =	3.97 in.	
	K =	1.5	
Pr	=	32.50 kips	
Pc	=	**145.00** kips	
Pr/Pc	=	0.22 >	0.2
Mr	=	33.49 k-ft	
Mc	=	**61.30** k-ft	

Eq #1: Pr/Pc + 8/9(Mr/Mc) < 1.0

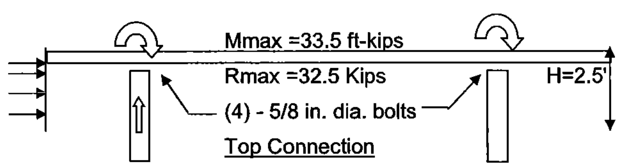
0.71 < 1 **Column O.K.**

Lateral Wind Loads (Service)

Lateral Wind Pressure	=	**37.34 psf**
Height of Exposed Fascia	=	**2.5 feet**
Total # of Lateral columns	=	**2**
Effective Column Height	=	17.05 feet
Lateral wind load to each column	=	1.62 kips
Moment due to wind loading	=	33.49 ft-kips
Worst Case Moment	=	**33.49 ft-kips**
Worst Case Lateral Load	=	**1.96 kips**

Eq #2: Pr/2Pc + Mr/Mc < 1.0

< 1 **Use Eqn # 1**



Number of Bolts =	4	Capacity of Connection
Size of Bolts =	**0.625**	32.625 ft-kips
<== Wind Loading Controls		**(o.k.)**

Earthquake Lateral/Longitudinal Load to Column - 2018 IBC

Risk Category	=	II	S_{DS}	=	0.128	
Seismic Design Category	=	B	S_{D1}	=	0.133	
I (SITE CLASS D)	=	1				
S_S	=	**12.00** %	V = Sds / [R/I]	=	0.102 W	(12.8-2)
S_1	=	**8.30** %	V = Sdi / [(R/I)T]	=	0.370 W	(12.8-3: T<TL)
S_{MS}	=	**0.192**	V = 0.044 * Sds * I > .(	=	0.010 W	(12.8-5)
S_{M1}	=	**0.199**	V = 0.5 * S1 / [R/I]	=	N.A. W	(12.8-6)
R (Cantilevered Steel Moment Frame)	=	**1.25** (Ordinary)				
T = 0.035 * (hn)^0.75	=	0.287	Total D.L. + 20% snow to Column	=	12.079 kips	
ρ Reliability/redundancy Factor	=	1	V (Controls) Cs	=	0.102	
Eh = ρ*Qe (Factored) .7 x S	=	**0.866 kips**	Ev = 0.2*Sds*D	=	0.179 kips	
Earthquake Moment to Column M	=	**14.763 ft-kips**				

Use: HSS10"x10"x1/4" Column (46 ksi) w/ (4) - 5/8 in. dia. bolts Column to Crossbeam

The most critical load condition is with moments due to lateral wind loading and DL only applied to the structure

S.E. Consultants
5800 E. Thomas Rd
Scottsdale, Az

Column Loads

COLUMNS C-5 THRU C-8

Gravity Loads

Dead Load	=	**15.00 psf**
Live Load	=	**100.00 psf**
Column Clear Height	=	**14.00 feet**
Tributary Width to Column	=	**29.95 feet**
Maximum Reaction to Column	=	**12.50 kips**
Effective Tributary to Column R/(LL+DL	=	108.70 ft^2
Total Dead Load to Column	=	**1.63 kips**

Wind Loads (Service)

Wind Uplift Pressure (Wp)	=	**13.96 psf**
Total Wind Uplift = Wp * Te	=	1.52 kips
Total Wind Uplift to Column	=	-0.11 kips

Column Check

Use Column		Fy =	46 ksi
HSS6"x6"x1/4"		Fa =	26.8 ksi
	Depth =	6	in.
	A =	5.24	in.^2
	S =	9.54	in.^3
	r =	2.34	in.
	K =	1.5	
Pr	=	12.50	kips
Pc	=	**73.60**	kips
Pr/Pc	=	0.17 <	0.2
Mr	=	20.11	k-ft
Mc	=	**25.70**	k-ft

