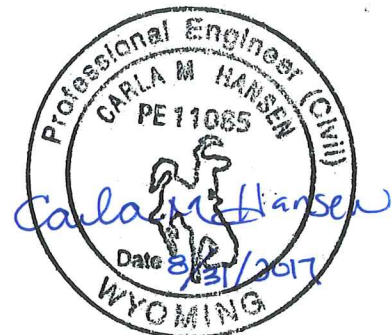


STRUCTURAL CALCULATIONS

PAVILION TEMPORARY FIRE STATION
305 WEST SNOW KING AVE
JACKSON WYOMING

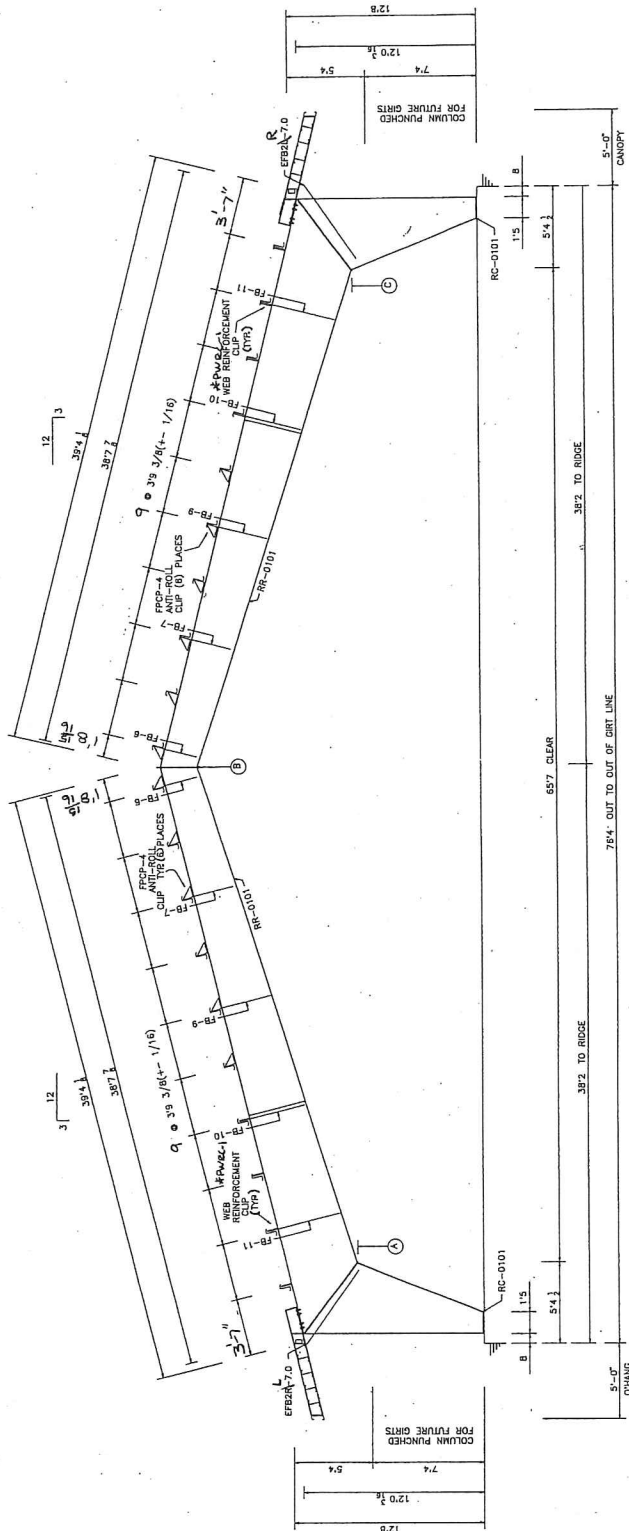
Prepared By
Nelson Engineering
Jackson, Wyoming



August 31, 2017
2017: NE Project No. 17-202-01
Original NE Project No. 95-183-01
Building Permit No. #_____

EXISTING END FRAME

SPICE ID	SPICE SIZE	SPICE BOLT TABLE	SPICE BOLT TABLE	SPICE BOLT TABLE
A	14	3/4 X 2	48 1/2	53 5/16
B	10	3/4 X 2	25 3/8	18 3/16
C	14	3/4 X 2	48 1/2	53 5/16



*SEE NOTE ON SH-E-01

RIGID FRAME CROSS SECTION

FRAME LINES 2 THRU 8

NO.	REVISION	BY	CHK.	DATE	NO.	REVISION	BY	CHK.	DATE	DRAWING SUBMITTAL STATUS	SCALE	DATE	DATE
1	RELEASED FOR CONSTRUCTION	DS	LS	5/18/96	1	RELEASED FOR CONSTRUCTION	DS	LS	5/18/96	FOR CONSTRUCTION	1/8" = 1'-0"	5/18/96	5/18/96
2					2					FOR PERMIT ONLY	1/8" = 1'-0"	5/18/96	5/18/96
3					3					PRELIMINARY ONLY	1/8" = 1'-0"	5/18/96	5/18/96

CARSON CITY	NEVADA
AMERICAN BUILDINGS	
DRAWING NUMBER	51852001 E-02
REVISED	

TETON COUNTY FAIR BOARD
TETON COUNTY FAIRGROUNDS
JACKSON, WY
GLENROCK CONCRETE

EXISTING MAIN FRAMES

Max Moments Summary for Moment Connection - Main Frame

• Haunch

D+C+S : $M = 630.18 \text{ k-ft}$ (Existing Max moment, frame was designed to)

D+1/4S+E : $M = 220.619 \text{ k-ft}$ (Max moment w/ updated loading)

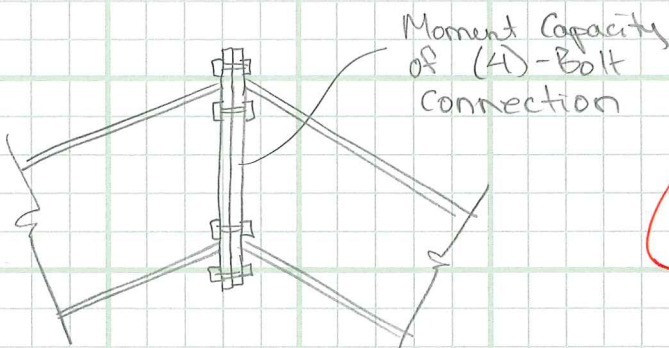
* Note since snow loading did not change & the max moment produced from loading that did increase (in seismic & wind) is still lower than original D+C+S loading the haunch connection is OK

* Per original design calcs $\rightarrow$ Max (@ haunch) = 650.2 k-ft

• Ridge

D+C+S : $M = 13.33 \text{ k-ft}$ (Existing Max moment, frame was designed for)

D+1/4S+E : $M = 30.671 \text{ k-ft}$ (Max moment w/ updated loading)



~~5/8" ϕ Bolts = 121.46 k-ft~~

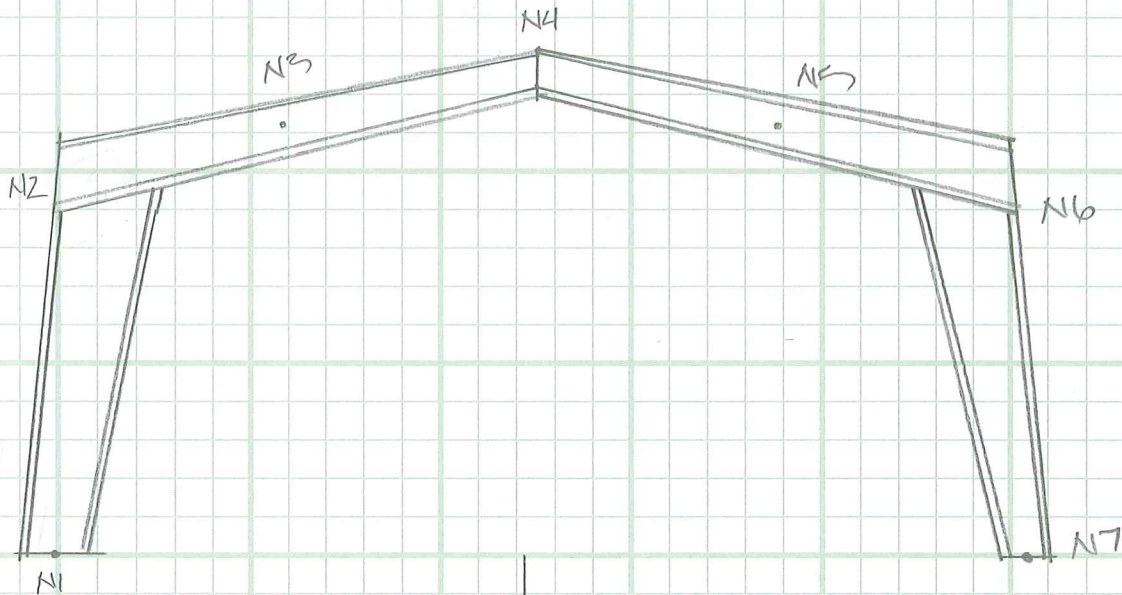
$> 30.671 \text{ k-ft}$ OK

$3/4" \phi$ Bolts = 170.43 k-ft

$> 30.671 \text{ k-ft}$ OK

Per original design calcs
Connection bolts = $3/4" \phi$
& Max = 115.3 k-ft

* See next page for load summaries



	OLD		NEW	
D+C+S :	X	Y	X	Y
Reactions:	67.50	102.04	68.68	76.41
Haunch M:	630.18	k-ft	582.605	k-ft
Ridge M:	13.33	k-ft	-114.534	

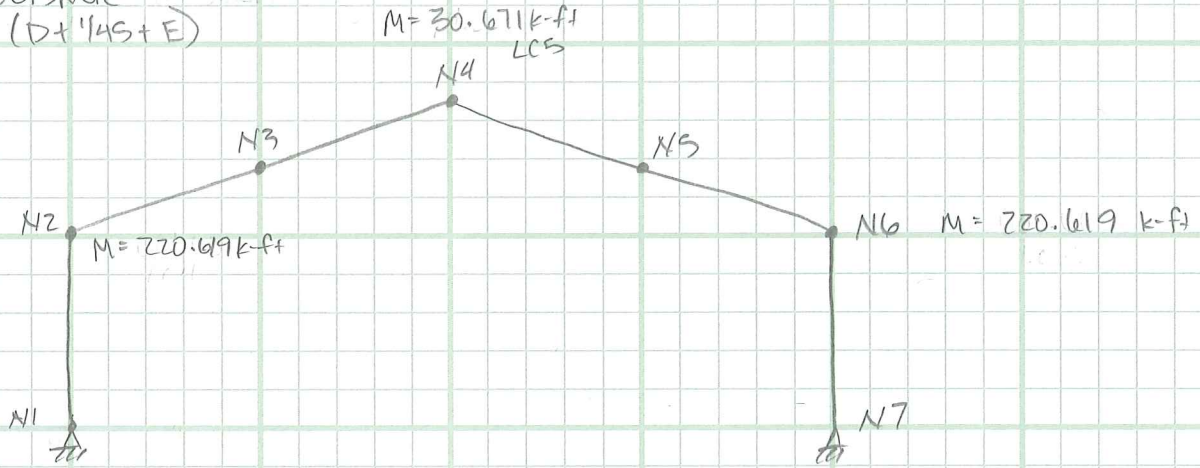
D+W :	X	Y	X	Y
Reactions:	N1 -8.52	-15.62	-14.306	-12.434
	N7 5.38	-8.52	14.292	-12.435
Haunch M:	88.34		-123.205	
Ridge M:	2.95		18.92	

D+E	X	Y	X	Y
Reactions:	N1 1.20	4.43	-0.853	3.684
	N7 -8.50	6.45	-10.097	6.724
Haunch M:	-4.79/-75.78		-14.959/91.905	
Ridge M:	0.05		-6.985	

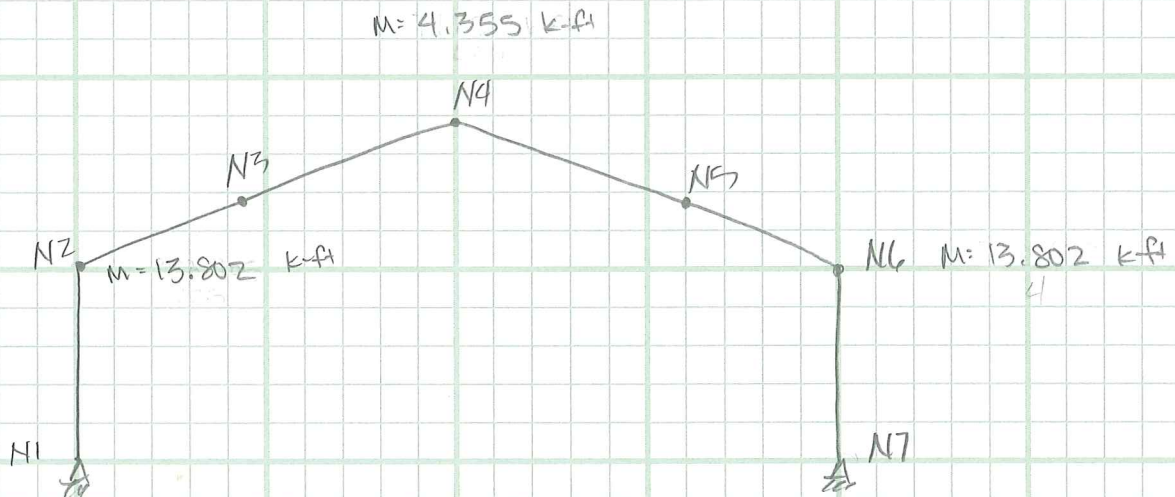
D+1/4S+E	X	Y	X	Y
Reactions:	N1 17.19	22.37	14.423	20.775
	N7 -24.49	24.59	-25.373	23.867
Haunch M:	138.10/208.95		111.662/220.619	
Ridge M:	0.23		-30.671	

Connection Forces

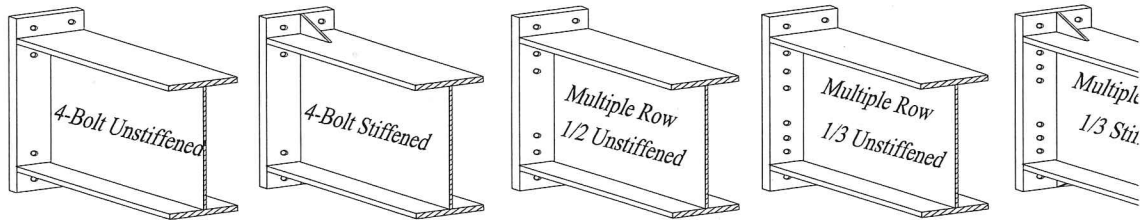
• Seismic
(D+1/4S+E)



• Wind (D+W)



RIDGE CONNECTION



☒ 4-Bolt Unstiffened
 ☐ 4-Bolt Stiffened
 ☐ Mul-Row 1/2 Unstiff..
 ☐ Mul-Row 1/3 Unstiff.
 ☐ Mul-F

Service Load Moment $M_w = 30.67$ kip-ft $< M_a = 170.43$ kip-ft

$F_{py} = 36$ psi End Plate $t_p = 1/2$ " Bolt Dia. = 3/4" A325

$b_p = 6.000$ in	$h = 29.250$ in
$t_f = 0.250$ in	$p_{f,o} = 1.500$ in
$g = 3.000$ in	$p_{f,i} = 1.500$ in
$p_{ext} = 3.000$ in	

Thin Plate Behavior W/ Prying Action Control !

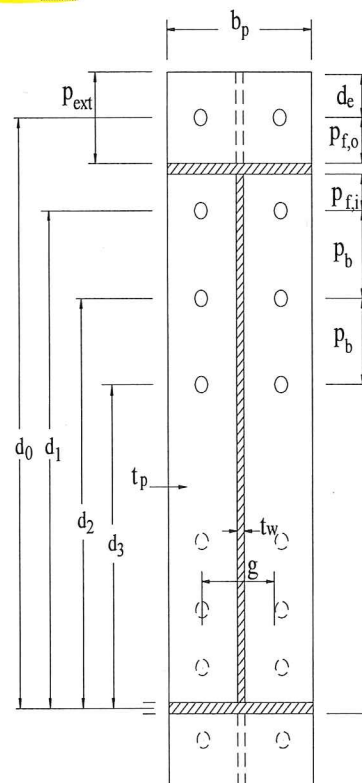
Connection Capacity OK !

$r_f = 1.00$	$s = 2.12$ in	$d_e = 1.50$ in
$h_0 = 30.75$ in	$h_1 = 27.50$ in	
$d_0 = 30.63$ in	$d_1 = 27.38$ in	
	$\Sigma d_n = 58.00$ in	
	$Y = 220.28$ in	

End-Plate Yielding:	$\phi_b M_{pl} = \phi_b F_{py} t^2 Y =$	148.69	k-ft
Bolt Rupture:	$P_t = (\pi d_b^2 / 4) F_t =$	39.76	kips
(No Prying Action)	$\phi M_{np} = \phi (2 P_t (\Sigma d_n)) =$	288.27	k-ft

Thin Plate Behavior(w/prying action)

$T_b =$	14.00	kips
$Q_{max,i} =$	4.50	kips
$Q_{max,o} =$	4.50	kips
$\phi M_q =$	255.65	k-ft



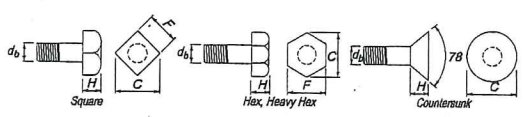
Fasteners,
unt

in.

1/8	1/4	3/8	1/2
—	—	—	—
—	—	—	—
48	197	—	—
54	205	261	333
60	212	270	344
67	220	279	355
74	229	290	366
81	237	300	379
88	246	310	391
95	255	321	403
102	263	332	416
109	272	342	428
116	280	353	441
123	289	363	453
130	298	374	465
137	306	384	478
144	315	395	490
151	324	405	503
158	332	416	515
165	341	426	527
172	349	437	540
179	358	447	552
186	367	458	565
193	375	468	577
200	384	479	589
207	392	489	602
214	401	500	614
221	410	510	626
228	418	521	639
235	427	531	651
242	435	542	664
—	444	552	676
—	453	563	689
28.0	34.4	42.5	49.7
11.3	13.8	16.8	20.0
34.0	31.6	31.2	32.9

7), updated for washer weights.

Table 7-20
Dimensions of Non-High-Strength Bolts, in.

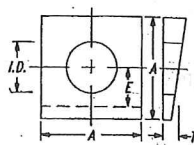
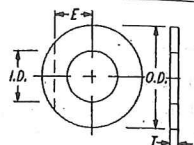


Bolts Dia <i>d_b</i> , in.		Square			Hex			Heavy Hex			Countersunk		Min, Thrd. Length, in.	
		<i>F</i> , in.	<i>C</i> , in.	<i>H</i> , in.	<i>F</i> , in.	<i>C</i> , in.	<i>H</i> , in.	<i>F</i> , in.	<i>C</i> , in.	<i>H</i> , in.	<i>C</i> , in.	<i>H</i> , in.	<i>L</i> ≤ 6 in.	<i>L</i> > 6 in.
Bolts	1/4	3/8	1/2	3/16	7/16	1/2	3/16	—	—	—	1/2	1/8	3/4	1
	3/8	9/16	13/16	1/4	9/16	5/8	1/4	—	—	—	11/16	3/16	1	1 1/4
	1/2	3/4	1 1/16	5/16	3/4	7/8	3/8	7/8	1	3/8	7/8	1/4	1 1/4	1 1/2
	5/8	15/16	1 5/16	7/16	15/16	1 1/16	7/16	1 1/16	1 1/4	7/16	1 1/8	5/16	1 1/2	1 3/4
	3/4	1 1/8	1 9/16	1/2	1 1/8	1 5/16	1/2	1 1/4	1 7/16	1/2	1 3/8	3/8	1 3/4	2
	7/8	1 5/16	1 7/8	5/8	1 5/16	1 1/2	9/16	1 7/16	1 11/16	9/16	1 9/16	7/16	2	2 1/4
	1	1 1/2	2 1/8	11/16	1 1/2	1 3/4	1 1/16	1 5/8	1 7/8	1 1/16	1 13/16	1/2	2 1/4	2 1/2
	1 1/8	1 11/16	2 3/8	3/4	1 11/16	1 15/16	3/4	1 13/16	2 1/16	3/4	2 1/16	9/16	2 1/2	2 3/4
	1 1/4	1 7/8	2 5/8	7/8	1 7/8	2 3/16	7/8	2	2 5/16	7/8	2 1/4	5/8	2 3/4	3
	1 3/8	2 1/16	2 15/16	15/16	2 1/16	2 3/8	15/16	2 3/16	2 1/2	15/16	2 1/2	1 1/16	3	3 1/4
	1 1/2	2 1/4	3 3/16	1	2 1/4	2 5/8	1	2 3/8	2 3/4	1	2 11/16	3/4	3 1/4	3 1/2
	1 3/4	—	—	—	2 5/8	3	1 3/16	2 3/4	3 3/16	1 3/16	—	—	3 3/4	4
	2	—	—	—	3	3 7/16	1 3/8	3 1/8	3 5/8	1 3/8	—	—	4 1/4	4 1/2
	2 1/4	—	—	—	3 3/8	3 7/8	1 1/2	3 1/2	4 1/16	1 1/2	—	—	4 3/4	5
	2 1/2	—	—	—	3 3/4	4 5/16	1 11/16	3 7/8	4 1/2	1 11/16	—	—	5 1/4	5 1/2
	2 3/4	—	—	—	4 1/8	4 3/4	1 13/16	4 1/4	4 15/16	1 13/16	—	—	5 3/4	6
3	—	—	—	4 1/2	5 3/16	2	4 5/8	5 5/16	2	—	—	6	6 1/2	
3 1/4	—	—	—	4 7/8	5 5/8	2 3/16	—	—	—	—	—	6	7	
3 1/2	—	—	—	5 1/4	6 1/16	2 5/16	—	—	—	—	—	6	7 1/2	
3 3/4	—	—	—	5 5/8	6 1/2	2 1/2	—	—	—	—	—	6	8	
4	—	—	—	6	6 15/16	2 11/16	—	—	—	—	—	6	8 1/2	

Notes:
For high-strength bolt and nut dimensions, refer to Table 7-15.
Square, hex, and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ANSI B18.2.1.
Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ANSI 18.5.
Minimum thread length = $2d_b + 1/4$ -in. for bolts up to 6 in. long, and $2d_b + 1/2$ -in. for bolts longer than 6 in.

SHEAR
FMC
MOM.
BRACE
COMF.
OTHER
SPEC.
WELDS
CONN.

asteners, in.



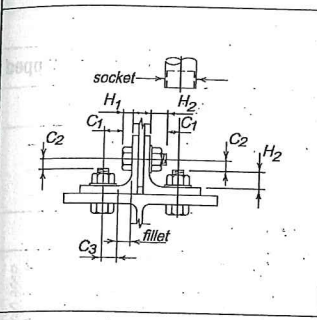
F, in

F, in	1 1/4	1 3/8	1 1/2
5/8	1 1/4	1 3/8	1 1/2
1/16	2	2 3/16	2 3/8
1/16	2 5/32	2 7/32	15/16
	2	2 1/4	2 1/4
1/2	15/8	1 3/4	1 7/8
1/16	2	2 3/16	2 3/8
64	17/32	1 11/32	1 15/32
1/4	2 1/2	2 3/4	3
1/4	1 3/8	1 1/2	1 5/8
36	0.136	0.136	0.136
77	0.177	0.177	0.177
	1 3/32	1 7/32	1 5/16
1/4	2 1/4	2 1/4	2 1/4
6	5/16	5/16	5/16
2	2:12	2:12	2:12
	1 3/32	1 7/32	1 5/16

+1/32
32

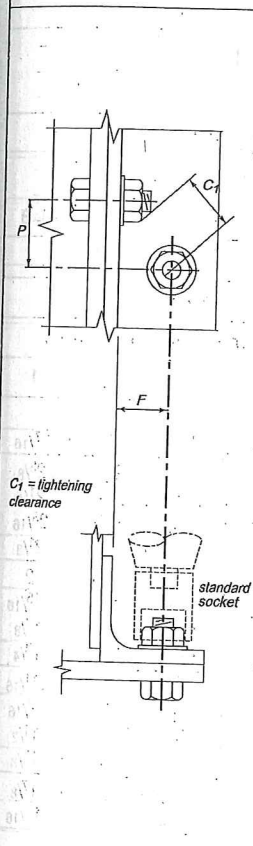
Table 7-16
Entering and Tightening Clearance, in.
Conventional ASTM A325 and A490 Bolts

Aligned Bolts



Nominal Bolt Dia.	Socket Dia.	H ₁	H ₂	C ₁	C ₂	C ₃	
						Circular	Clipped
5/8	1 3/4	25/64	1 1/4	1	1 1/16	1 1/16	9/16
3/4	2 1/4	15/32	1 3/8	1 1/4	3/4	3/4	1 1/16
7/8	2 1/2	35/64	1 1/2	1 3/8	7/8	7/8	13/16
1	2 5/8	39/64	1 5/8	1 7/16	15/16	1	7/8
1 1/8	2 7/8	11/16	1 7/8	1 9/16	1 1/16	1 1/8	1
1 1/4	3 1/8	25/32	2	1 11/16	1 1/8	1 1/4	1 1/8
1 3/8	3 1/4	27/32	2 1/8	1 3/4	1 1/4	1 3/8	1 1/4
1 1/2	3 1/2	15/16	2 1/4	1 7/8	1 5/16	1 1/2	1 5/16

Staggered Bolts



F	Stagger P, in.							
	Nominal Bolt Diameter, in.							
	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
1	15/8							
1 1/8	1 1/2							
1 1/4	1 1/2	1 15/16						
1 3/8	1 7/16	1 7/8	2 3/16					
1 1/2	1 1/4	1 13/16	2 1/8	25/16				
1 5/8	1 1/4	1 3/4	2 1/16	25/16	29/16			
1 3/4	1 3/16	1 11/16	2	2 1/4	29/16	2 13/16	3	
1 7/8	1 1/8	1 9/16	1 15/16	2 3/16	2 1/2	2 3/4	3	3 3/4
2	1 13/16	1 1/2	1 13/16	2 1/8	2 7/16	2 3/4	2 15/16	3 1/4
2 1/8	1 3/8	1 11/16	1 11/16	2	2 3/8	2 11/16	2 15/16	3 3/16
2 1/4	1 1/4	1 9/16	1 7/8	2 1/4	2 1/4	2 5/8	2 7/8	3 3/16
2 3/8	1 1/8	1 1/2	1 3/4	2 1/8	2 1/2	2 13/16	2 13/16	3 1/8
2 1/2		7/8	1 3/8	1 5/8	2	2 7/16	2 3/4	3 1/16
2 5/8			1 3/16	1 1/2	1 15/16	2 5/16	2 7/8	3
2 3/4			1 5/16	1 3/8	1 7/8	2 1/8	2 1/2	2 7/8
2 7/8				1 3/16	1 3/4	2 1/16	2 3/8	2 13/16
3				7/8	1 5/8	2	2 1/4	2 11/16
3 1/8					1 1/2	1 7/8	2 1/8	2 1/2
3 1/4					1 1/4	1 3/4	2	2 3/8
3 3/8					1 5/16	1 5/8	1 15/16	2 1/4
3 1/2						1 3/8	1 3/4	2 1/8
3 5/8						1 1/16	1 9/16	2
3 3/4							1 5/16	1 7/8
3 7/8								1 11/16
4								1 3/8

Notes:

H₁ = height of head
H₂ = maximum shank extension*
C₁ = clearance for tightening
C₂ = clearance for entering

C₃ = clearance for fillet*
P = bolt stagger

F = clearance for tightening staggered bolts

* Based on the use of one ASTM F436 washer.

SHEAR

FMC

MOM.

BRACE

COMP.

OTHER

SPEC.

WELDS

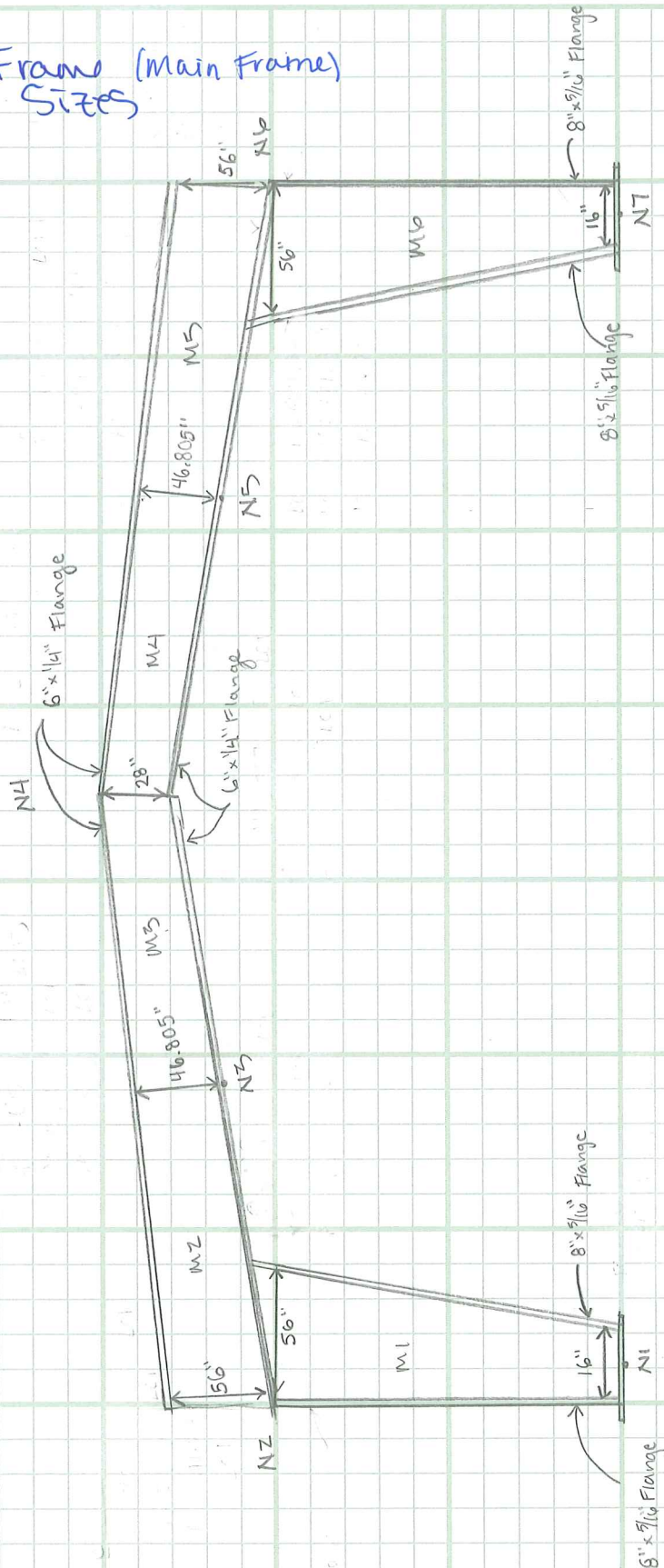
CONN.

Summary Page - Member Repairs - Main Frame

- @ column base: no repair needed; $M_{max} = 0 \text{ k-ft}$
- @ ~midpoint of col: no repair needed; $M_{max} = 92.232 \text{ k-ft}^{(10+1/4 \text{ ft } E)} < M_{max \text{ allow}} = 257.04 \text{ k-ft}$ using Sx
- @ top of column: no repair needed; $M_{max} = 220.619 \text{ k-ft}^{(10+1/4 \text{ ft } E)} < M_{max \text{ allow}} = 483.65 \text{ k-ft}$ using Sx
 (haunch)
- @ left/right ends: no repair needed; $M_{max} = 220.619 \text{ k-ft}^{(10+1/4 \text{ ft } E)} < M_{max \text{ allow}} = 383.556 \text{ k-ft}$ using Sx
 (haunch)
- @ N3/N5: no repair needed; $M_{max} = 35.259 \text{ k-ft}^{(10+1/4 \text{ ft } E)} < M_{max \text{ allow}} = 288.368 \text{ k-ft}$ using Sx
- @ Ridge: no repair needed; $M_{max} = 30.671 \text{ k-ft}^{(10+1/4 \text{ ft } E)} < M_{max \text{ allow}} = 133.11 \text{ k-ft}$ using Sx

* See following pages for moment frame member sizes & list of comparable loads summary.

*Moment Frame (Main Frame)
Member Sizes
Diagram



Load Summary:

LC	@ Base	@ Haunch	@ Ridge
① D+C+S	A = 86.518 k V = -55.551 k M = 0 k-ft	A = 86.326 V = 56.853 M = 582.605	A = 64.892 V = -17.502 M = -114.534
② D+W	A = -18.9 k V = 14.112 k M = 0 k-ft	A = -18.117 V = -12.38 M = -123.205	A = -12.06 V = 4.467 M = 18.92
③ D+E	A = 7.07 k V = -6.262 k M = 0 k-ft	A = 7.07 V = -6.262 M = 91.905	A = 4.581 V = 1.984 M = -6.985
④ D+1/4S+E	A = 25.191 k V = -15.796 k M = 0 k-ft	A = 25.191 V = -15.796 M = 220.619	A = 18.804 V = 5.29 M = -30.671
① D+C+S	A = 101.49 V = -53.86 M = 0	A = 87.57 V = 54.39 M = 614.40	A = 67.00 V = 18.76 M = -8.37
② D+W	A = -16.80 V = 5.88 M = 0	A = -8.62 V = -7.57 M = -88.34	A = -9.72 V = -2.46 M = -2.95
③ D+E	A = 4.57 V = -0.47 M = 0	A = 5.79 V = 2.68 M = -75.78	A = 4.94 V = 0.36 M = -0.05
④ D+1/4S+E	A = 24.86 V = -13.34 M = 0	A = 25.79 V = 14.72 M = 208.93	A = 20.31 V = 4.76 M = -0.23

New

original for comparison



NELSON ENGINEERING

Professional Engineers & Land Surveyors

Project Fairgrounds Job no. 17-202-01

sheet no. 4 of

By MML Date

JACKSON, WY • BUFFALO, WY

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Load Summary

@ N3/N5

	<u>New</u>	<u>Original</u>
① D+CIS	A= 78.351 V= 30.427 M= 16.261	A= 77.58 V= 25.78 M= 99.76
② D+W	A= -14.495 V= -6.337 M= 3.71	A= -9.01 V= -3.34 M= -13.46
③ D+E	A= 5.497 V= 2.716 M= 35.259	A= 5.24 V= 0.69 M= -17.72
④ D+1/4S+E	A= 22.735 V= 8.603 M= 3.998	A= 23.44 V= 6.18 M= 0.79

Load Summary

@ ~ midpoint of column:

	NEW	
① D+C+S	$A = 86.325 \text{ K}$ $V = -55.551 \text{ K}$ $M = 323.673 \text{ K} \cdot \text{ft}$	*no original values for ~ midpoint of column(s)
② D+W	$A = -18.554 \text{ K}$ $V = 11.999 \text{ K}$ $M = -74.521 \text{ K} \cdot \text{ft}$	
③ D+E	$A = 7.07 \text{ K}$ $V = -6.262 \text{ K}$ $M = 51.058 \text{ K} \cdot \text{ft}$	
④ D+1/4S+E	$A = 25.191 \text{ K}$ $V = -15.716 \text{ K}$ $M = 92.272 \text{ K} \cdot \text{ft}$	

Job Name C:\ABCP\FRAMES\51852001.01A
BUILDING -- RF-76'412'8-25

51852001

LOAD COMBINATION 1 --- D + C + S
ALLOWABLE STRESS FACTOR = 1.00

1989 AISC/ADS

UNIFORM LOADS

LIVE (PSF)	WIND (PSF)	DEAD (PSF)
75.00	0.00	8.70

WIND COEFFICIENTS			
C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS		LOCATION (FT.)
LOAD NO.	LINE NO.	
1	1	12.00
2	4	12.00
3	2	38.17

LOAD COMPONENTS			MOMENT (K-FT)
HORIZONTAL (KIPS)	VERTICAL (KIPS)		
0.00	-11.20		57.40
0.00	-11.20		-57.40
0.00	-0.75		0.00

MEMBER	NODES		AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	MEMBER END ACTIONS		
	I	J				AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	101.49	-53.86	0.00	-101.49	53.86	-554.67
2	2	3	87.57	54.39	614.48	-79.46	-25.55	-91.40
3	3	4	79.57	25.21	91.40	-67.00	18.76	8.37
4	4	5	67.00	18.76	-8.37	-79.57	25.21	-91.40
5	5	6	79.46	-25.55	91.40	-87.57	54.39	-614.48
6	6	7	101.49	53.86	554.67	-101.49	-53.86	0.00

SUPPORT	NODE	SUPPORT REACTIONS		
		HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	69.57	91.44	0.00
2	7	-69.57	91.44	0.00

Job Name C:\ABCP\FRAMES\51852001.01A
BUILDING -- RF-76'412'8-25

51852001

LOAD COMBINATION 2 --- D + W1->
ALLOWABLE STRESS FACTOR = 1.33

1989 AISC/ADS

UNIFORM LOADS

LIVE (PSF)	WIND (PSF)	DEAD (PSF)
0.00	18.53	5.70

WIND COEFFICIENTS			
C1	C2	C3	C4
0.80	-0.90	-0.70	-0.50

CONCENTRATED LOADS		LOCATION (FT.)
LOAD NO.	LINE NO.	
1	1	12.00
2	4	12.00

LOAD COMPONENTS			MOMENT (K-FT)
HORIZONTAL (KIPS)	VERTICAL (KIPS)		
0.00	5.46		-28.00
0.00	1.27		6.50

MEMBER	NODES		AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	MEMBER END ACTIONS		
	I	J				AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	-16.80	5.88	0.00	16.80	-5.88	60.51
2	2	3	-8.62	-7.57	-88.34	8.99	3.38	13.46
3	3	4	-9.01	-3.34	-13.46	9.55	-3.05	2.95
4	4	5	-9.72	-2.46	-2.95	9.11	-1.85	1.08
5	5	6	-9.10	1.89	-1.08	8.69	-4.71	46.82
6	6	7	-9.28	-3.93	-40.48	9.28	3.93	0.00

SUPPORT	NODE	SUPPORT REACTIONS		
		HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	-8.52	-15.62	0.00
2	7	5.38	-8.52	0.00

Job Name C:\ABCP\FRAMES\51852001.01A
BUILDING -- RF-76'412'8-25

51852001

LOAD COMBINATION 10 --- **D + E** →
ALLOWABLE STRESS FACTOR = 1.33

Base En ~ 2.738k / haunch.
1989 AISC/ADS

UNIFORM LOADS

LIVE WIND DEAD
(PSF) (PSF) (PSF)
0.00 0.00 5.70

1.5 = 1

* WIND COEFFICIENTS
* C1 C2 C3 C4
* 0.00 0.00 0.00 0.00

CONCENTRATED LOADS

LOAD NO. LINE NO. LOCATION
(FT.)
1 2 1.00
2 3 37.00

* LOAD COMPONENTS
* HORIZONTAL VERTICAL MOMENT
* (KIPS) (KIPS) (K-FT)
5.415 3.65 0.00 0.00
5.475 3.65 0.00 0.00

MEMBER END ACTIONS

MEMBER	NODES		AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	NODE J		
	I	J				AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	4.57	-0.47	0.00	-4.57	0.47	-4.79
2	2	3	5.79	2.68	4.95	-5.24	-0.72	17.22
3	3	4	5.24	0.69	-17.22	-4.38	2.30	0.05
4	4	5	4.94	0.36	-0.05	-5.79	2.63	-28.14
5	5	6	5.78	-2.66	28.14	-6.34	4.62	-75.78
6	6	7	7.74	7.34	75.62	-7.74	-7.34	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	1.20	4.43	0.00
2	7	-8.50	6.45	0.00

Job Name C:\ABCP\FRAMES\51852001.01A
BUILDING -- RF-76'412'8-25

51852001

LOAD COMBINATION 12 --- D + 1/4 S + E->
ALLOWABLE STRESS FACTOR = 1.33

1989 AISC/ADS

UNIFORM LOADS

LIVE (PSF)	WIND (PSF)	DEAD (PSF)
18.80	0.00	5.70

WIND COEFFICIENTS			
C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (FT.)
1	2	1.00
2	3	37.00

LOAD COMPONENTS			MOMENT (K-FT)
HORIZONTAL (KIPS)	VERTICAL (KIPS)		
5.475	3.65	0.00	0.00
5.475	3.65	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES		NODE I			NODE J		
	I	J	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	24.86	-13.34	0.00	-24.86	13.34	-137.39
2	2	3	25.79	14.72	138.10	-23.41	-6.28	-0.79
3	3	4	23.44	6.18	0.79	-19.76	6.70	0.23
4	4	5	20.31	4.76	-0.23	-23.99	8.11	-46.15
5	5	6	23.96	-8.22	46.15	-26.33	16.66	-208.93
6	6	7	28.03	20.22	208.22	-28.03	-20.22	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	17.19	22.37	0.00
2	7	-24.49	24.39	0.00

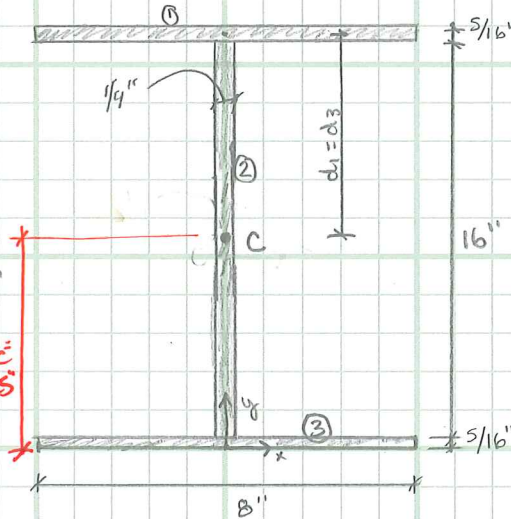
Column(s) - @ base

centroid:

$x_c = 0$

$y_c = 5/16" + \frac{16"}{2} = 8.3125"$

$y_c = 8.3125"$



- ① $A_1 = 5/16" \times 8" = 2.5 \text{ in}^2$
 $d_1 = (5/16" / 2) + \frac{16"}{2} = 8.15625"$
- ② $A_2 = 16" \times 1/4" = 4 \text{ in}^2$
 $d_2 = 0$
- ③ $A_3 = A_1 = 2.5 \text{ in}^2$
 $d_1 = d_3 = 8.15625"$

$$I_1 = I_{x1} + A_1 d_1^2 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{8(5/16")^3}{12} + (2.5 \text{ in}^2)(8.15625")^2 = 0.02035 \text{ in}^4 + 166.311 \text{ in}^4$$

$\therefore I_1 = 166.331 \text{ in}^4$

$$I_2 = I_{x2} + A_2 d_2^2 = \frac{bh^3}{12} = \frac{(1/4")(16")^3}{12} = 85.333 \text{ in}^4$$

$\therefore I_2 = 85.333 \text{ in}^4$

$I_3 = I_1$

$\therefore I_3 = 166.331 \text{ in}^4$

$$\Sigma I = 2(I_1) + I_2 = 2(166.331 \text{ in}^4) + 85.333 \text{ in}^4 = \underline{417.99 \text{ in}^4} = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{417.99 \text{ in}^4}{8.3125"} = \underline{50.285 \text{ in}^3} = S_x$$

$\Rightarrow M_{\text{max}} = 500.285 \text{ (ksi)} \div 12$
 $= 150.855 \text{ k-ft} \div 1.67 = 90.33 \text{ k-ft}$

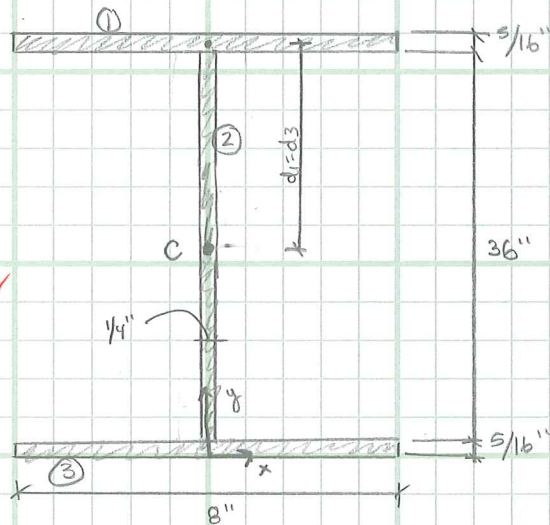
$M_{\text{max}} = 0 \text{ k (LC 2)}$

Column(s) - @ midpoint $\rightarrow h_{web} = \frac{56"_{max} + 16"_{min}}{2} = 36"$

centroid:

$$x_c = 0$$

$$y_c = \frac{5/16" + 36"}{2} = 18.3125" \checkmark$$



$$① A_1 = 2.5 \text{ in}^2$$

$$d_1 = \left(\frac{5/16"}{2}\right) + \frac{36"}{2} = 18.15625 \text{ in}$$

$$② A_2 = 36" \times \frac{1}{4}" = 9 \text{ in}^2$$

$$d_2 = 0$$

$$③ A_3 = A_1$$

$$d_3 = d_1$$

$$I_1 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{(8")(\frac{5}{16}")^3}{12} + 2.5 \text{ in}^2 (18.15625 \text{ in})^2 = 0.02035 \text{ in}^4 + 824.1235 \text{ in}^4$$

$$\therefore I_1 = 824.144 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{(\frac{1}{4}")(36")^3}{12} = 972 \text{ in}^4$$

$$\therefore I_2 = 972 \text{ in}^4$$

$$I_3 = I_1$$

$$\therefore I_3 = 824.144 \text{ in}^4$$

$$\Sigma I = (2)I_1 + I_2 = 2(824.144 \text{ in}^4) + 972 \text{ in}^4 = \underline{2620.288 \text{ in}^4} = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{2620.288 \text{ in}^4}{18.3125 \text{ in}} = \underline{143.087 \text{ in}^3} = S_{x_{act}} \checkmark$$

$$M_{max, allow} = S_x F_y = 143.087 \text{ in}^3 (36 \text{ ksi}) \times \frac{1 \text{ ft}}{12 \text{ in}} = 429.261 \text{ k} \cdot \text{ft} \times 1.67 \text{ (K)} = 257.04 \text{ k} \cdot \text{ft}$$

$$M_{max, allow} = 257.04 \text{ k} \cdot \text{ft} > M_{max} = 92.272 \text{ k} \cdot \text{ft} \text{ (D+1/4 S+E)}$$

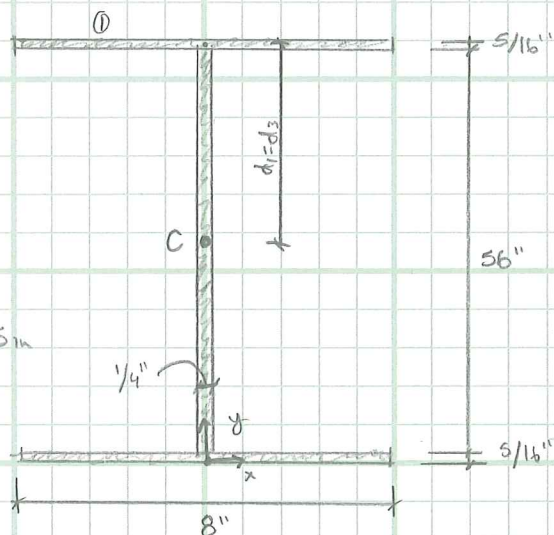
*Since snow loading remains the same (D+G+S) & the Max. moment from the new, additional loading (wind + seismic) is less than the Max. allow, the member @ $\sim 1/2$ of column is OK

Column(s) - @ top

centroid:

$$x_c = 0$$

$$y_c = \frac{5}{16}'' + \frac{56''}{2} = 28.3125''$$



$$\textcircled{1} A_1 = 2.5 \text{ in}^2$$

$$d_1 = \left(\frac{5}{16}'' \cdot \frac{1}{2}\right) + \frac{56''}{2} = 28.15625''$$

$$\textcircled{2} A_2 = \frac{1}{4}'' \times 56'' = 14 \text{ in}^2$$

$$d_2 = 0$$

$$\textcircled{3} A_3 = 2.5 \text{ in}^2$$

$$d_1 = d_3 = 28.15625''$$

$$I_1 = I_{x1} + A_1 d_1^2 = \frac{8'' \left(\frac{5}{16}''\right)^3}{12} + 2.5 \text{ in}^2 (28.15625'')^2 = 0.02035 \text{ in}^4 + 1981.936 \text{ in}^4$$

$$\therefore I_1 = 1981.956 \text{ in}^4$$

$$I_2 = I_{x2} + A_2 d_2^2 = \frac{1/4'' \times 56''^3}{12} = 3658.667 \text{ in}^4$$

$$\therefore I_2 = 3658.667 \text{ in}^4$$

$$I_1 = I_3 = 1981.956 \text{ in}^4$$

$$I_x = 2(I_1) + I_2 = 2(1981.956 \text{ in}^4) + 3658.667 \text{ in}^4 = 7622.579 \text{ in}^4 = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{7622.579 \text{ in}^4}{28.3125''} = 269.23 \text{ in}^3 = S_{x, \text{fact}} \Rightarrow M_{\text{max, allow}} = S_x F_y = 269.23 \text{ in}^3 (36 \text{ ksi}) \times 12 \text{ in/ft}$$

$$= 807.69 \text{ k} \cdot \text{ft} \times 0.6 = 483.65 \text{ k} \cdot \text{ft}$$

$$M_{\text{max, allow}} = 483.65 \text{ k} \cdot \text{ft} > M_{\text{max}} = 220.619 \text{ k} \cdot \text{ft} (D + 1/4 S + E)$$

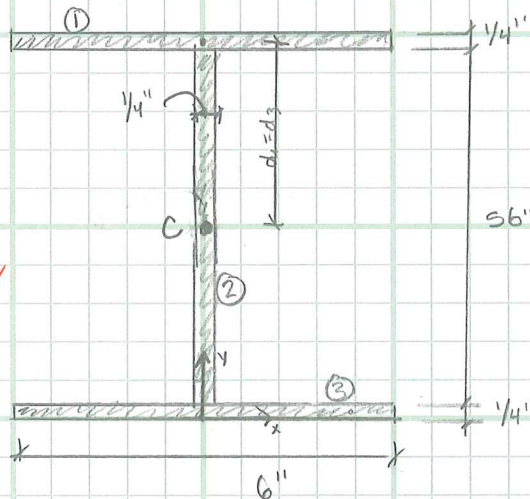
* since snow loading remains the same (D+crs) & the Maximum moment from the additional loading (seismic & wind) is less than the original D+crs loading the member @ the haunch is OK

Beams - @ left/right ends

Centroid:

$$x_c = 0$$

$$y_c = \frac{1}{4} + \frac{56}{2} = 28.25 \text{ in}$$



$$① A_1 = 6" \times \frac{1}{4}" = 1.5 \text{ in}^2$$

$$d_1 = (\frac{1}{4}" \times 2) + \frac{56"}{2} = 28.125 \text{ in}$$

$$② A_2 = \frac{1}{4}" \times 56" = 14 \text{ in}^2$$

$$d_2 = 0$$

$$③ A_3 = A_1 = 1.5 \text{ in}^2$$

$$d_3 = d_1 = 28.125 \text{ in}$$

$$I_1 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{6"(\frac{1}{4}")^3}{12} + 1.5 \text{ in}^2 (28.125 \text{ in})^2 = 0.00781 \text{ in}^4 + 1186.523 \text{ in}^4$$

$$\therefore I_1 = 1186.531 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{(\frac{1}{4}") (56")^3}{12} = 3658.667 \text{ in}^4$$

$$\therefore I_2 = 3658.667 \text{ in}^4$$

$$I_3 = I_1 = 1186.531 \text{ in}^4$$

$$\Sigma = 2(I_1) + I_2 = 2(1186.531 \text{ in}^4) + 3658.667 \text{ in}^4 = 6031.729 \text{ in}^4 = I_x$$

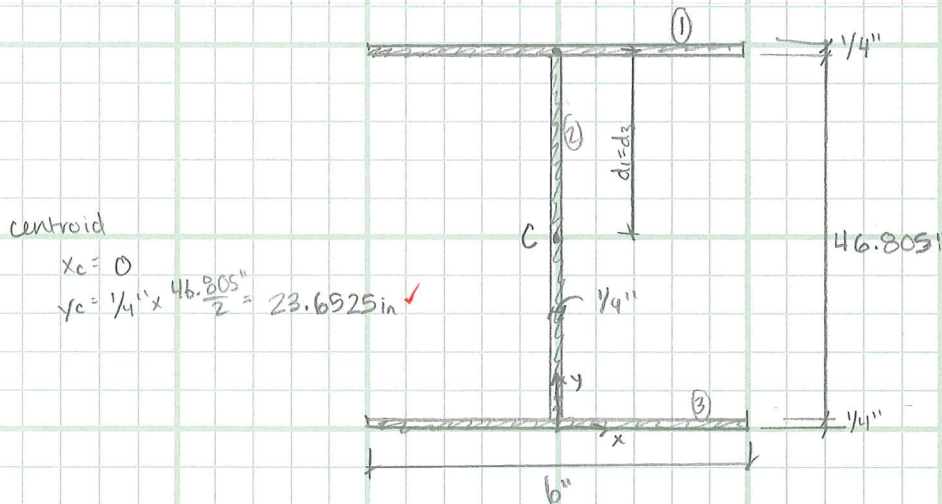
$$S_x = \frac{I_x}{y_c} = \frac{6031.729 \text{ in}^4}{28.25 \text{ in}} = 213.513 \text{ in}^3 = S_{x, \text{act}}$$

$$\begin{aligned} M_{\text{max}} &= S_x F_y = 213.513 \text{ in}^3 (36 \text{ ksi}) \times \frac{12}{17} \\ &= 640.539 \text{ k} \cdot \text{ft} \times 0.52 = 333.556 \text{ k} \cdot \text{ft} \end{aligned}$$

$$M_{\text{max, allow}} = 383.556 \text{ k} \cdot \text{ft} > M_{\text{max}} = 220.619 \text{ k} \cdot \text{ft} \quad (D + \frac{1}{4} S + E)$$

* Original Frame was designed for $M = 614.48 \text{ (D+C+S)}$
 & Since snow loading did not change & updated
 seismic & wind loading only produced $M = 162.29 \text{ k} \cdot \text{ft}$
 member @ left/right ends OK.

Beam(s) - @ N3/N5



centroid

$$x_c = 0$$

$$y_c = \frac{1}{4} \times \frac{46.805}{2} = 23.6925 \text{ in} \checkmark$$

$$\textcircled{1} A_1 = 6 \times \frac{1}{4} = 1.5 \text{ in}^2$$

$$d_1 = \left(\frac{1}{4} \times \frac{1}{2} \right) + \frac{46.805}{2} = 23.5275 \text{ in}$$

$$\textcircled{2} A_2 = \frac{1}{4} \times 46.805 = 11.701 \text{ in}^2$$

$$d_2 = 0$$

$$\textcircled{3} A_3 = A_1 = 1.5 \text{ in}^2$$

$$d_3 = d_1 = 23.5275 \text{ in}$$

$$I_1 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{6 \left(\frac{1}{4} \right)^3}{12} + 1.5 \text{ in}^2 (23.5275 \text{ in})^2 = 0.00781 \text{ in}^4 + 830.315 \text{ in}^4 =$$

$$\therefore I_1 = 830.3228 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{\frac{1}{4} (46.805 \text{ in})^3}{12} = 2136.169 \text{ in}^4$$

$$I_1 = I_3$$

$$\Sigma I = 2(I_1) + I_2 = 2(830.3228 \text{ in}^4) + 2136.169 \text{ in}^4 = \underline{3796.814 \text{ in}^4} = I_x \checkmark$$

$$S_x = \frac{I_x}{y_c} = \frac{3796.814 \text{ in}^4}{23.6925 \text{ in}} = \underline{160.525 \text{ in}^3} = S_{x, \text{act}} \checkmark$$

$$\Rightarrow M_{\text{max}} = 160.525 \text{ in}^3 (36 \text{ ksi}) \times \frac{12}{1} \text{ ft} = 481.575 \text{ K} \cdot \text{ft} \times 2 = 1.67$$

$$= 288.368 \text{ K} \cdot \text{ft}$$

* Since the snow loading remains the same (D+E)
+ the maximum moment produced from the
additional loading (seismic + wind) is less
than original D+E; loading on member
@ mid-points are OK

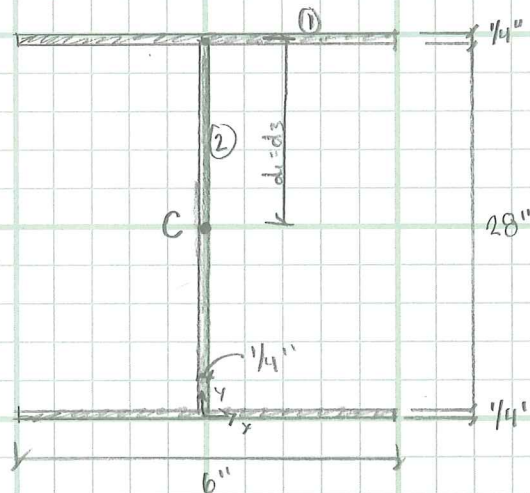
$$M_{\text{max,allow}} = 288.368 \text{ K} \cdot \text{ft} > M_{\text{max}} = 35.259 \text{ K} \cdot \text{ft} (D+E)$$

Beam(s) - @ ridge

centroid

$$x_c = 0$$

$$y_c = \frac{1}{4}'' + \frac{28''}{2} = 14.25'' \checkmark$$



$$① A_1 = 6'' \times \frac{1}{4}'' = 1.5 \text{ in}^2$$

$$d_1 = \left(\frac{1}{4}'' \times \frac{1}{2} \right) + \frac{28''}{2} = 14.125 \text{ in}$$

$$② A_2 = \frac{1}{4}'' \times 28'' = 7''$$

$$d_2 = 0$$

$$③ A_3 = A_1 = 1.5 \text{ in}^2$$

$$d_3 = d_1 = 14.125 \text{ in}$$

$$I_1 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{6''(\frac{1}{4}'')^3}{12} + 1.5 \text{ in}^2 (14.125 \text{ in})^2 = 0.00781 \text{ in}^4 + 299.273 \text{ in}^4$$

$$\therefore I_1 = 299.281 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{\frac{1}{4}''(28'')^3}{12} = 457.333 \text{ in}^4$$

$$I_3 = I_1$$

$$\Sigma I = 2(I_1) + I_2 = 2(299.281 \text{ in}^4) + 457.333 \text{ in}^4 = 1055.895 \text{ in}^4 = I_x \checkmark$$

$$S_x = \frac{I_x}{y_c} = \frac{1055.895 \text{ in}^4}{14.25 \text{ in}} = \underline{74.098 \text{ in}^3} \checkmark = S_x$$

$$\Rightarrow M_{\text{max}} = \frac{74.098 \text{ in}^3 \times 36 \text{ ksi} \times 1.12 \text{ ft}}{\text{allow}} = 222.294 \text{ k} \cdot \text{ft} \times 2 = 1.07$$

$$= 133.110 \text{ k} \cdot \text{ft}$$

$$* D + 1/4 S + E \text{ (New)} M = -30.67 \text{ k} \cdot \text{ft} < 133.110 \text{ k} \cdot \text{ft}$$

$$\therefore \underline{\underline{\text{OK}}}$$

Load Summary for Moment Connection - Endwall Frame

Load Combo	OLD		NEW	
D+C+S	X	Y	X	Y
Reactions	43.80	70.03	43.69	61.81
Haunch M	457.58		395.394	
Ridge M	7.64		104.159	
D+W	X	Y	X	Y
Reactions	① -5.29	-10.39	-11.627	-11.72
	② 3.21	-5.42	5.388	-8.025
Haunch M	-62.49		-87.708	
Ridge M	2.15		18.75	
D+E	X	Y	X	Y
Reactions	① 1.53	3.28	-0.523	2.66
	② -5.19	4.32	-6.737	4.736
Haunch M	11.66 / 48.64		9.255 / 64.652	
Ridge M	0.23		6.97	
D+1/4S+E	X	Y	X	Y
Reactions	① 12.05	15.19	9.182	14.154
	② -15.71	16.23	-16.442	16.26
Haunch M	105.89 / 143.35		76.956 / 152.033	
Ridge M	0.96		29.091	

Original Design
Calc Max

495.8 k-ft • Max Moment Haunch was designed for = 457.58 k-ft (D+C+S)

72.2 k-ft • Max Moment Ridge was designed for = 7.64 k-ft (D+C+S)

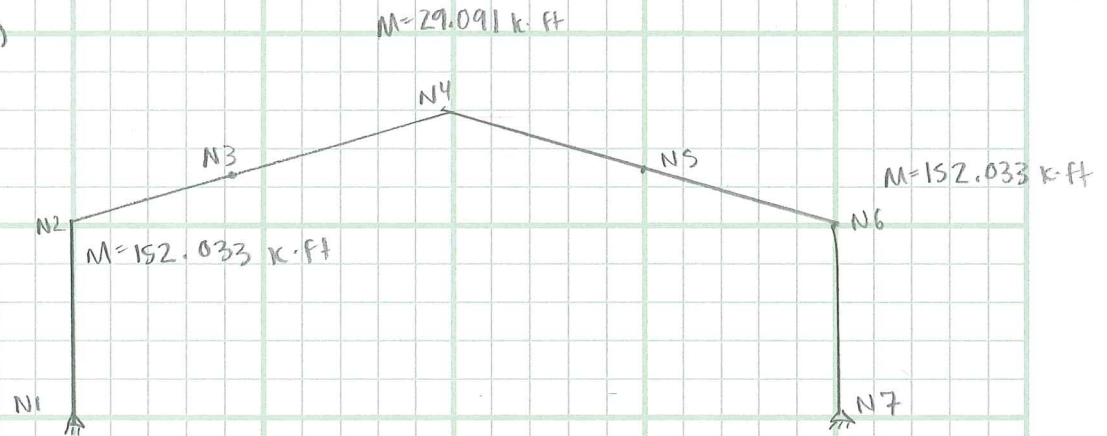
3/4" ϕ
(4) 5/8" ϕ Bolt Connection
M = ~~76.18~~ k-ft > 29.091 k-ft
104.32 k-ft

• Max new Moment at Haunch = 152.033 k-ft < ~~457.58~~ k-ft 495.8-A-k OK
(D+1/4S+E)

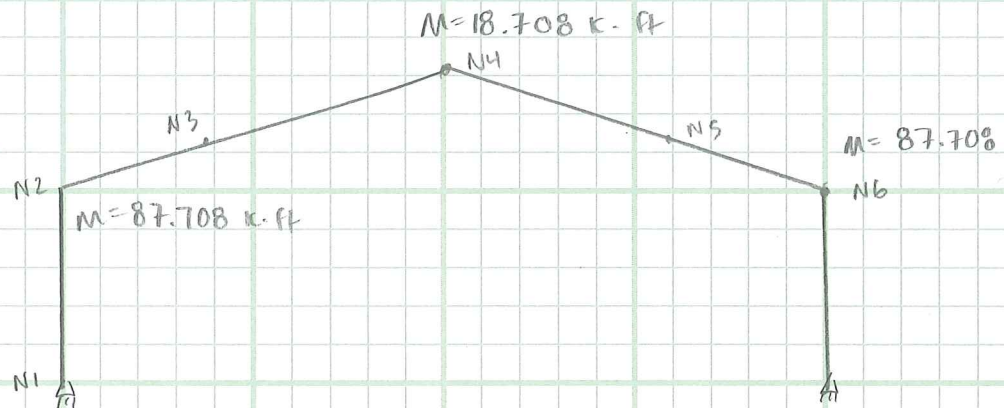
• Max new moment at ridge = 29.091 k-ft > 7.64 k-ft → Check Ridge Conn.
(D+1/4S+E)

Connection Forces

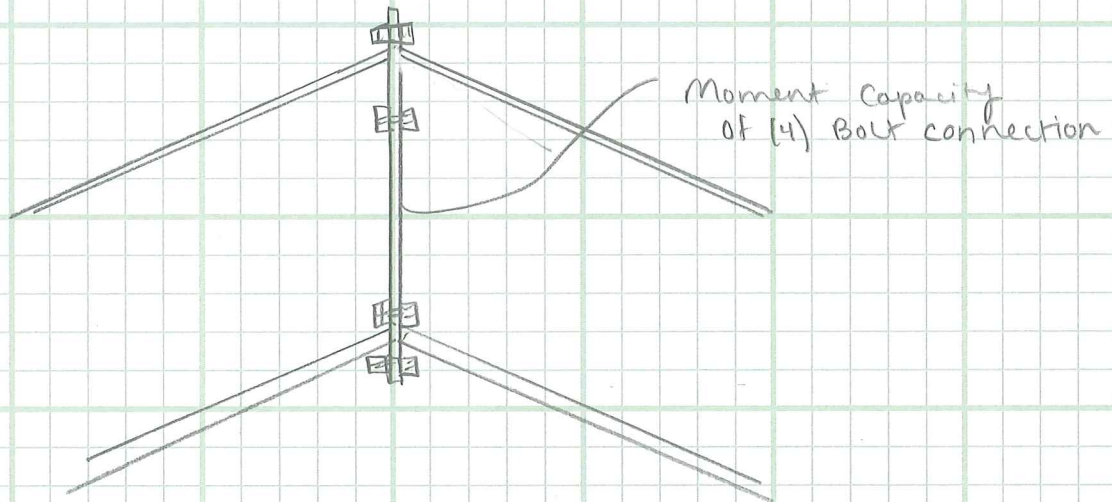
• Seismic
(D+1/4 S+E)



• Wind (D+W)



Connection @ Ridge:



$\frac{5}{8}'' \phi \text{ Bolts} = 76.18 \text{ k-ft}$

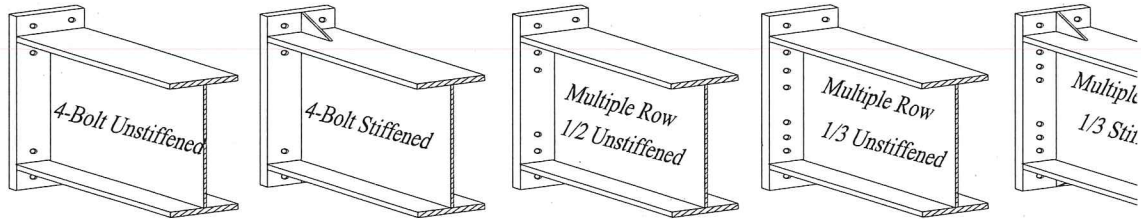
Per original design
calculations, connection
bolts are $\frac{3}{4}'' \phi$

$> 29.091 \text{ k-ft}$ OK

$M_{au} = 104.32 \text{ k-ft} > 29.091 \text{ k-ft}$ OK

$M_{au} (\text{per original calcs}) = 72.2 \text{ k-ft} > 29.091 \text{ k-ft}$ OK

RIDGE CONNECTION



☒ 4-Bolt Unstiffened
 ☐ 4-Bolt Stiffened
 ☐ Mul-Row 1/2 Unstiff..
 ☐ Mul-Row 1/3 Unstiff.
 ☐ Mul-f

Service Load Moment $M_w = 29.09$ kip-ft < $M_a = 104.32$ kip-ft

$F_{py} = 36$ psi
 End Plate $t_p = 1/2$ "
 Bolt Dia. = 3/4" A325

$b_p = 6.000$ in	$h = 18.000$ in
$t_f = 0.250$ in	$p_{f,o} = 1.500$ in
$g = 3.000$ in	$p_{f,i} = 1.500$ in
$p_{ext} = 3.000$ in	

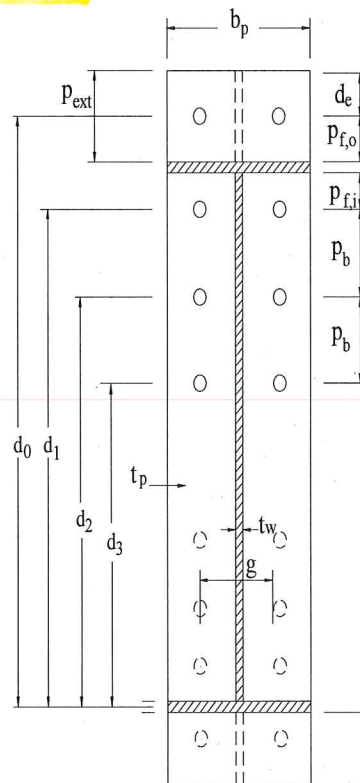
Thin Plate Behavior W/ Prying Action Control !
Connection Capacity OK !

$r_r = 1.00$	$s = 2.12$ in	$d_e = 1.50$ in
$h_0 = 19.50$ in	$h_1 = 16.25$ in	
$d_0 = 19.38$ in	$d_1 = 16.13$ in	
	$\Sigma d_n = 35.50$ in	
	$Y = 132.21$ in	

End-Plate Yielding:	$\phi_b M_{pl} = \phi_b F_{py} t^2 Y =$	89.24	k-ft
Bolt Rupture:	$P_t = (\pi d_b^2 / 4) F_t =$	39.76	kips
(No Prying Action)	$\phi M_{np} = \phi (2 P_t (\Sigma d_n)) =$	176.44	k-ft

Thin Plate Behavior(w/prying action)

$T_b =$	14.00	kips
$Q_{max,i} =$	4.50	kips
$Q_{max,o} =$	4.50	kips
$\phi M_q =$	156.47	k-ft

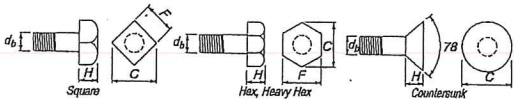


Fasteners,
unt

, in.			
1/8	1 1/4	1 3/8	1 1/2
—	—	—	—
48	197	—	—
54	205	261	333
60	212	270	344
67	220	279	355
74	229	290	366
81	237	300	379
88	246	310	391
95	255	321	403
102	263	332	416
109	272	342	428
116	280	353	441
123	289	363	453
130	298	374	465
137	306	384	478
144	315	395	490
151	324	405	503
158	332	416	515
165	341	426	527
172	349	437	540
179	358	447	552
186	367	458	565
193	375	468	577
200	384	479	589
207	392	489	602
214	401	500	614
221	410	510	626
228	418	521	639
235	427	531	651
242	435	542	664
—	444	552	676
—	453	563	689
28.0	34.4	42.5	49.7
1.3	13.8	16.8	20.0
4.0	31.6	31.2	32.9

), updated for washer weights.

Table 7-20
Dimensions of Non-High-Strength Bolts, in.



Bolts Dia d_b , in.		Square			Hex			Heavy Hex			Countersunk		Min. Thrd. Length, in.	
		F , in.	C , in.	H , in.	F , in.	C , in.	H , in.	F , in.	C , in.	H , in.	C , in.	H , in.	$L \leq 6$ in.	$L > 6$ in.
Bolts	1/4	3/8	1/2	3/16	7/16	1/2	3/16	—	—	—	1/2	1/8	3/4	1
	3/8	9/16	13/16	1/4	9/16	5/8	1/4	—	—	—	11/16	3/16	1	1 1/4
	1/2	3/4	1 1/16	5/16	3/4	7/8	3/8	7/8	1	3/8	7/8	1/4	1 1/4	1 1/2
	5/8	15/16	1 5/16	7/16	15/16	1 1/16	7/16	1 1/16	1 1/4	7/16	1 1/8	5/16	1 1/2	1 3/4
	3/4	1 1/8	1 9/16	1/2	1 1/8	1 5/16	1/2	1 1/4	1 7/16	1/2	1 3/8	3/8	1 3/4	2
	7/8	1 5/16	1 7/8	5/8	1 5/16	1 1/2	9/16	1 7/16	1 11/16	9/16	1 9/16	7/16	2	2 1/4
	1	1 1/2	2 1/8	1 1/16	1 1/2	1 3/4	1 1/16	1 5/8	1 7/8	1 1/16	1 13/16	1/2	2 1/4	2 1/2
	1 1/8	1 11/16	2 3/8	3/4	1 11/16	1 15/16	3/4	1 13/16	2 1/16	3/4	2 1/16	9/16	2 1/2	2 3/4
	1 1/4	1 7/8	2 5/8	7/8	1 7/8	2 3/16	7/8	2	2 5/16	7/8	2 1/4	5/8	2 3/4	3
	1 3/8	2 1/16	2 15/16	15/16	2 1/16	2 3/8	15/16	2 3/16	2 1/2	15/16	2 1/2	1 1/16	3	3 1/4
	1 1/2	2 1/4	3 3/16	1	2 1/4	2 5/8	1	2 3/8	2 3/4	1	2 11/16	3/4	3 1/4	3 1/2
	1 3/4	—	—	—	2 5/8	3	1 3/16	2 3/4	3 3/16	1 3/16	—	—	3 3/4	4
	2	—	—	—	3	3 7/16	1 3/8	3 1/8	3 5/8	1 3/8	—	—	4 1/4	4 1/2
	2 1/4	—	—	—	3 3/8	3 7/8	1 1/2	3 1/2	4 1/16	1 1/2	—	—	4 3/4	5
	2 1/2	—	—	—	3 3/4	4 5/16	1 11/16	3 7/8	4 1/2	1 11/16	—	—	5 1/4	5 1/2
	2 3/4	—	—	—	4 1/8	4 3/4	1 13/16	4 1/4	4 15/16	1 13/16	—	—	5 3/4	6
	3	—	—	—	4 1/2	5 3/16	2	4 5/8	5 5/16	2	—	—	6	6 1/2
	3 1/4	—	—	—	4 7/8	5 5/8	2 3/16	—	—	—	—	—	6	7
	3 1/2	—	—	—	5 1/4	6 1/16	2 5/16	—	—	—	—	—	6	7 1/2
	3 3/4	—	—	—	5 5/8	6 1/2	2 1/2	—	—	—	—	—	6	8
	4	—	—	—	6	6 15/16	2 11/16	—	—	—	—	—	6	8 1/2

Notes:
For high-strength bolt and nut dimensions, refer to Table 7-15.
Square, hex, and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ANSI B18.2.1.
Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ANSI 18.5.
Minimum thread length = 2d_b + 1/4-in. for bolts up to 6 in. long, and 2d_b + 1/2-in. for bolts longer than 6 in.

Notes:

For high-strength bolt and nut dimensions, refer to Table 7-15.
Square, hex, and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ANSI B18.2.1.
Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ANSI 18.5.
Minimum thread length = $2d_b + 1/4$ -in. for bolts up to 6 in. long, and $2d_b + 1/2$ -in. for bolts longer than 6 in.

SHEAR

FMC

MOM.

BRACE

COMP.

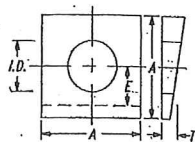
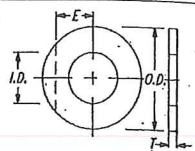
OTHER

SPEC.

WELDS

CONN.

fasteners, in.



in			
1/8	1 1/4	1 3/8	1 1/2
1/16	2	2 3/16	2 3/8
1/16	2 5/32	2 7/32	1 5/16
	2	2 1/4	2 1/4
1/2	1 5/8	1 3/4	1 7/8
1/16	2	2 3/16	2 3/8
3/4	1 7/32	1 11/32	1 15/32
1/4	2 1/2	2 3/4	3
1/4	1 3/8	1 1/2	1 5/8
3/16	0.136	0.136	0.136
7/8	0.177	0.177	0.177
	1 3/32	1 7/32	1 5/16
1/4	2 1/4	2 1/4	2 1/4
6	5/16	5/16	5/16
2	2:12	2:12	2:12
	1 3/32	1 7/32	1 5/16

 +1/32
32

Table 7-16
Entering and Tightening Clearance, in.
Conventional ASTM A325 and A490 Bolts

Aligned Bolts							
Nominal Bolt Dia.	Socket Dia.	H_1	H_2	C_1	C_2	C_3	
						Circular	Clipped
5/8	1 3/4	2 5/64	1 1/4	1	1 1/16	1 1/16	9/16
3/4	2 1/4	1 5/32	1 3/8	1 1/4	3/4	3/4	1 1/16
7/8	2 1/2	1 5/16	1 1/2	1 3/8	7/8	7/8	1 3/16
1	2 5/8	1 5/8	1 5/8	1 7/16	1 5/16	1	7/8
1 1/8	2 7/8	1 11/16	1 7/8	1 9/16	1 11/16	1 1/8	1
1 1/4	3 1/8	2 5/32	2	1 11/16	1 11/8	1 1/4	1 1/8
1 3/8	3 1/4	2 7/32	2 1/8	1 3/4	1 1/4	1 3/8	1 1/4
1 1/2	3 1/2	1 5/16	2 1/4	1 7/8	1 5/16	1 1/2	1 5/16

Staggered Bolts									
F	Stagger P, in.								
	Nominal Bolt Diameter, in.								
	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	
1	1 5/8								
1 1/8	1 1/2								
1 1/4	1 1/2	1 15/16							
1 3/8	1 7/16	1 7/8	2 3/16						
1 1/2	1 1/4	1 13/16	2 1/8	2 5/16					
1 5/8	1 1/4	1 3/4	2 1/16	2 5/16	2 9/16				
1 3/4	1 3/16	1 11/16	2	2 1/4	2 9/16	2 13/16	3		
1 7/8	1 1/8	1 9/16	1 15/16	2 3/16	2 1/2	2 3/4	3	3 3/4	
2	1	1 1/2	1 13/16	2 1/8	2 7/16	2 3/4	2 15/16	3 1/4	
2 1/8	1 3/16	1 3/8	1 11/16	2	2 3/8	2 11/16	2 15/16	3 3/16	
2 1/4		1 1/4	1 9/16	1 7/8	2 1/4	2 5/8	2 7/8	3 3/16	
2 3/8		1 1/8	1 1/2	1 3/4	2 1/8	2 1/2	2 13/16	3 1/8	
2 1/2		7/8	1 3/8	1 5/8	2	2 7/16	2 3/4	3 1/16	
2 5/8			1 3/16	1 1/2	1 15/16	2 5/16	2 7/8	3	
2 3/4			1 5/16	1 3/8	1 7/8	2 1/8	2 1/2	2 7/8	
2 7/8				1 3/16	1 3/4	2 1/16	2 3/8	2 13/16	
3				7/8	1 5/8	2	2 1/4	2 11/16	
3 1/8					1 1/2	1 7/8	2 1/8	2 1/2	
3 1/4					1 1/4	1 3/4	2	2 3/8	
3 3/8					1 5/16	1 5/8	1 15/16	2 1/4	
3 1/2						1 3/8	1 3/4	2 1/8	
3 5/8						1 1/16	1 9/16	2	
3 3/4							1 5/16	1 7/8	
3 7/8								1 11/16	
4								1 3/8	

Notes:

H_1 = height of head
 H_2 = maximum shank extension*
 C_1 = clearance for tightening
 C_2 = clearance for entering

C_3 = clearance for fillet*

P = bolt stagger

F = clearance for tightening staggered bolts

* Based on the use of one ASTM F436 washer.

SHEAR

FMC

MOM.

BRACE

COMP.

OTHER

SPEC.

WELDS

CONN.

Summary Page - Member Repairs - End Frame

@ column base - no repair needed; $M = 0 \text{ k}\cdot\text{ft}$

@ ~midpoint of col. - no repair needed; $M_{\max} = 84.463 \text{ k}\cdot\text{ft} < M_{\max_{\text{allow}}} = 187.506 \text{ k}\cdot\text{ft}$ using Sx

@ top of col. (haunch) - no repair needed; $M_{\max} = 152.034 \text{ k}\cdot\text{ft} < M_{\max_{\text{allow}}} = 351.604 \text{ k}\cdot\text{ft}$ using Sx

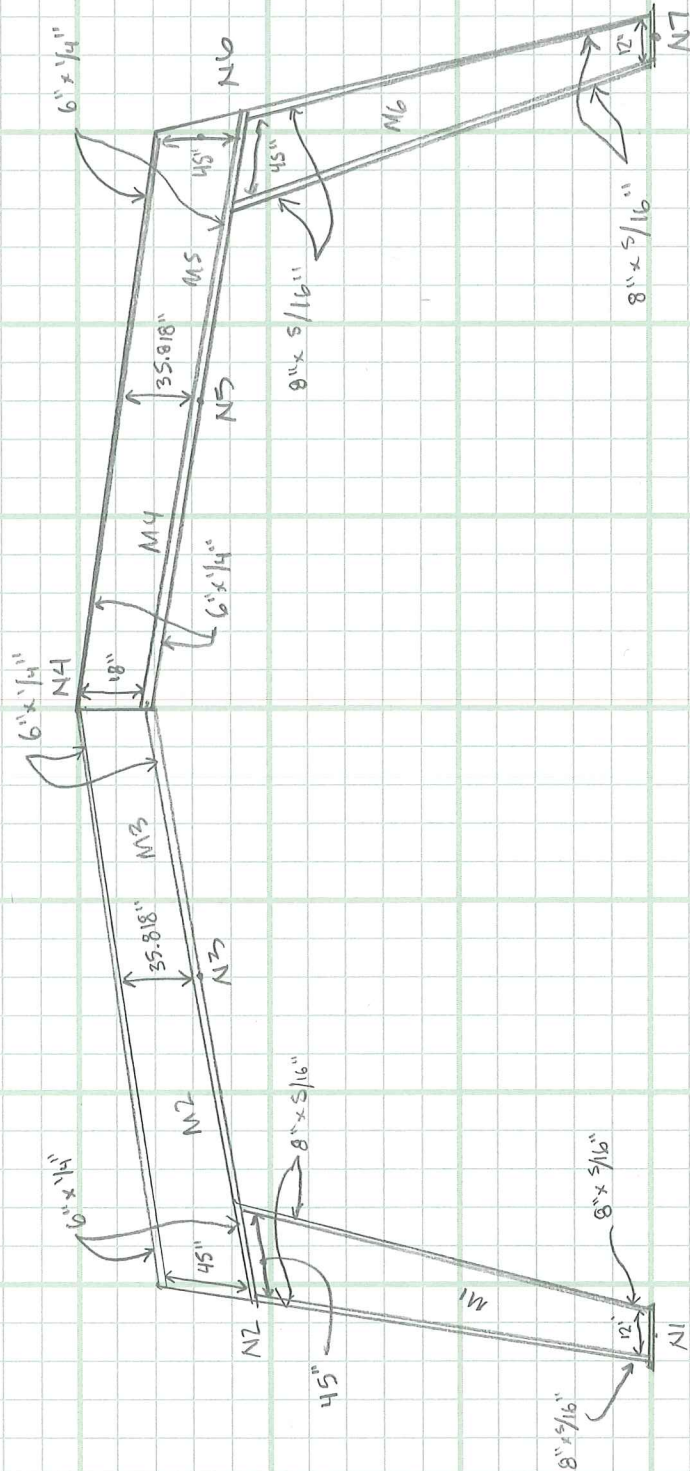
@ 1/2 ends (haunch) - no repair needed; $M_{\max} = 152.034 \text{ k}\cdot\text{ft} < M_{\max_{\text{allow}}} = 274.327 \text{ k}\cdot\text{ft}$ using Sx

@ N3/WS - no repair needed; $M_{\max} = 25.27 \text{ k}\cdot\text{ft} < M_{\max_{\text{allow}}} = 11.227 \text{ k}\cdot\text{ft}$ using Sx

@ Ridge - no repair needed; $M_{\max} = 29.091 \text{ k}\cdot\text{ft} < M_{\max_{\text{allow}}} = 72.11 \text{ k}\cdot\text{ft}$ using Sx

*see following pages for moment frame member sizes,
and list of comparable load summary

* Moment Frame (End Frame)
Member sizes
Diagram





NELSON ENGINEERING

Professional Engineers & Land Surveyors

Project Fairgrounds

Job no. 17-202-01

End Frame

sheet no. E3 of

By MPL

Date 7/25

JACKSON, WY • BUFFALO, WY

WWW.NELSONENGINEERING.NET

Load Summary:

LC	@ Base	@ Cal. midspan	@ Haunch	@ N3/MS	@ Ridge	
① D+C+S	A = 56.805 K V = -37.265 K M = 0 K-ft	A = 56.805 K V = -37.265 K M = 219.664 K-ft	A = 56.805 K V = 38.592 K M = 395.394 K-ft	A = 51.003 K V = 20.492 K M = -2.741 K-ft	A = 42.051 K V = 11.45 K M = -104.159 K-ft	NEW
② D+W	A = -13.128 V = 9.976 M = 0	A = -12.88 V = 8.076 M = -53.205	A = -12.681 V = -9.048 M = -87.708	A = -9.78 V = -4.558 M = 3.971	A = -8.165 V = -3.288 M = 18.755	
③ D+E	A = 5.569 V = -6.093 M = 0	A = 5.569 V = -6.093 M = 35.918	A = 5.569 V = -6.093 M = 64.652	A = 3.914 V = 2.422 M = -23.835	A = 3.272 V = 1.848 M = -6.97	
④ D+1/4S+E	A = 18.253 V = -14.329 M = 0	A = 18.253 V = -14.329 M = 84.463	A = 18.253 V = -14.329 M = 152.034	A = 15.252 V = 6.9 M = -25.27	A = 12.612 V = -4.486 M = -29.091	
① D+C+S	A = 67.21 V = -37.29 M = 0	n/a	A = 57.71 V = 36.60 M = 442.90	A = 52.38 V = 17.21 M = 74.44	A = 44.06 V = 12.12 M = -3.29	Original
② D+W	A = -10.99 V = 3.90 M = 0	n/a	A = -5.34 V = -5.00 M = -62.49	A = 9.61 V = -2.27 M = -11.04	A = -6.10 V = -1.56 M = -2.15	
③ D+E	A = -4.96 V = -4.58 M = 0	n/a	A = 4.96 V = 4.58 M = 48.64	A = 3.96 V = -1.69 M = 16.35	A = 3.37 V = 0.42 M = 0.23	
④ D+1/4S+E	A = -18.13 V = -13.47 M = 0	n/a	A = 18.13 V = 13.47 M = 143.35	A = 15.44 V = -5.42 M = 30.67	A = 13.50 V = 3.29 M = 0.96	

Job Name C:\ABCP\FRAMES\51852001.01B
BUILDING -- NONEXPANDABLE ENDWALLS

51852001

LOAD COMBINATION 1 --- D + C + S
ALLOWABLE STRESS FACTOR = 1.00

1989 AISC/ADS

UNIFORM LOADS

LIVE (PSF)	WIND (PSF)	DEAD (PSF)
75.00	0.00	9.00

WIND COEFFICIENTS			
C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS			LOAD COMPONENTS			MOMENT (K-FT)
LOAD NO.	LINE NO.	LOCATION (FT.)	HORIZONTAL (KIPS)	VERTICAL (KIPS)		
1	1	12.00	0.00	-8.22	45.91	
2	4	12.00	0.00	-8.22	-45.91	
3	2	38.17	0.00	-0.75	0.00	

MEMBER	NODES		MEMBER END ACTIONS			MEMBER END ACTIONS		
	I	J	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	67.21	-37.29	0.00	-67.21	37.29	-395.70
2	2	3	57.71	36.60	442.90	-52.32	-17.37	-74.44
3	3	4	52.38	17.21	74.44	-44.06	12.12	3.29
4	4	5	44.06	12.12	-3.29	-52.38	17.21	-74.44
5	5	6	52.32	-17.37	74.44	-57.71	36.60	-442.90
6	6	7	67.21	37.29	395.70	-67.21	-37.29	0.00

SUPPORT	NODE	SUPPORT REACTIONS		
		HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	45.69	61.81	0.00
2	7	-45.69	61.81	0.00

43.713 51.576
-43.713 51.576

Forces: Haunch: A=56.809
V=-37.215/38.592
M=395.394

Ridge: A=42.051
V=-11.45/11.45
M=-104.159

Job Name C:\ABCP\FRAMES\51852001.01B
BUILDING -- NONEXPANDABLE ENDWALLS

51852001

LOAD COMBINATION 2 --- D + W1->
ALLOWABLE STRESS FACTOR = 1.33

1989 AISC/ADS

UNIFORM LOADS

LIVE (PSF)	WIND (PSF)	DEAD (PSF)
0.00	18.53	6.00

WIND COEFFICIENTS			
C1	C2	C3	C4
0.80	-0.90	-0.70	-0.50

CONCENTRATED LOADS			LOCATION	LOAD COMPONENTS		MOMENT
LOAD NO.	LINE NO.		(FT.)	HORIZONTAL (KIPS)	VERTICAL (KIPS)	(K-FT)
1	1		12.00	0.00	3.79	-21.17
2	4		12.00	0.00	0.84	4.72

MEMBER	NODES		MEMBER END ACTIONS			MEMBER END ACTIONS		
	I	J	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	-10.99	3.90	0.00	10.99	-3.90	41.41
2	2	3	-5.34	-5.00	-62.49	5.61	2.29	11.04
3	3	4	-5.61	-2.27	-11.04	6.01	-1.87	2.15
4	4	5	-6.10	-1.56	-2.15	5.66	-1.19	0.77
5	5	6	-5.65	1.21	-0.77	5.36	-3.01	30.98
6	6	7	-5.79	-2.48	-26.35	5.79	2.48	0.00

SUPPORT	NODE	SUPPORT REACTIONS		
		HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	-5.29	-10.39	0.00
2	7	3.21	-5.42	0.00

-11.627 -11.72
5.388 -8.625

Forces: Haunch: A = -12.681 / -11.039
V = 6.557 / -9.048
M = -87.708

Ridge: A = -7.574 / -8.161
V = 3.287 / -1.219
M = 18.75

Job Name C:\ABCP\FRAMES\51852001.01B
BUILDING -- NONEXPANDABLE ENDWALLS

51852001

LOAD COMBINATION 10 --- D + E->
ALLOWABLE STRESS FACTOR = 1.33

1989 AISC/ADS

UNIFORM LOADS

LIVE WIND DEAD
(PSF) (PSF) (PSF)
0.00 0.00 6.00

* WIND COEFFICIENTS
*
* C1 C2 C3 C4
*
0.00 0.00 0.00 0.00

CONCENTRATED LOADS
LOAD NO. LINE NO.