**** NO INSPECTION HOLES TO BE USED ****

Eq #1: Pr/Pc + 8/9(Mr/Mc) < 1.0

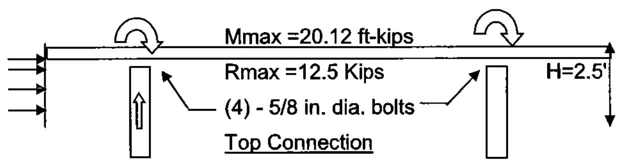
< 1 **Use Eqn. #2**

Eq #2: Pr/2Pc + Mr/Mc < 1.0

0.87 < 1 **Column O.K.**

Lateral Wind Loads (Service)

Lateral Wind Pressure	=	**22.40 psf**
Height of Exposed Fascia	=	**2.5 feet**
Total # of Lateral columns	=	**2**
Effective Column Height	=	17.05 feet
Lateral wind load to each column	=	0.84 kips
Moment due to wind loading	=	20.11 ft-kips
Worst Case Moment	=	**20.11 ft-kips**
Worst Case Lateral Load	=	**1.18 kips**



Number of Bolts =	4	Capacity of Connection	
Size of Bolts =	**0.625**	23.62 ft-kips	
<== Wind Loading Controls		**(o.k.)**	

Earthquake Lateral/Longitudinal Load to Column - 2018 IBC

Risk Category	=	**II**	S_{DS}	=	0.128	
Seismic Design Category	=	**B**	S_{D1}	=	0.133	
I (SITE CLASS D)	=	**1**				
S_S	=	**12.00** %	V = Sds / [R/I]	=	0.102 W	(12.8-2)
S_1	=	**8.30** %	V = Sdi / [(R/I)T]	=	0.370 W	(12.8-3: T<TL)
S_{MS}	=	**0.192**	V = 0.044 * Sds * I > .C	=	0.010 W	(12.8-5)
S_{M1}	=	**0.199**	V = 0.5 * S1 / [R/I]	=	N.A. W	(12.8-6)
R (Cantilevered Steel Moment Frame)	=	**1.25** (Ordinary)				
T = 0.035 * (hn)^0.75	=	0.287	Total D.L. + 20% snow to Column	=	3.804 kips	
ρ Reliability/redundancy Factor	=	**1**	V (Controls) Cs	=	0.102	
Eh = ρ*Qe (Factored) .7 x S	=	**0.273 kips**	Ev = 0.2*Sds*D	=	0.042 kips	
Earthquake Moment to Column M	=	**4.649 ft-kips**				

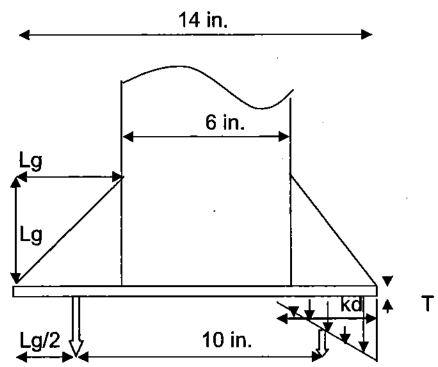
Use: HSS6"x6"x1/4" Column (46 ksi) w/ (4) - 5/8 in. dia. bolts Column to Crossbeam

The most critical load condition is with moments due to lateral wind loading and DL only applied to the structure

S.E. Consultants
5800 E. Thomas Rd
Scottsdale, Az

Connection At Footing

COLUMNS C-5 THRU C-8

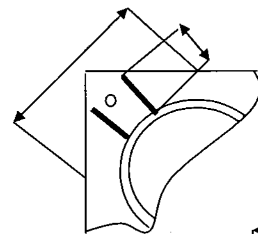


Column Loading

Gravity Load (P)	=	1.63 kips
Maximum Moment to Column	=	20.11 ft-kips
Wind / Seismic Reduction? (Y/N)	=	N
Gravity Load (P) * (.6)	=	**0.98 kips**
Moment to Column	=	**20.11 kip-ft**