1 2
2 3

LOCATION
(FT.)
1.00
37.00

* LOAD COMPONENTS
* HORIZONTAL VERTICAL MOMENT
* (KIPS) (KIPS) (K-FT)
-> 3.63 1.83 0.00 0.00
-> 3.63 1.83 0.00 0.00

MEMBER END ACTIONS

MEMBER	NODES		NODE I			NODE J		
	I	J	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	3.45	-1.09	0.00	-3.45	1.09	-11.57
2	2	3	4.07	2.06	11.66	-3.68	-0.69	7.21
3	3	4	3.68	0.68	-7.21	-3.09	1.42	-0.23
4	4	5	3.37	0.42	0.23	-3.97	1.68	-16.35
5	5	6	3.96	-1.69	16.35	-4.35	3.06	-48.73
6	6	7	4.96	4.58	48.64	-4.96	-4.58	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	1.53	3.28	0.00
2	7	-5.19	4.32	0.00

left right

Forces: Haunch: A=2.569/3.709 4.27/5.969
V=0.872/1.731 -3.719/6.093
M=-9.255 -64.652

Ridge: A=2.705/3.272
V=-1.848/-0.157
M=-6.97

Job Name C:\ABCP\FRAMES\51852001.01B
BUILDING -- NONEXPANDABLE ENDWALLS

51852001

LOAD COMBINATION 12 --- D + 1/4S + E->
ALLOWABLE STRESS FACTOR = 1.33

1989 AISC/ADS

UNIFORM LOADS

LIVE (PSF)	WIND (PSF)	DEAD (PSF)
18.80	0.00	6.00

WIND COEFFICIENTS			
C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS			LOCATION	LOAD COMPONENTS		MOMENT
LOAD NO.	LINE NO.		(FT.)	HORIZONTAL (KIPS)	VERTICAL (KIPS)	(K-FT)
1	2		1.00	→ 3.43	1.83	0.00
2	3		37.00	→ 3.43	1.83	0.00

MEMBER	NODES		MEMBER END ACTIONS			MEMBER END ACTIONS		
	I	J	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)	AXIAL (KIPS)	SHEAR (KIPS)	MOMENT (K-FT)
1	1	2	16.63	-9.98	0.00	-16.63	9.98	-105.89
2	2	3	17.25	10.09	106.28	-15.65	-4.42	-7.11
3	3	4	15.67	4.37	7.11	-13.21	4.29	-0.96
4	4	5	13.50	3.29	0.96	-15.95	5.37	-30.67
5	5	6	15.94	-5.42	30.67	-17.53	11.09	-143.35
6	6	7	18.13	13.47	142.97	-18.13	-13.47	0.00

SUPPORT	NODE	SUPPORT REACTIONS		
		HORIZ. (KIPS)	VERTICAL (KIPS)	MOMENT (K-FT)
1	1	12.05	15.19	0.00
2	7	-15.71	16.23	0.00

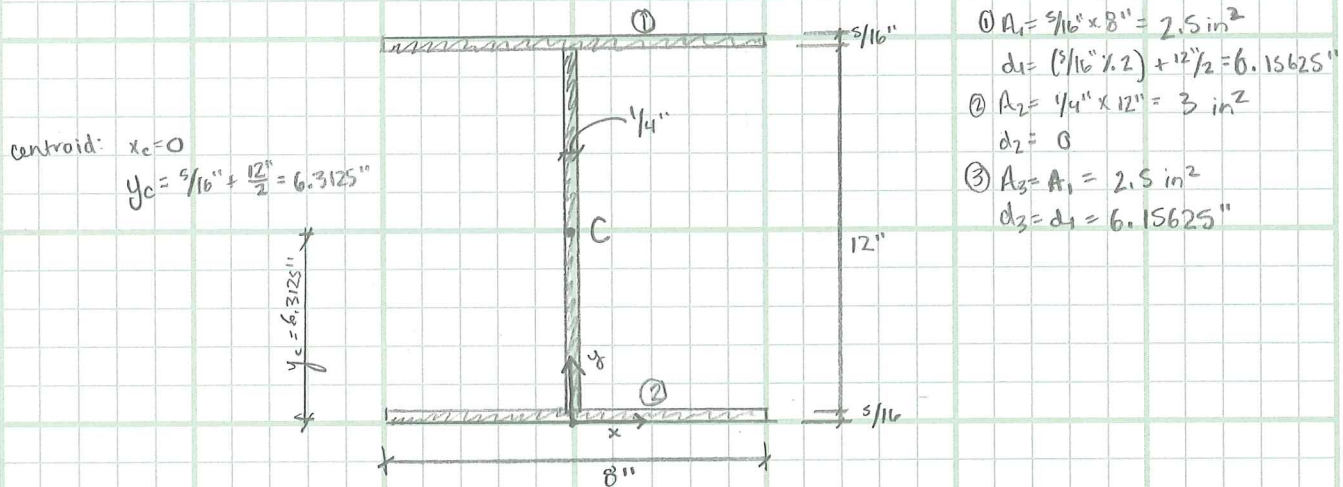
9.182 14.151
-16.442 14.210

Forces : Haunch: left
A= 15.225 / 14.151
V= -7.253 / 10.253
M= 76.956

right
A= 16.726 / 18.253
V= -12.237 / 14.329
M= 152.033

Ridge: A= 12.037 / 12.612
V= -4.486 / 2.429
M= -29.091

Column(s) - @ base (end frame)



$$I_1 = I_3 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{8''(\frac{5}{16}'')^3}{12} + 2.5 \text{ in}^2 (6.15625 \text{ in})^2 = 0.02035 \text{ in}^4 + 94.749 \text{ in}^4$$

$$\therefore I_1 = I_3 = 94.769 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{(\frac{1}{4}'')(12'')^3}{12} = 36 \text{ in}^4$$

$$\Sigma I = 2(I_1) + I_2 = 2(94.769 \text{ in}^4) + 36 \text{ in}^4 = \underline{225.538 \text{ in}^4} = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{225.538 \text{ in}^4}{6.3125 \text{ in}} = \underline{35.729 \text{ in}^3} = S_x \Rightarrow M_{\text{max}} = S_x F_y = 35.729 \text{ in}^3 (36 \text{ ksi}) \times \frac{1}{12}$$

$$= 107.186 \text{ k} \cdot \text{ft} \times 1.67 (\Omega)$$

$$= \underline{164.183 \text{ k} \cdot \text{ft}}$$

$$M_{\text{max}} = 0 \text{ k} \cdot \text{ft}$$



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Project Fairgrounds

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Section Properties

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Date 3/25

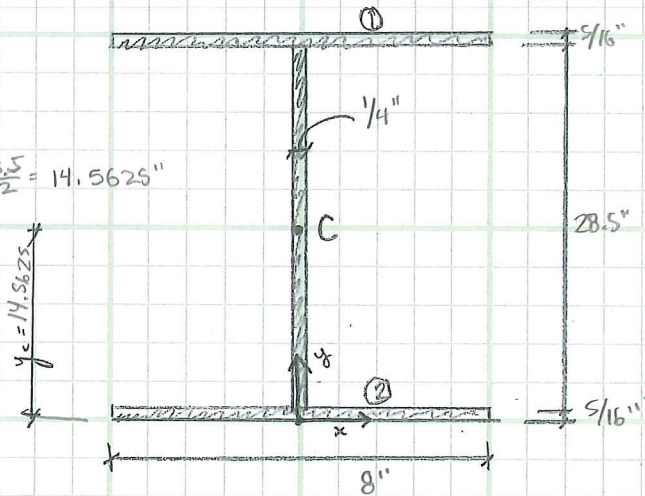
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Column(s) ① ~ midpoint (end frame)

$$h_{web} = \frac{12" \text{ min} + 45" \text{ max}}{2} = 28.5"$$

centroid: $x_c = 0$

$$y_c = \frac{5/16" + 28.5"}{2} = 14.5625"$$



$$① A_1 = 2.5 \text{ in}^2$$

$$d_1 = \left(\frac{5/16" \cdot 2}{2} \right) + \frac{28.5"}{2} = 14.40625 \text{ in}$$

$$② A_2 = 1/4" \times 28.5" = 7.125 \text{ in}^2$$

$$d_2 = 0$$

$$③ A_3 = A_1 = 2.5 \text{ in}^2$$

$$d_3 = d_1 = 14.40625 \text{ in}$$

$$I_1 = I_3 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{8" \left(\frac{5}{16}" \right)^3}{12} + 2.5 \text{ in}^2 (14.40625 \text{ in})^2 = 0.02035 \text{ in}^4 + 518.85 \text{ in}^4$$

$$\therefore I_1 = I_3 = 518.87 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{1/4" (28.5")^3}{12} = 482.273 \text{ in}^4$$

$$I_x = 2(I_1) + I_2 = 2(518.87 \text{ in}^4) + 482.273 \text{ in}^4 = 1520.013 \text{ in}^4 = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{1520.013 \text{ in}^4}{14.5625 \text{ in}} = 104.379 \text{ in}^3 = S_x$$

$$\Rightarrow M_{\text{max allow}} = S_x F_y = 104.379 \text{ in}^3 (36 \text{ ksi}) \cdot \frac{12"}{ft} = 313.136 \text{ k} \cdot \text{ft} \cdot \frac{1}{1.67 \Omega} = 187.506 \text{ k} \cdot \text{ft}$$

$$M_{\text{max}} = \frac{(D + 1/4 S + E)}{2} = 84.463 \text{ k} \cdot \text{ft} < M_{\text{max allow}}$$

* since snow loading remains the same (D + (1/4 S) & the max moment (with new, additional loading (wind & seismic) is less than the max. allow moment, the member @ 1/2 column is OK



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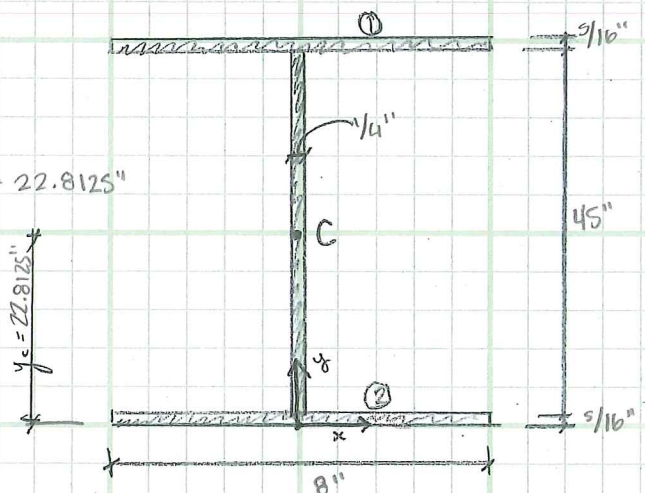
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Column(s) - @ top (end frame)

Centroid: $x_c = 0$

$$y_c = \frac{5}{16} + \frac{45}{2} = 22.8125"$$



$$① A_1 = 2.5 \text{ in}^2$$

$$d_1 = \left(\frac{5}{16} \cdot \frac{1}{2}\right) + \frac{45}{2} = 22.65625 \text{ in}$$

$$② A_2 = \frac{1}{4} \times 45 = 11.25 \text{ in}^2$$

$$d_2 = 0$$

$$③ A_3 = A_1 = 2.5 \text{ in}^2$$

$$d_3 = d_1 = 22.65625 \text{ in}$$

$$I_1 = I_3 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{8 \left(\frac{5}{16}\right)^3}{12} + 2.5 \text{ in}^2 (22.65625 \text{ in})^2 = 0.02035 \text{ in}^4 + 1283.264 \text{ in}^4$$

$$\therefore I_1 = I_3 = 1283.285 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{\frac{1}{4} \times (45)^3}{12} = 1898.4375 \text{ in}^4$$

$$\Sigma I = 2(I_1) + I_2 = 2(1283.285 \text{ in}^4) + 1898.4375 \text{ in}^4 = \underline{4465.008 \text{ in}^4} = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{4465.008 \text{ in}^4}{22.8125"} = \underline{195.726 \text{ in}^3} = S_x \Rightarrow M_{\text{max}} = S_x F_y = 195.726 \text{ in}^3 (36 \text{ ksi}) \times \frac{12}{17} \text{ ft} = 587.179 \text{ k} \cdot \text{ft} \times 1.6772 = \underline{351.604 \text{ k} \cdot \text{ft}}$$

$$(D + \frac{1}{4}S + E) M_{\text{max}} = 152.034 \text{ k} \cdot \text{ft} < M_{\text{max}}^{\text{allow}}$$

* Since snow loading remains the same (D+C+S) & the max. moment from the new, additional loading (wind & seismic) is less than the original D+C+S loading, the member @ the haunch is OK



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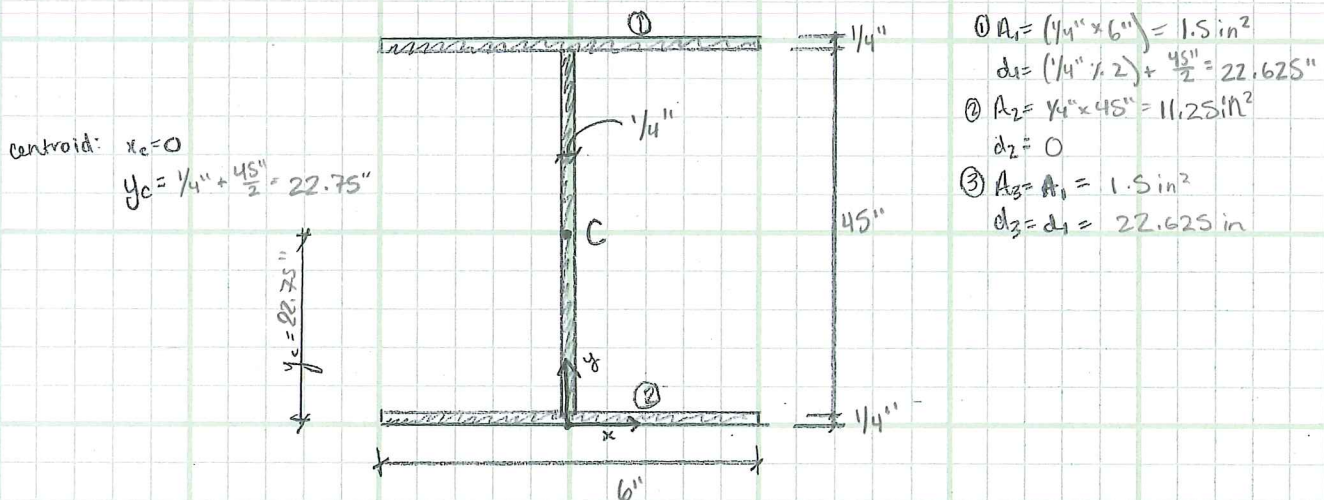
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Beams(s) - @ L/R ends (end frame)



$$I_1 = I_3 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{6(1/4)^3}{12} + 1.5 \text{ in}^2 (22.625 \text{ in})^2 = 0.0078125 \text{ in}^4 + 767.836 \text{ in}^4$$

$$\therefore I_1 = I_3 = 787.844 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{1/4(45)^3}{12} = 1898.4375 \text{ in}^4$$

$$\Sigma I = 2(I_1) + I_2 = 2(787.844 \text{ in}^4) + 1898.4375 \text{ in}^4 = 3474.125 \text{ in}^4 = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{3474.125 \text{ in}^4}{22.75 \text{ in}} = 152.709 \text{ in}^3 = S_x \Rightarrow M_{\text{max}} = S_x F_y = 152.709 \text{ in}^3 (36 \text{ ksi}) \times 12/14$$

$$= 458.126 \text{ k}\cdot\text{ft} \times 1.67 \Omega$$

$$= \boxed{274.327 \text{ k}\cdot\text{ft}}$$

$$(D+1/45+E)$$

$$M_{\text{max}} = 152.034 \text{ k}\cdot\text{ft} < M_{\text{max}}^{\text{allow}}$$

* since the original frame was designed for $M = 442.9$ (D+crs) & since snow loading remains the same & the updated wind & seismic loading produced $M_{\text{max}} = 152.034 \text{ k}\cdot\text{ft}$; members @ haunch(s) are OK



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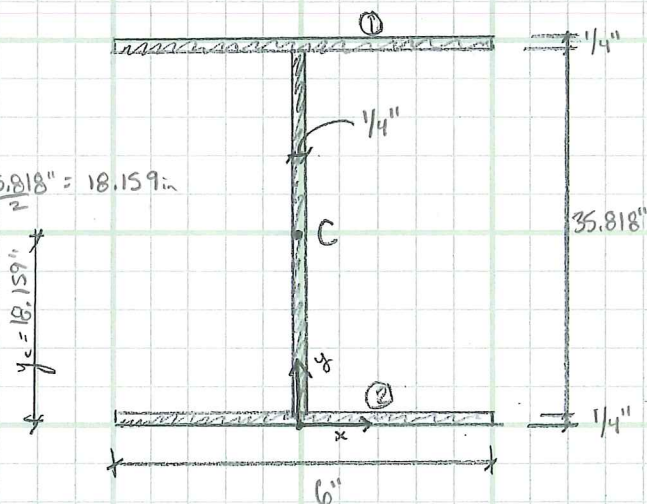
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Beams) - @ N3/N5 (end frame)

centroid: $x_c = 0$

$$y_c = \frac{1}{4} + \frac{35.818}{2} = 18.159 \text{ in}$$



$$① A_1 = 1.5 \text{ in}^2$$

$$d_1 = \left(\frac{1}{4} \times 1.2 \right) + \frac{35.818}{2} = 18.034 \text{ in}$$

$$② A_2 = \frac{1}{4} \times 35.818 = 8.9545 \text{ in}^2$$

$$d_2 = 0$$

$$③ A_3 = A_1 = 1.5 \text{ in}^2$$

$$d_3 = d_1 = 18.034 \text{ in}$$

$$I_1 = I_3 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{(6)(\frac{1}{4})^3}{12} + 1.5 \text{ in}^2 (18.034 \text{ in})^2 = 0.0078125 \text{ in}^4 + 487.838 \text{ in}^4$$

$$\therefore I_1 = I_3 = 487.846 \text{ in}^4$$

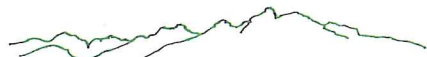
$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{\frac{1}{4} (35.818 \text{ in})^3}{12} = 957.332 \text{ in}^4$$

$$\Sigma I = 2(I_1) + I_2 = 2(487.846 \text{ in}^4) + 957.332 \text{ in}^4 = 1933.024 \text{ in}^4 = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{1933.024 \text{ in}^4}{18.159 \text{ in}} = 106.45 \text{ in}^3 = S_x \Rightarrow M_{\text{max}} = S_x F_y = 106.45 \text{ in}^3 (36 \text{ ksi}) \times \frac{1}{12} \text{ ft} = 319.35 \text{ k} \cdot \text{ft} \times 1.67 \text{ ft} = 191.227 \text{ k} \cdot \text{ft}$$

$$M_{\text{max}} = 25.27 \text{ k} \cdot \text{ft} < M_{\text{max}}^{\text{allow}} \quad (\text{D} + \frac{1}{4} \text{S} + \text{E})$$

*since snow loading remains the same (D+C+S) & the max. moment from the new, additional loading (wind & seismic) is less than the original D+C+S; members @ N3/N5 are OK



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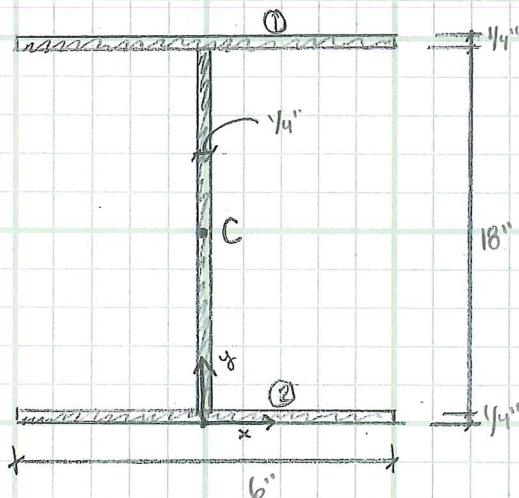
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Beam(s) - @ ridge (end frame)

centroid: $x_c = 0$

$$y_c = \frac{1}{4} + \frac{18}{2} = 9.25"$$

$$y_c = 9.25"$$



$$① A_1 = 1.5 \text{ in}^2$$

$$d_1 = \left(\frac{1}{4} \times \frac{1}{2}\right) + \frac{18}{2} = 9.125 \text{ in}$$

$$② A_2 = \frac{1}{4} \times 18 = 4.5 \text{ in}^2$$

$$d_2 = 0$$

$$③ A_3 = A_1 = 1.5 \text{ in}^2$$

$$d_3 = d_1 = 9.125 \text{ in}$$

$$I_1 = I_3 = \frac{bh^3}{12} + A_1 d_1^2 = \frac{6 \left(\frac{1}{4}\right)^3}{12} + 1.5 \text{ in}^2 (9.125 \text{ in})^2 = 0.0078125 \text{ in}^4 + 124.898 \text{ in}^4$$

$$\therefore I_1 = I_3 = 124.90625 \text{ in}^4$$

$$I_2 = \frac{bh^3}{12} + A_2 d_2^2 = \frac{1/4 (18)^3}{12} = 121.5 \text{ in}^4$$

$$I_x = 2(I_1) + I_2 = 2(124.90625 \text{ in}^4) + 121.5 \text{ in}^4 = 371.3125 \text{ in}^4 = I_x$$

$$S_x = \frac{I_x}{y_c} = \frac{371.3125 \text{ in}^4}{9.25 \text{ in}} = 40.142 \text{ in}^3 = S_x \Rightarrow M_{\text{max}} = S_x \times F_y = 40.142 \text{ in}^3 (36 \text{ ksi}) \times 12 = 120.426 \text{ k} \cdot \text{ft} \times 1.67 \text{ ft} = 32.111 \text{ k} \cdot \text{ft}$$

$$M_{\text{max}} = \frac{(D + 1/4 S + E)}{2} = 29.091 \text{ k} \cdot \text{ft} < M_{\text{max}}^{\text{allow}}$$

$\therefore$ member @ ridge is OK

Basic Wind Speed
 120mph = V Risk Category III & IV Figure 26.5-1c

$K_d = 0.85$ Table 26.6-1

$K_{zt} = 1.0$ Figure 26.8-1 (Eqn 26.8-1)

Gust Effect Factor = 0.85 (26.9.1)

$G_{Cpi} = +0.18$ Enclosed Buildings Table 26.11-1
 -0.18

Chapter 30: Wind Loads - Components & cladding

(Eqn 30.3-1)

$$q_z = 0.00256 K_z K_{zt} K_d V^2 = 0.00256 \times 0.92 \times 1.0 \times 0.85 \times 120^2 = 28.83 \text{ psf}$$

$K_z = \sim 0.92$ (Table 30.3-1)
 $h \sim 23'$ @ Ridge
 Exposure C

Low Rise Building (30.4)

p = Pressure on Building

$$P = q_h (G_{Cp} - G_{Cpi}) \text{ lb/ft}^2$$

$$q_h = q_z$$

25' x 12' = 300ft² — Say 100ft² Area: $G_{Cp} = \begin{matrix} \text{Zone 4} \\ +0.8 \\ -0.9 \end{matrix} \quad \begin{matrix} \text{Zone 5} \\ +0.8 \\ -1.1 \end{matrix}$

Table 30.7-2:

20' = h ; Gable Roof: $LC 1 - \text{Zone 4} = \frac{120\text{mph}}{-36.2 \times 0.8} = -28.96$ Zone 5
 $LC 2 - \text{Zone 4} = \frac{33.3 \times 0.8}{100\text{ft}^2} = 26.64$ $-56 \times 0.8 = -44.8$
 $30.5 \times 0.8 = 24.4$

$$P = 28.83 \text{ psf} (+0.8 - (-0.18)) = 28.253 \text{ psf}$$

$$28.83 \text{ psf} (-1.1 - 0.18) = 36.90 \text{ psf}$$

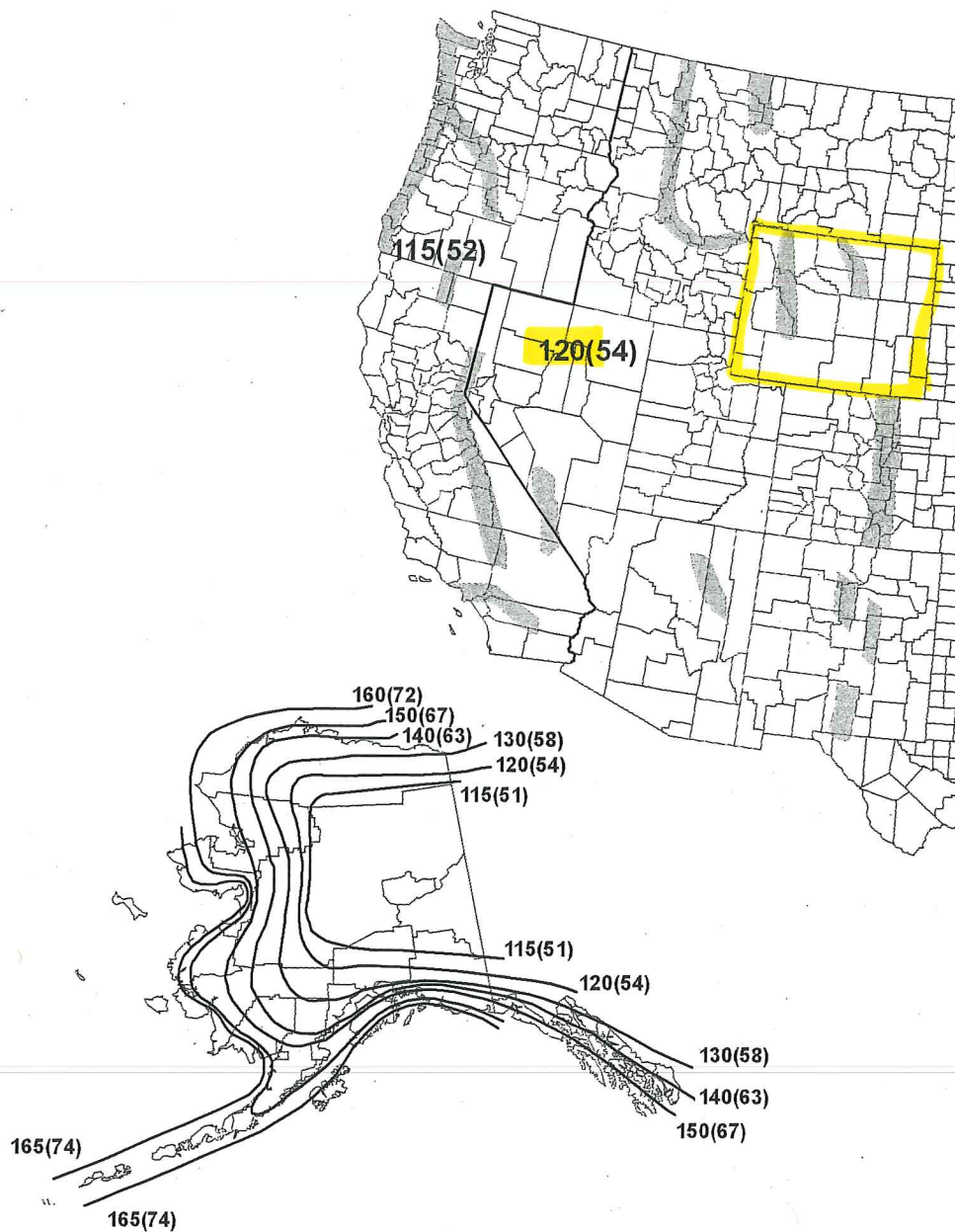


FIGURE 26.5-1B Basic Wind Speeds for Risk Category III and IV Buildings and Other Structures

Notes:

1. Values are design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1700 years).

Compression & Bending

$$F_{CEX} = 498$$

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{bx}}{F'_{bx}\left(1 - \frac{f_c}{F_{CEX}}\right)}$$

$$f_c \sim 100 \text{ lb} / (1.5 \times 7.25) = 9.2$$

$$\Rightarrow 10 \text{ psi}$$

$$\frac{10 \text{ psi}}{498} = 0.02$$

$$1 - 0.02 = 0.98$$

Since $\frac{f_c}{F'_c} \sim 0.02$

Use bending only.

$$36.2 \text{ psf} \times 2' = 72.4 \text{ plf}$$

$$M = \frac{wl^2}{8} = \frac{72.4 \text{ plf} \times (12')^2}{8} = 1303.2 \text{ lb-ft} \times 12 = 15,638.4 \text{ lb-in}$$

$$S_x = 13.14 \text{ in}^3$$

$$f_b = \frac{15,638.4 \text{ lb-in}}{13.14 \text{ in}^3} = 1190.14 \text{ psi}$$

$$F_b = 1500 \text{ psi} \times C_D \times F_b = 1500 \text{ psi} \times 1.6 \times 1.2 = 2880 \text{ psi}$$

$$\frac{f_b}{F_b} = \frac{1190.1 \text{ psi}}{2880 \text{ psi}} = 0.4132$$

Enercalc S.R. = 0.386 w/ Bracing in Weak Axis @ 4'-0" o.c.

$$\Delta = 0.4216" \text{ (Per Enercalc)}$$

$$12' = 144" / 0.4216" = 341.6 \text{ -ok-}$$

$\Delta = T_L/240$: Brittle
 $T_L/120$: Non Brittle

$$f_c = \frac{P}{A} \leq F'_c$$

$$F'_c = F_c C_D C_M C_t C_F C_i C_p$$

$$C_p = \frac{1 + \frac{F_{CE}}{F_c^*}}{2c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F_c^*}}{2c} \right)^2 - \frac{F_{CE}}{F_c^* c}}$$

$$F_{CE} = \frac{K_{CE} E'}{\left(\frac{l_e}{d} \right)^2}$$

$$\frac{l_e}{d} \max$$

$$l_{e \text{ minor}} = \frac{48''}{1.5''} = 32$$

$$l_{e \text{ major}} = \frac{(12' \times 12' / 16)}{7.25''} = 19.86$$

$$F_{CE} = \frac{0.3 \times 1,100,000}{(32)^2} = 498$$

$$F_c^* = F_c C_D C_M C_t^{1.05} C_F C_i = 1500 \text{ psi} \times 1.6 \times 1.0 \times 1.0 \times \sim 2400 \text{ psi}$$

$$\frac{F_{CE}}{F_c^*} = \frac{498}{2400} = 0.2075$$

$$C = 0.8$$

$$C_p = \left(\frac{1 + 0.2075}{2 \times 0.8} \right) - \sqrt{\left(\frac{1 + 0.2075}{2 \times 0.8} \right)^2 - \frac{0.2075}{0.8}} =$$

$$C_p = 0.7546 - 0.5569 = 0.198$$

$$F'_c = F_c^* \times C_p = 2400 \text{ psi} \times 0.198 = 475 \text{ psi}$$

Since Deflection not calculated - See "Beam Design"

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 7:07AM

Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 12'-0" tall 2x8 wood studs at 24" spacing with 36.2 psf

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		2x8	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber	
Overall Column Height		12.0 ft		Wood Member Type		Sawn	
(Used for non-slender calculations)							
Wood Species		Douglas Fir - Larch		Exact Width		1.50 in	
Wood Grade		No.1		Exact Depth		7.250 in	
Fb +		1,000.0 psi	Fv	180.0 psi	Area		10.875 in^2
Fb -		1,000.0 psi	Ft	675.0 psi	Ix		47.635 in^4
Fc - Prll		1,500.0 psi	Density	31.20 pcf	Iy		2.039 in^4
Fc - Perp		625.0 psi					
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			
Basic		1,700.0	1,700.0	1,700.0 ksi			
Minimum		620.0	620.0				
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis : Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, K= 1.0			
				Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 12 ft, K= 1.0			

Applied Loads

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

Column self weight included: 28.275 lbs * Dead Load Factor

BENDING LOADS...

Lat. Uniform Load creating Mx-x, W = 0.07240 k/ft ✓

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.3862 : 1	Maximum SERVICE Lateral Load Reactions . .			
Load Combination	+D+0.60W	Top along Y-Y	0.4344 k	Bottom along Y-Y	0.4344 k
Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X	0.0 k
Location of max.above base	5.960 ft	Maximum SERVICE Load Lateral Deflections . .			
At maximum location values are . . .		Along Y-Y	0.4216 in	at	6.040 ft above base
Applied Axial	0.02828 k	for load combination: W Only			
Applied Mx	0.7819 k-ft	Along X-X	0.0 in	at	0.0 ft above base
Applied My	0.0 k-ft	for load combination: n/a			
Fc : Allowable	475.570 psi	Other Factors used to calculate allowable stresses . .			
PASS Maximum Shear Stress Ratio =	0.1248 : 1				
Load Combination	+D+0.60W				
Location of max.above base	12.0 ft				
Applied Design Shear	35.950 psi				
Allowable Shear	288.0 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.321	0.005718	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+0.60W	1.600	0.189	0.3862	PASS	5.960 ft	0.1248	PASS	12.0 ft
+D+0.450W	1.600	0.189	0.2897	PASS	5.960 ft	0.09362	PASS	12.0 ft
+0.60D+0.60W	1.600	0.189	0.3859	PASS	5.960 ft	0.1248	PASS	12.0 ft
+0.60D	1.600	0.189	0.003280	PASS	0.0 ft	0.0	PASS	12.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top	@ Base	@ Top
D Only						0.028				
+D+0.60W				0.261	0.261	0.028				

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 12'-0" tall 2x8 wood studs at 24" spacing with 36.2 psf

Maximum Reactions

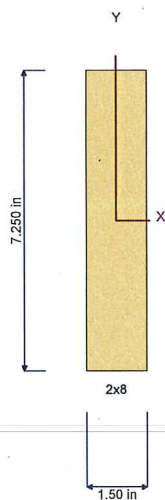
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+D+0.450W				0.195	0.195	0.028					
+0.60D+0.60W				0.261	0.261	0.017					
+0.60D						0.017					
W Only				0.434	0.434						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.253 in	6.040 ft
+D+0.450W	0.0000 in	0.000 ft	0.190 in	6.040 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.253 in	6.040 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.422 in	6.040 ft

Sketches



Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 9:41AM

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 2X8 WOOD STUD

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set: ASCE 7-10

Material Properties

Analysis Method: Allowable Stress Design
Load Combination: ASCE 7-10

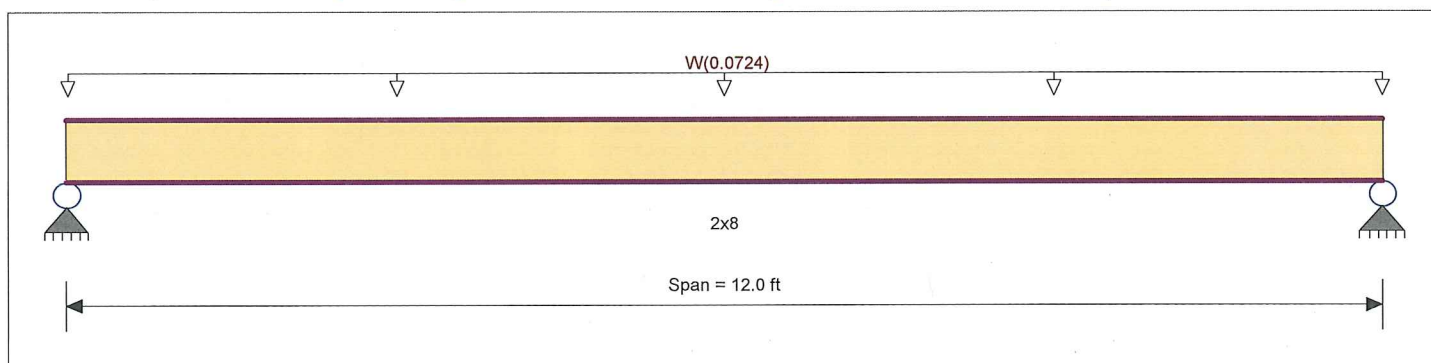
Wood Species: Douglas Fir - Larch
Wood Grade: Stud

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

F_b + 700.0 psi
F_b - 700.0 psi
F_c - Prll 850.0 psi
F_c - Perp 625.0 psi
F_v 180.0 psi
F_t 450.0 psi

E: Modulus of Elasticity
E_{bend} - xx 1,400.0 ksi
E_{minbend} - xx 510.0 ksi

Density 31.20 pcf
Repetitive Member Stress Increase



Applied Loads

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

Uniform Load: W = 0.03620 ksf, Tributary Width = 2.0 ft, (WINDWARD LOADING)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	0.462	1	Maximum Shear Stress Ratio	=	0.113	1
Section used for this span		2x8		Section used for this span		2x8	
fb: Actual	=	714.05	psi	fv: Actual	=	32.54	psi
FB: Allowable	=	1,545.60	psi	Fv: Allowable	=	288.00	psi
Load Combination		+D+0.60W+H		Load Combination		+D+0.60W+H	
Location of maximum on span	=	6.000	ft	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		0.509	in	Ratio =		282	<360
Max Upward Transient Deflection		0.000	in	Ratio =		0	<360
Max Downward Total Deflection		0.306	in	Ratio =		471	>=180
Max Upward Total Deflection		0.000	in	Ratio =		0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
+D+H	Length = 12.0 ft	1			0.90	1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
+D+L+H	Length = 12.0 ft	1			1.00	1.200	1.00	1.15	1.00	1.00	1.00			869.40	0.00	0.00	162.00
+D+Lr+H	Length = 12.0 ft	1			1.00	1.200	1.00	1.15	1.00	1.00	1.00			966.00	0.00	0.00	180.00
+D+S+H	Length = 12.0 ft	1			1.25	1.200	1.00	1.15	1.00	1.00	1.00			1207.50	0.00	0.00	225.00
+D+0.750Lr+0.750L+H	Length = 12.0 ft	1			1.15	1.200	1.00	1.15	1.00	1.00	1.00			1110.90	0.00	0.00	207.00
+D+0.750Lr+0.750S+H	Length = 12.0 ft	1			1.25	1.200	1.00	1.15	1.00	1.00	1.00			1207.50	0.00	0.00	225.00
+D+0.750L+0.750S+H	Length = 12.0 ft	1			1.15	1.200	1.00	1.15	1.00	1.00	1.00			1110.90	0.00	0.00	207.00

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 2X8 WOOD STUD

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
+D+0.60W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1		0.462	0.113	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.78	714.05	1545.60	0.24	32.54	288.00
+D+0.70E+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1				1.60	1.200	1.00	1.15	1.00	1.00	1.00			1545.60	0.00	0.00	288.00
+D+0.750Lr+0.750L+0.450W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1		0.346	0.085	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.59	535.54	1545.60	0.18	24.40	288.00
+D+0.750L+0.750S+0.450W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1		0.346	0.085	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.59	535.54	1545.60	0.18	24.40	288.00
+D+0.750L+0.750S+0.5250E+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1				1.60	1.200	1.00	1.15	1.00	1.00	1.00			1545.60	0.00	0.00	288.00
+0.60D+0.60W+0.60H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1		0.462	0.113	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.78	714.05	1545.60	0.24	32.54	288.00
+0.60D+0.70E+0.60H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1				1.60	1.200	1.00	1.15	1.00	1.00	1.00			1545.60	0.00	0.00	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.5095	6.044		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

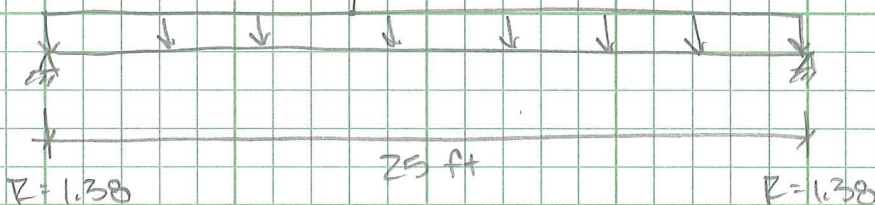
Load Combination	Support 1	Support 2
Overall MAXimum	0.434	0.434
Overall MINimum	0.195	0.195
+D+H		
+D+L+H		
+D+Lr+H		
+D+S+H		
+D+0.750Lr+0.750L+H		
+D+0.750L+0.750S+H		
+D+0.60W+H	0.261	0.261
+D+0.70E+H		
+D+0.750Lr+0.750L+0.450W+H	0.195	0.195
+D+0.750L+0.750S+0.450W+H	0.195	0.195
+D+0.750L+0.750S+0.5250E+H		
+0.60D+0.60W+0.60H	0.261	0.261
+0.60D+0.70E+0.60H		
D Only		
Lr Only		
L Only		
S Only		
W Only	0.434	0.434
E Only		
H Only		

Header @ Top of Moment Frame

$W = 36.2 \text{ psf}$ Comp & Cladding

For Areas $> 100 \text{ ft}^2 \Rightarrow 28.96$, USE 30 psf

$$w = 30 \text{ psf} \times \frac{12'}{2} = 180 \text{ plf}$$



C6x13 : SR: $M = 0.394$ $\Delta_L = 1.936" = L/154$ NG
 $V = 0.024$ $\Delta_T = 1.162" = L/258$

MC6x18 : SR $M = 0.245$ $\Delta_L = 1.124" = L/266$ NG
 $V = 0.028$ $\Delta_T = 0.677" = L/443$

$d = 8"$
 $b_f = 2.527"$
 $t_f = 0.390"$

C8x18.7 ✓ SR $M = 0.338$ $\Delta_L = 0.762" = L/393$ OK
 $V = 0.027$ $\Delta_T = 0.458" = L/655$

MC7x19.1 ✓ SR $M = 0.324$ $\Delta_L = 1.271" = L/235$ -ok
 $V = 0.042$

$d = 7"$
 $b_f = 3.452"$
 $t_f = 0.5"$

MC8x18.7 SR $M = 0.184$ $\Delta_L = 0.638" = L/469$
 $V = 0.023$ $\Delta_T = 0.383" = L/782$

$d = 7"$
 $b_f = 2\frac{1}{4}"$
 $t_f = 3/8"$
 $t_w = 0.306"$
 $t_w = 0.419"$

C7x14.75 : S.R: $M = 0.482$ $\Delta_{TL} = 2.014"$
 $V = 0.036$ $= 7L/148$
-OK PER ARCH
FOR NON BRITTLE

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Steel Beam

Printed: 29 AUG 2017, 7:11AM
File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29
Licensee : nelson engineering

Lic. # : KW-06004539

Description : HEADER @ TOP OF MOMENT FRAME

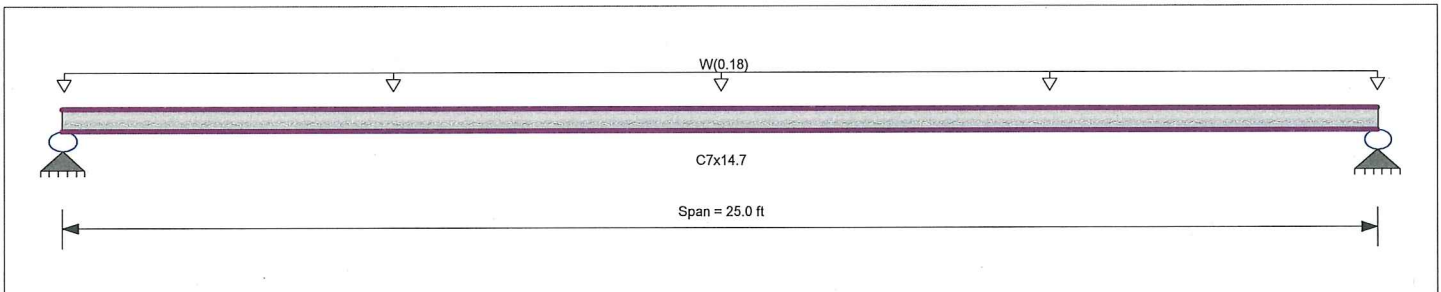
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

Fy : Steel Yield : 36.0 ksi
E : Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : W = 0.030 ksf, Tributary Width = 6.0 ft, (WIND)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio =	0.482 : 1	Maximum Shear Stress Ratio =	0.036 : 1
Section used for this span	C7x14.7	Section used for this span	C7x14.7
Ma : Applied	8.438 k-ft	Va : Applied	1.350 k
Mn / Omega : Allowable	17.515 k-ft	Vn/Omega : Allowable	37.936 k
Load Combination	+D+0.60W+H	Load Combination	+D+0.60W+H
Location of maximum on span	12.500ft	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	2.014 in	Ratio =	148 < 240.0
Max Upward Transient Deflection	0.000 in	Ratio =	0 < 240.0
Max Downward Total Deflection	1.209 in	Ratio =	248 >= 180
Max Upward Total Deflection	0.000 in	Ratio =	0 < 180

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	2.0148	12.571		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.250	2.250
Overall MINimum	1.013	1.013
+D+H		
+D+L+H		
+D+Lr+H		
+D+S+H		
+D+0.750Lr+0.750L+H		
+D+0.750L+0.750S+H		
+D+0.60W+H	1.350	1.350
+D+0.70E+H		
+D+0.750Lr+0.750L+0.450W+H	1.013	1.013
+D+0.750L+0.750S+0.450W+H	1.013	1.013
+D+0.750L+0.750S+0.5250E+H		
+0.60D+0.60W+0.60H	1.350	1.350
+0.60D+0.70E+0.60H		
D Only		
Lr Only		
L Only		
S Only		
W Only	2.250	2.250
E Only		

OK
> TL/120
For Non Brittle

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 7:11AM

Steel Beam

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : HEADER @ TOP OF MOMENT FRAME

Vertical Reactions			Support notation : Far left is #1	Values in KIPS
Load Combination	Support 1	Support 2		
H Only				

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 30 AUG 2017, 6:17PM

Steel Beam

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK~A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: HEADER @ TOP OF MOMENT FRAME

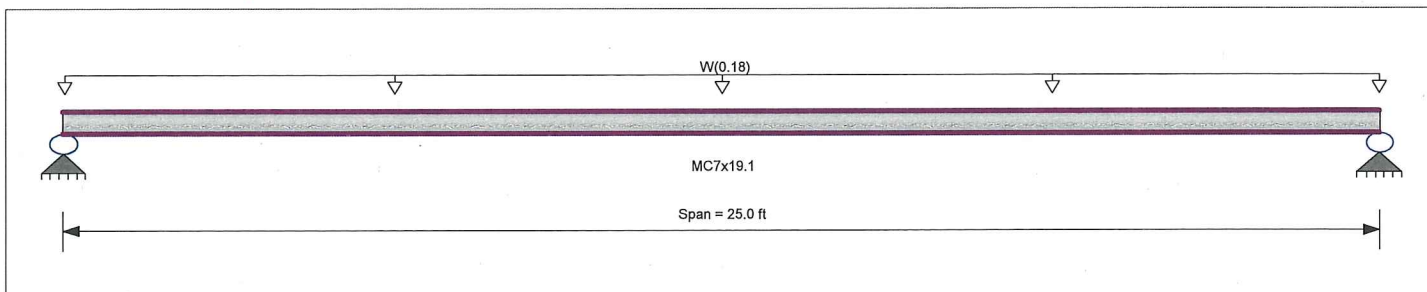
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set: ASCE 7-10

Material Properties

Analysis Method: Allowable Strength Design
Beam Bracing: Beam is Fully Braced against lateral-torsional buckling
Bending Axis: Major Axis Bending

Fy: Steel Yield: 36.0 ksi
E: Modulus: 29,000.0 ksi



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load: $W = 0.030$ ksf, Tributary Width = 6.0 ft, (WIND)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio =	0.324 : 1	Maximum Shear Stress Ratio =	0.042 : 1
Section used for this span	MC7x19.1	Section used for this span	MC7x19.1
Ma: Applied	8.438 k-ft	Va: Applied	1.350 k
Mn / Omega: Allowable	26.048 k-ft	Vn / Omega: Allowable	31.870 k
Load Combination	+D+0.60W+H	Load Combination	+D+0.60W+H
Location of maximum on span	12.500 ft	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	1.271 in	Ratio =	235 < 240.0
Max Upward Transient Deflection	0.000 in	Ratio =	0 < 240.0
Max Downward Total Deflection	0.763 in	Ratio =	393 >= 180
Max Upward Total Deflection	0.000 in	Ratio =	0 < 180

2 ok
for Non Brittle
Per Arch.

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	1.2715	12.571		0.0000	0.000

Vertical Reactions

Support notation: Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.250	2.250
Overall MINimum	1.013	1.013
+D+H		
+D+L+H		
+D+Lr+H		
+D+S+H		
+D+0.750Lr+0.750L+H		
+D+0.750L+0.750S+H		
+D+0.60W+H	1.350	1.350
+D+0.70E+H		
+D+0.750Lr+0.750L+0.450W+H	1.013	1.013
+D+0.750L+0.750S+0.450W+H	1.013	1.013
+D+0.750L+0.750S+0.5250E+H		
+0.60D+0.60W+0.60H	1.350	1.350
+0.60D+0.70E+0.60H		
D Only		
Lr Only		
L Only		
S Only		
W Only	2.250	2.250
E Only		

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 30 AUG 2017, 6:17PM

Steel Beam

File = S:\Proj2017\2KZYV1~\SU53VC-6\DLLGK~A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : HEADER @ TOP OF MOMENT FRAME

Vertical Reactions			Support notation : Far left is #1	Values in KIPS
Load Combination	Support 1	Support 2		
H Only				

Title Block Line 6

Printed: 25 AUG 2017, 12:27PM

Steel Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: HEADER @ TOP OF MOMENT FRAME

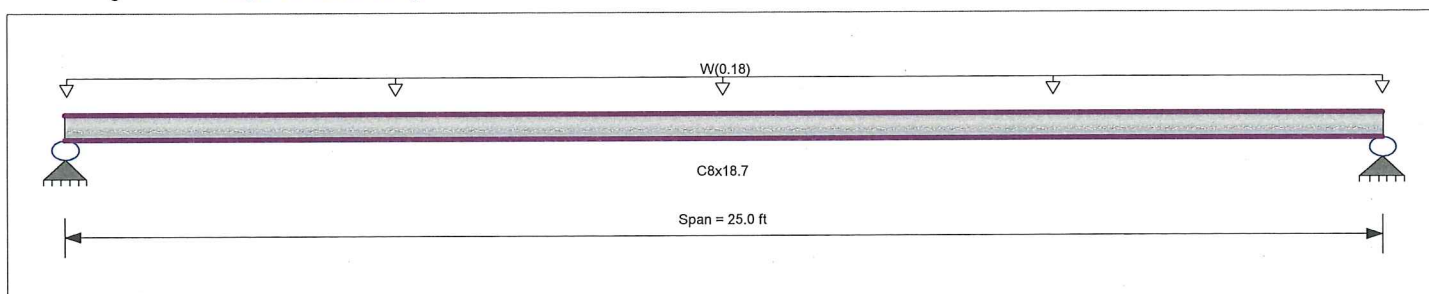
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set: ASCE 7-10

Material Properties

Analysis Method: Allowable Strength Design
Beam Bracing: Beam is Fully Braced against lateral-torsional buckling
Bending Axis: Major Axis Bending

Fy : Steel Yield : 36.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load: W = 0.030 ksf, Tributary Width = 6.0 ft, (WIND)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.338 : 1	Maximum Shear Stress Ratio =	0.027 : 1
Section used for this span	C8x18.7	Section used for this span	C8x18.7
Ma : Applied	8.438 k-ft	Va : Applied	1.350 k
Mn / Omega : Allowable	24.970 k-ft	Vn/Omega : Allowable	50.391 k
Load Combination	+D+0.60W+H	Load Combination	+D+0.60W+H
Location of maximum on span	12.500 ft	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	1.245 in	Ratio =	240 >= 240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 < 240.0
Max Downward Total Deflection	0.749 in	Ratio =	401 >= 180
Max Upward Total Deflection	0.000 in	Ratio =	0 < 180

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	1.2483	12.571		0.0000	0.000

Vertical Reactions

Support notation: Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.250	2.250
Overall MINimum	1.013	1.013
+D+H		
+D+L+H		
+D+Lr+H		
+D+S+H		
+D+0.750Lr+0.750L+H		
+D+0.750L+0.750S+H		
+D+0.60W+H	1.350	1.350
+D+0.70E+H		
+D+0.750Lr+0.750L+0.450W+H	1.013	1.013
+D+0.750L+0.750S+0.450W+H	1.013	1.013
+D+0.750L+0.750S+0.5250E+H		
+0.60D+0.60W+0.60H	1.350	1.350
+0.60D+0.70E+0.60H		
D Only		
Lr Only		
L Only		
S Only		
W Only	2.250	2.250
E Only		

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:
Project ID: 17-202-01

Title Block Line 6

Steel Beam

Lic. # : KW-06004539

Description : HEADER @ TOP OF MOMENT FRAME

Printed: 25 AUG 2017, 12:27PM
File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29
Licensee : nelson engineering

Vertical Reactions			Support notation : Far left is #1	Values in KIPS
Load Combination	Support 1	Support 2		
H Only				

KWIK Bolt TZ Expansion Anchor 3.3.5

3.3.5.1 Product description

The KWIK Bolt TZ (KB-TZ) is a torque controlled expansion anchor which is especially suited to seismic and cracked concrete applications. This anchor line is available in carbon steel, type 304 and type 316 stainless steel versions. The anchor diameters range from 3/8-, 1/2-, 5/8- and 3/4-inch in a variety of lengths. Applicable base materials include normal-weight concrete, structural lightweight concrete, lightweight concrete over metal deck and grout-filled concrete masonry.

Guide specifications

Torque controlled expansion anchors shall be KWIK Bolt TZ (KB-TZ) supplied by Hilti meeting the description in Federal Specification A-A 1923A, type 4. The anchor bears a length identification mark embossed into the impact section (dog point) of the anchor surrounded by four embossed notches identifying the anchor as a Hilti KWIK Bolt TZ. Anchors are manufactured to meet one of the following conditions:

- The carbon steel anchor body, nut, and washer have an electroplated zinc coating conforming to ASTM B633 to a minimum thickness of 5 µm. The stainless steel expansion sleeve conforms to type 316.
- Stainless steel anchor body, nut and washer conform to type 304. Stainless steel expansion sleeve conforms to type 316.
- Stainless steel anchor body, nut, washer, and expansion sleeve conform to type 316 stainless steel.

Product features

- Product and length identification marks facilitate quality control after installation.
- Through fixture installation and variable thread lengths improve productivity and accommodate various base plate thicknesses.
- Type 316 stainless steel wedges provide superior performance in cracked concrete.
- Ridges on expansion wedges provide increased reliability.
- Mechanical expansion allows immediate load application.
- Raised impact section (dog point) prevents thread damage during installation.
- Bolt meets ductility requirements of ACI 318-14 Section 2.3.
- ACI 349-01 Nuclear Design Guide is available. Call Hilti Technical Support.

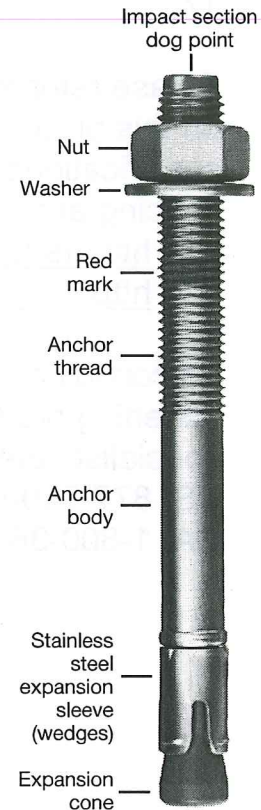
3.3.5.1 Product description

3.3.5.2 Material specifications

3.3.5.3 Technical data

3.3.5.4 Installation instructions

3.3.5.5 Ordering information



3.3.5

Listings/Approvals

ICC-ES (International Code Council)
ESR-1917, ESR-3785, ESL-1067

City of Los Angeles
Research Report No. 25701
Research Report No. 26057

FM (Factory Mutual)
Pipe Hanger Components for Automatic
Sprinkler Systems for 3/8 through 3/4

UL LLC
Pipe Hanger Equipment for Fire
Protection Services for 3/8 through 3/4



Independent code evaluation

IBC® / IRC® 2015

IBC® / IRC® 2012

IBC® / IRC® 2009

IBC® / IRC® 2006

**National Building Code
of Canada 2015**

**National Building Code
of Canada 2010**

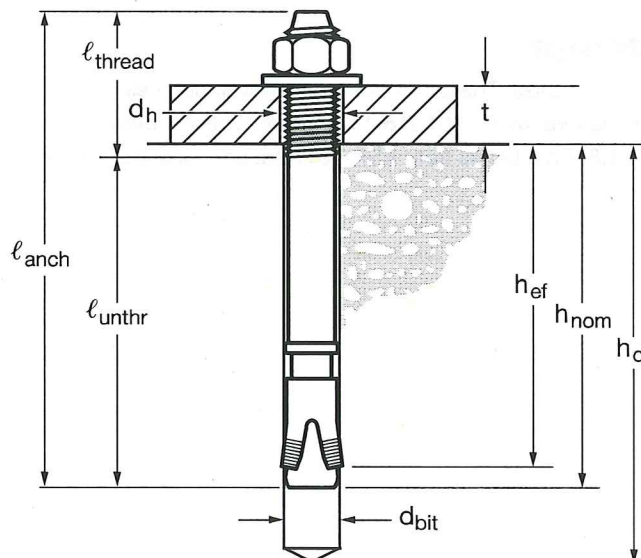
KWIK Bolt TZ Expansion Anchor 3.3.5

Table 1 - Hilti KWIK Bolt TZ carbon steel specifications

Setting information	Symbol	Units	Nominal anchor diameter d _a														
			3/8			1/2				5/8				3/4			
Nominal bit diameter	d _{bit}	in.	3/8			1/2				5/8				3/4			
Minimum nominal embedment	h _{nom}	in. (mm)	1-13/16 (46)	2-5/16 (59)	3-1/16 (78)	2-3/8 (60)	3-5/8 (91)		3-9/16 (91)	4-7/16 (113)		3-13/16 (97)	4-5/16 (110)		5-5/16 (135)		
Effective minimum embedment	h _{ef}	in. (mm)	1-1/2 (38)	2 (51)	2-3/4 (70)	2 (51)	3-1/4 (83)		3-1/8 (79)	4 (102)		3-1/4 (83)	3-3/4 (95)		4-3/4 (121)		
Min. hole depth	h _o	in. (mm)	2 (51)	2-5/8 (67)	3-3/8 (83)	2-5/8 (67)	4 (102)		3-3/4 (95)	4-3/4 (121)		4 (102)	4-5/8 (117)		5-3/4 (146)		
Min. thickness of fixture ¹	t _{min}	in. (mm)	0 (0)	0 (0)	0 (0)	3/4 (19)	1/4 (6)		3/8 (9)	3/4 (19)		0 (0)	0 (0)		7/8 (23)		
Max. thickness of fixture	t _{max}	in. (mm)	2-13/16 (71)	2-5/16 (59)	1-9/16 (40)	4 (101)	2-3/4 (70)		5-5/8 (143)	4-3/4 (121)		5-9/16 (141)	4-15/16 (125)		3-15/16 (100)		
Installation torque (concrete)	T _{inst}	ft-lb (Nm)	25 (34)			40 (54)			60 (81)				110 (149)				
Installation torque (masonry)	T _{inst}	ft-lb (Nm)	n/a	15 (20)	n/a	25 (34)			35 (47)				n/a	70 (95)			
Fixture hole diameter	d _h	in. (mm)	7/16 (11.1)			9/16 (14.3)				11/16 (17.5)				13/16 (20.6)			
Available anchor lengths	ℓ _{anch}	in. (mm)	3 (76)	3-3/4 (95)	5 (127)	3-3/4 (95)	4-1/2 (114)	5-1/2 (140)	7 (178)	4-3/4 (121)	6 (152)	8-1/2 (216)	10 (254)	5-1/2 (140)	7 (178)	8 (203)	10 (254)
Threaded length including dog point	ℓ _{thread}	in. (mm)	1-1/2 (38)	2-1/4 (57)	3-1/2 (89)	1-5/8 (41)	2-3/8 (60)	3-3/8 (86)	4-7/8 (124)	1-1/2 (38)	2-3/4 (70)	5-1/4 (133)	6-3/4 (171)	2-1/2 (63)	4 (103)	5 (128)	7 (179)
Unthreaded length	ℓ _{unthr}	in. (mm)	1-1/2 (39)			2-1/8 (54)				3-1/4 (83)				3 (77)			

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¹ Minimum thickness of fixture is a concern only when the anchor is installed at the minimum nominal embedment. When KWIK Bolt TZ anchors are installed at this embedment, the anchor threading ends near the surface of the concrete. If the fixture is sufficiently thin, it could be possible to run the nut to the bottom of the threading during application of the installation torque. If fixtures are thin, it is recommended that embedment be increased accordingly.

Figure 1 - Hilti KWIK Bolt TZ specifications


KWIK Bolt TZ Expansion Anchor 3.3.5

Table 4 - Steel strength for Hilti KWIK Bolt TZ carbon steel anchors^{1,2}

Nominal anchor diameter in	Effective embedment in. (mm)			Tensile ϕN_{sa} ³ lb (kN)	Shear ϕV_{sa} ⁴ lb (kN)	Seismic shear $\phi V_{sa,eq}$ ⁵ lb (kN)
3/8	1-1/2 (38)			4,875 (21.7)	1,415 (6.3)	1,415 (6.3)
	2 (51)	2-3/4 (70)		4,875 (21.7)	2,335 (10.4)	1,465 (6.5)
1/2	2 (51)	3-1/4 (83)		8,030 (35.7)	3,570 (15.9)	3,570 (15.9)
5/8	3-1/8 (79)	4 (102)		12,880 (57.3)	5,260 (23.4)	4,940 (22.0)
3/4	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)	18,840 (83.8)	8,890 (39.5)	7,635 (34.0)

1 See section 3.1.8.6 to convert design strength value to ASD value.

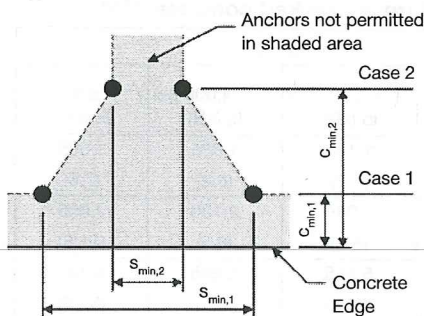
2 Hilti KWIK Bolt TZ carbon steel anchors are to be considered ductile steel elements.

3 Tensile $\phi N_{sa} = \phi A_{se,N} f_{uta}$ as noted in ACI 318-14 Chapter 17.

4 Shear values determined by static shear tests with $\phi V_{sa} < \phi 0.60 A_{se,V} f_{uta}$ as noted in ACI 318-14 Chapter 17.

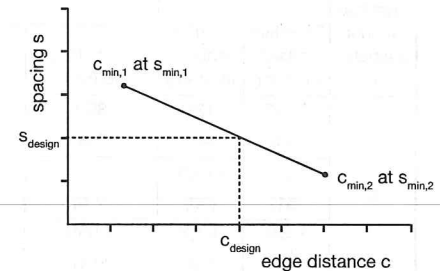
5 Seismic shear values determined by seismic shear tests with $\phi V_{sa,eq} < \phi 0.60 A_{se,V} f_{uta}$ as noted in ACI 318-14 Chapter 17. See section 3.1.8.7 for additional information on seismic applications.

3.3.5

Figure 2


For a specific edge distance, the permitted spacing is calculated as follows:

$$s \geq s_{min,2} + \frac{(s_{min,1} - s_{min,2})}{(c_{min,1} - c_{min,2})} (c - c_{min,2})$$


Table 5 - Carbon steel Hilti KWIK Bolt TZ installation parameters¹

Setting information	Symbol	Units	Nominal anchor diameter d _a											
			3/8			1/2			5/8			3/4		
Effective minimum embedment	h_{ef}	in. (mm)	1-1/2 (38)	2 (51)	2-3/4 (70)	2 (51)	3-1/4 (83)	3-1/8 (79)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)		
Min. member thickness	h_{min}	in. (mm)	3-1/4 (83)	4 (102)	5 (127)	4 (102)	6 (152)	6 (152)	8 (203)	5 (127)	6 (152)	8 (203)		
Case 1	$c_{min,1}$	in. (mm)	8 (203)	2-1/2 (64)	2-1/2 (64)	2-3/4 (70)	2-3/8 (60)	3-5/8 (92)	3-1/4 (83)	9-1/2 (241)	4-3/4 (121)	4-1/8 (105)		
	for $s_{min,1} \geq$	in. (mm)	8 (203)	5 (127)	5 (127)	5-3/4 (146)	5-3/4 (146)	6-1/8 (156)	5-7/8 (149)	5 (127)	10-1/2 (267)	8-7/8 (225)		
Case 2	$c_{min,2}$	in. (mm)	8 (203)	3-5/8 (92)	3-5/8 (92)	4-1/8 (105)	3-1/2 (89)	4-3/4 (121)	4-1/4 (108)	9-1/2 (241)	9-1/2 (241)	7-3/4 (197)		
	for $s_{min,2} \geq$	in. (mm)	8 (203)	2-1/2 (64)	2-1/2 (64)	2-3/4 (70)	2-3/8 (60)	3-1/2 (89)	3 (76)	5 (127)	5 (127)	4 (102)		

1 Linear interpolation is permitted to establish an edge distance and spacing combination between Case 1 and Case 2.

Linear interpolation for a specific edge distance c, where $c_{min,1} < c < c_{min,2}$, will determine the permissible spacings.

3.3.5 KWIK Bolt TZ Expansion Anchor

Table 2 - Hilti KWIK Bolt TZ carbon steel design strength with concrete / pullout failure in uncracked concrete^{1,2,3,4,5}

Nominal anchor diameter in.	Effective embed. in. (mm)	Nominal embed. in. (mm)	Tension - ϕN_n				Shear - ϕV_n			
			$f'_c = 2,500$ psi lb (kN)	$f'_c = 3,000$ psi lb (kN)	$f'_c = 4,000$ psi lb (kN)	$f'_c = 6,000$ psi lb (kN)	$f'_c = 2,500$ psi lb (kN)	$f'_c = 3,000$ psi lb (kN)	$f'_c = 4,000$ psi lb (kN)	$f'_c = 6,000$ psi lb (kN)
3/8	1-1/2 (38)	1-13/16 (46)	1,185 (5.3)	1,300 (5.8)	1,500 (6.7)	1,835 (8.2)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2-5/16 (59)	1,635 (7.3)	1,790 (8.0)	2,070 (9.2)	2,535 (11.3)	2,375 (10.6)	2,605 (11.6)	3,005 (13.4)	3,680 (16.4)
	2-3/4 (70)	3-1/16 (78)	2,670 (11.9)	2,925 (13.0)	3,380 (15.0)	4,140 (18.4)	7,660 (34.1)	8,395 (37.3)	9,690 (43.1)	11,870 (52.8)
1/2	2 (51)	2-3/8 (60)	2,205 (9.8)	2,415 (10.7)	2,790 (12.4)	3,420 (15.2)	2,375 (10.6)	2,605 (11.6)	3,005 (13.4)	3,680 (16.4)
	3-1/4 (83)	3-5/8 (91)	3,585 (15.9)	3,925 (17.5)	4,535 (20.2)	5,555 (24.7)	9,845 (43.8)	10,785 (48.0)	12,450 (55.4)	15,250 (67.8)
5/8	3-1/8 (79)	3-9/16 (91)	4,310 (19.2)	4,720 (21.0)	5,450 (24.2)	6,675 (29.7)	9,280 (41.3)	10,165 (45.2)	11,740 (52.2)	14,380 (64.0)
	4 (102)	4-7/16 (113)	5,945 (26.4)	6,510 (29.0)	7,520 (33.5)	9,210 (41.0)	13,440 (59.8)	14,725 (65.5)	17,000 (75.6)	20,820 (92.6)
3/4	3-1/4 (83)	3-13/16 (97)	4,570 (20.3)	5,005 (22.3)	5,780 (25.7)	7,080 (31.5)	9,845 (43.8)	10,785 (48.0)	12,450 (55.4)	15,250 (67.8)
	3-3/4 (95)	4-5/16 (110)	5,380 (23.9)	5,895 (26.2)	6,810 (30.3)	8,340 (37.1)	12,200 (54.3)	13,365 (59.5)	15,430 (68.6)	18,900 (84.1)
	4-3/4 (121)	5-9/16 (142)	6,940 (30.9)	7,605 (33.8)	8,780 (39.1)	10,755 (47.8)	17,390 (77.4)	19,050 (84.7)	22,000 (97.9)	26,945 (119.9)

Table 3 - Hilti KWIK Bolt TZ carbon steel design strength with concrete / pullout failure in cracked concrete^{1,2,3,4,5}

Nominal anchor diameter in.	Effective embed. in. (mm)	Nominal embed. in. (mm)	Tension - ϕN_n				Shear - ϕV_n			
			$f'_c = 2,500$ psi lb (kN)	$f'_c = 3,000$ psi lb (kN)	$f'_c = 4,000$ psi lb (kN)	$f'_c = 6,000$ psi lb (kN)	$f'_c = 2,500$ psi lb (kN)	$f'_c = 3,000$ psi lb (kN)	$f'_c = 4,000$ psi lb (kN)	$f'_c = 6,000$ psi lb (kN)
3/8	1-1/2 (38)	1-13/16 (46)	860 (3.8)	940 (4.2)	1,085 (4.8)	1,330 (5.9)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)	1,695 (7.5)
	2 (51)	2-3/8 (60)	1,565 (7.0)	1,710 (7.6)	1,975 (8.8)	2,420 (10.8)	1,685 (7.5)	1,845 (8.2)	2,130 (9.5)	2,605 (11.6)
	2-3/4 (70)	3-1/8 (79)	2,050 (9.1)	2,245 (10.0)	2,595 (11.5)	3,175 (14.1)	5,425 (24.1)	5,945 (26.4)	6,865 (30.5)	8,405 (37.4)
1/2	2 (51)	2-3/8 (60)	1,565 (7.0)	1,710 (7.6)	1,975 (8.8)	2,420 (10.8)	1,685 (7.5)	1,845 (8.2)	2,130 (9.5)	2,605 (11.6)
	3-1/4 (83)	3-5/8 (91)	3,195 (14.2)	3,500 (15.6)	4,040 (18.0)	4,950 (22.0)	6,970 (31.0)	7,640 (34.0)	8,820 (39.2)	10,800 (48.0)
5/8	3-1/8 (79)	3-9/16 (91)	3,050 (13.6)	3,345 (14.9)	3,860 (17.2)	4,730 (21.0)	6,575 (29.2)	7,200 (32.0)	8,315 (37.0)	10,185 (45.3)
	4 (102)	4-7/16 (113)	4,420 (19.7)	4,840 (21.5)	5,590 (24.9)	6,845 (30.4)	9,520 (42.3)	10,430 (46.4)	12,040 (53.6)	14,750 (65.6)
3/4	3-1/4 (83)	3-13/16 (97)	3,235 (14.4)	3,545 (15.8)	4,095 (18.2)	5,015 (22.3)	6,970 (31.0)	7,640 (34.0)	8,820 (39.2)	10,800 (48.0)
	3-3/4 (95)	4-5/16 (110)	4,010 (17.8)	4,395 (19.5)	5,075 (22.6)	6,215 (27.6)	8,640 (38.4)	9,465 (42.1)	10,930 (48.6)	13,390 (59.6)
	4-3/4 (121)	5-9/16 (142)	5,720 (25.4)	6,265 (27.9)	7,235 (32.2)	8,860 (39.4)	12,320 (54.8)	13,495 (60.0)	15,585 (69.3)	19,085 (84.9)

1 See section 3.1.8.6 to convert design strength value to ASD value.

2 Linear interpolation between embedment depths and concrete compressive strengths is not permitted.

3 Apply spacing, edge distance, and concrete thickness factors in tables 6 to 13 as necessary. Compare to the steel values in table 4. The lesser of the values is to be used for the design.

4 Tabular values are for normal weight concrete only. For lightweight concrete multiply design strength by λ_s as follows: for sand-lightweight, $\lambda_s = 0.68$; for all-lightweight, $\lambda_s = 0.60$.

5 Tabular values are for static loads only. Seismic design is not permitted for uncracked concrete. For seismic tension loads, multiply cracked concrete tabular values in tension only by $\alpha_{N,seis} = 0.75$.

No reduction needed for seismic shear. See section 3.1.8.7 for additional information on seismic applications.

$$\phi N_n = 5590 \times \alpha_s \times f_{cr}$$

$$\Rightarrow \text{Allow} = 5590 \times 0.66 \times 0.75$$

$$= 2767 \text{ \#}$$

3.3.5 KWIK Bolt TZ Expansion Anchor

Table 10 - Load adjustment factors for 5/8-in. diameter carbon steel Hilti KWIK Bolt TZ in uncracked concrete^{1,2}

5/8-in. KB-TZ CS uncracked concrete		Spacing factor in tension f_{AN}		Edge distance factor in tension f_{RN}		Spacing factor in shear ³ f_{AV}		Edge distance in shear				Conc. thickness fac- tor in shear ⁴ f_{HV}	
								⊥ toward edge f_{RV}		to and away from edge f_{RV}			
Effective embed. h_{ef}	in. (mm)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)
Nominal embed. h_{nom}	in. (mm)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)
Spacing (s) / edge distance (c_e) / concrete thickness (h) - in. (mm)	3 (76)	n/a	0.63	n/a	n/a	n/a	0.55	n/a	n/a	n/a	n/a	n/a	n/a
	3-1/4 (83)	n/a	0.64	n/a	0.46	n/a	0.55	n/a	0.17	n/a	0.34	n/a	n/a
	3-1/2 (89)	0.69	0.65	n/a	0.48	0.57	0.56	n/a	0.19	n/a	0.38	n/a	n/a
	3-5/8 (92)	0.69	0.65	0.60	0.48	0.57	0.56	0.28	0.20	0.56	0.40	n/a	n/a
	4 (102)	0.71	0.67	0.64	0.51	0.58	0.56	0.32	0.23	0.64	0.47	n/a	n/a
	4-1/4 (108)	0.73	0.68	0.67	0.53	0.58	0.57	0.35	0.26	0.67	0.51	n/a	n/a
	4-1/2 (114)	0.74	0.69	0.70	0.56	0.59	0.57	0.38	0.28	0.70	0.56	n/a	n/a
	4-3/4 (121)	0.75	0.70	0.73	0.58	0.59	0.58	0.42	0.30	0.73	0.58	n/a	n/a
	5 (127)	0.77	0.71	0.77	0.60	0.60	0.58	0.45	0.33	0.77	0.60	0.63	n/a
	5-1/2 (140)	0.79	0.73	0.85	0.64	0.61	0.59	0.52	0.38	0.85	0.64	0.66	n/a
	5-7/8 (149)	0.81	0.74	0.90	0.67	0.62	0.59	0.57	0.42	0.90	0.67	0.68	n/a
	6 (152)	0.82	0.75	0.92	0.69	0.62	0.59	0.59	0.43	0.92	0.69	0.69	0.62
	6-1/8 (156)	0.83	0.76	0.94	0.70	0.62	0.60	0.61	0.44	0.94	0.70	0.69	0.62
	8 (203)	0.93	0.83	1.00	0.91	0.66	0.63	0.91	0.66	1.00	0.91	0.79	0.71
	10 (254)	1.00	0.92		1.00	0.70	0.66	1.00	0.92		1.00	0.89	0.80
	12 (305)		1.00			0.74	0.69		1.00			0.97	0.87
	14 (356)					0.77	0.72					1.00	0.94
	16 (406)					0.81	0.75						1.00
	18 (457)					0.85	0.78						
	20 (508)					0.89	0.82						
	22 (559)					0.93	0.85						
	> 24 (610)					0.97	0.88						

Table 11 - Load adjustment factors for 5/8-in. diameter carbon steel Hilti KWIK Bolt TZ in cracked concrete^{1,2}

5/8-in. KB-TZ CS cracked concrete		Spacing factor in tension f_{AN}		Edge distance factor in tension f_{RN}		Spacing factor in shear ³ f_{AV}		Edge distance in shear				Conc. thickness fac- tor in shear ⁴ f_{HV}	
								┴ toward edge f_{RV}		to and away from edge f_{RV}			
Effective embed. h_{ef}	in. (mm)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)	3-1/8 (79)	4 (102)
Nominal embed. h_{nom}	in. (mm)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)	3-9/16 (90)	4-7/16 (113)
Spacing (s) / edge distance (c_e) / concrete thickness (h) - in. (mm)	3 (76)	n/a	0.63	n/a	n/a	n/a	0.55	n/a	n/a	n/a	n/a	n/a	n/a
	3-1/4 (83)	n/a	0.64	n/a	0.66	n/a	0.55	n/a	0.17	n/a	0.35	n/a	n/a
	3-1/2 (89)	0.69	0.65	n/a	0.69	0.57	0.56	n/a	0.19	n/a	0.39	n/a	n/a
	3-5/8 (92)	0.69	0.65	0.83	0.71	0.57	0.56	0.28	0.20	0.56	0.41	n/a	n/a
	4 (102)	0.71	0.67	0.89	0.75	0.58	0.56	0.33	0.24	0.65	0.47	n/a	n/a
	4-1/4 (108)	0.73	0.68	0.93	0.78	0.58	0.57	0.36	0.26	0.71	0.52	n/a	n/a
	4-1/2 (114)	0.74	0.69	0.97	0.81	0.59	0.57	0.39	0.28	0.78	0.56	n/a	n/a
	4-3/4 (121)	0.75	0.70	1.00	0.84	0.59	0.58	0.42	0.31	0.84	0.61	n/a	n/a
	5 (127)	0.77	0.71		0.87	0.60	0.58	0.45	0.33	0.91	0.66	0.63	n/a
	5-1/2 (140)	0.79	0.73		0.93	0.61	0.59	0.52	0.38	1.00	0.76	0.66	n/a
	5-7/8 (149)	0.81	0.74		0.98	0.62	0.59	0.58	0.42		0.84	0.68	n/a
	6 (152)	0.82	0.75		1.00	0.62	0.60	0.60	0.43		0.87	0.69	0.62
	6-1/8 (156)	0.83	0.76			0.62	0.60	0.62	0.45		0.89	0.69	0.62
	8 (203)	0.93	0.83			0.66	0.63	0.92	0.67		1.00	0.79	0.71
	10 (254)	1.00	0.92			0.70	0.66	1.00	0.93			0.89	0.80
	12 (305)		1.00			0.74	0.69		1.00			0.97	0.87
	14 (356)					0.78	0.72					1.00	0.94
	16 (406)					0.82	0.75						1.00
	18 (457)					0.85	0.79						
	20 (508)					0.89	0.82						
	22 (559)					0.93	0.85						
	> 24 (610)					0.97	0.88						

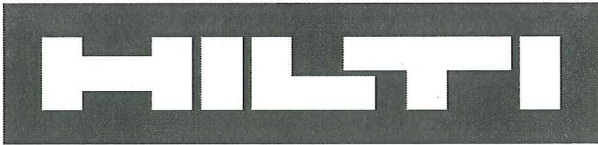
1 Linear interpolation not permitted.

2 When combining multiple load adjustment factors (e.g. for a 4 anchor pattern in a corner with thin concrete member) the design can become very conservative. To optimize the design, use Hilti PROFIS Anchor Design software or perform anchor calculation using design equations from ACI 318-14 Chapter 17.

3 Spacing factor reduction in shear, f_{AV} , assumes an influence of a nearby edge. If no edge exists, then $f_{AV} = f_{AN}$.

4 Concrete thickness reduction factor in shear, f_{HV} , assumes an influence of a nearby edge. If no edge exists, then $f_{HV} = 1.0$.

□ If a reduction factor value is in a shaded cell, this indicates that this specific edge distance may not be permitted with a certain spacing (or vice versa). Check with table 5 and figure 2 of this section to calculate permissible edge distance, spacing and concrete thickness combinations.



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- 5. Kwik Bolt TZ carbon steel
- 6. Exp anchor KB-TZ 5/8" x 8-1/2"

EXP ANCHOR KB-TZ 5/8" X 8-1/2"

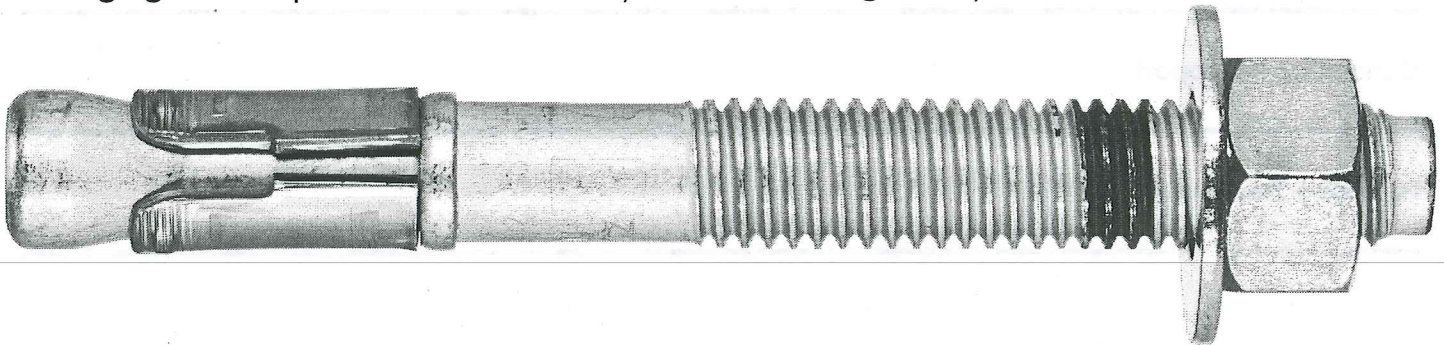
Ultimate-performance wedge anchor for cracked concrete and seismic (CS)

Item number 387518

> 1 RATINGS

SHARE

Packaging unit: 15 pc Drill bit diameter: 5/8 in Anchor length: 8-1/2 in



Move cursor over image to zoom.