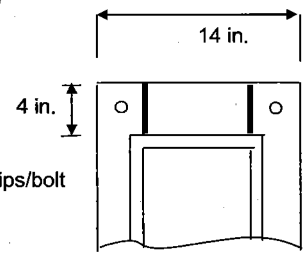
$$\Sigma Fv = Tb + P - fc \cdot Wb \cdot \frac{kd}{2}$$

$$\Sigma M = Sbolts + fc \cdot Wplate \cdot \frac{kd}{2} \cdot \left(\frac{Wplate}{2} - \frac{kd}{3}\right) - M \cdot 12$$



$$Tb = \left(kd \cdot fc \cdot \frac{Wplate}{2}\right) - P$$

Allowable Comp. to Concrete (fc)	=	**1** ksi	
Column Width (Cw)	=	6 in.	
Gusset Length (Lg)	=	**4** in.	
Gusset Height (Lg)	=	**8** in.	
Base Plate Width (Wplate)	=	14 in.	
Bolt Spacing	=	10 in.	
Solve above formulas for kd:	=	32.78 in.	**3.22 in.**
2		out	in
Maximum Bolt Tension (Tb)	=	**21.56 kips**	1 1/4" Dia A-36 Bolts = 23.4 kips/bolt
/ (2) Bolts	=	**10.78 kips/bolt**	
		2 Bolts per side	



Use 1 1/4" x 1'-11" Embed F1554 (grade 36) Anchor Bolts (Headed Studs)

Base Plate & Gusset

kd < Lg

Use Eqn. #1=> $Mp = fc \cdot \frac{kd}{2} \cdot Wplate \cdot \left[(Lg - kd) + \frac{2}{3} \cdot kd\right]$

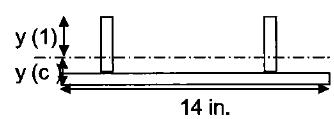
Gusset Thickness	=	**0.5 in.**
Plate Thickness (T)	=	**0.75 in.**
Maximum Moment to Plate (Mp)	=	65.92 kips-in
Fb	=	**36 ksi**
Fb (allowable)	=	27 ksi
Sx Required (Mp/Fb)	=	**2.44 in^3**

$$Mp = \left[\frac{(kd - Lg)}{kd} \cdot Lg^2 \cdot \frac{Wplate}{2}\right] + \left(1 - \frac{kd - Lg}{2}\right) \cdot \frac{Lg^2 \cdot Wplate}{3}$$

$$Mp = Tb \cdot Lb$$

Section Properties of "Double Tee" Gusset

Part		A	Y	Ay	Ay^2	Io	Io + Ay^2
Base Plate 0.75 in. x 14 in. Pl		10.50	0.38	3.94	1.48	0.49	1.97
Gusset (2) 0.5 in. x 4 in. Pl		4	2.75	11	30.25	5.33	35.58
	=	14.50		14.94			37.55
						Ay(c)^2	15.39
						Ix =	**22.16 in^4**



y(c)	=	1.0302 in.
y(1)	=	3.7198 in.
Sx=Ix / y(1)	=	**5.96 in^3**

Base Plate O.K.

Use 0.75 in. x 14 in. x 14 in. Base Plate w (2) 4 in. x 8 in. x 0.5 in. Gusset each side

*** FOOTINGS THAT ARE DESIGNED BY OTHERS ARE ASSUMED ADEQUATE FOR DESIGN REQUIREMENTS OF THESE ANCHORS.**

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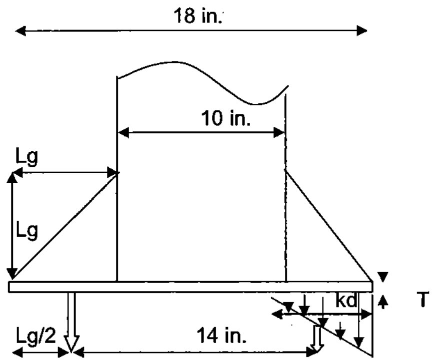
Scottsdale, Az