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Price/ Item: \$68.34 / 1 Box

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KEY TECHNICAL DATA

Base materials: Concrete (cracked), Concrete (light weight), Concrete (light weight over metal deck), Concrete (uncracked), Masonry (grout-filled CMU)

Item number	Drill bit diameter	Anchor length	Packaging unit	Price / Unit	Quantity
Exp anchor KB-TZ 5/8" x 6" Item number 387517 More details >	5/8 in 6 in 15 pc	6 in	15 pc	\$47.55 / 1 Box Ordering & product details >	Box 1
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HIT-HY 200 Adhesive Anchoring System 3.2.3

Table 40 - Hilti HIT-HY 200 adhesive design strength with concrete / bond failure for threaded rod
in cracked concrete 1,2,3,4,5,6,7,8,9

Nominal anchor diameter in.	Effective embedment in. (mm)	Tension — ΦN_n or N_r				Shear — ΦV_n or V_r			
		$f'_c = 2500$ psi (17.2 Mpa) lb (kN)	$f'_c = 3000$ psi (20.7 Mpa) lb (kN)	$f'_c = 4000$ psi (27.6 Mpa) lb (kN)	$f'_c = 6000$ psi (41.4 Mpa) lb (kN)	$f'_c = 2500$ psi (17.2 Mpa) lb (kN)	$f'_c = 3000$ psi (20.7 Mpa) lb (kN)	$f'_c = 4000$ psi (27.6 Mpa) lb (kN)	$f'_c = 6000$ psi (41.4 Mpa) lb (kN)
3/8	2-3/8 (60)	1,900 (8.5)	1,900 (8.5)	1,900 (8.5)	2,015 (9.0)	2,045 (9.1)	2,045 (9.1)	2,045 (9.1)	2,165 (9.6)
	3-3/8 (86)	2,700 (12.0)	2,700 (12.0)	2,700 (12.0)	2,860 (12.7)	5,810 (25.8)	5,810 (25.8)	5,810 (25.8)	6,160 (27.4)
	4-1/2 (114)	3,600 (16.0)	3,600 (16.0)	3,600 (16.0)	3,815 (17.0)	7,750 (34.5)	7,750 (34.5)	7,750 (34.5)	8,215 (36.5)
	7-1/2 (191)	5,995 (26.7)	5,995 (26.7)	5,995 (26.7)	6,355 (28.3)	12,915 (57.4)	12,915 (57.4)	12,915 (57.4)	13,690 (60.9)
	10 (254)	10,730 (47.7)	10,730 (47.7)	10,730 (47.7)	11,375 (50.6)	23,115 (102.8)	23,115 (102.8)	23,115 (102.8)	24,500 (109.0)
1/2	2-3/4 (70)	2,520 (11.2)	2,760 (12.3)	2,950 (13.1)	3,130 (13.9)	5,425 (24.1)	5,945 (26.4)	6,355 (28.3)	6,735 (30.0)
	4-1/2 (114)	4,830 (21.5)	4,830 (21.5)	4,830 (21.5)	5,120 (22.8)	10,400 (46.3)	10,400 (46.3)	10,400 (46.3)	11,025 (49.0)
	6 (152)	6,440 (28.6)	6,440 (28.6)	6,440 (28.6)	6,825 (30.4)	13,870 (61.7)	13,870 (61.7)	13,870 (61.7)	14,700 (65.4)
	10 (254)	10,730 (47.7)	10,730 (47.7)	10,730 (47.7)	11,375 (50.6)	23,115 (102.8)	23,115 (102.8)	23,115 (102.8)	24,500 (109.0)
	12-1/2 (318)	16,865 (75.0)	16,865 (75.0)	16,865 (75.0)	17,875 (79.5)	36,320 (161.6)	36,320 (161.6)	36,320 (161.6)	38,500 (171.3)
5/8	3-1/8 (79)	3,050 (13.6)	3,345 (14.9)	3,860 (17.2)	4,470 (19.9)	6,575 (29.2)	7,200 (32.0)	8,315 (37.0)	9,625 (42.8)
	5-5/8 (143)	7,370 (32.8)	7,590 (33.8)	7,590 (33.8)	8,045 (35.8)	15,875 (70.6)	16,345 (72.7)	16,345 (72.7)	17,325 (77.1)
	7-1/2 (191)	10,120 (45.0)	10,120 (45.0)	10,120 (45.0)	10,725 (47.7)	21,790 (96.9)	21,790 (96.9)	21,790 (96.9)	23,100 (102.8)
	10 (254)	14,640 (65.1)	14,640 (65.1)	14,640 (65.1)	15,515 (69.0)	31,530 (140.3)	31,530 (140.3)	31,530 (140.3)	33,420 (148.7)
	12-1/2 (318)	16,865 (75.0)	16,865 (75.0)	16,865 (75.0)	17,875 (79.5)	36,320 (161.6)	36,320 (161.6)	36,320 (161.6)	38,500 (171.3)
3/4	3-1/2 (89)	3,620 (16.1)	3,965 (17.6)	4,575 (20.4)	5,605 (24.9)	7,790 (34.7)	8,535 (38.0)	9,855 (43.8)	12,070 (53.7)
	6-3/4 (171)	9,690 (43.1)	10,615 (47.2)	10,980 (48.8)	11,635 (51.8)	20,870 (92.8)	22,860 (101.7)	23,645 (105.2)	25,065 (111.5)
	9 (229)	14,640 (65.1)	14,640 (65.1)	14,640 (65.1)	15,515 (69.0)	31,530 (140.3)	31,530 (140.3)	31,530 (140.3)	33,420 (148.7)
	12-1/2 (318)	16,865 (75.0)	16,865 (75.0)	16,865 (75.0)	17,875 (79.5)	36,320 (161.6)	36,320 (161.6)	36,320 (161.6)	38,500 (171.3)
	15 (381)	24,395 (108.5)	24,395 (108.5)	24,395 (108.5)	25,860 (115.0)	52,550 (233.8)	52,550 (233.8)	52,550 (233.8)	55,700 (247.8)
7/8	3-1/2 (89)	3,620 (16.1)	3,965 (17.6)	4,575 (20.4)	5,605 (24.9)	7,790 (34.7)	8,535 (38.0)	9,855 (43.8)	12,070 (53.7)
	6-3/4 (171)	9,690 (43.1)	10,615 (47.2)	10,980 (48.8)	11,635 (51.8)	20,870 (92.8)	22,860 (101.7)	23,645 (105.2)	25,065 (111.5)
	9 (229)	14,640 (65.1)	14,640 (65.1)	14,640 (65.1)	15,515 (69.0)	31,530 (140.3)	31,530 (140.3)	31,530 (140.3)	33,420 (148.7)
	12-1/2 (318)	16,865 (75.0)	16,865 (75.0)	16,865 (75.0)	17,875 (79.5)	36,320 (161.6)	36,320 (161.6)	36,320 (161.6)	38,500 (171.3)
	15 (381)	24,395 (108.5)	24,395 (108.5)	24,395 (108.5)	25,860 (115.0)	52,550 (233.8)	52,550 (233.8)	52,550 (233.8)	55,700 (247.8)
1	3-1/2 (89)	3,620 (16.1)	3,965 (17.6)	4,575 (20.4)	5,605 (24.9)	7,790 (34.7)	8,535 (38.0)	9,855 (43.8)	12,070 (53.7)
	6-3/4 (171)	9,690 (43.1)	10,615 (47.2)	10,980 (48.8)	11,635 (51.8)	20,870 (92.8)	22,860 (101.7)	23,645 (105.2)	25,065 (111.5)
	9 (229)	14,640 (65.1)	14,640 (65.1)	14,640 (65.1)	15,515 (69.0)	31,530 (140.3)	31,530 (140.3)	31,530 (140.3)	33,420 (148.7)
	12-1/2 (318)	16,865 (75.0)	16,865 (75.0)	16,865 (75.0)	17,875 (79.5)	36,320 (161.6)	36,320 (161.6)	36,320 (161.6)	38,500 (171.3)
	15 (381)	24,395 (108.5)	24,395 (108.5)	24,395 (108.5)	25,860 (115.0)	52,550 (233.8)	52,550 (233.8)	52,550 (233.8)	55,700 (247.8)
1-1/4	3-1/2 (89)	3,620 (16.1)	3,965 (17.6)	4,575 (20.4)	5,605 (24.9)	7,790 (34.7)	8,535 (38.0)	9,855 (43.8)	12,070 (53.7)
	6-3/4 (171)	9,690 (43.1)	10,615 (47.2)	10,980 (48.8)	11,635 (51.8)	20,870 (92.8)	22,860 (101.7)	23,645 (105.2)	25,065 (111.5)
	9 (229)	14,640 (65.1)	14,640 (65.1)	14,640 (65.1)	15,515 (69.0)	31,530 (140.3)	31,530 (140.3)	31,530 (140.3)	33,420 (148.7)
	12-1/2 (318)	16,865 (75.0)	16,865 (75.0)	16,865 (75.0)	17,875 (79.5)	36,320 (161.6)	36,320 (161.6)	36,320 (161.6)	38,500 (171.3)
	15 (381)	24,395 (108.5)	24,395 (108.5)	24,395 (108.5)	25,860 (115.0)	52,550 (233.8)	52,550 (233.8)	52,550 (233.8)	55,700 (247.8)

3.2.3

1 See section 3.1.7 for explanation on development of load values.

2 See section 3.1.7.3 to convert design strength (factored resistance) value to ASD value.

3 Linear interpolation between embedment depths and concrete compressive strengths is not permitted.

4 Apply spacing, edge distance, and concrete thickness factors in tables 42 - 55 as necessary. Compare to the steel values in table 41.

The lesser of the values is to be used for the design.

5 Data is for temperature range A: Max. short term temperature = 104° F (40° C), max. long term temperature = 75° F (24° C).

For temperature range B: Max. short term temperature = 176° F (80° C), max. long term temperature = 122° F (50° C) multiply above value by 0.80.

For temperature range C: Max. short term temperature = 248° F (120° C), max. long term temperature = 162° F (72° C) multiply above value by 0.70.

Short term elevated concrete temperatures are those that occur over brief intervals, e.g., as a result of diurnal cycling. Long term concrete temperatures are roughly constant over significant periods of time.

6 Tabular values are for dry concrete conditions. For water saturated concrete multiply design strength (factored resistance) by 0.85.

7 Tabular values are for short term loads only. For sustained loads including overhead use, see section 3.1.7.5.

8 Tabular values are for normal weight concrete only. For lightweight concrete, multiply design strength (factored resistance) by λ_s as follows:

For sand-lightweight, $\lambda_s = 0.51$. For all-lightweight, $\lambda_s = 0.45$.

9 Tabular values are for static loads only. For seismic loads, multiply cracked concrete tabular values by the following reduction factors:

3/8-in to 3/4-in diameter - $\alpha_{seis} = 0.60$

7/8-in to 1-1/4-in diameter - $\alpha_{seis} = 0.75$

See section 3.1.7.4 for additional information on seismic applications.

HIT-HY 200 Adhesive Anchoring System 3.2.3

Table 46 - Load adjustment factors for 5/8-in. diameter threaded rods in uncracked concrete^{1,2,3}

5/8-in. uncracked concrete	Embedment h_{ef} in. (mm)	Spacing factor in tension f_{AN}				Edge distance factor in tension f_{RN}				Spacing factor in shear ⁴ f_{AV}				Edge distance in shear								Concrete thickness factor in shear ⁵ f_{HV}			
														⊥ Toward edge f_{RV}				 To edge f_{RV}							
		3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)
Spacing (s) / Edge distance (c) / Concrete thickness (h), - in. (mm)	1-3/4 (44)	n/a	n/a	n/a	n/a	0.33	0.25	0.18	0.11	n/a	n/a	n/a	n/a	0.09	0.04	0.03	0.01	0.19	0.08	0.06	0.03	n/a	n/a	n/a	n/a
	3-1/8 (79)	0.59	0.59	0.57	0.54	0.44	0.30	0.22	0.13	0.56	0.54	0.53	0.52	0.22	0.10	0.07	0.04	0.44	0.20	0.13	0.07	n/a	n/a	n/a	n/a
	4 (102)	0.62	0.62	0.59	0.55	0.52	0.33	0.25	0.14	0.58	0.55	0.53	0.52	0.32	0.15	0.10	0.05	0.52	0.29	0.19	0.10	n/a	n/a	n/a	n/a
	4-5/8 (117)	0.63	0.63	0.60	0.56	0.59	0.36	0.27	0.16	0.59	0.55	0.54	0.53	0.40	0.18	0.12	0.06	0.59	0.36	0.24	0.13	0.60	n/a	n/a	n/a
	5 (127)	0.65	0.65	0.61	0.57	0.63	0.38	0.28	0.16	0.60	0.56	0.54	0.53	0.45	0.21	0.13	0.07	0.63	0.38	0.27	0.14	0.63	n/a	n/a	n/a
	6 (152)	0.68	0.68	0.63	0.58	0.76	0.42	0.31	0.18	0.62	0.57	0.55	0.53	0.59	0.27	0.18	0.09	0.76	0.42	0.35	0.19	0.69	n/a	n/a	n/a
	7 (178)	0.70	0.70	0.66	0.59	0.86	0.47	0.34	0.20	0.64	0.58	0.56	0.54	0.75	0.34	0.22	0.12	0.88	0.47	0.38	0.24	0.74	n/a	n/a	n/a
	7-1/8 (181)	0.71	0.71	0.66	0.60	0.87	0.47	0.35	0.20	0.64	0.58	0.56	0.54	0.77	0.35	0.23	0.12	0.90	0.47	0.38	0.24	0.75	0.57	n/a	n/a
	8 (203)	0.73	0.73	0.68	0.61	0.95	0.52	0.38	0.22	0.66	0.59	0.57	0.55	0.91	0.41	0.27	0.14	1.00	0.52	0.41	0.29	0.79	0.61	n/a	n/a
	9 (229)	0.76	0.76	0.70	0.62	1.00	0.57	0.42	0.25	0.68	0.60	0.58	0.55	1.00	0.50	0.32	0.17		0.57	0.45	0.32	0.84	0.65	0.56	n/a
	10 (254)	0.79	0.79	0.72	0.63		0.64	0.47	0.27	0.70	0.62	0.59	0.56		0.58	0.38	0.20		0.64	0.48	0.34	0.89	0.68	0.59	n/a
	11 (279)	0.82	0.82	0.74	0.65		0.70	0.52	0.30	0.72	0.63	0.60	0.56		0.67	0.43	0.23		0.70	0.52	0.36	0.93	0.71	0.62	n/a
	12 (305)	0.85	0.85	0.77	0.66		0.77	0.56	0.33	0.74	0.64	0.60	0.57		0.76	0.50	0.26		0.77	0.56	0.38	0.97	0.75	0.65	n/a
	14 (356)	0.91	0.91	0.81	0.69		0.89	0.66	0.38	0.77	0.66	0.62	0.58		0.96	0.62	0.33		0.89	0.66	0.41	1.00	0.81	0.70	0.57
	16 (406)	0.97	0.97	0.86	0.71		1.00	0.75	0.44	0.81	0.69	0.64	0.59		1.00	0.76	0.41		1.00	0.75	0.46		0.86	0.75	0.61
	18 (457)	1.00	1.00	0.90	0.74			0.84	0.49	0.85	0.71	0.66	0.60			0.91	0.49			0.84	0.50		0.91	0.79	0.64
	20 (508)			0.94	0.77			0.94	0.55	0.89	0.73	0.67	0.61			1.00	0.57			0.94	0.55		0.96	0.83	0.68
	22 (559)			0.99	0.79			1.00	0.60	0.93	0.75	0.69	0.63				0.66			1.00	0.60		1.00	0.87	0.71
	24 (610)			1.00	0.82				0.66	0.97	0.78	0.71	0.64				0.75				0.66			0.91	0.74
	26 (660)				0.85				0.71	1.00	0.80	0.73	0.65				0.84				0.71			0.95	0.77
	28 (711)				0.87				0.77		0.82	0.74	0.66				0.94				0.77			0.99	0.80
	30 (762)				0.90				0.82		0.85	0.76	0.67				1.00				0.82			1.00	0.83
	36 (914)				0.98				0.99		0.92	0.81	0.71								0.99				0.91
	>48 (1219)				1.00				1.00		1.00	0.92	0.77								1.00				1.00

3.2.3

Table 47 - Load adjustment factors for 5/8-in. diameter threaded rods in cracked concrete^{1,2,3}

5/8-in. cracked concrete	Embedment h_{ef} in. (mm)	Spacing factor in tension f_{AN}				Edge distance factor in tension f_{RN}				Spacing factor in shear ⁴ f_{AV}				Edge distance in shear								Concrete thickness factor in shear ⁵ f_{HV}			
														⊥ Toward edge f_{RV}				 To edge f_{RV}							
		3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)	3-1/8 (79)	5-5/8 (143)	7-1/2 (191)	12-1/2 (318)
Spacing (s) / Edge distance (c) / Concrete thickness (h), - in. (mm)	1-3/4 (44)	n/a	n/a	n/a	n/a	0.46	0.46	0.43	0.40	n/a	n/a	n/a	n/a	0.09	0.04	0.03	0.02	0.19	0.09	0.06	0.04	n/a	n/a	n/a	n/a
	3-1/8 (79)	0.59	0.59	0.57	0.54	0.55	0.55	0.50	0.44	0.56	0.54	0.53	0.52	0.22	0.10	0.07	0.04	0.45	0.20	0.15	0.09	n/a	n/a	n/a	n/a
	4 (102)	0.62	0.62	0.59	0.55	0.62	0.62	0.55	0.46	0.58	0.55	0.54	0.53	0.33	0.15	0.11	0.06	0.65	0.30	0.22	0.13	n/a	n/a	n/a	n/a
	4-5/8 (117)	0.63	0.63	0.60	0.56	0.66	0.66	0.58	0.48	0.59	0.55	0.54	0.53	0.40	0.18	0.13	0.08	0.81	0.37	0.27	0.16	0.60	n/a	n/a	n/a
	5 (127)	0.65	0.65	0.61	0.57	0.69	0.69	0.60	0.49	0.60	0.56	0.55	0.53	0.45	0.21	0.15	0.09	0.91	0.41	0.30	0.18	0.63	n/a	n/a	n/a
	6 (152)	0.68	0.68	0.63	0.58	0.77	0.77	0.66	0.53	0.62	0.57	0.56	0.54	0.60	0.27	0.20	0.12	1.00	0.54	0.40	0.24	0.69	n/a	n/a	n/a
	7 (178)	0.70	0.70	0.66	0.59	0.86	0.86	0.72	0.56	0.64	0.58	0.57	0.55	0.75	0.34	0.25	0.15		0.68	0.50	0.30	0.74	n/a	n/a	n/a
	7-1/8 (181)	0.71	0.71	0.66	0.60	0.87	0.87	0.73	0.56	0.64	0.58	0.57	0.55	0.77	0.35	0.26	0.15		0.70	0.51	0.31	0.75	0.58	n/a	n/a
	8 (203)	0.73	0.73	0.68	0.61	0.95	0.95	0.78	0.59	0.66	0.59	0.58	0.55	0.92	0.42	0.30	0.18		0.84	0.61	0.37	0.79	0.61	n/a	n/a
	9 (229)	0.76	0.76	0.70	0.62	1.00	1.00	0.85	0.62	0.68	0.60	0.58	0.56	1.00	0.50	0.36	0.22		1.00	0.73	0.44	0.84	0.65	0.58	n/a
	10 (254)	0.79	0.79	0.72	0.63			0.91	0.66	0.70	0.62	0.59	0.57		0.58	0.43	0.26		1.00	0.85	0.51	0.89	0.68	0.61	n/a
	11 (279)	0.82	0.82	0.74	0.65			0.98	0.69	0.72	0.63	0.60	0.57		0.67	0.49	0.29			0.98	0.59	0.93	0.72	0.64	n/a
	12 (305)	0.85	0.85	0.77	0.66			1.00	0.73	0.74	0.64	0.61	0.58		0.77	0.56	0.34			1.00	0.67	0.97	0.75	0.67	n/a
	14 (356)	0.91	0.91	0.81	0.69				0.81	0.78	0.66	0.63	0.59		0.97	0.71	0.42				0.81	1.00	0.81	0.73	0.61
	16 (406)	0.97	0.97	0.86	0.71				0.89	0.82	0.69	0.65	0.61		1.00	0.86	0.52				0.89		0.86	0.78	0.66
	18 (457)	1.00	1.00	0.90	0.74				0.97	0.85	0.71	0.67	0.62			1.00	0.62				0.97		0.92	0.82	0.70
	20 (508)			0.94	0.77				1.00	0.89	0.73	0.69	0.63				0.72				1.00		0.97	0.87	0.73
	22 (559)			0.99	0.79					0.93	0.76	0.71	0.65				0.83					1.00	0.91	0.77	
	24 (610)			1.00	0.82					0.97	0.78	0.73	0.66				0.95							0.95	0.80
	26 (660)				0.85					1.00	0.80	0.75	0.67				1.00							0.99	0.84
	28 (711)				0.87						0.83	0.76	0.69											1.00	0.87
	30 (762)				0.90						0.85	0.78	0.70												0.90
	36 (914)				0.98						0.92	0.84	0.74												0.98
	>48 (1219)				1.00						1.00	0.95	0.82												1.00

1 Linear interpolation not permitted

2

Cold Formed Steel Studs

4/360 limit ; 20 psf lateral load
30 psf lat load 4/240

550162-54 @ 24"oc $l_{max} = 15'-4"$

600162-54 @ 24"oc $l_{max} = 16'-6"$
-33 @ 24"oc $l_{max} = 13'-9"$ e requires web stiffeners

600200-54 @ 24"oc $l_{max} = 17'-4"$
-33 @ 24"oc $l_{max} = 14'-4"$ e ~~16'-4" e~~
11'-8" e

800162-54 @ 24"oc $l_{max} = 20'-9"$ e
-33 $l_{max} = 15'-3"$ e

800200-54 @ 24"oc $l_{max} = 21'-9"$ e
-33 $l_{max} = 16'-9"$ e

g Wall Heights — Curtain Wall 1-sp

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

Permitting Wall Heights — Curtain Wall 1-span

	Fy (ksi)	Spacing (in) o.c.	5 psf			15 psf			20 psf			25 psf			30 psf			35 psf			40 psf			50 psf		
			L/120	L/240	L/360	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600	L/240	L/360	L/600
600S137-68	33	12	45' 3"	35' 11"	31' 4"	28' 0"	24' 6"	20' 8"	25' 6"	22' 3"	18' 9"	23' 8"	20' 8"	17' 5"	22' 3"	19' 5"	16' 5"	21' 2"	18' 5"	15' 7"	20' 2"	17' 8"	14' 11"	18' 9"	16' 5"	13' 10"
600S137-68	33	16	41' 1"	32' 7"	28' 6"	25' 6"	22' 3"	18' 9"	23' 2"	20' 2"	17' 1"	21' 6"	18' 9"	15' 10"	20' 2"	17' 8"	14' 11"	19' 2"	16' 9"	14' 2"	18' 4"	16' 0"	13' 6"	16' 4"	14' 11"	12' 7"
600S137-68	33	24	35' 11"	28' 6"	24' 11"	22' 3"	19' 5"	16' 5"	20' 2"	17' 6"	14' 11"	18' 8"	16' 5"	13' 10"	17' 3"	15' 5"	13' 0"	16' 0"	14' 8"	12' 4"	14' 11"	14' 0"	11' 10"	13' 4"	13' 0"	11' 0"
600S200-68	33	12	47' 7"	37' 9"	33' 0"	29' 6"	25' 9"	21' 9"	26' 9"	23' 5"	19' 9"	24' 10"	21' 9"	18' 4"	23' 5"	20' 5"	17' 3"	22' 3"	19' 5"	16' 4"	21' 3"	18' 7"	15' 8"	19' 9"	17' 3"	14' 6"
600S200-68	33	16	43' 2"	34' 3"	29' 11"	26' 9"	23' 5"	19' 9"	24' 4"	21' 3"	17' 11"	22' 7"	19' 9"	16' 8"	21' 3"	18' 7"	15' 8"	20' 2"	17' 8"	14' 11"	19' 4"	16' 10"	14' 3"	17' 5"	15' 8"	13' 2"
600S200-68	33	24	37' 9"	29' 11"	26' 2"	23' 5"	20' 5"	17' 3"	21' 3"	18' 7"	15' 8"	19' 9"	17' 3"	14' 6"	18' 5"	16' 3"	13' 8"	17' 0"	15' 5"	13' 0"	15' 11"	14' 9"	12' 5"	14' 3"	13' 8"	11' 6"
600S137-97	33	12	47' 11"	38' 0"	33' 2"	29' 8"	25' 11"	21' 10"	27' 0"	23' 7"	19' 10"	25' 0"	21' 10"	18' 5"	23' 7"	20' 7"	17' 4"	22' 5"	19' 7"	16' 6"	21' 5"	18' 8"	15' 9"	19' 10"	17' 4"	14' 8"
600S137-97	33	16	43' 6"	34' 6"	30' 2"	27' 0"	23' 7"	19' 10"	24' 6"	21' 5"	18' 1"	22' 9"	19' 10"	16' 9"	21' 5"	18' 8"	15' 9"	20' 4"	17' 9"	15' 0"	19' 5"	17' 0"	14' 4"	18' 1"	15' 9"	13' 4"
600S137-97	33	24	38' 0"	30' 2"	26' 4"	23' 7"	20' 7"	17' 4"	21' 5"	18' 8"	15' 9"	19' 10"	17' 4"	14' 6"	18' 8"	16' 4"	13' 9"	17' 9"	15' 6"	13' 1"	16' 11"	14' 10"	12' 6"	15' 2"	13' 9"	11' 7"
600S162-97	33	12	50' 1"	39' 9"	34' 9"	31' 1"	27' 2"	22' 11"	28' 3"	24' 8"	20' 9"	26' 2"	22' 11"	19' 4"	24' 8"	21' 6"	18' 2"	23' 5"	20' 5"	17' 3"	22' 5"	19' 7"	16' 6"	20' 9"	18' 2"	15' 4"
600S162-97	33	16	45' 6"	36' 2"	31' 7"	28' 3"	24' 8"	20' 9"	25' 8"	22' 5"	18' 11"	23' 10"	20' 9"	17' 6"	22' 5"	19' 7"	16' 6"	21' 3"	18' 7"	15' 8"	20' 4"	17' 9"	15' 0"	18' 11"	16' 6"	13' 11"
600S162-97	33	24	39' 9"	31' 7"	27' 7"	24' 8"	21' 6"	18' 2"	22' 5"	19' 7"	16' 6"	20' 9"	18' 2"	15' 4"	19' 7"	17' 1"	14' 5"	18' 7"	16' 3"	13' 8"	17' 9"	15' 6"	13' 1"	16' 0"	14' 5"	12' 2"
600S200-97	33	12	52' 10"	41' 11"	36' 7"	32' 9"	28' 7"	24' 1"	29' 9"	26' 0"	21' 11"	27' 7"	24' 1"	20' 4"	26' 0"	22' 8"	19' 2"	24' 8"	21' 7"	18' 2"	23' 7"	20' 7"	17' 5"	21' 11"	19' 2"	16' 2"
600S200-97	33	16	48' 0"	38' 1"	33' 3"	29' 9"	26' 0"	21' 11"	27' 0"	23' 7"	19' 11"	25' 1"	21' 11"	18' 6"	23' 7"	20' 7"	17' 5"	22' 5"	19' 7"	16' 6"	21' 5"	18' 9"	15' 10"	19' 11"	17' 5"	14' 8"
600S200-97	33	24	41' 11"	33' 3"	29' 1"	26' 0"	22' 8"	19' 2"	23' 7"	20' 7"	17' 5"	21' 11"	19' 2"	16' 2"	20' 7"	18' 0"	15' 2"	19' 7"	17' 1"	14' 5"	18' 9"	16' 4"	13' 10"	17' 0"	15' 2"	12' 10"
600S162-118	33	12	52' 11"	42' 0"	36' 8"	32' 10"	28' 8"	24' 2"	29' 10"	26' 0"	21' 11"	27' 8"	24' 2"	20' 5"	26' 0"	22' 8"	19' 2"	24' 9"	21' 7"	18' 3"	23' 8"	20' 8"	17' 5"	21' 11"	19' 2"	16' 2"
600S162-118	33	16	48' 1"	38' 2"	33' 4"	29' 10"	26' 0"	21' 11"	27' 1"	23' 8"	19' 11"	25' 2"	21' 11"	18' 6"	23' 8"	20' 8"	17' 5"	22' 6"	19' 8"	16' 7"	21' 6"	18' 9"	15' 10"	19' 11"	17' 5"	14' 8"
600S162-118	33	24	42' 0"	33' 4"	29' 1"	26' 0"	22' 9"	19' 2"	23' 8"	20' 8"	17' 5"	21' 11"	19' 2"	16' 2"	20' 8"	18' 1"	15' 3"	19' 8"	17' 2"	14' 6"	18' 9"	16' 5"	13' 10"	17' 5"	15' 3"	12' 10"
600S200-118	33	12	55' 10"	44' 4"	38' 9"	34' 7"	30' 3"	25' 6"	31' 5"	27' 6"	23' 2"	29' 2"	25' 6"	21' 6"	27' 6"	24' 0"	20' 3"	26' 1"	22' 10"	19' 3"	25' 0"	21' 10"	18' 5"	23' 2"	20' 3"	17' 1"
600S200-118	33	16	50' 9"	40' 3"	35' 2"	31' 5"	27' 6"	23' 2"	28' 7"	25' 0"	21' 1"	26' 6"	23' 2"	19' 7"	25' 0"	21' 10"	18' 5"	23' 9"	20' 9"	17' 6"	22' 8"	19' 10"	16' 9"	21' 1"	18' 5"	15' 6"
600S200-118	33	24	44' 4"	35' 2"	30' 9"	27' 6"	24' 0"	20' 3"	25' 0"	21' 10"	18' 5"	23' 2"	20' 3"	17' 1"	21' 10"	19' 1"	16' 1"	20' 9"	18' 1"	15' 3"	19' 10"	17' 4"	14' 7"	18' 5"	16' 1"	13' 7"
800S137-33	33	12	40' 6"	34' 0"	29' 8"	23' 5"	23' 2"	19' 7"	20' 3"	20' 3"	17' 9"	18' 1"	18' 1"	16' 5"	16' 6"	16' 6"	15' 6"	15' 4"	15' 4"	14' 9"	14' 4"	14' 4"	14' 1"	12' 10"	12' 10"	12' 10"
800S137-33	33	16	35' 1"	30' 11"	27' 0"	20' 3"	20' 3"	17' 9"	17' 6"	17' 6"	16' 2"	15' 8"	15' 8"	15' 0"	14' 4"	14' 4"	14' 1"	13' 3"	13' 3"	13' 3"	12' 5"	12' 5"	12' 5"	11' 1"	11' 1"	11' 1"
800S137-33	33	24	28' 6"	27' 0"	23' 7"	16' 6"	16' 6"	15' 6"	14' 4"	14' 4"	14' 1"	12' 10"	12' 10"	12' 10"	11' 8"	11' 8"	11' 8"	10' 10"	10' 10"	10' 10"	10' 10"	10' 10"	10' 10"	9' 1"	9' 1"	9' 1"
800S162-33	33	12	43' 3"	35' 5"	30' 11"	25' 0"	24' 2"	20' 4"	21' 7"	21' 7"	18' 6"	19' 4"	19' 4"	17' 2"	17' 8"	17' 8"	16' 2"	16' 4"	16' 4"	15' 4"	15' 3"	15' 3"	14' 8"	13' 8"	13' 8"	13' 8"
800S162-33	33	16	37' 5"	32' 2"	28' 1"	21' 7"	21' 7"	18' 6"	18' 9"	18' 9"	16' 10"	16' 9"	16' 9"	15' 7"	15' 3"	15' 3"	14' 8"	14' 2"	14' 2"	13' 11"	13' 3"	13' 3"	13' 3"	11' 10"	11' 10"	11' 10"
800S162-33	33	24	30' 7"	28' 1"	24' 7"	17' 8"	17' 8"	16' 2"	15' 3"	15' 3"	14' 8"	13' 8"	13' 8"	13' 8"	12' 6"	12' 6"	12' 6"	11' 7"	11' 7"	11' 7"	10' 10"	10' 10"	10' 10"	9' 6"	9' 6"	9' 6"
800S200-33	33	12	46' 4"	37' 9"	33' 0"	26' 9"	25' 9"	21' 8"	23' 2"	23' 2"	19' 9"	20' 9"	20' 9"	18' 4"	18' 11"	18' 11"	17' 3"	17' 6"	17' 6"	16' 4"	16' 5"	16' 5"	15' 8"	14' 8"	14' 8"	14' 8"
800S200-33	33	16	40' 2"	34' 3"	29' 11"	23' 2"	23' 2"	19' 9"	20' 1"	20' 1"	17' 11"	17' 11"	17' 11"	16' 8"	16' 5"	16' 5"	15' 8"	15' 2"	15' 2"	14' 10"	14' 2"	14' 2"	14' 2"	12' 6"	12' 6"	12' 6"
800S200-33	33	24	32' 9"	29' 11"	26' 2"	18' 11"	18' 11"	17' 3"	16' 5"	16' 5"	15' 8"	14' 8"	14' 8"	14' 6"	13' 5"	13' 5"	13' 5"	12' 5"	12' 5"	12' 5"	11' 7"	11' 7"	11' 7"	9' 6"	9' 6"	9' 6"
800S137-43	33	12	47' 2"	37' 5"	32' 8"	28' 1"	25' 6"	21' 6"	24' 3"	23' 2"	19' 7"	21' 9"	21' 6"	18' 2"	19' 10"	19' 10"	17' 1"	18' 4"	18' 4"	16' 3"	17' 2"	17' 2"	15' 6"	15' 4"	15' 4"	15' 4"
800S137-43	33	16	42' 1"	34' 0"	29' 9"	24' 3"	23' 2"	19' 7"	21' 0"	21' 0"	17' 9"	18' 10"	18' 10"	16' 6"	17' 2"	17' 2"	15' 6"	15' 11"	15' 11"	14' 9"	14' 10"	14' 10"	14' 1"	13' 4"	13' 4"	13' 4"
800S137-43	33	24	34' 4"	29' 9"	25' 11"	19' 10"	19' 10"	17' 1"	17' 2"	17' 2"	15' 6"	15' 4"	15' 4"	14' 5"	14' 0"	14' 0"	13' 7"	13' 0"	13' 0"	12' 11"	12' 2"	12' 2"	12' 2"	10' 10"	10' 10"	10' 10"
800S162-43	33	12	49' 1"	38' 11"	34' 0"	29' 11"	26' 7"	22' 5"	25' 11"	24' 2"	20' 4"	23' 2"	22' 5"	18' 11"	21' 2"	21' 1"	17' 9"	19' 7"	19' 7"	16' 11"	18' 4"	18' 4"	16' 2"	16' 5"	16' 5"	16' 5"
800S162-43	33	16	44' 7"	35' 4"	30' 11"	25' 11"	24' 2"	20' 4"	25' 11"	24' 2"	18' 6"	18' 5"	18' 5"	16' 6"	18' 4"	18' 4"	16' 2"	17' 0"	17' 0"	15' 4"	15' 10"	15' 10"	14' 8"	14' 2"	14' 2"	14' 2"</

6'-0" @ Side Walls -

Door headers

7'-0"
12'-0" Eave

> 9'-4" to top of transom

$$\begin{array}{r} 12'-0" \\ - 3.5" \\ - 3.5" \\ - 3" \\ \hline 11'-2" \end{array} - 9'-4" = 1'-10"$$

Studs @ each side: 6'-2" Opening

$$\frac{6.167'}{2} + \frac{2'}{2} = 4.0835' \times 36.2 \text{ psf} = 147.82 \text{ pef}$$

w/ fully Unsupported to 12': (2) 2x8 DF-L #2

S.R.: $M = 0.3922$ -ok

3'-0" @ End wall

$$\frac{7'-2"}{2} + \frac{5'-6"}{2} = 6.333' \text{ Trib} \times 45 \text{ psf} = 285 \text{ pef} \times 0.6 = 171 \text{ lb/ft}$$

$$\div 12 = 14.25 \text{ lb/in}$$

A single 2x6 weak axis S.R. = $M = 0.814$
 $V = 0.117$

$$S_y = 2.063 \text{ in}^3 \quad (0.6) \frac{285 \text{ pef} \times 3.5^2}{8} = 261.8 \text{ lb-ft} \times \frac{12 \text{ in}}{\text{ft}} = 3,142.125 \text{ lb-in}$$

$$I_y = 1.547 \text{ in}^4$$

$$3,142.125 \text{ lb-in} \div 2.063 \text{ in}^3 = 1523 \text{ psi}$$

(Enercalc = Same w/ $F_b' = 1872 \text{ psi}$)

$$\Delta = \frac{5 w l^4}{384 E I} = \frac{5 \times 14.25 \text{ lb/in} \times (42 \text{ in})^4}{384 \times 1,600,000 \text{ psi} \times 1.547 \text{ in}^4} = 0.233" = \frac{1}{80} \checkmark \quad 0.6w$$

$$w/I_y = 1.547 \text{ in}^4 \times 3 = 4.641 \text{ in}^4 \quad \Delta = 0.077" = \frac{1}{540} \quad 0.6w \quad \frac{1/324}{w} \checkmark$$



NELSON ENGINEERING

Professional Engineers & Land Surveyors

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Project Temporary Fire House Job no. 17-202-01

Comp & Cladding WL sheet no. _____ of _____

By CH Date _____

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3'-0" @ End wall

Columns @ ends of Header

Max height = 13'-0"

$$\text{Loading: } \frac{2'}{2} + \frac{3'}{2} = 2.5' \times 45\text{psf} = 112.5\text{plf}$$

$$285\text{plf} \times \frac{3.5'}{2} = 498.75\text{lb}$$

(2) 2x8 Kings OK - however need Bracing @ 4'-0"

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 8:16AM

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: Header above 3'-0" door openings at Endwall

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

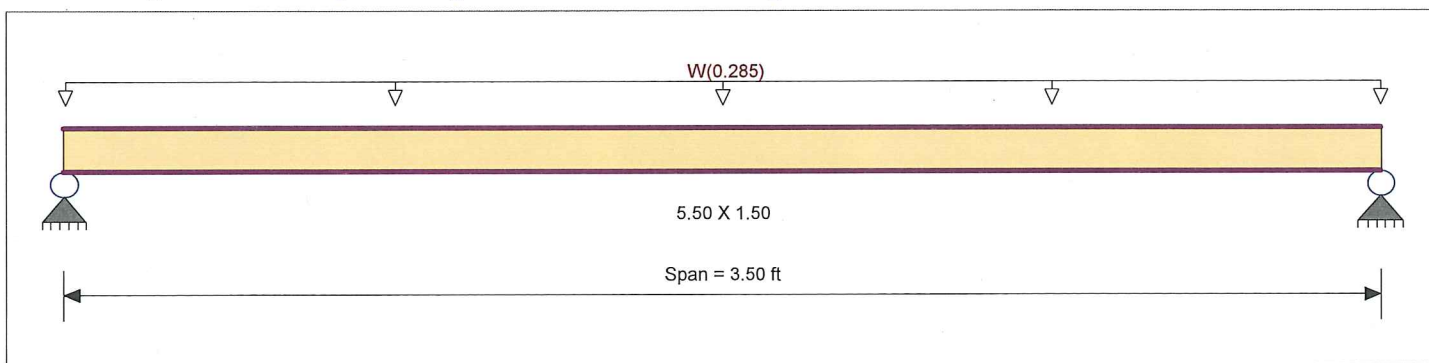
Material Properties

Analysis Method: Allowable Stress Design
Load Combination ASCE 7-10

Wood Species: Douglas Fir - Larch
Wood Grade: No.2

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Fb +	900 psi	E: Modulus of Elasticity	
Fb -	900 psi	Ebend- xx	1600ksi
Fc - Prll	1350 psi	Eminbend - xx	580ksi
Fc - Perp	625 psi		
Fv	180 psi		
Ft	575 psi	Density	31.2pcf



Applied Loads

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

Uniform Load: W = 0.2850, Tributary Width = 1.0 ft, (Wind Loading from Zone 5)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	0.814	1	Maximum Shear Stress Ratio	=	0.177	: 1
Section used for this span		5.50 X 1.50		Section used for this span		5.50 X 1.50	
fb: Actual	=	1,523.45psi		fv: Actual	=	50.83 psi	
FB: Allowable	=	1,872.00psi		Fv: Allowable	=	288.00 psi	
Load Combination		+0.60W		Load Combination		+0.60W	
Location of maximum on span	=	1.750ft		Location of maximum on span	=	3.385 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.391 in	Ratio =	107	<	240.0	
Max Upward Transient Deflection		0.000 in	Ratio =	0	<	240.0	
Max Downward Total Deflection		0.235 in	Ratio =	179	<	180	
Max Upward Total Deflection		0.000 in	Ratio =	0	<	180	

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios							Moment Values			Shear Values			
			M	V	C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	M	fb	F'b	V	fv
+0.60W	Length = 3.50 ft	1			0.90	1.300	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
						1.300	1.00	1.00	1.00	1.00			1053.00	0.00	0.00	162.00
							1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
+0.450W	Length = 3.50 ft	1	0.814	0.177	1.60	1.300	1.00	1.00	1.00	1.00	0.26	1,523.45	1872.00	0.28	50.83	288.00
						1.300	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
							1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
	Length = 3.50 ft	1	0.610	0.132	1.60	1.300	1.00	1.00	1.00	1.00	0.20	1,142.59	1872.00	0.21	38.13	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.3910	1.763		0.0000	0.000

Nelson Engineering
P.O. Box 1599
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Jackson Wyoming

Project Title: TOJ Fairground Building
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Title Block Line 6

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Wood Beam

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ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : Header above 3'-0" door openings at Endwall

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.499	0.499
Overall MINimum	0.224	0.224
+0.60W	0.299	0.299
+0.450W	0.224	0.224
W Only	0.499	0.499

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 13'-0" tall Door King Studs with 45 psf

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		2-2x8		
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber		
Overall Column Height		13.0 ft		Wood Member Type		Sawn		
(Used for non-slender calculations)				Exact Width		3.0 in		
Wood Species		Douglas Fir - Larch		Exact Depth		7.250 in		
Wood Grade		No.2		Area		21.750 in^2		
Fb +		900.0 psi	Fv	180.0 psi	Ix	95.270 in^4	Cf or Cv for Bending	1.20
Fb -		900.0 psi	Ft	575.0 psi	Iy	16.313 in^4	Cf or Cv for Compression	1.050
Fc - Prll		1,350.0 psi	Density	31.20 pcf			Cf or Cv for Tension	1.20
Fc - Perp		625.0 psi					Cm : Wet Use Factor	1.0
							Ct : Temperature Factor	1.0
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			Cfu : Flat Use Factor	1.0
		Basic	1,600.0	1,600.0	1,600.0 ksi		Kf : Built-up columns	1.0
		Minimum	580.0	580.0			Use Cr : Repetitive ?	Yes
NDS 15.3.2								
Brace condition for deflection (buckling) along columns :								
X-X (width) axis :				Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, K = 1.0				
Y-Y (depth) axis :				Unbraced Length for X-X Axis buckling = 13 ft, K = 1.0				

Applied Loads

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

Column self weight included: 61.263 lbs * Dead Load Factor

BENDING LOADS . . .

Wind Loading on King Studs: Lat. Uniform Load creating Mx-x, W = 0.1125 k/ft

Header Load: Lat. Point Load at 7.0 ft creating Mx-x, W = 0.4980 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.5565 : 1	Maximum SERVICE Lateral Load Reactions . .			
	Load Combination	+D+0.60W	Top along Y-Y	0.9994 k	Bottom along Y-Y	0.9611 k
	Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X	0.0 k
	Location of max. above base	6.980 ft	Maximum SERVICE Load Lateral Deflections . . .			
	At maximum location values are . . .		Along Y-Y	0.7382 in at	6.631 ft	above base
	Applied Axial	0.06126 k	for load combination: W Only			
	Applied Mx	2.381 k-ft	Along X-X	0.0 in at	0.0 ft	above base
	Applied My	0.0 k-ft	for load combination: n/a			
	Fc : Allowable	908.36 psi	Other Factors used to calculate allowable stresses . . .			
PASS	Maximum Shear Stress Ratio =	0.1436 : 1	Bending	Compression	Tension	
	Load Combination	+D+0.60W				
	Location of max. above base	13.0 ft				
	Applied Design Shear	41.355 psi				
	Allowable Shear	288.0 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.613	0.003602	PASS	0.0 ft	0.0	PASS	13.0 ft
+D+0.60W	1.600	0.401	0.5565	PASS	6.980 ft	0.1436	PASS	13.0 ft
+D+0.450W	1.600	0.401	0.4173	PASS	6.980 ft	0.1077	PASS	13.0 ft
+0.60D+0.60W	1.600	0.401	0.5558	PASS	6.980 ft	0.1436	PASS	13.0 ft
+0.60D	1.600	0.401	0.001860	PASS	0.0 ft	0.0	PASS	13.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only						0.061				

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 13'-0" tall Door King Studs with 45 psf

Note: Only non-zero reactions are listed.

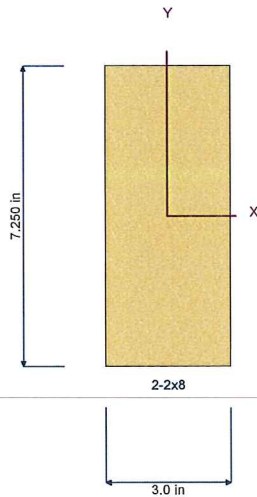
Maximum Reactions

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+D+0.60W				0.577	0.600	0.061					
+D+0.450W				0.432	0.450	0.061					
+0.60D+0.60W				0.577	0.600	0.037					
+0.60D						0.037					
W Only				0.961	0.999						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.443 in	6.631 ft
+D+0.450W	0.0000 in	0.000 ft	0.332 in	6.631 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.443 in	6.631 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.738 in	6.631 ft

Sketches





NELSON ENGINEERING

Professional Engineers & Land Surveyors

JACKSON, WY • BUFFALO, WY

Project Temporary Fire House Job no. 17-202-01

Components & Cladding sheet no. _____ of _____

By CH Date _____

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Full height Studs

$$I_x = 47.63 \text{ in}^4$$

Max Studs : 18'-10" height

Wind Loading: 36.2 psf (Zone 4)

$$@ 16" \text{ Spacing} = 48.27 \text{ plf} \div 12 \text{ in/ft} = 4.02 \text{ lb/in}$$

$$\Delta: \frac{5wL^4}{384EI} = \frac{5 \times 4.02 \text{ lb/in} \times (226)^4}{384 \times 1,600,000 \text{ psi} \times 47.63 \text{ in}^4} = 1.792"$$

DF-L #2

$$226 / 1.792 = L/126$$

$$12" \text{ Spacing: } 36.2 \text{ psf} \div 12 \text{ in/ft} = 3.02 \text{ lb/in}$$

$$\Delta = \frac{5 \times 3.02 \text{ lb/in} \times (226)^4}{384 \times 1,600,000 \text{ psi} \times 47.63 \text{ in}^4} = 1.346"$$

$$226 / 1.346 = L/168$$

2 in. x 8 in. x 20 ft. Premium #2 and Better Douglas Fir Lumber

★★★★★ [Write the first Review](#) [Questions & Answers \(3\)](#)

- Meets high grading standards for strength and appearance
- Easy to cut, fasten and paint, perfect for many building projects
- Renewable resource from environmentally managed forests

\$15²⁶ /each



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Product Overview

- Meets high grading standards for strength and appearance

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Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 18'-0" tall 2x8 wood studs at 16" spacing with 36.2 psf

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		2x8	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber	
Overall Column Height		18.0 ft		Wood Member Type		Sawn	
(Used for non-slender calculations)							
Wood Species		Douglas Fir - Larch		Exact Width		1.50 in	
Wood Grade		No.1		Exact Depth		7.250 in	
Fb +		1,000.0 psi	Fv	180.0 psi	Area		10.875 in^2
Fb -		1,000.0 psi	Ft	675.0 psi	Ix		47.635 in^4
Fc - Prll		1,500.0 psi	Density	31.20 pcf	Iy		2.039 in^4
Fc - Perp		625.0 psi					
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			
Basic		1,700.0	1,700.0	1,700.0 ksi			
Minimum		620.0	620.0				
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis :		Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, 16 ft, K = 1.0	
				Y-Y (depth) axis :		Unbraced Length for X-X Axis buckling = 18 ft, K = 1.0	
						Allow Stress Modification Factors	
						Cf or Cv for Bending	1.20
						Cf or Cv for Compression	1.050
						Cf or Cv for Tension	1.20
						Cm : Wet Use Factor	1.0
						Ct : Temperature Factor	1.0
						Cfu : Flat Use Factor	1.0
						Kf : Built-up columns	1.0 NDS 15.3.2
						Use Cr : Repetitive ?	Yes

Applied Loads

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

Column self weight included: 42.413 lbs * Dead Load Factor

BENDING LOADS...