Connection At Footing

COLUMNS C-1 THRU C-4

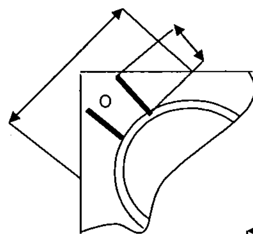


Column Loading

Gravity Load (P)	=	6.97 kips
Maximum Moment to Column	=	33.49 ft-kips
Wind / Seismic Reduction? (Y/N)	=	N
Gravity Load (P) * (.6)	=	**4.18 kips**
Moment to Column	=	**33.49 kip-ft**

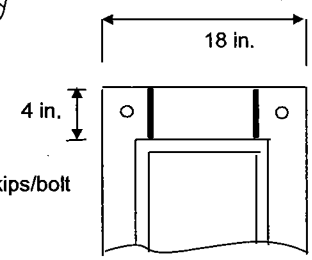
$$\Sigma Fv = Tb + P - fc \cdot Wb \cdot \frac{kd}{2}$$

$$\Sigma M = Sbolts + fc \cdot Wplate \cdot \frac{kd}{2} \cdot \left(\frac{Wplate}{2} - \frac{kd}{3}\right) - M \cdot 12$$



$$Tb = \left(kd \cdot fc \cdot \frac{Wplate}{2}\right) - P$$

Allowable Comp. to Concrete (fc)	=	1 ksi		
Column Width (Cw)	=	10 in.		
Gusset Length (Lg)	=	**4** in.		
Gusset Height (Lg)	=	**8** in.		
Base Plate Width (Wplate)	=	18 in.		
Bolt Spacing	=	14 in.		
Solve above formulas for kd:	=	44.79 in.	**3.21 in.**	
2		out	in	
Maximum Bolt Tension (Tb)	=	**24.70 kips**	1 1/4" Dia A-36 Bolts =	23.4 kips/bolt
/ (2) Bolts	=	**12.35 kips/bolt**		
		2 Bolts per side		



Use 1 1/4" x 1'-11" Embed F1554 (grade 36) Anchor Bolts (Headed Studs)

Base Plate & Gusset

kd < Lg

Use Eqn. #1=> $Mp = fc \cdot \frac{kd}{2} \cdot Wplate \cdot \left[(Lg - kd) + \frac{2}{3} \cdot kd\right]$

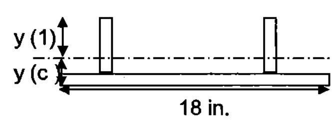
Gusset Thickness	=	**0.5 in.**
Plate Thickness (T)	=	**0.75 in.**
Maximum Moment to Plate (Mp)	=	84.57 kips-in
Fb	=	**36 ksi**
Fb (allowable)	=	27 ksi
Sx Required (Mp/Fb)	=	**3.13 in^3**

$$Mp = \left[\frac{(kd - Lg)}{kd} \cdot Lg^2 \cdot \frac{Wplate}{2}\right] + \left(1 - \frac{kd - Lg}{2}\right) \cdot \frac{Lg^2 \cdot Wplate}{3}$$

$$Mp = Tb \cdot Lb$$

Section Properties of "Double Tee" Gusset

Part		A	Y	Ay	Ay^2	Io	Io + Ay^2
Base Plate 0.75 in. x 18 in. Pl		13.50	0.38	5.06	1.90	0.63	2.53
Gusset (2) 0.5 in. x 4 in. Pl		4	2.75	11	30.25	5.33	35.58
	=	17.50		16.06			38.11
						Ay(c)^2	14.74
y(c)	=	0.9179 in.				**Ix =**	**23.37 in^4**
y(1)	=	3.8321 in.					
Sx=Ix / y(1)	=	**6.10 in^3**	**Base Plate O.K.**				



Use 0.75 in. x 18 in. x 18 in. Base Plate w (2) 4 in. x 8 in. x 0.5 in. Gusset each side

*** FOOTINGS THAT ARE DESIGNED BY OTHERS ARE ASSUMED ADEQUATE FOR DESIGN REQUIREMENTS OF THESE ANCHORS.**

S.E. Consultants
5800 E. Thomas Rd
Scottsdale, Az

Spread Footing Design

COLUMNS C-2, C-4, C-6, AND C-8 ONLY

Footing Resistance Against Overturning

Applied Overturning Moment	=	33.49 ft-kips
D.L. of Structure	=	12.97 kips
Length of Footing	=	**6.75 feet**
Width of Footing	=	**6.75 feet**
Thickness of Footing	=	**30 in.**
Dead Load of Footing	=	17.09 kips
Volume of Earth in 30 deg Cone	=	48.77 ft^3
Dead Load of Earth	=	4.88 kips
Total D.L.	=	34.94 kips
D.L. Resisting Moment (DL+WL)	=	**117.91 ft-kips**
D.L. Res Mom (DL+.75(WL+SL))	=	**182.53 ft-kips**
Required Factor of Safety	=	**1.67**
F. of S. Against Overturning	=	**3.52**

Note: F.of S. Over 1.67 satisfies .6DL combo

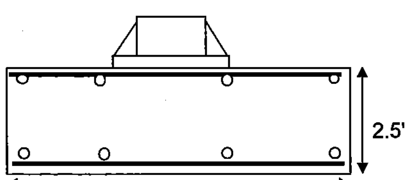
Overturning O.K.

Dead Load Resistance Against Uplift

Applied Uplift Load - Total Wind	=	9.74 kips
D.L. of Structure	=	12.97 kips
Volume of Earth in 30 deg Cone	=	48.77 ft^3
Dead Load of Footing	=	17.09 kips
Dead Load of Earth	=	4.88 kips
Total Dead Load Resistance	=	**34.94 kips**
F. of S. Against Uplift	=	**3.59**

Uplift O.K.

Weight of Column Surround = 6.00 kips



6.75' x 6.75'

* BOTTOM OF FOOTING TO BE AT FROST DEPTH

* NO SOILS REPORT KNOWN

Soil Bearing Check

Allowable Soil Pressure	**1500 psf**
Increases for Width OR Depth	0
Allowable S. P. w/ Increases	1500 psf
Applied Overturning Moment	33.49 ft-kips
Snow or Roof Live Load	25.53 kips
D.L. of Structure	12.97 kips
Footing D.L. - Existing Soils	5.69 kips
Total Vert. Load	44.19 kips
e' = (Mr - OTM) / Total Vert. Load	4.163 feet
e = L / 2 - e'	-0.788 feet
L / 6 =	1.125 feet
RESULTANT IN MIDDLE 1/3	
S.P. = P/A +/- M/S	
Soil Pressure (overturning)	**1460.0 psf**
Soil Pressure (gravity)	**845.0 psf**

Soil Bearing O.K.

* Worst Case Combination: DL+.75(WL+SL/RLL)

FOOTING REINFORCING

f'c		**2500 psi**
fy		**60000 psi**
d		26 inches
M, Moment		33.492 ft.-kips
Mu, Ultimate Moment = 1.7 M		56.937 ft.-kips
Assume a one foot strip		
Mu, = 1.7 M / Width of Footing		8.435 ft.-kips
Beta		0.85
Phi		0.85
rho, balanced		0.018
m		28.235
Rn	49.545 psi =	0.050 ksi
rho, (act)		0.0008
rho, (req'd.)		0.0009
Req'd. As		0.324 sq. in. / ft.
Total As Req'd.		**2.187 sq. in.**
Total As Req'd. (Wind or Seismic)		**2.187 sq. in.**
USE: 8	6 's As	**3.53 sq.in.**

Reinforcing O.K.,

Use: 6.75' x 6.75' x 30 in. Spread Footing
With (8) # 6 Bars (12 in. Max. Spacing)
ORS. **Horizontally each way at the top and bottom of footing. (3" Min Clearance)**