Lat. Uniform Load creating Mx-x, W = 0.04830 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.5826 : 1	Maximum SERVICE Lateral Load Reactions . .				
	Load Combination	+D+0.60W	Top along Y-Y	0.4347 k	Bottom along Y-Y	0.4347 k	
	Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X	0.0 k	
	Location of max.above base	9.060 ft	Maximum SERVICE Load Lateral Deflections . .				
	At maximum location values are . . .		Along Y-Y	1.424 in	at	9.060 ft above base	
	Applied Axial	0.04241 k	for load combination : W Only				
	Applied Mx	1.174 k-ft	Along X-X	0.0 in	at	0.0 ft above base	
	Applied My	0.0 k-ft	for load combination : n/a				
Fc : Allowable	475.570 psi	Other Factors used to calculate allowable stresses . .					
PASS	Maximum Shear Stress Ratio =	0.1249 : 1	<u>Bending</u>			<u>Compression</u>	<u>Tension</u>
	Load Combination	+D+0.60W					
	Location of max.above base	18.0 ft					
	Applied Design Shear	35.975 psi					
	Allowable Shear	288.0 psi					

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.321	0.008576	PASS	0.0 ft	0.0	PASS	18.0 ft
+D+0.60W	1.600	0.189	0.5826	PASS	9.060 ft	0.1249	PASS	18.0 ft
+D+0.450W	1.600	0.189	0.4370	PASS	8.940 ft	0.09369	PASS	0.0 ft
+0.60D+0.60W	1.600	0.189	0.5809	PASS	9.060 ft	0.1249	PASS	18.0 ft
+0.60D	1.600	0.189	0.004920	PASS	0.0 ft	0.0	PASS	18.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only					0.042				
+D+0.60W			0.261	0.261	0.042				

Nelson Engineering
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Engineer: WAA
Project Descr:

Project ID: 17-202-01

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee : nelson engineering

Description : 18'-0" tall 2x8 wood studs at 16" spacing with 36.2 psf

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+D+0.450W				0.196	0.196	0.042					
+0.60D+0.60W				0.261	0.261	0.025					
+0.60D						0.025					
W Only				0.435	0.435						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.854 in	9.060 ft
+D+0.450W	0.0000 in	0.000 ft	0.641 in	9.060 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.854 in	9.060 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	1.424 in	9.060 ft

Sketches



Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\ISU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 20'-0" tall 2x8 wood studs at 16" spacing with 36.2 psf

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method : Allowable Stress Design				Wood Section Name 2x8	
End Fixities Top & Bottom Pinned				Wood Grading/Manuf. Graded Lumber	
Overall Column Height		20.0 ft		Wood Member Type Sawn	
(Used for non-slender calculations)					
Wood Species Douglas Fir - Larch		Exact Width 1.50 in Allow Stress Modification Factors			
Wood Grade No.1		Exact Depth 7.250 in Cf or Cv for Bending 1.20			
Fb +	1,000.0 psi	Fv	180.0 psi	Area	10.875 in^2 Cf or Cv for Compression 1.050
Fb -	1,000.0 psi	Ft	675.0 psi	Ix	47.635 in^4 Cf or Cv for Tension 1.20
Fc - Prll	1,500.0 psi	Density	31.20 pcf	Iy	2.039 in^4 Cm : Wet Use Factor 1.0
Fc - Perp	625.0 psi				Ct : Temperature Factor 1.0
E : Modulus of Elasticity ...		x-x Bending	y-y Bending	Axial	Cfu : Flat Use Factor 1.0
	Basic	1,700.0	1,700.0	1,700.0 ksi	Kf : Built-up columns 1.0 NDS 15.3.2
	Minimum	620.0	620.0		Use Cr : Repetitive ? Yes
Brace condition for deflection (buckling) along columns :					
		X-X (width) axis : Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, 16 ft, K = 1.0			
		Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 20 ft, K = 1.0			

Applied Loads

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

Column self weight included: 47.125 lbs * Dead Load Factor
BENDING LOADS ...
Lat. Uniform Load creating Mx-x, W = 0.04830 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.7211 : 1	Maximum SERVICE Lateral Load Reactions . .			
Load Combination	+D+0.60W	Top along Y-Y	0.4830 k	Bottom along Y-Y	0.4830 k
Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X	0.0 k
Location of max. above base	9.933 ft	Maximum SERVICE Load Lateral Deflections . .			
At maximum location values are ...		Along Y-Y	2.170 in	at	10.067 ft above base
Applied Axial	0.04713 k	for load combination: W Only			
Applied Mx	1.449 k-ft	Along X-X	0.0 in	at	0.0 ft above base
Applied My	0.0 k-ft	for load combination: n/a			
Fc : Allowable	445.897 psi	Other Factors used to calculate allowable stresses . .			
PASS Maximum Shear Stress Ratio =	0.1388 : 1				
Load Combination	+D+0.60W	Bending Compression Tension			
Location of max. above base	20.0 ft				
Applied Design Shear	39.972 psi				
Allowable Shear	288.0 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.302	0.01012	PASS	0.0 ft	0.0	PASS	20.0 ft
+D+0.60W	1.600	0.177	0.7211	PASS	9.933 ft	0.1388	PASS	20.0 ft
+D+0.450W	1.600	0.177	0.5408	PASS	10.067 ft	0.1041	PASS	20.0 ft
+0.60D+0.60W	1.600	0.177	0.7183	PASS	9.933 ft	0.1388	PASS	20.0 ft
+0.60D	1.600	0.177	0.005831	PASS	0.0 ft	0.0	PASS	20.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top	@ Base	@ Top
D Only						0.047				
+D+0.60W				0.290	0.290	0.047				

Nelson Engineering
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Wood Column

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ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 20'-0" tall 2x8 wood studs at 16" spacing with 36.2 psf

Note: Only non-zero reactions are listed.

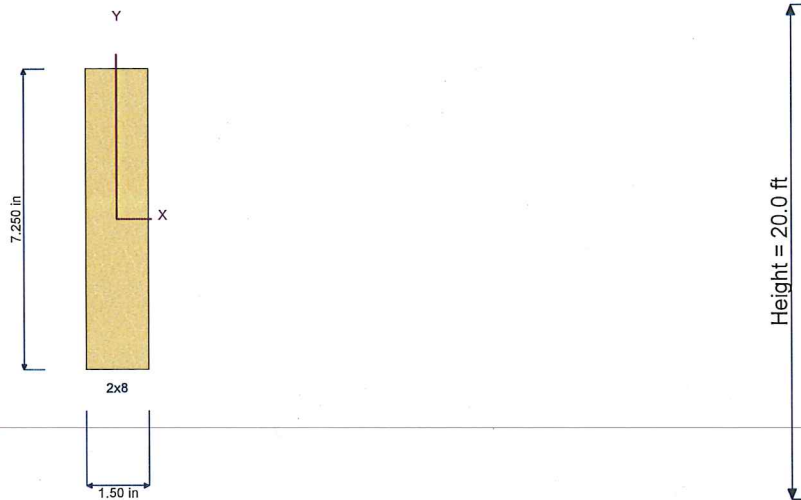
Maximum Reactions

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+D+0.450W				0.217	0.217	0.047					
+0.60D+0.60W				0.290	0.290	0.028					
+0.60D						0.028					
W Only				0.483	0.483						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	1.302 in	10.067 ft
+D+0.450W	0.0000 in	0.000 ft	0.977 in	10.067 ft
+0.60D+0.60W	0.0000 in	0.000 ft	1.302 in	10.067 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	2.170 in	10.067 ft

Sketches



Stud wall frame to Purlin Ldg

Wind Ldg: Use $36.2 \text{ psf} \times \sim \frac{1.6}{2} = 362 \text{ psf}$
 (Tallest Section)

Existing Purlins @ $\sim 3'-9" \text{ o.c. } \pm$

$362 \text{ psf} \times \sim 4' = 1448^\#$ Max Rv @ Purlins

(2) Screws

If 20 ga. Purlins: #7 = $500^\#$ Shear
 #8-10: $565^\#$ Shear

#12 $2\frac{1}{2}$

$2\frac{3}{4}$ - Gives $1\frac{1}{4}$ into Purlin

Purlins Likely at least 16 Gauge Based on Span Charts

16 ga ~ 0.0625

$\Rightarrow$ Allow Shear $\sim 430^\#$ /screw

0.6 WL
 $1448^\# \times 0.6 = 868.8^\#$ /Pulin (conserv)

$\div 2$ Screws = $434.4^\#$ /screw

-OK-

20' is worst case @ Ridge & won't happen
 along length of Endwall

TABLE 4—ALLOWABLE SHEAR (BEARING) CAPACITY (P_{NS}/Ω), pounds-force^{1, 2, 3, 4, 5}

Steel $F_u = 45$ ksi, Applied Factor of Safety, $\Omega=3.0$													
Screw Designation	Nominal Diameter (in.)	Design Thickness of Member Not in Contact with the Screw Head (in)	Design Thickness of Member in Contact with the Screw Head (in)										
			0.018	0.024	0.030	0.036	0.048	0.060	0.075	0.105	0.125	0.187	0.250
10-16	0.190	0.018	66	66	66	66	66	66	66	66	66		
		0.024	102	102	102	102	102	102	102	102	102		
		0.030	111	143	143	143	143	143	143	143	143		
		0.036	120	152	185	188	188	188	188	188	188		
		0.048	139	168	199	228	289	289	289	289	289		
		0.060	139	185	213	239	327	404	404	404	404		
		0.075	139	185	231	251	337	427	564	564	564		
		0.105	139	185	231	277	356	436	570	808	808		
12-14 12-24	0.216	0.018	71	71	71	71	71	71	71	71	71	71	71
		0.024	109	109	109	109	109	109	109	109	109	109	109
		0.030	125	152	152	152	152	152	152	152	152	152	152
		0.036	136	170	205	200	200	200	200	200	200	200	200
		0.048	157	190	223	253	308	308	308	308	308	308	308
		0.060	157	210	240	266	362	430	430	430	430	430	430
		0.075	157	210	262	282	375	468	601	601	601	601	601
		0.105	157	210	262	315	402	483	624	919	919	919	919
		0.125	157	210	262	315	420	494	629	919	1094	1094	1094
		0.187	157	210	262	315	420	525	642	919	1094	1636	1636
1/4-14 1/4-28 ⁶	0.250	0.018	76	76	76	76	76	76	76	76	76	76	76
		0.024	117	117	117	117	117	117	117	117	117	117	117
		0.030	142	164	164	164	164	164	164	164	164	164	164
		0.036	156	193	215	215	215	215	215	215	215	215	215
		0.048	182	218	253	283	331	331	331	331	331	331	331
		0.060	182	243	276	300	406	463	463	463	463	463	463
		0.075	182	243	304	322	424	521	647	647	647	647	647
		0.105	182	243	304	365	461	544	694	1063	1063	1063	1063
		0.125	182	243	304	365	486	560	703	1063	1266	1266	1266
		0.187	182	243	304	365	486	608	731	1063	1266	1893	1893
		0.250	182	243	304	365	486	608	759	1063	1266	1893	2531

For SI: 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

¹The lower of the allowable shear (bearing) and the allowable fastener shear strength found in Tables 4 and 5, respectively, must be used for design.²ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.³The allowable bearing capacity for other member thickness can be determined by interpolating within the table.⁴To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.⁵For $F_u = 58$ ksi, multiply values by 1.29; for $F_u = 65$ ksi, multiply values by 1.44.⁶Shear values do not apply to 5, 6 and 8-inch-long 1/4-28 screws, due to the fact that they are not fully threaded.TABLE 5—FASTENER STRENGTH OF SCREWS^{1, 2, 3, 4, 5}

SCREW DESIGNATION	DIAMETER (in.)	ALLOWABLE FASTENER STRENGTH		NOMINAL FASTENER STRENGTH	
		Tensile, P_{ts}/Ω (lb)	Shear, P_{ss}/Ω (lb)	Tensile, P_{ts} (lb)	Shear, P_{ss} (lb)
10-16	0.190	885	573	2654	1718
12-14	0.216	1184	724	3551	2171
12-24	0.216	1583	885	4750	2654
1/4-14	0.250	1605	990	4816	2970
1/4-28	0.250	1922	1308	5767	3925

For SI: 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

¹For tension connections, the least of the allowable pull-out, pullover, and fastener tension strength found in Tables 2, 3, and 5, respectively, must be used for design.²For shear connection, the lower of the allowable shear (bearing) and the allowable fastener shear strength found in Table 4 and 5, respectively, must be used for design.³See Section 4.1 for fastener spacing and end distance requirements.⁴Nominal strengths are based on laboratory tests;⁵To calculate LRFD values, multiply nominal strength values by the LRFD Φ factor of 0.5.

TABLE 1—TESK® SELF-DRILLING TAPPING SCREWS¹

DESCRIPTION (nom. size-tpi x length)	NOMINAL DIAMETER (inch)	HEAD STYLE	HEAD DIAMETER (inch)	DRILL POINT	DRILLING CAPACITY ³ (in.)		LENGTH OF LOAD BEARING AREA ⁴ (inch)	COATING
					Min.	Max.		
10-16 x $\frac{3}{4}$ "	0.190	HWH	0.400	TEKS/1	0.018	0.095	0.220	Climaseal
12-14 x $\frac{3}{4}$ "	0.216	HWH	0.415	TEKS/1	0.018	0.095	0.205	Climaseal
$\frac{1}{4}$ -14 x $\frac{7}{8}$ "	0.250	HWH	0.415	TEKS/1	0.018	0.095	0.380	Climaseal
10-16 x $\frac{1}{2}$ "	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.150	Climaseal
10-16 x $\frac{5}{8}$ "	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.200	Climaseal
10-16 x $\frac{3}{4}$ "	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.325	Climaseal
10-16 x $\frac{1}{2}$ "	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.150	Climaseal
10-16 x $\frac{5}{8}$ "	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.200	Climaseal
10-16 x $\frac{3}{4}$ "	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.325	Climaseal
10-16 x 1"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.575	Climaseal
10-16 x 1"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.575	Climaseal
10-16 x $1\frac{1}{4}$ "	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.825	Climaseal
10-16 x $1\frac{1}{2}$ "	0.190	HWH	0.400	TEKS/3	0.036	0.175	1.075	Climaseal
10-16 x $\frac{3}{4}$ "	0.190	HWH ²	0.435	TEKS/3	0.036	0.175	0.323	E-Zinc
12-14 x $\frac{3}{4}$ "	0.216	HWH	0.415	TEKS/3	0.036	0.210	0.270	Climaseal
12-14 x 1"	0.216	HWH	0.415	TEKS/3	0.036	0.210	0.520	Climaseal
12-14 x $1\frac{1}{4}$ "	0.216	HWH	0.415	TEKS/2	0.036	0.210	0.550	Climaseal
12-14 x $1\frac{1}{2}$ "	0.216	HWH	0.415	TEKS/2	0.036	0.210	0.800	Climaseal
12-14 x 2"	0.216	HWH	0.415	TEKS/3	0.036	0.210	1.450	Climaseal
12-14 x $2\frac{1}{2}$ "	0.216	HWH	0.415	TEKS/3	0.036	0.210	1.950	Climaseal
12-14 x 3"	0.216	HWH	0.415	TEKS/3	0.036	0.210	2.450	Climaseal
12-14 x 4"	0.216	HWH	0.415	TEKS/3	0.036	0.210	3.450	Climaseal
$\frac{1}{4}$ -14 x $\frac{3}{4}$ "	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.210	Climaseal
$\frac{1}{4}$ -14 x 1"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.400	Climaseal
$\frac{1}{4}$ -14 x $1\frac{1}{4}$ "	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.650	Climaseal
$\frac{1}{4}$ -14 x $1\frac{1}{2}$ "	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.900	Climaseal
$\frac{1}{4}$ -14 x 2"	0.250	HWH	0.500	TEKS/3	0.036	0.210	1.400	Climaseal
$\frac{1}{4}$ -14 x $2\frac{1}{2}$ "	0.250	HWH	0.500	TEKS/3	0.036	0.210	1.900	Climaseal
$\frac{1}{4}$ -14 x 3"	0.250	HWH	0.500	TEKS/3	0.036	0.210	2.400	Climaseal
$\frac{1}{4}$ -14 x 4"	0.250	HWH	0.500	TEKS/3	0.036	0.210	3.400	Climaseal
$\frac{1}{4}$ -14 x $\frac{3}{4}$ "	0.250	HWH ²	0.610	TEKS/3	0.036	0.210	0.250	Climaseal
$\frac{1}{4}$ -14 x 1"	0.250	HWH ²	0.610	TEKS/3	0.036	0.210	0.500	Climaseal
12-24 x $\frac{7}{8}$ "	0.216	HWH	0.415	TEKS/4	0.125	0.250	0.325	Climaseal
12-24 x $1\frac{1}{4}$ "	0.216	HWH	0.415	TEKS/4.5	0.125	0.375	0.575	Climaseal
12-24 x $1\frac{1}{4}$ "	0.216	HWH	0.415	TEKS/5	0.125	0.500	0.375	Climaseal
12-24 x $1\frac{1}{2}$ "	0.216	HWH	0.415	TEKS/5	0.125	0.500	0.625	Climaseal
12-24 x 2"	0.216	HWH	0.415	TEKS/5	0.125	0.500	1.125	Climaseal
$\frac{1}{4}$ -28 x 3"	0.250	HWH	0.415	TEKS/5	0.125	0.500	2.150	Climaseal
$\frac{1}{4}$ -28 x 4"	0.250	HWH	0.415	TEKS/5	0.125	0.500	3.150	Climaseal
$\frac{1}{4}$ -28 x 5 ^{ns}	0.250	HWH	0.605	TEKS/5	0.125	0.500	4.150	Climaseal
$\frac{1}{4}$ -28 x 6 ^{ns}	0.250	HWH	0.605	TEKS/5	0.125	0.500	5.150	Climaseal
$\frac{1}{4}$ -28 x 8 ^{ns}	0.250	HWH	0.605	TEKS/5	0.125	0.500	7.150	Climaseal

For SI: 1 inch = 25.4 mm.

¹ Screw dimensions comply with ASME B18.6.4 (nom. size = nominal screw size, tip = threads per inch, length = inches).² HWH with serrations.³ Drilling capacity refers to the minimum and maximum total allowable thicknesses of material the fastener is designed to drill through, including any space between the layers.⁴ Length of load-bearing area is the total screw length minus the length from the screw point to the third full thread. See Figure 4.⁵ Partially threaded.

Specifications

Dimensions

Screw Length	2-3/4 in
--------------	----------

Details

ACQ Rated Fastener	Yes
--------------------	-----

Coated	N
--------	---

Drive Style	Phillips
-------------	----------

Driver bit included	No
---------------------	----

Fastener Callout Size	#12 x 2-3/4 In.
-----------------------	-----------------

Fastener Plating	Zinc
------------------	------

Fastener Thread Type	Fine
----------------------	------

Fastener Type	Self-Drilling Screw
---------------	---------------------

Fastener/Connector Material	Steel
-----------------------------	-------

Finish	Zinc
--------	------

Finish Family	Metallic
---------------	----------

Head Style	Flat
------------	------

Indoor/Outdoor	Indoor
----------------	--------

Measurement Standard	SAE
----------------------	-----

Package Quantity	40
------------------	----

Primary Use	Wood to metal fastening
-------------	-------------------------

Product Weight (lb.)	.02lb
----------------------	-------

Returnable	90-Day
------------	--------

Self drilling	Yes
---------------	-----

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Teks #12 2-3/4 in. Phillips Flat-Head Self-Drilling Screws (40-Pack)

★★★★★ (8)

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- Secure fastening without the need to pre-drill
- No tool slippage or cam-outs
- Drill, tap and fasten in one motion

\$7⁷⁸
/each

Quantity

-

1

+

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Self-Drilling Screws 3.6.2

3.6.2.1 Product Description

Hilti self-drilling screws are designed to drill their own hole in steel base materials up to 1/2" thick. These screws are available in a variety of head styles, thread lengths and drill-flute lengths for screw diameters #6 through 1/4". Hilti self-drilling screws meet ASTM C1513, ASTM C954 and SAE J78 standards, as applicable.

Product Features:

- Hex head for metal-to-metal applications
- Flush head for wood-to-metal applications
- For metal from 0.035" to 0.500" thick
- Winged reamers for wood over 1/2" thick
- Stitch screws for light gauge metal-to-metal
- Sealing screws for water resistant fastenings

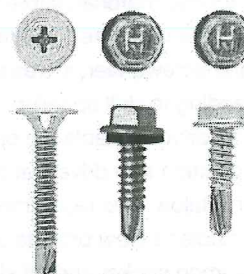
3.6.2.1 Product Description

3.6.2.2 Material Specifications

3.6.2.3 Technical Data

3.6.2.4 Installation Instructions

3.6.2.5 Ordering Information



3.6.2.2 Material Specifications

Material	ASTM A510 Grade 1018-1022
Heat Treatment	Case hardened and tempered • Sizes 8, 10 and 12: 0.004" to 0.009" case depth • Size 1/4": 0.005" to 0.011" case depth
Plating	Refer to Section 3.6.2.5 for screw coating information.

Listings/Approvals

ICC-ES (International Code Council)
ESR-2196
COLA (City of Los Angeles)
RR 25678



Warning: Because of the potential for delayed hydrogen assisted stress corrosion cracking, many hardened steel fasteners are not recommended for use with dissimilar metals or chemically treated wood when moisture may be present or in corrosive environments. For further information, contact Hilti Technical Support at 1-877-749-6337.

ICC-ES ESR-2196, provides IBC recognition of Hilti's Self-Drilling Screw fasteners for most common applications (e.g. CFS connections, gypsum to CFS, etc.), including HWH, HHWH, PPH, PPFH, PBH, PWH, PTH, PPCH, TPCH and PFTH head style screws.

3.6.2.3 Technical Data

Ultimate Tensile Strengths – Pullout (Tension), lb (kN)^{1,2,3,4,5,6,7}

Screw Designation	Nominal Diameter in.	Thickness of steel member not in contact with the screw head, ga (in.)					
		20 (0.036)	18 (0.048)	16 (0.060)	14 (0.075)	12 (0.105)	10 (0.135)
#6	0.138	190 (0.85)	250 (1.11)	320 (1.42)	395 (1.76)	555 (2.47)	715 (3.18)
#7	0.151	210 (0.93)	275 (1.22)	345 (1.53)	435 (1.93)	605 (2.69)	780 (3.47)
#8	0.164	225 (1.00)	300 (1.33)	375 (1.67)	470 (2.09)	660 (2.94)	845 (3.76)
#10	0.190	260 (1.16)	350 (1.56)	435 (1.93)	545 (2.42)	765 (3.40)	980 (4.36)
#12	0.216	295 (1.31)	395 (1.76)	495 (2.20)	620 (2.76)	870 (3.87)	1120 (4.98)
1/4 in.	0.250	345 (1.53)	460 (2.05)	575 (2.56)	715 (3.18)	1000 (4.45)	1290 (5.74)

- 1 The lower of the ultimate pullout, pullover, and tension fastener strength of screw should be used for design.
- 2 Load values based upon calculations done in accordance with Section E4 of the AISI S100.
- 3 AISI S100 recommends a safety factor of 3.0 be applied for allowable strength design, a Φ factor of 0.5 be applied for LRFD design or a Φ factor of 0.4 be applied for LSD design.
- 4 ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.
- 5 The screw diameters in the table above are available in head styles of pan, hex washer, pancake, flat, wafer and bugle.
- 6 The load data in the table is based upon sheet steel with $F_u = 45$ ksi. For $F_u = 55$ ksi steel, multiply values by 1.22. For $F_u = 65$ ksi steel, multiply values by 1.44.
- 7 Refer to Section 3.6.2.5 to ensure drilling capacities.

Self-Drilling Screws 3.6.2

Ultimate Shear Strengths – Bearing (Shear), lb (kN)^{1,2,3,4,5,6,7}

Screw Designation	Nominal Diameter in.	Thickness of steel member in contact with screw head ga (in.)	Thickness of steel member not in contact with the screw head, ga (in.)				
			20 (0.036)	18 (0.048)	16 (0.060)	14 (0.075)	≥ 12 (0.105)
#7	0.151	20 (0.036)	500 (2.22)	660 (2.94)	660 (2.94)	660 (2.94)	660 (2.94)
		18 (0.048)	500 (2.22)	660 (2.94)	880 (3.91)	880 (3.91)	880 (3.91)
		≥ 16 (0.060)	500 (2.22)	660 (2.94)	890 (3.96)	890 (3.96)	890 (3.96)
#8	0.164	20 (0.036)	525 (2.34)	715 (3.18)	715 (3.18)	715 (3.18)	715 (3.18)
		18 (0.048)	525 (2.34)	805 (3.58)	955 (4.25)	955 (4.25)	955 (4.25)
		≥ 16 (0.060)	525 (2.34)	805 (3.58)	1120 (4.98)	1170 (5.20)	1170 (5.20)
#10-12	0.190	20 (0.036)	565 (2.51)	830 (3.69)	830 (3.69)	830 (3.69)	830 (3.69)
		18 (0.048)	565 (2.51)	865 (3.85)	1110 (4.94)	1110 (4.94)	1110 (4.94)
		16 (0.060)	565 (2.51)	865 (3.85)	1210 (5.38)	1390 (6.18)	1390 (6.18)
		≥ 14 (0.075)	565 (2.51)	865 (3.85)	1210 (5.38)	1645 (7.32)	1645 (7.32)
#10-16	0.190	20 (0.036)	565 (2.51)	830 (3.69)	830 (3.69)	830 (3.69)	830 (3.69)
		18 (0.048)	565 (2.51)	865 (3.85)	1110 (4.94)	1110 (4.94)	1110 (4.94)
		≥ 16 (0.060)	565 (2.51)	865 (3.85)	1210 (5.38)	1215 (5.40)	1215 (5.40)
#10-18	0.190	20 (0.036)	565 (2.51)	830 (3.69)	830 (3.69)	830 (3.69)	830 (3.69)
		18 (0.048)	565 (2.51)	865 (3.85)	1110 (4.94)	1110 (4.94)	1110 (4.94)
		16 (0.060)	565 (2.51)	865 (3.85)	1210 (5.38)	1390 (6.18)	1390 (6.18)
		≥ 14 (0.075)	565 (2.51)	865 (3.85)	1210 (5.38)	1645 (7.32)	1645 (7.32)
#12-14	0.216	20 (0.036)	600 (2.67)	930 (4.14)	945 (4.20)	945 (4.20)	945 (4.20)
		18 (0.048)	600 (2.67)	925 (4.11)	1260 (5.60)	1260 (5.60)	1260 (5.60)
		16 (0.060)	600 (2.67)	925 (4.11)	1290 (5.74)	1570 (6.98)	1570 (6.98)
		≥ 14 (0.075)	600 (2.67)	925 (4.11)	1290 (5.74)	1800 (8.00)	1880 (8.36)
#12-24	0.216	20 (0.036)	600 (2.67)	930 (4.14)	945 (4.20)	945 (4.20)	945 (4.20)
		18 (0.048)	600 (2.67)	925 (4.11)	1260 (5.60)	1260 (5.60)	1260 (5.60)
		16 (0.060)	600 (2.67)	925 (4.11)	1290 (5.74)	1570 (6.98)	1570 (6.98)
		14 (0.075)	600 (2.67)	925 (4.11)	1290 (5.74)	1800 (8.00)	1970 (8.76)
		≥ 12 (0.090)	600 (2.67)	925 (4.11)	1290 (5.74)	1800 (8.00)	2285 (10.16)
1/4 in.	0.250	20 (0.036)	645 (2.87)	1020 (4.54)	1090 (4.85)	1090 (4.85)	1090 (4.85)
		18 (0.048)	645 (2.87)	995 (4.43)	1400 (6.23)	1460 (6.49)	1460 (6.49)
		16 (0.060)	645 (2.87)	995 (4.43)	1390 (6.18)	1820 (8.10)	1820 (8.10)
		14 (0.075)	645 (2.87)	995 (4.43)	1390 (6.18)	1940 (8.63)	2280 (10.14)
		≥ 12 (0.090)	645 (2.87)	995 (4.43)	1390 (6.18)	1940 (8.63)	2440 (10.85)

- 1 The lower of the ultimate shear bearing and shear fastener strength of screw should be used for design.
- 2 Load values based upon calculations done in accordance with Section E4 of AISI S100.
- 3 AISI S100 recommends a safety factor of 3.0 be applied for allowable strength design, a Φ factor of 0.5 be applied for LRFD design or a Φ factor of 0.4 be applied for LSD design.
- 4 ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.
- 5 Load values in table are for Hex Washer Head (HWH and HHWH), Phillips Pan Head (PPH), Phillips Truss Head (PTH), Phillips Pancake Head (PPCH), and Phillips Flat Truss Head (PFTH) style screws. Phillips Bugle Head (PBH) and Phillips Wafer Head (PWH) styles are not covered by this table because they are not intended for attachment of steel to steel.
- 6 The load data in the table is based upon sheet steel with $F_u = 45$ ksi. For $F_u = 55$ ksi steel, multiply values by 1.22. For $F_u = 65$ ksi steel, multiply values by 1.44.
- 7 Refer to Section 3.6.2.5 to ensure drilling capacities.

3.6.2.4 Installation Instructions

For general discussion of Hilti screw fastener installation, reference Section 3.6.1.7.

For allowable diaphragm shear loads and stiffness values for steel roof or floor deck utilizing Hilti self-drilling screws as frame or sidelap fasteners, reference Section 3.5 and

download Hilti's Profis DF software at www.us.hilti.com/decking (US); or www.hilti.ca (Canada).

To estimate the number of sidelap screws on a steel roof or floor deck project, reference Section 3.5.1.6.

Warning: Because of the potential for delayed hydrogen assisted stress corrosion cracking, many hardened steel fasteners are not recommended for use with dissimilar metals or chemically treated wood when moisture may be present or in corrosive environments. For further information, contact Hilti Technical Support at 1-877-749-6337.

3.6.2 Self-Drilling Screws

Ultimate Tensile Strengths – Pullover (Tension), lb (kN)^{1,2,3,4,5,6,7}

Screw Designation	Washer or Head Diameter in.	Thickness of steel member in contact with the screw head, ga (in.)						
		22 (0.030)	20 (0.036)	18 (0.048)	16 (0.060)	14 (0.075)	12 (0.105)	10 (0.135)
Hex Washer Head (HWH)								
#8	0.335	675 (3.00)	815 (3.63)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#10	0.399	805 (3.58)	970 (4.31)	1290 (5.74)	1370 (6.09)	1370 (6.09)	1370 (6.09)	1370 (6.09)
#12-14	0.415	835 (3.71)	1010 (4.49)	1340 (5.96)	1680 (7.47)	2100 (9.34)	2325 (10.34)	2325 (10.34)
#12-24	0.415	835 (3.71)	1010 (4.49)	1340 (5.96)	1680 (7.47)	2100 (9.34)	2940 (13.08)	3780 (16.81)
1/4 in.	0.500	1010 (4.49)	1220 (5.43)	1620 (7.21)	2030 (9.03)	2530 (11.25)	3540 (13.75)	4560 (20.28)
Phillips Pan Head (PPH)								
#7	0.303	615 (2.74)	735 (3.27)	980 (4.36)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#8	0.311	630 (2.80)	755 (3.36)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#10	0.364	740 (3.29)	885 (3.94)	1180 (5.25)	1370 (6.09)	1370 (6.09)	1370 (6.09)	1370 (6.09)
Phillips Truss Head (PTH)								
#8	0.433	875 (3.89)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#10	0.411	830 (3.69)	1000 (4.45)	1330 (5.92)	1390 (6.18)	1390 (6.18)	1390 (6.18)	1390 (6.18)
Phillips Pancake Head (PPCH)								
#10, #12	0.409	830 (3.69)	995 (4.43)	1325 (5.89)	1370 (6.09)	1370 (6.09)	1370 (6.09)	1370 (6.09)
Phillips Flat Truss Head (PFTH)								
#10	0.364	740 (3.29)	885 (3.94)	1180 (5.25)	1475 (6.56)	1840 (8.18)	2170 (9.65)	2170 (9.65)

1. The lower of the ultimate pullout, pullover, and tension fastener strength of screw should be used for design.
2. Load values based upon calculations done in accordance with Section E4 of the AISI S100.
3. AISI S100 recommends a safety factor of 3.0 be applied for allowable strength design, a Φ factor of 0.5 be applied for LRFD design or a Φ factor of 0.4 be applied for LSD design.
4. ANSI/ASME standard screw head diameters were used in the calculations and are listed in the tables.
5. Phillips Bugle Head (PBH) and Phillips Wafer Head (PWH) styles are not covered by this table because they are not intended for attachment of steel to steel.
6. The load data in the table is based upon sheet steel with $F_u = 45$ ksi. For $F_u = 55$ ksi steel, multiply values by 1.22. For $F_u = 65$ ksi steel, multiply values by 1.44.
7. Refer to Section 3.6.2.5 for drilling capacities.

Nominal Ultimate Fastener Strength of Screw

Screw Designation	Nominal Diameter (in.)	Nominal Fastener Strength	
		Tension, P_{ts} lb (kN) ¹	Shear, P_{ss} lb (kN) ^{2,3,4}
#6-20	0.138	1000 (4.45)	890 (3.96)
#7-18	0.151	1000 (4.45)	890 (3.96)
#8-18	0.164	1000 (4.45)	1170 (5.20)
#10-12	0.190	2170 (9.65)	1645 (7.32)
#10-16	0.190	1370 (6.09)	1215 (5.40)
#10-18	0.190	1390 (6.18)	1645 (7.32)
#12-14	0.216	2325 (10.34)	1880 (8.36)
#12-24	0.216	3900 (17.35)	2285 (10.16)
1/4 in.	0.250	4580 (20.37)	2440 (10.85)

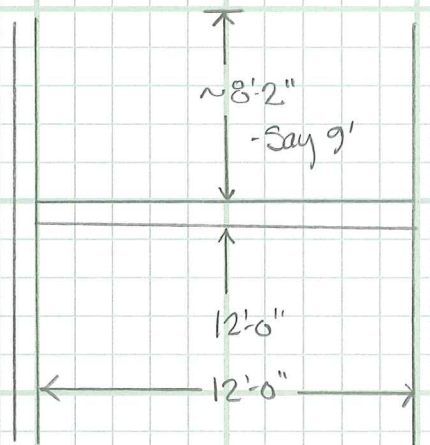
1. The lower of the ultimate pullout, pullover, and tension fastener strength of screw should be used for design. The Pullout and Pullover tables in this section have already been adjusted where screw strength governs.
2. The lower of the ultimate shear fastener strength and shear bearing should be used for design. The Shear Bearing table in this section has already been adjusted where screw strength governs.
3. AISI S100 recommends a safety factor of 3.0 be applied for allowable strength design, a Φ factor of 0.5 be applied for LRFD design or a Φ factor of 0.4 be applied for LSD design.
4. When the distance to the end of the connected part is parallel to the line of the applied force the allowable shear fastener strength must be reduced for end distance, when necessary, in accordance with E4.3.2 of Appendix A of AISI S100.

Torsional Strength^{1,2}

Size	Min. Torsional Strength in-lb (Nm)
6-20	24 (2.7)
7-18	38 (4.3)
8-18	42 (4.8)
10-12	61 (6.9)
10-16	61 (6.9)
10-18	61 (6.9)
10-24	65 (7.3)
12-14	92 (10.4)
12-24	100 (11.3)
1/4-14	150 (17.0)
1/4-20	156 (17.6)

1. Based on screw only. Does not consider base material limitations.
2. Values in table are ultimate torsional strengths. To obtain maximum setting torque, multiply values in table by 0.66.

Endwall w/ Member @ 12'-0" Door



$$\frac{9'}{2} + \frac{12'}{2} = 10.5' \text{ tri} \times 36.2 \text{ psf} = 380.1 \text{ plf}$$

$$M = \frac{380 \text{ plf} \times 12.5^2}{8} = 7,423.83 \text{ lb-ft} \times 0.6 = 4454.3 \text{ lb-ft}$$

$$= 53,452 \text{ lb-in}$$

(3) 2x8

$$S_y = 2.719 \text{ in}^3$$

$$I_y = 2.039 \text{ in}^4$$

$$\times 3 = 8.157$$

$$53,452 \text{ lb-in} \div 2.719 \text{ in}^3 = 19,658 \text{ psi}$$

$$53,452 \text{ lb-in} \div 8.157 \text{ in}^3 = 6552.9 \text{ psi}$$

$$F_b' = 1728 \text{ psi}$$

DF-L #2 8x8

S.R: $M = 0.528$
 $\Delta = 0.299" = 1/502 \quad 0.6 \text{ WL}$

$$R_v = 2.38^k \text{ WL (low L)}$$

HUC88 - Allow = 2085 1.0
 $\times 1.6 = 3336 \checkmark$

• If hdr only spans ~ 7'-0" - (2) 2x8 (Laid flat)

S.R: 0.369
 $R_v = 1,330^{\#} \div 2 = 665^{\#}$

HGA10
 Allow = 1165[#]

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 8:52AM

Wood Beam

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: Header above 12'-0" door openings at Endwall

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set: ASCE 7-10

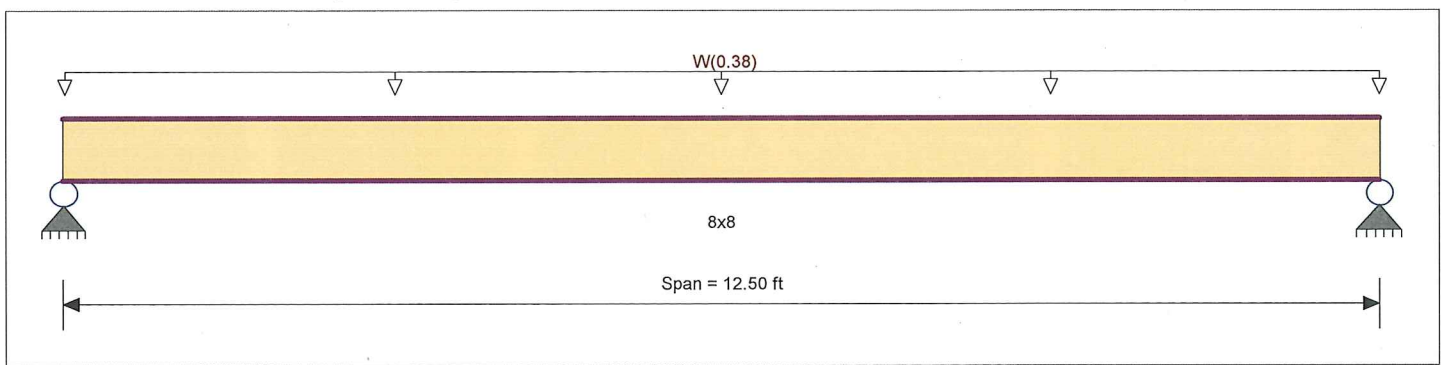
Material Properties

Analysis Method: Allowable Stress Design
Load Combination: ASCE 7-10

Wood Species: Douglas Fir - Larch
Wood Grade: No.2

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

F_b + 900 psi
F_b - 900 psi
F_c - Prll 1350 psi
F_c - Perp 625 psi
F_v 180 psi
F_t 575 psi
E: Modulus of Elasticity
E_{bend-xx} 1600 ksi
E_{minbend-xx} 580 ksi
Density 31.2 pcf



Applied Loads

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

Uniform Load: W = 0.380, Tributary Width = 1.0 ft, (Wind Loading from Zone 4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.528	1	Maximum Shear Stress Ratio	=	0.119	1
Section used for this span		8x8		Section used for this span		8x8	
fb: Actual	=	760.00	psi	fv: Actual	=	34.39	psi
FB: Allowable	=	1,440.00	psi	Fv: Allowable	=	288.00	psi
Load Combination		+0.60W		Load Combination		+0.60W	
Location of maximum on span	=	6.250	ft	Location of maximum on span	=	11.907	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.498	in	Ratio =		301	>=240.
Max Upward Transient Deflection		0.000	in	Ratio =		0	<240.0
Max Downward Total Deflection		0.299	in	Ratio =		502	>=180
Max Upward Total Deflection		0.000	in	Ratio =		0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios								Moment Values			Shear Values			
			M	V	C _d	C _{FN}	C _i	C _r	C _m	C _t	C _L	M	fb	F'b	V	fv	F'v
+0.60W	Length = 12.50 ft	1			0.90	1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
+0.450W	Length = 12.50 ft	1	0.528	0.119	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.45	760.00	1440.00	1.29	34.39	288.00
						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
	Length = 12.50 ft	1	0.396	0.090	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.34	570.00	1440.00	0.97	25.80	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.4977	6.296		0.0000	0.000

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : Header above 12'-0" door openings at Endwall

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.375	2.375
Overall MINimum	1.069	1.069
+0.60W	1.425	1.425
+0.450W	1.069	1.069
W Only	2.375	2.375

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 30 AUG 2017, 7:21PM

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53V\6-TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 7' Header above 12'-0" door openings at Endwall

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

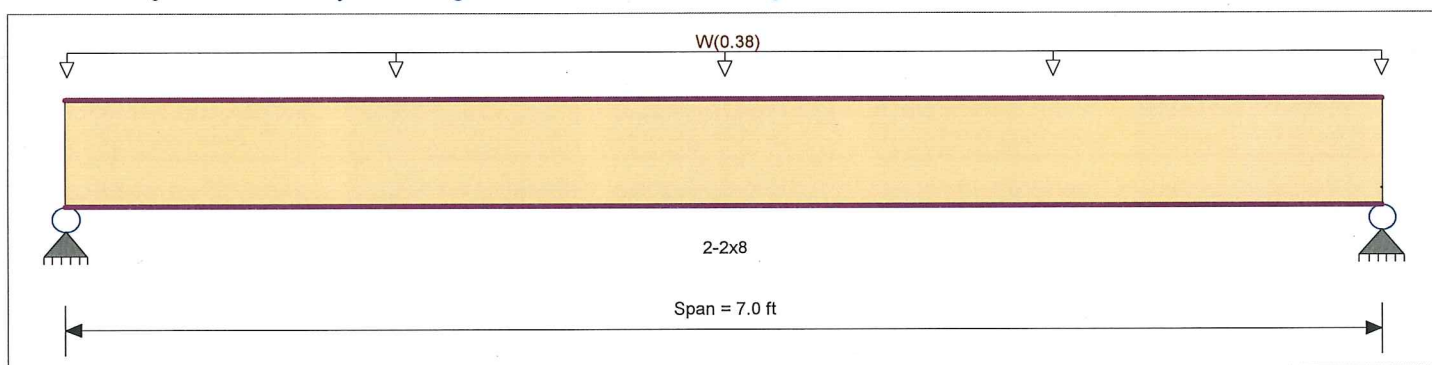
Material Properties

Analysis Method: Allowable Stress Design
Load Combination ASCE 7-10

Wood Species: Douglas Fir - Larch
Wood Grade: No.2

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Fb +	900.0 psi	E: Modulus of Elasticity	
Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Fc - Perp	625.0 psi		
Fv	180.0 psi		
Ft	575.0 psi	Density	31.20 pcf



Applied Loads

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

Uniform Load: W = 0.380, Tributary Width = 1.0 ft, (Wind Loading from Zone 4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.369	1	Maximum Shear Stress Ratio	=	0.159	: 1
Section used for this span		2-2x8		Section used for this span		2-2x8	
fb: Actual	=	637.64	psi	fv: Actual	=	45.80	psi
FB: Allowable	=	1,728.00	psi	Fv: Allowable	=	288.00	psi
Load Combination		+0.60W		Load Combination		+0.60W	
Location of maximum on span	=	3.500	ft	Location of maximum on span	=	6.412	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.135	in	Ratio =		620	>=240.
Max Upward Transient Deflection		0.000	in	Ratio =		0	<240.0
Max Downward Total Deflection		0.081	in	Ratio =		1033	>=180
Max Upward Total Deflection		0.000	in	Ratio =		0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
	Length = 7.0 ft	1			0.90	1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
+0.60W						1.200	1.00	1.00	1.00	1.00	1.00			972.00	0.00	0.00	162.00
	Length = 7.0 ft	1	0.369	0.159	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.40	637.64	1728.00	0.66	45.80	288.00
+0.450W						1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
	Length = 7.0 ft	1	0.277	0.119	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.05	478.23	1728.00	0.50	34.35	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.1355	3.526		0.0000	0.000

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 7' Header above 12'-0" door openings at Endwall

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.330	1.330
Overall MINimum	0.599	0.599
+0.60W	0.798	0.798
+0.450W	0.599	0.599
W Only	1.330	1.330

H Seismic & Hurricane Ties

The hurricane tie series features various configurations of wind and seismic ties for trusses and rafters.