S.E. Job #: 0890-21

Arning Job #: 17704

Preliminary Unless Sealed on Cover Sheet

S.E. Consultants
5800 E. Thomas Rd
Scottsdale, Az

Cube Footing Design

COLUMNS C-2, C-4, C-6, AND C-8 ONLY

Footing Resistance Against Wind

Footing Width x Length	=	**4**	feet
Footing Depth (d)	=	**11**	feet
Column Height (H)	=	17.05	feet
Maximum Lateral Load (p)	=	1.96	kips
Footing Diagonal Length (b)	=	5.66	feet
S1 = (d / 3) * Pa	=	1.1	
A = 2.34*p / (S1 * b)	=	0.74	feet

Allowable lateral soil bearing (Pa)	=	**0.3**	ksf
Allowable Increase Per Foot of Depth	=	0	ksf / ft
Average Allowable w/ Increases	=	**0.300**	ksf

Designed per IBC formula:

$$d=\frac{A}{2}\cdot 1+\sqrt{1+\frac{4.36\cdot h}{A}}$$

d required = 4.09 feet (Min)

Depth O.K.

Steel Reinforcement

Steel Tensile Capacity	=	21.6	ksi
Max Tension to Footinç	=	24.70	kips
Area req per side. (As)	=	1.44	in^2

Use:	As
(9) #4 bars per side, 32 Total	1.764 in^2
(6) #5 bars per side, 20 Total	1.842 in^2
(5) #6 bars per side, 16 Total	2.205 in^2

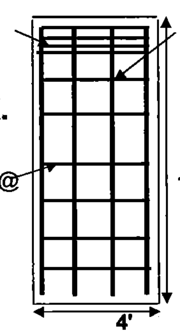
All veritical rebar require standard hooks

Soil Bearing Check

Area of Footing	=	16	ft.^2
Allowable soil bearing pressure	=	1500	psf
Allowable w/ depth increase	=	1500	psf
Load to Footing	=	38.50	kips
Volume of Footing	=	176	ft^2
Max pressure at footing	=	**2406**	**psf**
Allowable Skin Friction (Pa/6)	=	250.00	lb/ft
Skin Surface (Excludes top 12")	=	160.00	ft^2
Skin Friction	=	**40.00**	**kips**

Soil Bearing Fails, See Skin Friction

Skin Friction O.K.



* NO SOILS REPORT KNOWN

Uplift Resistance

DL of Structure	=	12.97	kips
Volume of earth within 30 deg. Cone	=	876.17	ft^3
Min of D.L of Earth or Skin Fricion	=	40.00	kips
Dead load of footing	=	26.4	kips
Total resistance against uplift	=	**79.37**	**kips**
Uplift to column	=	9.74	kips

Uplift O.K.

Use: 4' x 4' x 11' Cube Footing
With (9) #4 bars per side, 32 Total

S.E. Consultants
5800 E. Thomas Rd
Scottsdale, Az

Caisson Footing Design

COLUMNS C-2, C-4, C-6, AND C-8 ONLY

Footing Resistance Against Wind

Footing Diameter	=	**3**	feet
Footing Depth (d)	=	**17.5**	feet
Column Height (H)	=	17.05	feet
Maximum Lateral Load (p)	=	1.96	kips
Footing Diagonal Length (b)	=	3.00	feet

Allowable lateral soil bearing (Pa)	=	**0.3**	ksf
Allowable Increase Per Foot of Depth	=	0	ksf / ft
Average Allowable w/ Increases	=	**0.300**	ksf

S1 = (d / 3) * Pa	=	1.2	
A = 2.34*p / (S1 * b)	=	1.28	feet

Designed per IBC formula:

$$d = \frac{A}{2} \cdot 1 + \sqrt{1 + \frac{4.36 \cdot h}{A}}$$

d required = **5.55 feet (Min)**

Depth O.K.