The H16 series has a presloped seat of 5/12 for double trusses.

The presloped 5/12 seat of the H16 provides for a tight fit and reduced deflection. The strap length provides for various truss height up to a maximum of 13 1/2" (H16 series). Minimum heel height for H16 series is 4".

The HGA10 attaches to gable trusses and provides good lateral wind resistance. The HS24 attaches the bottom chord of a truss or rafter at pitches from 0/12 to 4/12 to double 2x4 top plates. Double shear nailing allows for higher lateral resistance.

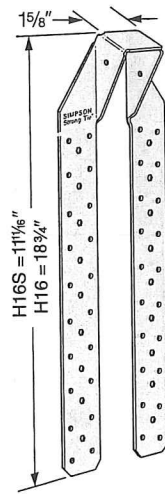
MATERIAL: See table

FINISH: Galvanized. See Corrosion Information, pages 13-15.

INSTALLATION: • Use all specified fasteners. See General Notes.

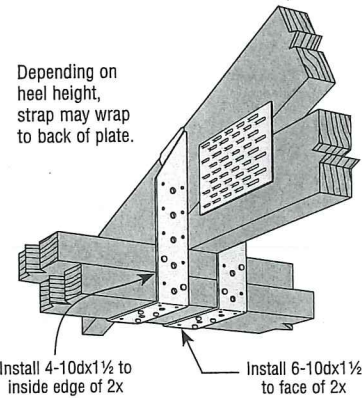
- HGA10KT: sold as a kit with (10) HGA10 connectors.
- Simpson Strong-Tie® Strong-Drive® SDS Heavy-Duty Connector screws are included.
- HS24 requires slant nailing only when bottom chord of truss or rafter has no slope.

CODES: See page 12 for Code Reference Key Chart.

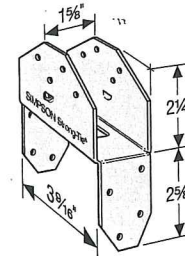


H16 and H16S

Presloped at 5/12. Truss/Rafter Pitch of 3/12 to 7/12 is acceptable

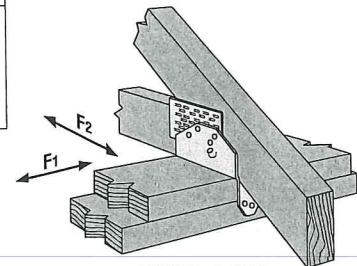


H16 Installation



HS24

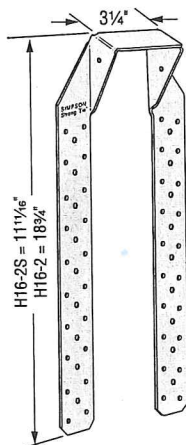
U.S. Patents 5,603,580



HS24 Installation

Model No.	Ga	Fasteners			DF/SP Allowable Loads ¹				SPF/HF Allowable Loads ¹				Code Ref.
		To Rafters/Truss	To Plates	To Studs	Uplift (160)	F ₁	F ₂	F ₃	Uplift (160)	F ₁	F ₂	F ₃	
HGA10KT	14	4-SDS 1/4"x1 1/2"	4-SDS 1/4"x3"	—	695	1165	940	780	500	840	675	495	F26, L18
HS24	18	8-8dx1 1/2 & 2-8d slant	8-8d	—	605 ³	645 ³	1025 ³	—	520 ³	555 ³	880 ³	—	I17, F16, L6
H16	18	2-10dx1 1/2	10-10dx1 1/2	—	1470	—	—	—	1265	—	—	—	F26
H16S	18	2-10dx1 1/2	10-10dx1 1/2	—	1470	—	—	—	1265	—	—	—	
H16-2	18	2-10dx1 1/2	10-10dx1 1/2	—	1470	—	—	—	1265	—	—	—	
H16-2S	18	2-10dx1 1/2	10-10dx1 1/2	—	1470	—	—	—	1265	—	—	—	

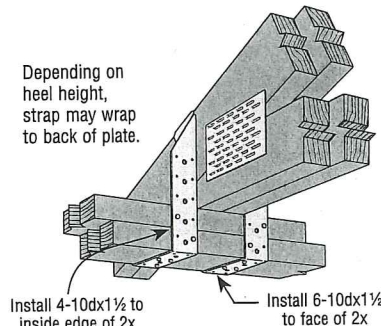
1. Loads have been increased for wind or earthquake loading with no further increase allowed; reduce where other loads govern.
2. When cross-grain bending or cross-grain tension cannot be avoided, mechanical reinforcement to resist such forces should be considered.
3. HS24 DF/SP allowable loads without slant nailing are 605 lbs. (uplift), 590 lbs. (F₁), 640 lbs. (F₂). For SPF/HF loads multiply these values by 0.86.
4. For H16-2S, S = short.
5. Allowable loads in the F₁ direction are not intended to replace diaphragm boundary members or prevent cross grain bending of the truss or rafter members. Additional shear transfer elements shall be considered where there may be effects of cross grain bending or tension.
6. **NAILS:** 10dx1 1/2 = 0.148" dia. x 1 1/2" long, 8d = 0.131" dia. x 2 1/2" long, 8dx1 1/2 = 0.131" dia. x 1 1/2" long. See page 22-23 for other nail sizes and information.



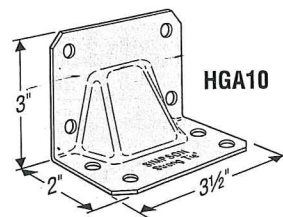
H16-2 and H16-2S

Presloped at 5/12. Pitch of 3/12 to 7/12 is acceptable

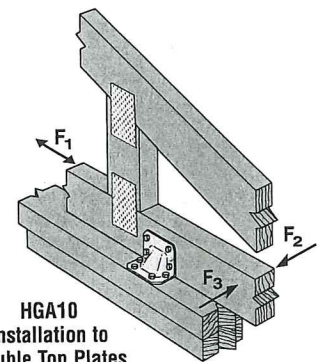
Depending on heel height, strap may wrap to back of plate.



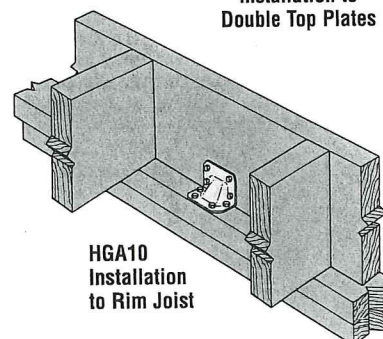
H16-2 Installation



HGA10

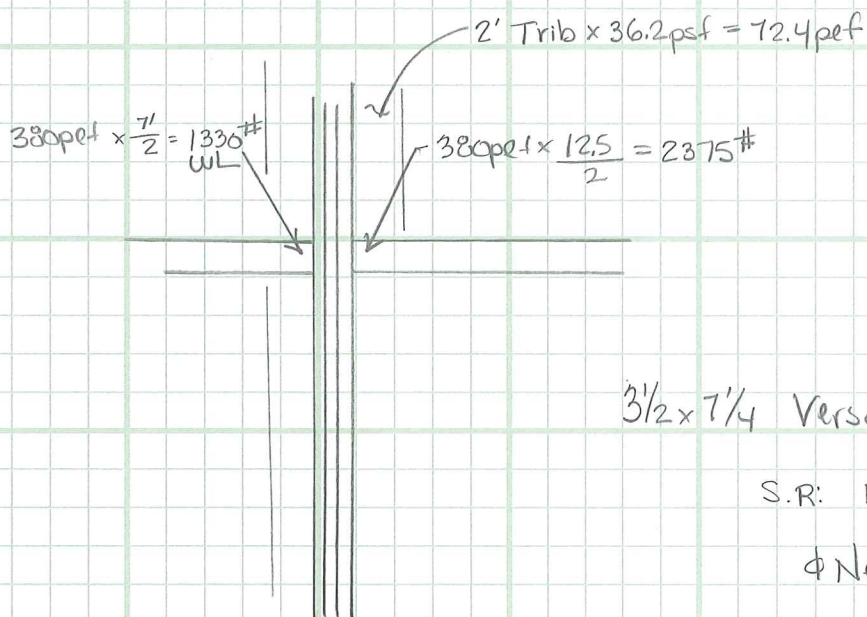


HGA10 Installation to Double Top Plates



HGA10 Installation to Rim Joist

King Studs @ 12'-0" Garage Doors



$3\frac{1}{2} \times 7\frac{1}{4}$ Versa Stud LVL

S.R: $M = 0.9586$

⌀ Needs Blocking @ 4' o.c.

$5\frac{1}{4} \times 7\frac{1}{4}$ Versa Stud LVL

S.R = 0.6516

⌀ OK w/ fully unbraced.

$5\frac{1}{4} \times 7\frac{1}{4}$ Versa Stud

w/ Loading noted: S.R: $M = 0.717$

$V = 0.155$

$\Delta TL = 2.224" = \frac{1}{102}$

$7\frac{1}{4} \times 7\frac{1}{4}$ Versa Stud : S.R: $M = 0.538$

$\Delta TL = 1.668" = \frac{1}{136}$ o.k.w

$> \frac{1}{120}$ -OK

$R_v = 2.05^k$ R_{v1}
 3.03^k R_{v2}

w/ only Garage Door Header: $5\frac{1}{4} \times 7\frac{1}{4}$ LVL S.R- $M = 0.444$

$\Delta TL = 1.362" = \frac{1}{167}$ o.k.w

$3\frac{1}{2} \times 7\frac{1}{4}$ No Good for Slenderness $3\frac{1}{2} \times 7\frac{1}{4}$ $\Delta TL = 2.043" = \frac{1}{111}$ -No Good

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 8:58AM

Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 19'-0" tall Door King Studs with 36.2 psf

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		3.5x7.25	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Boise Cascade	
Overall Column Height		19.0 ft		Wood Member Type		VersaLam	
(Used for non-slender calculations)							
Wood Species		Boise Cascade		Exact Width		3.50 in	
Wood Grade		Versa Lam Column 1.7 2650 V		Exact Depth		7.250 in	
Fb +		2650 psi	Fv	285 psi	Area		25.375 in^2
Fb -		2650 psi	Ft	1500 psi	Ix		111.148 in^4
Fc - Prll		3000 psi	Density	41.75 pcf	Iy		25.904 in^4
Fc - Perp		750 psi					
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			
Basic		1700	1700	1700 ksi			
Minimum		506024096	506024096				
Allow Stress Modification Factors							
Cf or Cv for Bending 1.0							
Cf or Cv for Compression 1.0							
Cf or Cv for Tension 1.0							
Cm : Wet Use Factor 1.0							
Ct : Temperature Factor 1.0							
Cfu : Flat Use Factor 1.0							
Kf : Built-up columns 1.0 NDS 15.3.2							
Use Cr : Repetitive ? Yes							
Brace condition for deflection (buckling) along columns :							
X-X (width) axis :				Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, 16 ft, K = 1.0			
Y-Y (depth) axis :				Unbraced Length for X-X Axis buckling = 19 ft, K = 1.0			

Applied Loads

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

Column self weight included: 139.783 lbs * Dead Load Factor

BENDING LOADS ...

Wind Loading on King Studs: Lat. Uniform Load creating Mx-x, W = 0.07240 k/ft

Header Load 12' Header: Lat. Point Load at 12.0 ft creating Mx-x, W = 2.375 k

Header Load 7' Header: Lat. Point Load at 12.0 ft creating Mx-x, W = 1.330 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.9586 : 1	Maximum SERVICE Lateral Load Reactions . .			
	Load Combination	+D+0.60W	Top along Y-Y	3.028 k	Bottom along Y-Y	2.053 k
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X	0.0 k
	Location of max.above base	11.987 ft	Maximum SERVICE Load Lateral Deflections . .			
	At maximum location values are . . .		Along Y-Y	5.584 in	at	10.074 ft above base
	Applied Axial	0.1398 k	for load combination : W Only			
	Applied Mx	11.643 k-ft	Along X-X	0.0 in	at	0.0 ft above base
	Applied My	0.0 k-ft	for load combination : n/a			
	Fc : Allowable	719.79 psi	Other Factors used to calculate allowable stresses . .			
				<u>Bending</u>	<u>Compression</u>	<u>Tension</u>
PASS	Maximum Shear Stress Ratio =	0.2355 : 1				
	Load Combination	+D+0.60W				
	Location of max.above base	19.0 ft				
	Applied Design Shear	107.390 psi				
	Allowable Shear	456.0 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.262	0.007787	PASS	0.0 ft	0.0	PASS	19.0 ft
+D+0.60W	1.600	0.150	0.9586	PASS	11.987 ft	0.2355	PASS	19.0 ft
+D+0.450W	1.600	0.150	0.7190	PASS	11.987 ft	0.1766	PASS	19.0 ft
+0.60D+0.60W	1.600	0.150	0.9557	PASS	11.987 ft	0.2355	PASS	19.0 ft
+0.60D	1.600	0.150	0.004592	PASS	0.0 ft	0.0	PASS	19.0 ft

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1~\ISU53VC-6\TDLLGK~A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 19'-0" tall Door King Studs with 36.2 psf

Maximum Reactions

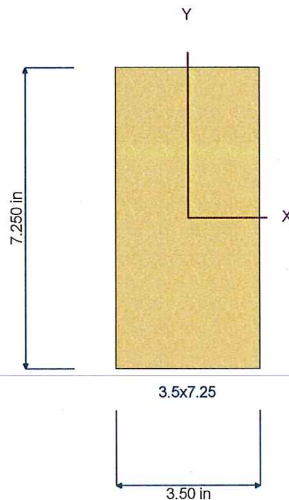
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						0.140					
+D+0.60W				1.232	1.817	0.140					
+D+0.450W				0.924	1.363	0.140					
+0.60D+0.60W				1.232	1.817	0.084					
+0.60D						0.084					
W Only				2.053	3.028						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	3.351 in	10.074 ft
+D+0.450W	0.0000 in	0.000 ft	2.513 in	10.074 ft
+0.60D+0.60W	0.0000 in	0.000 ft	3.351 in	10.074 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	5.584 in	10.074 ft

Sketches



Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 19'-0" tall Door King Studs with 36.2 psf

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method : Allowable Stress Design			Wood Section Name		5.25x7.25	
End Fixities Top & Bottom Pinned			Wood Grading/Manuf.		Boise Cascade	
Overall Column Height 19.0 ft			Wood Member Type		VersaLam	
(Used for non-slender calculations)						
Wood Species Boise Cascade			Exact Width		5.250 in Allow Stress Modification Factors	
Wood Grade Versa Lam Column 1.7 2650 W			Exact Depth		7.250 in Cf or Cv for Bending 1.0	
Fb + 2650 psi Fv 285 psi			Area		38.063 in^2 Cf or Cv for Compression 1.0	
Fb - 2650 psi Ft 1500 psi			Ix		166.722 in^4 Cf or Cv for Tension 1.0	
Fc - Prll 3000 psi Density 41.75 pcf			Iy		87.425 in^4 Cm : Wet Use Factor 1.0	
Fc - Perp 750 psi			Ct : Temperature Factor 1.0			
E : Modulus of Elasticity . . .			Axial		Cfu : Flat Use Factor 1.0	
x-x Bending y-y Bending			1700 ksi		Kf : Built-up columns 1.0 NDS 15.3.2	
Basic 1700 1700			Use Cr : Repetitive ? Yes			
Minimum 506024096 506024096						
Brace condition for deflection (buckling) along columns :						
X-X (width) axis : Unbraced Length for X-X Axis buckling = 19 ft, K = 1.0						
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 19 ft, K = 1.0						

Applied Loads

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

Column self weight included: 209.677 lbs * Dead Load Factor

BENDING LOADS ...

Wind Loading on King Studs: Lat. Uniform Load creating Mx-x, W = 0.07240 k/ft

Header Load 12' Header: Lat. Point Load at 12.0 ft creating Mx-x, W = 2.375 k

Header Load 7' Header: Lat. Point Load at 12.0 ft creating Mx-x, W = 1.330 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.6576 : 1	Maximum SERVICE Lateral Load Reactions ...			
Load Combination	+D+0.60W	Top along Y-Y	3.028 k	Bottom along Y-Y	2.053 k
Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X	0.0 k
Location of max.above base	11.987 ft	Maximum SERVICE Load Lateral Deflections ...			
At maximum location values are ...		Along Y-Y	3.723 in	at	10.074 ft above base
Applied Axial	0.2097 k	for load combination: W Only			
Applied Mx	11.643 k-ft	Along X-X	0.0 in	at	0.0 ft above base
Applied My	0.0 k-ft	for load combination: n/a			
Fc : Allowable	380.819 psi	Other Factors used to calculate allowable stresses ...			
PASS Maximum Shear Stress Ratio =	0.1570 : 1				
Load Combination	+D+0.60W	Bending Compression Tension			
Location of max.above base	19.0 ft				
Applied Design Shear	71.593 psi				
Allowable Shear	456.0 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.140	0.01458	PASS	0.0 ft	0.0	PASS	19.0 ft
+D+0.60W	1.600	0.079	0.6576	PASS	11.987 ft	0.1570	PASS	19.0 ft
+D+0.450W	1.600	0.079	0.4933	PASS	11.987 ft	0.1178	PASS	19.0 ft
+0.60D+0.60W	1.600	0.079	0.6555	PASS	11.987 ft	0.1570	PASS	19.0 ft
+0.60D	1.600	0.079	0.008679	PASS	0.0 ft	0.0	PASS	19.0 ft

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 19'-0" tall Door King Studs with 36.2 psf

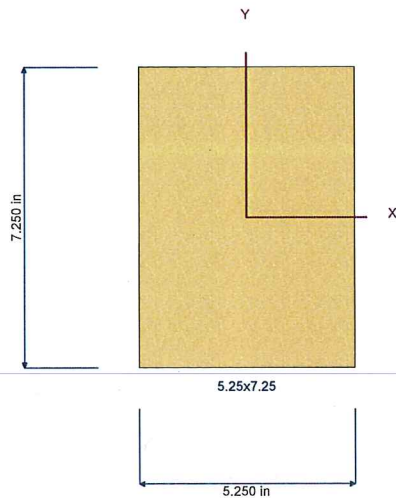
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						0.210					
+D+0.60W				1.232	1.817	0.210					
+D+0.450W				0.924	1.363	0.210					
+0.60D+0.60W				1.232	1.817	0.126					
+0.60D						0.126					
W Only				2.053	3.028						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	2.234 in	10.074 ft
+D+0.450W	0.0000 in	0.000 ft	1.675 in	10.074 ft
+0.60D+0.60W	0.0000 in	0.000 ft	2.234 in	10.074 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	3.723 in	10.074 ft

Sketches



Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1~\ISU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee : nelson engineering

Description : 19'-0" tall king Studs at Overhead Door with HALF Loading- DEFLECTION CALC

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-10

Fb + 2,650.0 psi
Fb - 2,650.0 psi
Fc - Prll 3,000.0 psi
Fc - Perp 750.0 psi
Fv 285.0 psi
Ft 1,500.0 psi

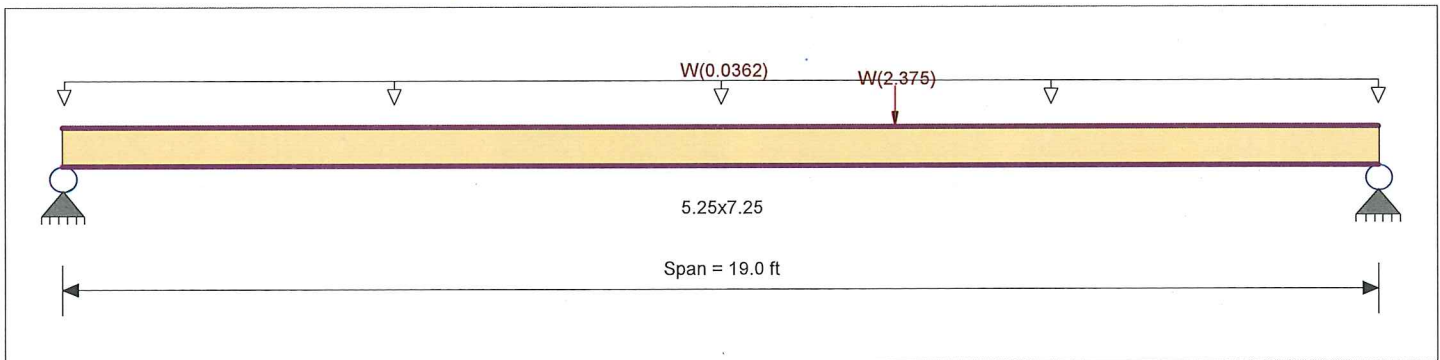
E : Modulus of Elasticity

Ebend- xx 1,700.0 ksi
Eminbend - xx 881.30 ksi

Wood Species : Boise Cascade
Wood Grade : Versa Lam Column 1.7 2650 West

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Density 41.750 pcf



Applied Loads

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

Uniform Load : W = 0.03620 ksf, Tributary Width = 1.0 ft, (1' Trib Wind Loading)

Point Load : W = 2.375 k @ 12.0 ft, (12" Garage Door Header Loading)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	0.444	1	Maximum Shear Stress Ratio	=	0.095	1
Section used for this span		5.25x7.25		Section used for this span		5.25x7.25	
fb : Actual	=	1,881.33 psi		fv : Actual	=	43.12 psi	
FB : Allowable	=	4,240.00 psi		Fv : Allowable	=	456.00 psi	
Load Combination		+0.60W		Load Combination		+0.60W	
Location of maximum on span	=	11.996 ft		Location of maximum on span	=	18.445 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		2.270 in	Ratio =	100	<	240.0	
Max Upward Transient Deflection		0.000 in	Ratio =	0	<	240.0	
Max Downward Total Deflection		1.362 in	Ratio =	167	<	180	
Max Upward Total Deflection		0.000 in	Ratio =	0	<	180	

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{F/V}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
	Length = 19.0 ft	1			0.90	1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
+0.60W						1.000	1.00	1.00	1.00	1.00	1.00			2385.00	0.00	0.00	256.50
	Length = 19.0 ft	1	0.444	0.095	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.21	1,881.33	4240.00	1.09	43.12	456.00
+0.450W						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
	Length = 19.0 ft	1	0.333	0.071	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.41	1,410.99	4240.00	0.82	32.34	456.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	2.2703	10.124		0.0000	0.000

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 19'-0" tall king Studs at Overhead Door with HALF Loading- DEFLECTION CALC

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.219	1.844
Overall MINimum	0.549	0.830
+0.60W	0.731	1.106
+0.450W	0.549	0.830
W Only	1.219	1.844

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1~\ISU53VC-6\TDLLGK~A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 19'-0" tall king Studs at Overhead Door - DEFLECTION CALC

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design

Load Combination ASCE 7-10

Wood Species : Boise Cascade

Wood Grade : Versa Lam Column 1.7 2650 West

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 2650 psi

Fb - 2650 psi

Fc - Prll 3000 psi

Fc - Perp 750 psi

Fv 285 psi

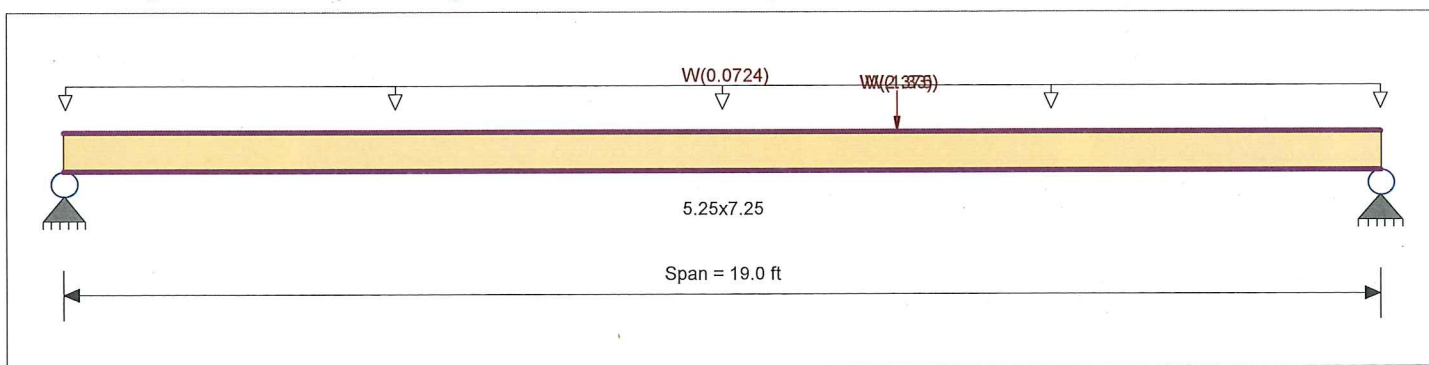
Ft 1500 psi

E : Modulus of Elasticity

Ebend- xx 1700ksi

Eminbend - xx 506024096ksi

Density 41.75pcf



Applied Loads

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

Uniform Load : W = 0.03620 ksf, Tributary Width = 2.0 ft, (2' Trib Wind Loading)

Point Load : W = 2.375 k @ 12.0 ft, (12" Garage Door Header Loading)

Point Load : W = 1.330 k @ 12.0 ft, (12" Garage Door Header Loading)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	0.717 : 1	Maximum Shear Stress Ratio	=	0.155 : 1
Section used for this span		5.25x7.25	Section used for this span		5.25x7.25
fb : Actual	=	3,039.62psi	fv : Actual	=	70.64 psi
FB : Allowable	=	4,240.00psi	Fv : Allowable	=	456.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	11.996ft	Location of maximum on span	=	18.445 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection		3.707 in	Ratio =		61 < 240.0
Max Upward Transient Deflection		0.000 in	Ratio =		0 < 240.0
Max Downward Total Deflection		2.224 in	Ratio =		102 < 180
Max Upward Total Deflection		0.000 in	Ratio =		0 < 180

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{F/V}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
	Length = 19.0 ft	1			0.90	1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
+0.60W						1.000	1.00	1.00	1.00	1.00	1.00		2385.00		0.00	0.00	256.50
	Length = 19.0 ft	1	0.717	0.155	1.60	1.000	1.00	1.00	1.00	1.00	1.00	11.65	3,039.62	4240.00	1.79	70.64	456.00
+0.450W						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
	Length = 19.0 ft	1	0.538	0.116	1.60	1.000	1.00	1.00	1.00	1.00	1.00	8.74	2,279.71	4240.00	1.34	52.98	456.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	3.7066	10.124		0.0000	0.000

Nelson Engineering
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Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK~A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 19'-0" tall king Studs at Overhead Door - DEFLECTION CALC

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.053	3.028
Overall MINimum	0.924	1.363
+0.60W	1.232	1.817
+0.450W	0.924	1.363
W Only	2.053	3.028

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 19'-0" tall Door King Studs OH Door ONLY with 36.2 psf

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		5.25x7.25	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Boise Cascade	
Overall Column Height		19.0 ft		Wood Member Type		VersaLam	
(Used for non-slender calculations)							
Wood Species		Boise Cascade		Exact Width		5.250 in	
Wood Grade		Versa Lam Column 1.7 2650 V		Exact Depth		7.250 in	
Fb +		2,650.0 psi		Fv		285.0 psi	
Fb -		2,650.0 psi		Ft		1,500.0 psi	
Fc - Prll		3,000.0 psi		Density		41.750 pcf	
Fc - Perp		750.0 psi					
E : Modulus of Elasticity . . .		x-x Bending		y-y Bending		Axial	
Basic		1,700.0		1,700.0		1,700.0 ksi	
Minimum		881.30		881.30			
Brace condition for deflection (buckling) along columns :							
X-X (width) axis :				Unbraced Length for X-X Axis buckling = 19 ft, K = 1.0			
Y-Y (depth) axis :				Unbraced Length for X-X Axis buckling = 19 ft, K = 1.0			
						Allow Stress Modification Factors	
						Cf or Cv for Bending	1.0
						Cf or Cv for Compression	1.0
						Cf or Cv for Tension	1.0
						Cm : Wet Use Factor	1.0
						Ct : Temperature Factor	1.0
						Cfu : Flat Use Factor	1.0
						Kf : Built-up columns	1.0 NDS 15.3.2
						Use Cr : Repetitive ?	Yes

Applied Loads

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

Column self weight included : 209.677 lbs * Dead Load Factor

BENDING LOADS :

Wind Loading on King Studs: Lat. Uniform Load creating Mx-x, W = 0.03620 k/ft

Header Load 12' Header: Lat. Point Load at 12.0 ft creating Mx-x, W = 2.375 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.4071 : 1	Maximum SERVICE Lateral Load Reactions . .		
	Load Combination	+D+0.60W	Top along Y-Y	1.844 k	Bottom along Y-Y 1.219 k
	Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X 0.0 k
	Location of max.above base	11.987 ft	Maximum SERVICE Load Lateral Deflections . . .		
	At maximum location values are . . .		Along Y-Y	2.280 in at 10.201 ft	above base
	Applied Axial	0.2097 k	for load combination : W Only		
	Applied Mx	7.206 k-ft	Along X-X	0.0 in at 0.0 ft	above base
	Applied My	0.0 k-ft	for load combination : n/a		
	Fc : Allowable	380.819 psi	Other Factors used to calculate allowable stresses . . .		
PASS	Maximum Shear Stress Ratio =	0.09561 : 1	Bending	Compression	Tension
	Load Combination	+D+0.60W			
	Location of max.above base	19.0 ft			
	Applied Design Shear	43.60 psi			
	Allowable Shear	456.0 psi			

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.140	0.01458	PASS	0.0 ft	0.0	PASS	19.0 ft
+D+0.60W	1.600	0.079	0.4071	PASS	11.987 ft	0.09561	PASS	19.0 ft
+D+0.450W	1.600	0.079	0.3054	PASS	11.987 ft	0.07171	PASS	19.0 ft
+0.60D+0.60W	1.600	0.079	0.4057	PASS	11.987 ft	0.09561	PASS	19.0 ft
+0.60D	1.600	0.079	0.008679	PASS	0.0 ft	0.0	PASS	19.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	k	Y-Y Axis Reaction @ Base	k	Axial Reaction @ Base	My - End Moments @ Base	k-ft	Mx - End Moments @ Base	k-ft
D Only					0.210				

Nelson Engineering
P.O. Box 1599
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Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Wood Beam

Lic. #: KW-06004539

Printed: 29 AUG 2017, 9:03AM
File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee: nelson engineering

Description: 19'-0" tall king Studs at Overhead Door - DEFLECTION CALC

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set: ASCE 7-10

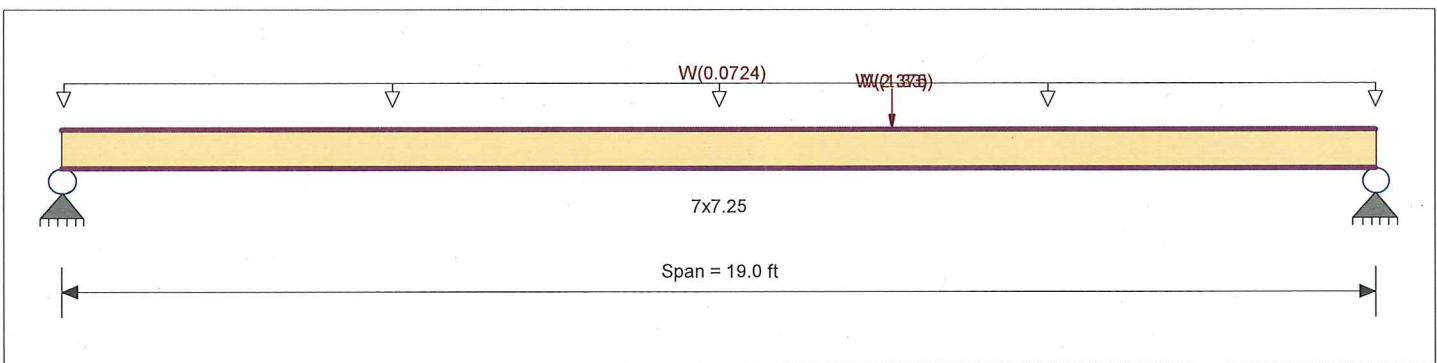
Material Properties

Analysis Method: Allowable Stress Design
Load Combination: ASCE 7-10

Wood Species: Boise Cascade
Wood Grade: Versa Lam Column 1.7 2650 West

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

F_b + 2650 psi
F_b - 2650 psi
F_c - Prll 3000 psi
F_c - Perp 750 psi
F_v 285 psi
F_t 1500 psi
E: Modulus of Elasticity
E_{bend} - xx 1700 ksi
E_{minbend} - xx 506024096 ksi
Density 41.75 pcf



Applied Loads

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

Uniform Load: W = 0.03620 ksf, Tributary Width = 2.0 ft, (2' Trib Wind Loading)
Point Load: W = 2.375 k @ 12.0 ft, (12" Garage Door Header Loading)
Point Load: W = 1.330 k @ 12.0 ft, (12" Garage Door Header Loading)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	0.538	1	Maximum Shear Stress Ratio	=	0.116	: 1
Section used for this span		7x7.25		Section used for this span		7x7.25	
fb: Actual	=	2,279.71	psi	fv: Actual	=	52.98	psi
FB: Allowable	=	4,240.00	psi	Fv: Allowable	=	456.00	psi
Load Combination		+0.60W		Load Combination		+0.60W	
Location of maximum on span	=	11.996	ft	Location of maximum on span	=	18.445	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		2.780	in	Ratio =		82	<240.0
Max Upward Transient Deflection		0.000	in	Ratio =		0	<240.0
Max Downward Total Deflection		1.668	in	Ratio =		136	<180
Max Upward Total Deflection		0.000	in	Ratio =		0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios							Moment Values			Shear Values				
			M	V	C _d	C _{FN}	C _i	C _r	C _m	C _t	C _L	M	fb	F'b	V	fv	F'v
+0.60W	Length = 19.0 ft	1			0.90	1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
						1.000	1.00	1.00	1.00	1.00	1.00		2385.00	0.00	0.00	256.50	
+0.450W	Length = 19.0 ft	1	0.538	0.116	1.60	1.000	1.00	1.00	1.00	1.00	1.00	11.65	2,279.71	4240.00	1.79	52.98	456.00
						1.000	1.00	1.00	1.00	1.00	1.00		0.00	0.00	0.00	0.00	0.00
	Length = 19.0 ft	1	0.403	0.087	1.60	1.000	1.00	1.00	1.00	1.00	1.00	8.74	1,709.78	4240.00	1.34	39.74	456.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	2.7800	10.124		0.0000	0.000

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 19'-0" tall king Studs at Overhead Door - DEFLECTION CALC

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.053	3.028
Overall MINimum	0.924	1.363
+0.60W	1.232	1.817
+0.450W	0.924	1.363
W Only	2.053	3.028

Nelson Engineering
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430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 9:10AM

Wood Column

File = S:\Proj2017\2KZYV1~\ISU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 19'-0" tall Door King Studs OH Door ONLY with 36.2 psf

Maximum Reactions

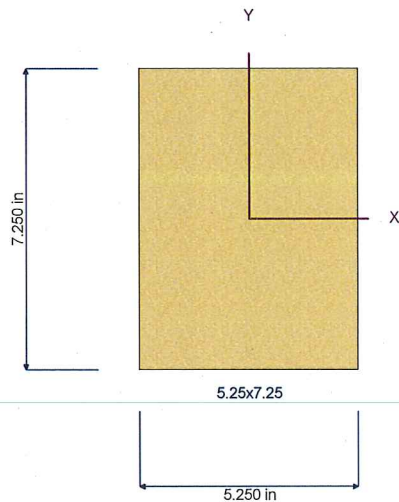
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+D+0.60W				0.731	1.106	0.210					
+D+0.450W				0.549	0.830	0.210					
+0.60D+0.60W				0.731	1.106	0.126					
+0.60D						0.126					
W Only				1.219	1.844						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	1.368 in	10.201 ft
+D+0.450W	0.0000 in	0.000 ft	1.026 in	10.201 ft
+0.60D+0.60W	0.0000 in	0.000 ft	1.368 in	10.201 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	2.280 in	10.201 ft

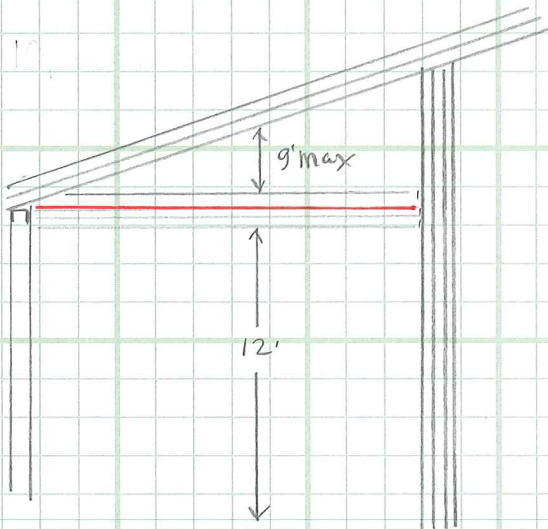
Sketches



2x6 Studs x 20' will not be very straight.

→ Recommend Interior full height Columns & 2 wall heights.

Building Width $\sim 75' \div 5 = 15'$



$$\frac{12'}{2} + \frac{9'}{2} = 10.5' \times 10 \text{psf} = 105 \text{psf}$$

(3) 2x6 S.R. = M = 1.335

Curtain Wall (Interior Walls Only)
5psf LL = 52.5psf

(3) 2x6 D.F.L #2

S.R. M = 0.668

$R_v = 0.39^k$

Full height studs

If put 5psf Ldg & $R_v = 0.39^k \times 2 = 0.78^k$

as Wind Loading : (2) 2x6 D.F.L #2 S.R. = 0.9027

w/ Blocking @ 4'

If put loads on as Live Load (4) 2x6 S.R. = 1.191

$3\frac{1}{2} \times 5\frac{1}{2}$ LVL S.R. = 0.9056 as a Beam (as LL)

$5\frac{1}{4} \times 5\frac{1}{4}$ LVL S.R. = 0.6591

S.R. = 1.020
 $\Delta = 2.814" = +\frac{7}{8}$

w/ Curtain Ldg as WL & as a Beam: $3\frac{1}{2} \times 5\frac{1}{2}$ S.R. = 0.383
 $\Delta_{TL} = 1.688" = +\frac{1}{4}$

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Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 30 AUG 2017, 4:29PM

Wood Column

File = S:\Proj2017\2KZYV1-1ISU53VC-6ITDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 2x6 Interior Curtain wall Studs

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		2x6	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber	
Overall Column Height		12.0 ft		Wood Member Type		Sawn	
(Used for non-slender calculations)							
Wood Species		Douglas Fir - Larch		Exact Width		1.50 in	
Wood Grade		No.1		Exact Depth		5.50 in	
Fb +		1000 psi	Fv	180 psi	Area		8.250 in^2
Fb -		1000 psi	Ft	675 psi	Ix		20.797 in^4
Fc - Prll		1500 psi	Density	31.2 pcf	Iy		1.547 in^4
Fc - Perp		625 psi					
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			
Basic		1700	1700	1700 ksi			
Minimum		620	620				
Allow Stress Modification Factors							
Cf or Cv for Bending 1.30							
Cf or Cv for Compression 1.10							
Cf or Cv for Tension 1.30							
Cm : Wet Use Factor 1.0							
Ct : Temperature Factor 1.0							
Cfu : Flat Use Factor 1.0							
Kf : Built-up columns 1.0 NDS 15.3.2							
Use Cr : Repetitive ? No							
Brace condition for deflection (buckling) along columns :							
X-X (width) axis : Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, K = 1.0							
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 12 ft, K = 1.0							

Applied Loads

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

Column self weight included : 21.450 lbs * Dead Load Factor

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, L = 0.0050 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.1138 : 1	Maximum SERVICE Lateral Load Reactions . .		
	Load Combination	+D+L	Top along Y-Y	0.030 k	Bottom along Y-Y 0.030 k
	Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X 0.0 k
	Location of max. above base	5.960 ft	Maximum SERVICE Load Lateral Deflections . . .		
	At maximum location values are . . .		Along Y-Y	0.06669 in	at 6.040 ft above base
	Applied Axial	0.02145 k	for load combination : +D+L		
	Applied Mx	0.090 k-ft	Along X-X	0.0 in	at 0.0 ft above base
	Applied My	0.0 k-ft	for load combination : n/a		
	Fc : Allowable	461.799 psi	Other Factors used to calculate allowable stresses . . .		
PASS	Maximum Shear Stress Ratio =	0.03030 : 1			
	Load Combination	+D+L	Bending Compression Tension		
	Location of max. above base	0.0 ft			
	Applied Design Shear	5.455 psi			
	Allowable Shear	180.0 psi			

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.308	0.005689	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+L	1.000	0.280	0.1138	PASS	5.960 ft	0.03030	PASS	0.0 ft
+D+0.750L	1.250	0.228	0.06926	PASS	5.960 ft	0.01818	PASS	12.0 ft
+0.60D	1.600	0.181	0.003273	PASS	0.0 ft	0.0	PASS	12.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only					0.021				
+D+L			0.030	0.030	0.021				
+D+0.750L			0.023	0.023	0.021				

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 30 AUG 2017, 4:40PM

Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 20' tall - 2x6 Interior Curtain wall Studs

*W 5psf Ldg
& studs @ 24" o.c. Spacing*

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		2x6		
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber		
Overall Column Height		20.0 ft		Wood Member Type		Sawn		
(Used for non-slender calculations)								
Wood Species		Douglas Fir - Larch		Exact Width		1.50 in		
Wood Grade		No.1		Exact Depth		5.50 in		
Fb +		1,000.0 psi	Fv	180.0 psi	Area	8.25 in^2	Cf or Cv for Bending	1.30
Fb -		1,000.0 psi	Ft	675.0 psi	Ix	20.797 in^4	Cf or Cv for Compression	1.10
Fc - Prll		1,500.0 psi	Density	31.20 pcf	Iy	1.547 in^4	Cf or Cv for Tension	1.30
Fc - Perp		625.0 psi					Cm : Wet Use Factor	1.0
							Ct : Temperature Factor	1.0
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			Cfu : Flat Use Factor	1.0
		Basic	1,700.0	1,700.0	1,700.0 ksi		Kf : Built-up columns	1.0
		Minimum	620.0	620.0			Use Cr : Repetitive ?	No
Brace condition for deflection (buckling) along columns :								
X-X (width) axis :					Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, 16 ft, K = 1.0			
Y-Y (depth) axis :					Unbraced Length for X-X Axis buckling = 20 ft, K = 1.0			

Applied Loads

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

Column self weight included: 35.750 lbs * Dead Load Factor

BENDING LOADS ...