Steel Reinforcement

Steel Tensile Capacity	=	21.6	ksi
Tension to Footing per half	=	24.70	kips
Steel req., 1 side (As)	=	1.14	in^2
Total Steel req'd.	=	2.54	in^2
Use:		As	
(14) #4 bars		2.744	in^2
(10) #5 bars		3.07	in^2
(6) #6 bars		2.646	in^2

All veritical rebar require standard hooks

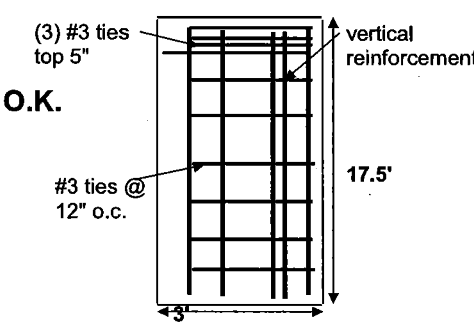
Soil Bearing Check

Area of Footing	=	7.07	ft.^2
Allowable soil bearing pressure	=	1500	psf
Allowable w/ depth increase	=	1500	psf
Volume of Footing	=	123.68	ft^2
Load to Footing	=	38.50	kips
Max pressure at footing	=	**5447**	**psf**

Soil Bearing Fails, See Skin Friction

Allowable Skin Friction (Pa/6)	=	250.00	lb/ft
Skin Surface (Excludes top 12")	=	155.51	ft^2
Skin Friction	=	**38.88**	**kips**

Skin Friction O.K.



* NO SOILS REPORT KNOWN

Uplift Resistance

DL of Structure	=	12.97	kips
Volume of earth within 30 deg. Cone	=	2706.79	ft^3
Min of D.L of Earth or Skin Fricion	=	38.88	kips
Dead load of footing	=	18.55	kips
Total resistance against uplift	=	70.40	kips
Uplift to column	=	9.74	kips

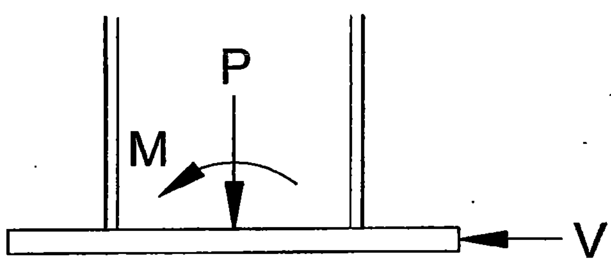
Uplift O.K.

Use: 3' Diameter x 17.5' Caisson Footing
With (14) #4 bars vertical.

S.E. Job #: 0890-21
Arning Job #: 17704

Preliminary Unless Sealed on Cover Sheet

ASD SERVICE LEVEL REACTIONS FOR FOUNDATION DESIGN			
LOAD CASE COLUMN LOCATIONS C1, C3, C5, C7	SHEAR V (KIPS)	VERTICAL P (KIPS)	MOMENT M (KIP-FT)
DEAD	0	3.10	0
LIVE	0	6.25	0
SNOW	0	24.20	0
WIND 1 - TRANSVERSE	.73	4.76	14.51
WIND 2 - TRANSVERSE	.73	-4.37	14.51
WIND 3 - LONGITUDINAL	.56	4.76	11.62
WIND 4 - LONGITUDINAL	.56	-4.37	11.62
SEISMIC	.57	.08	9.71



NOTES:
1. SIGN CONVENTION SHOWN DENOTES POSITIVE FORCES.
2. THE REACTIONS PROVIDED DO NOT INCLUDE THE DEAD WEIGHT OR WIND/SEISMIC REACTIONS DUE TO CANOPY COLUMN WRAPS, IF PRESENT.
3. ALL REACTIONS PROVIDED REPRESENT ASD SERVICE LEVEL REACTIONS. WIND LOAD REACTIONS ARE PROVIDED AS 0.6*ULTIMATE VALUE AND SEISMIC LOAD REACTIONS ARE PROVIDED AS 0.7*ULTIMATE VALUES.
4. ALL REACTIONS SHOWN ARE AS A WORST CASE CONDITION.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 17704 Date: 5-21 By: NJW Job No.: 0890-21 SH.: 9

S.E. CONSULTANTS, INC.