Lat. Uniform Load creating Mx-x, L = 0.010 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.6406 : 1	Maximum SERVICE Lateral Load Reactions ..	
	Load Combination	+D+L	Top along Y-Y	0.10 k Bottom along Y-Y 0.10 k
	Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k Bottom along X-X 0.0 k
	Location of max.above base	9.933 ft	Maximum SERVICE Load Lateral Deflections ...	
	At maximum location values are ...		Along Y-Y	1.029 in at 10.067 ft above base
	Applied Axial	0.03575 k	for load combination: +D+L	
	Applied Mx	0.50 k-ft	Along X-X	0.0 in at 0.0 ft above base
	Applied My	0.0 k-ft	for load combination: n/a	
	Fc: Allowable	258.079 psi	Other Factors used to calculate allowable stresses ...	
PASS	Maximum Shear Stress Ratio =	0.1010 : 1	Bending	Compression Tension
	Load Combination	+D+L		
	Location of max.above base	20.0 ft		
	Applied Design Shear	18.182 psi		
	Allowable Shear	180.0 psi		

*20' MAX = 240"
÷ 1.029
= 233.*

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.173	0.01687	PASS	0.0 ft	0.0	PASS	20.0 ft
+D+L	1.000	0.156	0.6406	PASS	9.933 ft	0.1010	PASS	20.0 ft
+D+0.750L	1.250	0.126	0.3899	PASS	10.067 ft	0.06061	PASS	20.0 ft
+0.60D	1.600	0.099	0.009928	PASS	0.0 ft	0.0	PASS	20.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top	@ Base	@ Top
D Only						0.036				
+D+L				0.100	0.100	0.036				
+D+0.750L				0.075	0.075	0.036				

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 30 AUG 2017, 4:41PM

Wood Column

File = S:\Proj\2017\2KZYV1-1SU53VC-6ITDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 2x6 Interior Curtain wall Studs

4/ 5psf Ldg
\$ studs @ 24" o.c. spacing

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		2x6											
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber											
Overall Column Height		12.0 ft		Wood Member Type		Sawn											
(Used for non-slender calculations)						Exact Width		1.50 in		Allow Stress Modification Factors							
Wood Species		Douglas Fir - Larch		Exact Depth		5.50 in		Cf or Cv for Bending		1.30							
Wood Grade		No.1		Area		8.250 in^2		Cf or Cv for Compression		1.10							
Fb +		1,000.0 psi		Fv		180.0 psi		Ix		20.797 in^4		Cf or Cv for Tension		1.30			
Fb -		1,000.0 psi		Ft		675.0 psi		Iy		1.547 in^4		Cm : Wet Use Factor		1.0			
Fc - Prll		1,500.0 psi		Density		31.20 pcf						Ct : Temperature Factor		1.0			
Fc - Perp		625.0 psi										Cfu : Flat Use Factor		1.0			
E : Modulus of Elasticity . . .		x-x Bending		y-y Bending		Axial						Kf : Built-up columns		1.0			
		Basic		1,700.0		1,700.0		1,700.0 ksi				Use Cr : Repetitive ?		No			
		Minimum		620.0		620.0											
														Brace condition for deflection (buckling) along columns :			
														X-X (width) axis :		Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, K = 1.0	
														Y-Y (depth) axis :		Unbraced Length for X-X Axis buckling = 12 ft, K = 1.0	

Applied Loads

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

Column self weight included : 21.450 lbs * Dead Load Factor

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, L = 0.010 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.2276 : 1	Maximum SERVICE Lateral Load Reactions . .		
	Load Combination	+D+L	Top along Y-Y	0.060 k	Bottom along Y-Y 0.060 k
	Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X 0.0 k
	Location of max.above base	5.960 ft	Maximum SERVICE Load Lateral Deflections . . .		
	At maximum location values are . . .		Along Y-Y	0.1334 in at 6.040 ft	above base
	Applied Axial	0.02145 k	for load combination : +D+L		
	Applied Mx	0.180 k-ft	Along X-X	0.0 in at 0.0 ft	above base
	Applied My	0.0 k-ft	for load combination : n/a		
	Fc : Allowable	461.799 psi	Other Factors used to calculate allowable stresses . . .		
PASS	Maximum Shear Stress Ratio =	0.06061 : 1	Bending Compression Tension		
	Load Combination	+D+L			
	Location of max.above base	0.0 ft			
	Applied Design Shear	10.909 psi			
	Allowable Shear	180.0 psi			

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.308	0.005689	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+L	1.000	0.280	0.2276	PASS	5.960 ft	0.06061	PASS	0.0 ft
+D+0.750L	1.250	0.228	0.1385	PASS	5.960 ft	0.03636	PASS	12.0 ft
+0.60D	1.600	0.181	0.003273	PASS	0.0 ft	0.0	PASS	12.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only						0.021				
+D+L				0.060	0.060	0.021				
+D+0.750L				0.045	0.045	0.021				

Titen HD® Design Information — Concrete

SIMPSON

Strong-Tie®



Titen HD® Allowable Tension Loads in Normal-Weight Concrete

Size (in.)	Drill Bit Dia. (in.)	Embed. Depth in. (mm)	Critical Edge Dist. in. (mm)	Critical Spacing in. (mm)	Tension Load							
					f' _c ≥ 2,000 psi (13.8 MPa Concrete)			f' _c ≥ 3,000 psi (20.7 MPa Concrete)		f' _c ≥ 4,000 psi (27.6 MPa Concrete)		
					Ultimate lb. (kN)	Std. Dev. lb. (kN)	Allowable lb. (kN)	Allowable lb. (kN)	Ultimate lb. (kN)	Std. Dev. lb. (kN)	Allowable lb. (kN)	
5/8 (9.5)	3/8	1 1/2 (38)	6 (152)	4 (102)	2,070 (9.2)	—	520 (2.3)	635 (2.8)	2,974 (13.2)	—	745 (3.3)	
		2 3/4 (70)	3 (76)	6 (152)	4,297 (19.1)	—	1,075 (4.8)	1,315 (5.8)	6,204 (27.6)	—	1,550 (6.9)	
		3 3/4 (95)			7,087 (31.5)	347 (1.5)	1,770 (7.9)	2,115 (9.4)	9,820 (43.7)	1,434 (6.4)	2,455 (10.9)	
1/2 (12.7)	1/2	2 3/4 (70)	4 (102)	8 (203)	4,610 (20.5)	—	1,155 (5.1)	1,400 (6.2)	6,580 (29.3)	—	1,645 (7.3)	
		3 3/8 (92)			7,413 (33.0)	412 (1.8)	1,855 (8.3)	2,270 (10.1)	10,742 (47.8)	600 (2.7)	2,685 (11.9)	
		5 3/4 (146)			10,278 (45.7)	297 (1.3)	2,570 (11.4)	3,240 (14.4)	15,640 (69.6)	2,341 (10.4)	3,910 (17.4)	
5/8 (15.9)	5/8	2 3/4 (70)	5 (127)	10 (254)	4,610 (20.5)	—	1,155 (5.1)	1,400 (6.2)	6,580 (29.3)	—	1,645 (7.3)	
		4 1/8 (105)			8,742 (38.9)	615 (2.7)	2,185 (9.7)	2,630 (11.7)	12,286 (54.7)	1,604 (7.1)	3,070 (13.7)	
		5 3/4 (146)			12,953 (57.6)	1,764 (7.8)	3,240 (14.4)	3,955 (17.6)	18,680 (83.1)	—	4,670 (20.8)	
3/4 (19.1)	3/4	2 3/4 (70)	6 (152)	12 (305)	4,674 (20.8)	—	1,170 (5.2)	1,405 (6.3)	6,580 (29.3)	—	1,645 (7.3)	
		4 5/8 (117)			10,340 (46.0)	1,096 (4.9)	2,585 (11.5)	3,470 (15.4)	17,426 (77.5)	1,591 (7.1)	4,355 (19.4)	
		5 3/4 (146)			13,765 (61.2)	1,016 (4.5)	3,440 (15.3)	4,055 (18.0)	18,680 (83.1)	1,743 (7.8)	4,670 (20.8)	

- The allowable loads listed are based on a safety factor of 4.0.
- Refer to allowable load-adjustment factors for spacing and edge distance on pages 198 and 199.
- The minimum concrete thickness is 1 1/2 times the embedment depth.
- Tension and shear loads for the Titen HD anchor may be combined using the elliptical interaction equation ($n=5/8$). Allowable load may be interpolated for concrete compressive strengths between 2,000 psi and 4,000 psi.

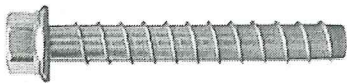


Titen HD® Allowable Shear Loads in Normal-Weight Concrete

Size (in.)	Drill Bit Dia. (in.)	Embed. Depth in. (mm)	Critical Edge Dist. in. (mm)	Critical Spacing in. (mm)	Shear Load						
					$f'_c \geq 2,000$ psi (13.8 MPa Concrete)			$f'_c \geq 3,000$ psi (20.7 MPa Concrete)		$f'_c \geq 4,000$ psi (27.6 MPa Concrete)	
					Ultimate lb. (kN)	Std. Dev. lb. (kN)	Allowable lb. (kN)	Allowable lb. (kN)	Ultimate lb. (kN)	Std. Dev. lb. (kN)	Allowable lb. (kN)
5/8 (9.5)	3/8	1 1/2 (38)	6 (152)	4 (102)	2,912 (13.0)	—	730 (3.2)	825 (3.7)	3,668 (16.3)	—	915 (4.1)
		2 3/4 (70)	4 1/2 (114)	6 (152)	6,353 (28.3)	—	1,585 (7.1)	1,665 (7.4)	—	—	1,740 (7.7)
		3 3/4 (95)			6,377 (28.4)	1,006 (4.5)	1,595 (7.1)	1,670 (7.4)	—	—	1,740 (7.7)
1/2 (12.7)	1/2	2 3/4 (70)	6 (152)	8 (203)	6,435 (28.6)	—	1,605 (7.1)	2,050 (9.1)	9,987 (44.4)	—	2,495 (7.8)
		3 3/8 (92)			9,324 (41.5)	1,285 (5.7)	2,330 (10.4)	2,795 (12.4)	13,027 (57.9)	597 (2.7)	3,255 (14.5)
		5 3/4 (146)			11,319 (50.3)	1,245 (5.5)	2,830 (12.6)	3,045 (13.5)	—	—	3,255 (14.5)
5/8 (15.9)	5/8	2 3/4 (70)	7 1/2 (191)	10 (254)	7,745 (34.5)	—	1,940 (8.6)	2,220 (9.9)	9,987 (44.4)	—	2,495 (7.8)
		4 1/8 (105)			8,706 (38.7)	1,830 (8.1)	2,175 (9.7)	3,415 (15.2)	18,607 (82.8)	1,650 (7.3)	4,650 (20.7)
		5 3/4 (146)			12,498 (55.6)	2,227 (9.9)	3,125 (13.9)	3,890 (17.3)	—	—	4,650 (20.7)
3/4 (19.1)	3/4	2 3/4 (70)	9 (229)	12 (305)	7,832 (34.8)	—	1,960 (8.7)	2,415 (10.7)	11,460 (51.0)	—	2,865 (12.7)
		4 5/8 (117)			11,222 (49.9)	2,900 (12.9)	2,805 (12.5)	4,490 (20.0)	24,680 (109.8)	2,368 (10.5)	6,170 (27.4)
		5 3/4 (146)			19,793 (88.0)	3,547 (15.8)	4,950 (22.0)	5,560 (24.7)	24,680 (109.8)	795 (3.5)	6,170 (27.4)

- The allowable loads listed are based on a safety factor of 4.0.
- Refer to allowable load-adjustment factors for spacing and edge distance on pages 198 and 199.
- The minimum concrete thickness is 1 1/2 times the embedment depth.
- Tension and shear loads for the Titen HD anchor may be combined using the elliptical interaction equation ($n=5/8$). Allowable load may be interpolated for concrete compressive strengths between 2,000 psi and 4,000 psi.

* See page 12 for an explanation of the load table icons.



- Code listed by ICC-ES for cracked and uncracked concrete applications
- Patented thread design with serrated cutting teeth for easy installation
- Installs with 50% less torque than competitors
- **Recommended Fixture Hole Size (in.): 3/8"-7/16"**

Simpson Strong-Tie Titen HD Shear And Tension Load Table

Item	Size	Drill Bit Size	Wrench Size	Package Qty	Price	Qty
THDB25178H	1/4" x 1-7/8"	1/4"	3/8"	100	\$45.86	0
THDB25234H	1/4" x 2-3/4"	1/4"	3/8"	50	\$26.23	0
THDB25300H	1/4" x 3"	1/4"	3/8"	50	\$27.09	0
THD37134H	3/8" x 1-3/4"	3/8"	9/16"	50	\$27.69	0
THD37212H	3/8" x 2-1/2"	3/8"	9/16"	50	\$31.54	0
THD37300H	3/8" x 3"	3/8"	9/16"	50	\$34.62	0
THD37400H	3/8" x 4"	3/8"	9/16"	50	\$35.38	0
THD37500H	3/8" x 5"	3/8"	9/16"	50	\$51.54	0
THD37600H	3/8" x 6"	3/8"	9/16"	50	\$57.69	0
THD50300H	1/2" x 3"	1/2"	3/4"	25	\$27.31	0
THD50400H	1/2" x 4"	1/2"	3/4"	20	\$24.31	0
THD50500H	1/2" x 5"	1/2"	3/4"	20	\$31.69	0
THD50600H	1/2" x 6"	1/2"	3/4"	20	\$32.92	0
THD50612H	1/2" x 6-1/2"	1/2"	3/4"	20	\$36.62	0
THD50800H	1/2" x 8"	1/2"	3/4"	20	\$52.92	0
THD501200H	1/2" x 12"	1/2"	3/4"	5	\$36.57	0
THD501300H	1/2" x 13"	1/2"	3/4"	5	\$37.14	0
THD501400H	1/2" x 14"	1/2"	3/4"	5	\$37.36	0
THD501500H	1/2" x 15"	1/2"	3/4"	5	\$38.29	0
THDB62400H	5/8" x 4"	5/8"	15/16"	10	\$20.92	0
THDB62500H	5/8" x 5"	5/8"	15/16"	10	\$23.38	0
THDB62600H	5/8" x 6"	5/8"	15/16"	10	\$25.54	0
THDB62612H	5/8" x 6-1/2"	5/8"	15/16"	10	\$26.48	0
THDB62800H	5/8" x 8"	5/8"	15/16"	10	\$48.46	0
THD75100H	3/4" x 10"	3/4"	1-1/8"	5	\$43.08	0
THD75400H	3/4" x 4"	3/4"	1-1/8"	10	\$31.69	0

5/8 x 4" - 1 1/2" R = 2 1/2" embed.

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: Tripple Plate at Interior Wall 15' span

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

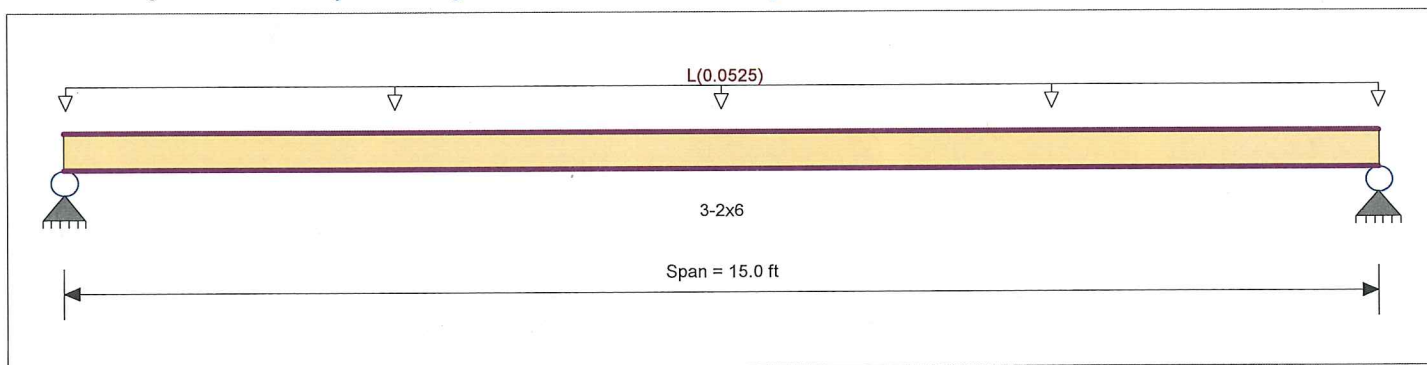
Material Properties

Analysis Method: Allowable Stress Design
Load Combination ASCE 7-10

Fb +	900 psi	E : Modulus of Elasticity	
Fb -	900 psi	Ebend- xx	1600 ksi
Fc - Prll	1350 psi	Eminbend - xx	580 ksi
Fc - Perp	625 psi		
Fv	180 psi		
Ft	575 psi	Density	31.2 pcf

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Uniform Load : L = 0.0050 ksf, Tributary Width = 10.50 ft, (Interior Loading)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.668	1	Maximum Shear Stress Ratio	=	0.125	: 1
Section used for this span		3-2x6		Section used for this span		3-2x6	
fb : Actual	=	780.99	psi	fv : Actual	=	22.47	psi
FB : Allowable	=	1,170.00	psi	Fv : Allowable	=	180.00	psi
Load Combination		L Only		Load Combination		L Only	
Location of maximum on span	=	7.500	ft	Location of maximum on span	=	14.562	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.000	in	Ratio =		0	<240.0
Max Upward Transient Deflection		0.000	in	Ratio =		0	<240.0
Max Downward Total Deflection		0.603	in	Ratio =		298	>=180
Max Upward Total Deflection		0.000	in	Ratio =		0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L		M	fb	F'b	V	fv	F'v
Length = 15.0 ft	1			0.90	1.300	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
L Only					1.300	1.00	1.00	1.00	1.00	1.00				1053.00	0.00	0.00	162.00
Length = 15.0 ft	1	0.668	0.125	1.00	1.300	1.00	1.00	1.00	1.00	1.00	1.48	780.99	1170.00	0.37	22.47	180.00	
+0.750L					1.300	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 15.0 ft	1	0.401	0.075	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.11	585.74	1462.50	0.28	16.85	225.00	

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
L Only	1	0.6026	7.555		0.0000	0.000

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Wood Beam

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK~A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : Tripple Plate at Interior Wall 15' span

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.394	0.394
Overall MINimum	0.295	0.295
L Only	0.394	0.394
+0.750L	0.295	0.295

Nelson Engineering
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430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 20'-0" tall 2x6 wood studs at Interior Curtain Wall

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method : Allowable Stress Design				Wood Section Name 2-2x6	
End Fixities Top & Bottom Pinned				Wood Grading/Manuf. Graded Lumber	
Overall Column Height		20.0 ft		Wood Member Type Sawn	
(Used for non-slender calculations)					
Wood Species Douglas Fir - Larch		Exact Width 3.0 in Allow Stress Modification Factors			
Wood Grade No.2		Exact Depth 5.50 in Cf or Cv for Bending 1.30			
Fb +	900 psi	Fv	180 psi	Area	16.50 in^2 Cf or Cv for Compression 1.10
Fb -	900 psi	Ft	575 psi	Ix	41.594 in^4 Cf or Cv for Tension 1.30
Fc - Prll	1350 psi	Density	31.2 pcf	Iy	12.375 in^4 Cm : Wet Use Factor 1.0
Fc - Perp	625 psi				Ct : Temperature Factor 1.0
E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial		Cfu : Flat Use Factor 1.0
	Basic	1600	1600	1600 ksi	Kf : Built-up columns 1.0 NDS 15.3.2
	Minimum	580	580		Use Cr : Repetitive ? Yes
Brace condition for deflection (buckling) along columns :					
X-X (width) axis :				Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, 16 ft, K = 1.0	
Y-Y (depth) axis :				Unbraced Length for X-X Axis buckling = 20 ft, K = 1.0	

Applied Loads

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

Column self weight included : 71.50 lbs * Dead Load Factor
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, W = 0.0050 k/ft
Lat. Point Load at 12.0 ft creating Mx-x, W = 0.780 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.9027 : 1	Maximum SERVICE Lateral Load Reactions . .		
	Load Combination	+D+0.60W	Top along Y-Y	0.5180 k	Bottom along Y-Y 0.3620 k
	Governing NDS Formula	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X 0.0 k
	Location of max.above base	11.946 ft	Maximum SERVICE Load Lateral Deflections . . .		
	At maximum location values are . . .		Along Y-Y	3.504 in at 10.604 ft	above base
	Applied Axial	0.07150 k	for load combination : W Only		
	Applied Mx	2.381 k-ft	Along X-X	0.0 in at 0.0 ft	above base
	Applied My	0.0 k-ft	for load combination : n/a		
	Fc : Allowable	244.760 psi	Other Factors used to calculate allowable stresses . . .		
PASS	Maximum Shear Stress Ratio =	0.09811 : 1	Bending	Compression	Tension
	Load Combination	+D+0.60W			
	Location of max.above base	20.0 ft			
	Applied Design Shear	28.255 psi			
	Allowable Shear	288.0 psi			

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.179	0.01806	PASS	0.0 ft	0.0	PASS	20.0 ft
+D+0.60W	1.600	0.103	0.9027	PASS	11.946 ft	0.09811	PASS	20.0 ft
+D+0.450W	1.600	0.103	0.6771	PASS	11.946 ft	0.07358	PASS	20.0 ft
+0.60D+0.60W	1.600	0.103	0.8962	PASS	11.946 ft	0.09811	PASS	20.0 ft
+0.60D	1.600	0.103	0.01062	PASS	0.0 ft	0.0	PASS	20.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					0.072				

Nelson Engineering
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Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 20'-0" tall 2x6 wood studs at Interior Curtain Wall

Note: Only non-zero reactions are listed.

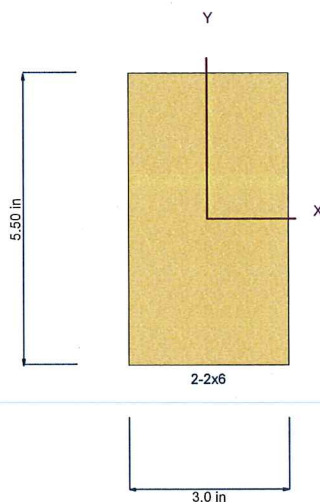
Maximum Reactions

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+D+0.60W				0.217	0.311	0.072					
+D+0.450W				0.163	0.233	0.072					
+0.60D+0.60W				0.217	0.311	0.043					
+0.60D						0.043					
W Only				0.362	0.518						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance	Max. Y-Y Deflection		Distance
D Only	0.0000 in	0.000 ft		0.000 in	0.000 ft	
+D+0.60W	0.0000 in	0.000 ft		2.102 in	10.604 ft	
+D+0.450W	0.0000 in	0.000 ft		1.577 in	10.604 ft	
+0.60D+0.60W	0.0000 in	0.000 ft		2.102 in	10.604 ft	
+0.60D	0.0000 in	0.000 ft		0.000 in	0.000 ft	
W Only	0.0000 in	0.000 ft		3.504 in	10.604 ft	

Sketches



Nelson Engineering
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Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

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Wood Column

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: 20'-0" tall 2x6 wood studs at Interior Curtain Wall

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		5.25x5.25	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Boise Cascade	
Overall Column Height		20.0 ft		Wood Member Type		VersaLam	
(Used for non-slender calculations)							
Wood Species		Boise Cascade		Exact Width		5.250 in Allow Stress Modification Factors	
Wood Grade		Versa Lam Column 1.7 2650 V		Exact Depth		5.250 in Cf or Cv for Bending 1.0	
Fb +		2650 psi	Fv	285 psi	Area	27.563 in^2	Cf or Cv for Compression 1.0
Fb -		2650 psi	Ft	1500 psi	Ix	63.308 in^4	Cf or Cv for Tension 1.0
Fc - Prll		3000 psi	Density	41.75 pcf	Iy	63.308 in^4	Cm : Wet Use Factor 1.0
Fc - Perp		750 psi					Ct : Temperature Factor 1.0
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			Cfu : Flat Use Factor 1.0
Basic		1700	1700	1700 ksi			Kf : Built-up columns 1.0 NDS 15.3.2
Minimum		506024096	506024096				Use Cr : Repetitive ? Yes
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis :		Lu for X-X Axis buckling : 4 ft, 8 ft, 12 ft, 16 ft, K = 1.0	
				Y-Y (depth) axis :		Unbraced Length for X-X Axis buckling = 20 ft, K = 1.0	

Applied Loads

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

Column self weight included : 159.827 lbs * Dead Load Factor
BENDING LOADS ...
Lat. Uniform Load creating Mx-x, L = 0.0050 k/ft
Lat. Point Load at 12.0 ft creating Mx-x, L = 0.780 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.6591 : 1	Maximum SERVICE Lateral Load Reactions . .			
	Load Combination	+D+L	Top along Y-Y	0.5180 k	Bottom along Y-Y	0.3620 k
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3	Top along X-X	0.0 k	Bottom along X-X	0.0 k
	Location of max.above base	11.946 ft	Maximum SERVICE Load Lateral Deflections . . .			
	At maximum location values are . . .		Along Y-Y	2.167 in	at	10.604 ft above base
	Applied Axial	0.1598 k	for load combination : +D+L			
	Applied Mx	3.968 k-ft	Along X-X	0.0 in	at	0.0 ft above base
	Applied My	0.0 k-ft	for load combination : n/a			
	Fc : Allowable	342.244 psi	Other Factors used to calculate allowable stresses . . .			
				<u>Bending</u>	<u>Compression</u>	<u>Tension</u>
PASS	Maximum Shear Stress Ratio =	0.09891 : 1				
	Load Combination	+D+L				
	Location of max.above base	20.0 ft				
	Applied Design Shear	28.190 psi				
	Allowable Shear	285.0 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.127	0.01697	PASS	0.0 ft	0.0	PASS	20.0 ft
+D+L	1.000	0.114	0.6591	PASS	11.946 ft	0.09891	PASS	20.0 ft
+D+0.750L	1.250	0.092	0.3956	PASS	11.946 ft	0.05935	PASS	20.0 ft
+0.60D	1.600	0.072	0.01011	PASS	0.0 ft	0.0	PASS	20.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only					0.160				
+D+L			0.362	0.518	0.160				

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 9:27AM

Wood Column

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : 20'-0" tall 2x6 wood studs at Interior Curtain Wall

Maximum Reactions

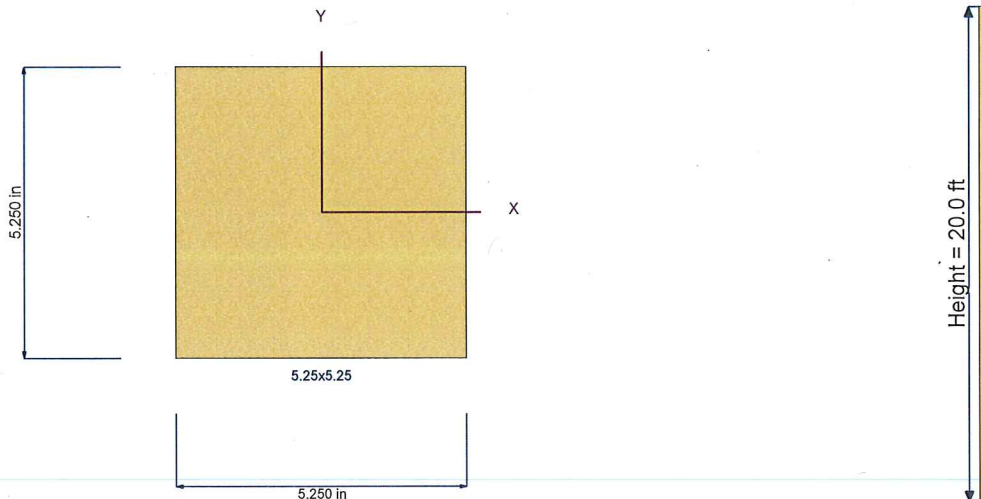
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+D+0.750L				0.272	0.389	0.160					
+0.60D						0.096					
L Only				0.362	0.518						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	2.167 in	10.604 ft
+D+0.750L	0.0000 in	0.000 ft	1.625 in	10.604 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	2.167 in	10.604 ft

Sketches



Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 9:37AM

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53V\6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: VersaLam Column at Curtain Wall for DEFLECTION - with curtain loading as LL

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

Material Properties

Analysis Method: Allowable Stress Design
Load Combination: ASCE 7-10

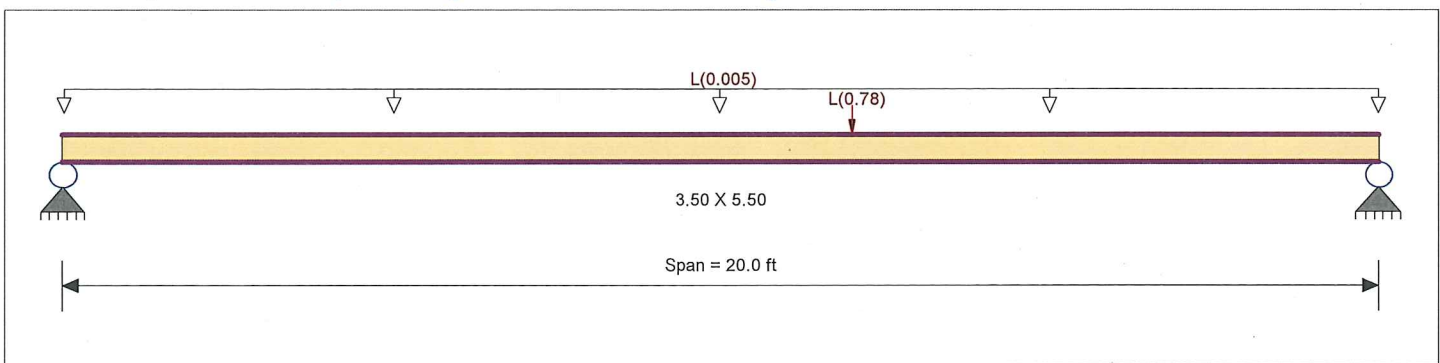
Fb + 2650 psi
Fb - 2650 psi
Fc - Prll 3000 psi
Fc - Perp 750 psi
Fv 285 psi
Ft 1500 psi

E: Modulus of Elasticity
Ebend-xx 1700 ksi
Eminbend-xx 306024096 ksi

Wood Species: Boise Cascade
Wood Grade: Versa Lam Column 1.7 2650 West

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Density 41.75 pcf



Applied Loads

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

Uniform Load: L = 0.0050 ksf, Tributary Width = 1.0 ft, (5psf Curtain Wall Loading)

Point Load: L = 0.780 k @ 12.0 ft, (triple wall plates)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	1.020	1	Maximum Shear Stress Ratio	=	0.141	1
Section used for this span		3.50 X 5.50		Section used for this span		3.50 X 5.50	
fb: Actual	=	2,703.31	psi	fv: Actual	=	40.19	psi
FB: Allowable	=	2,650.00	psi	Fv: Allowable	=	285.00	psi
Load Combination		L Only		Load Combination		L Only	
Location of maximum on span	=	11.971	ft	Location of maximum on span	=	19.562	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.000	in	Ratio =		0 < 240.0	
Max Upward Transient Deflection		0.000	in	Ratio =		0 < 240.0	
Max Downward Total Deflection		2.814	in	Ratio =		85 < 180	
Max Upward Total Deflection		0.000	in	Ratio =		0 < 180	

OK
Curtain Wall Ldg isnt Live
either Seismic or wind

Maximum Forces & Stresses for Load Combinations

Load Combination		Span #	Max Stress Ratios								Moment Values			Shear Values			
Segment Length			M	V	C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	M	fb	F'b	V	fv	F'v
Length = 20.0 ft L Only	1				0.90	1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 20.0 ft +0.750L	1	1.020	0.141	1.00	1.000	1.00	1.00	1.00	1.00	1.00	1.00	3.98	2,703.31	2650.00	0.52	40.19	285.00
						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 20.0 ft	1	0.612	0.085	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.00	2.98	2,027.48	3312.50	0.39	30.14	356.25

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
L Only	1	2.8142	10.584		0.0000	0.000

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 9:37AM

Wood Beam

File = S:\Proj2017\2KZYV1~1\SU53VC-6\TDLLGK~A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : VersaLam Column at Curtain Wall for DEFLECTION - with curtain loading as LL

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.362	0.518
Overall MINimum	0.272	0.389
L Only	0.362	0.518
+0.750L	0.272	0.389

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 9:38AM

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: nelson engineering

Description: VersaLam Column at Curtain Wall for DEFLECTION - with curtain loading as WL

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-10

Wood Species : Boise Cascade

Wood Grade : Versa Lam Column 1.7 2650 West

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 2,650.0 psi

Fb - 2,650.0 psi

Fc - Prll 3,000.0 psi

Fc - Perp 750.0 psi

Fv 285.0 psi

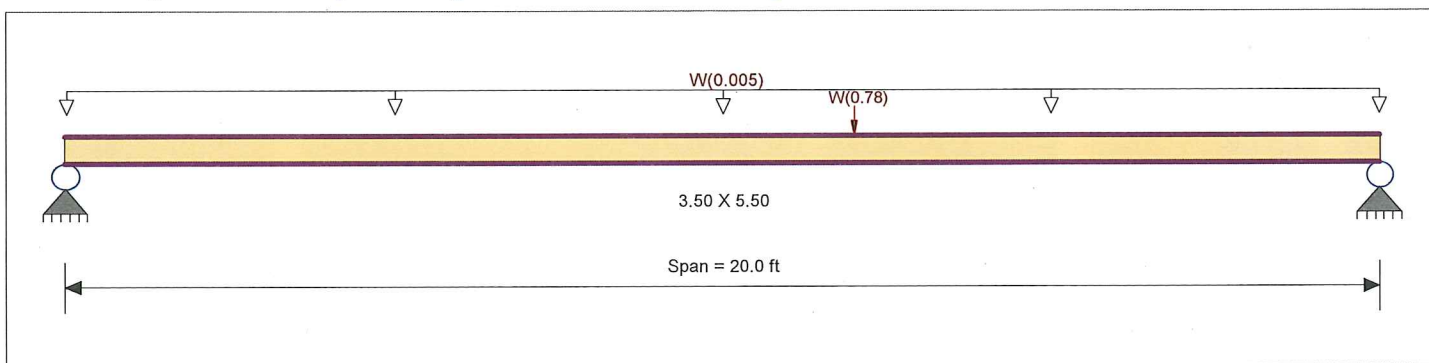
Ft 1,500.0 psi

E : Modulus of Elasticity

Ebend- xx 1,700.0 ksi

Eminbend - xx 881.30 ksi

Density 41.750 pcf



Applied Loads

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

Uniform Load : W = 0.0050 ksf, Tributary Width = 1.0 ft, (5psf Curtain Wall Loading)

Point Load : W = 0.780 k @ 12.0 ft, (triple wall plates)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	0.383	1	Maximum Shear Stress Ratio	=	0.053	1
Section used for this span	=	3.50 X 5.50		Section used for this span	=	3.50 X 5.50	
fb : Actual	=	1,621.99	psi	fv : Actual	=	24.12	psi
FB : Allowable	=	4,240.00	psi	Fv : Allowable	=	456.00	psi
Load Combination	=	+0.60W		Load Combination	=	+0.60W	
Location of maximum on span	=	11.971	ft	Location of maximum on span	=	19.562	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		2.814	in	Ratio =		85	<240.0
Max Upward Transient Deflection		0.000	in	Ratio =		0	<240.0
Max Downward Total Deflection		1.688	in	Ratio =		142	<180
Max Upward Total Deflection		0.000	in	Ratio =		0	<180

Look for Non Brittle Per Arch

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios								Moment Values			Shear Values			
			M	V	C _d	C _{F/V}	C _i	C _r	C _m	C _t	C _L	M	fb	F'b	V	fv	F'v
+0.60W	Length = 20.0 ft	1			0.90	1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
+0.450W	Length = 20.0 ft	1	0.383	0.053	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.39	1,621.99	4240.00	0.31	24.12	456.00
						1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
	Length = 20.0 ft	1	0.287	0.040	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.79	1,216.49	4240.00	0.23	18.09	456.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	2.8142	10.584		0.0000	0.000

Nelson Engineering
P.O. Box 1599
430 South Cache Street
Jackson Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Title Block Line 6

Printed: 29 AUG 2017, 9:38AM

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06004539

Licensee : nelson engineering

Description : VersaLam Column at Curtain Wall for DEFLECTION - with curtain loading as WL

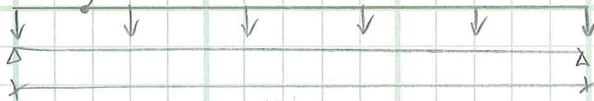
Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.362	0.518
Overall MINimum	0.163	0.233
+0.60W	0.217	0.311
+0.450W	0.163	0.233
W Only	0.362	0.518

Ceiling Joists



$l = 13'$

→ 2x6 DF-L#1 @ 16" o.c. w/ 20 psf DL ★

$$F_b/F_b = 0.709 \quad \Delta U/299$$

$$F_v/F_v = 0.194$$

→ 2x8 DF-L#1 @ 16" o.c.

→ 2x8 DF-L#1 @ 16" o.c. w/ 20 psf DL & 30 psf LL

$$F_b/F_b = 0.965 \quad \Delta U/488$$

$$F_v/F_v = 0.314 \quad \Delta U/283$$

→ 2x12 DF-L#1 @ 16" o.c.

w/ JOISTS ORIENTED OPPOSITE DIRECTION:

$l = \sim 10.5' \text{ (max)}$

→ 2x6 DF-L#1 @ 16" o.c. w/ 20 psf DL & 20 psf LL

$$F_b/F_b = 0.806 \quad \Delta U/607$$

$$F_v/F_v = 0.269 \quad \Delta U/293$$

→ 2x8 DF-L#1 @ 16" o.c. w/ 20 psf DL & 60 psf LL

$$F_b/F_b = 0.994 \quad \Delta U/463$$

$$F_v/F_v = 0.390 \quad \Delta U/340$$

★ Per Mechanical Engineer: Loading upon ceiling joists for Units & Maintenance will not exceed 20 psf

⇒ Use 2x6 Joists @ 16" o.c.

Simpson LU26- Allow = 835# > 185# Rv ✓ OK

Wood Beam

File = S:\Proj2017\2KZYV1-1SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set: IBC 2015

Material Properties

Analysis Method: Allowable Stress Design
Load Combination: IBC 2015

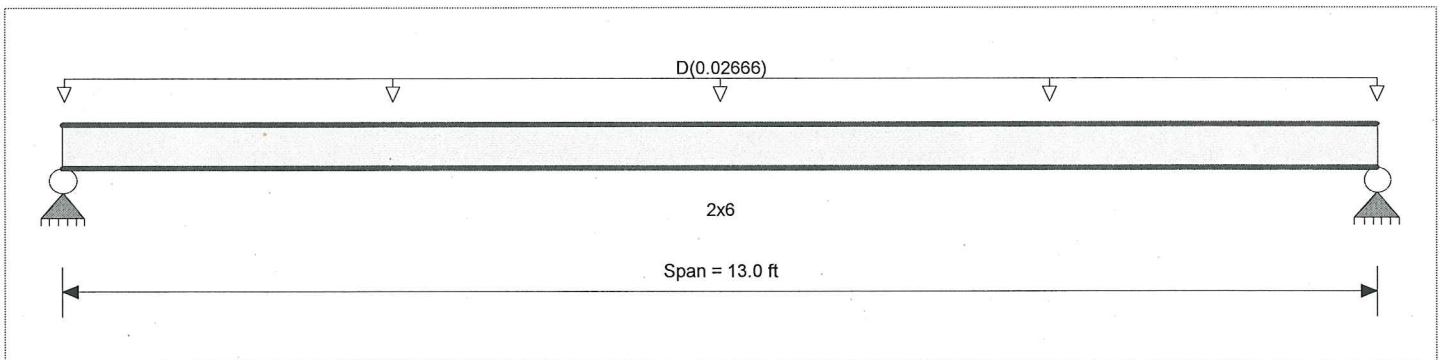
Fb + 1,000.0 psi
Fb - 1,000.0 psi
Fc - Pll 1,500.0 psi
Fc - Perp 625.0 psi
Fv 180.0 psi
Ft 675.0 psi

E: Modulus of Elasticity
Ebend-xx 1,700.0 ksi
Eminbend-xx 620.0 ksi

Wood Species: Douglas Fir - Larch
Wood Grade: No.1

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Density 31.20pcf
Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loads

Uniform Load: D = 0.020 ksf, Tributary Width = 1.333 ft, (Storage)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.709	1	Maximum Shear Stress Ratio	=	0.194	: 1
Section used for this span		2x6		Section used for this span		2x6	
fb: Actual	=	953.58	psi	fv: Actual	=	31.41	psi
FB: Allowable	=	1,345.50	psi	Fv: Allowable	=	162.00	psi
Load Combination		+D+H		Load Combination		+D+H	
Location of maximum on span	=	6.500	ft	Location of maximum on span	=	12.573	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.000	in	Ratio =		0	<360
Max Upward Transient Deflection		0.000	in	Ratio =		0	<360
Max Downward Total Deflection		0.520	in	Ratio =		299	>=240.
Max Upward Total Deflection		0.000	in	Ratio =		0	<240.0

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios									Moment Values			Shear Values		
		M	V	C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	M	fb	F'b	V	fv	F'v
+D+H Length = 13.0 ft	1	0.709	0.194	0.90	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	1345.50	0.17	31.41	162.00
+D+L+H Length = 13.0 ft	1	0.638	0.175	1.00	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	1495.00	0.17	31.41	180.00
+D+Lr+H Length = 13.0 ft	1	0.510	0.140	1.25	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	1868.75	0.17	31.41	225.00
+D+S+H Length = 13.0 ft	1	0.555	0.152	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	1719.25	0.17	31.41	207.00
+D+0.750Lr+0.750L+H Length = 13.0 ft	1	0.510	0.140	1.25	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	1868.75	0.17	31.41	225.00
+D+0.750L+0.750S+H					1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Nelson Engineering
P.O. Box 1599
430 S. Cache St.
Jackson, Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Wood Beam

File = S:\Proj2017\2KZYV1-1SU53VC-6ITDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

Load Combination	Segment Length	Span #	Max Stress Ratios		C_d	C_{FV}	C_i	C_r	C_m	C_t	C_L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
Length = 13.0 ft	1		0.555	0.152	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	1719.25	0.17	31.41	207.00
+D+0.60W+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.399	0.109	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	2392.00	0.17	31.41	288.00
+D+0.70E+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.399	0.109	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	2392.00	0.17	31.41	288.00
+D+0.750Lr+0.750L+0.450W+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.399	0.109	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	2392.00	0.17	31.41	288.00
+D+0.750L+0.750S+0.450W+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.399	0.109	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	2392.00	0.17	31.41	288.00
+D+0.750L+0.750S+0.5250E+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.399	0.109	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.60	953.58	2392.00	0.17	31.41	288.00
+0.60D+0.60W+0.60H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.239	0.065	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.36	572.15	2392.00	0.10	18.85	288.00
+0.60D+0.70E+0.60H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.239	0.065	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.36	572.15	2392.00	0.10	18.85	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.5201	6.547		0.0000	0.000

Vertical Reactions

Support notation: Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.185	0.185
Overall MINimum	0.111	0.111
+D+H	0.185	0.185
+D+L+H	0.185	0.185
+D+Lr+H	0.185	0.185
+D+S+H	0.185	0.185
+D+0.750Lr+0.750L+H	0.185	0.185
+D+0.750L+0.750S+H	0.185	0.185
+D+0.60W+H	0.185	0.185
+D+0.70E+H	0.185	0.185
+D+0.750Lr+0.750L+0.450W+H	0.185	0.185
+D+0.750L+0.750S+0.450W+H	0.185	0.185
+D+0.750L+0.750S+0.5250E+H	0.185	0.185
+0.60D+0.60W+0.60H	0.111	0.111
+0.60D+0.70E+0.60H	0.111	0.111
D Only	0.185	0.185
Lr Only		
L Only		
S Only		
W Only		
E Only		
H Only		

FACE MOUNT HANGERS – SOLID SAWN LUMBER (DF/SP)

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

These products are approved for installation with the Strong-Drive® SD Connector screw. See page 27 for more information.

Joist Size	Model No.	Ga	Dimensions (in.)				Min/Max	Fasteners		DF/SP Allowable Loads				Installed Cost Index (ICI)	Code Ref.	
			W	H	B			Header	Joist	Uplift (160)	Floor (100)	Snow (115)	Roof (125)			
SAWN LUMBER SIZES																
TPL 2x6	LUS26-3	18	4 5/8	4 1/4	2			4-16d	4-16d	1165	1030	1180	1280	*	160	
	U26-3	16	4 5/8	4 1/4	2			8-16d	4-10d	740	1150	1305	1410	*		
	HU26-3/HUC26-3	14	4 1 1/16	5 1/2	2 1/2	Min		8-16d	4-10d	760	1190	1345	1445	*		
		14	4 1 1/16	5 1/2	2 1/2	Max		12-16d	6-10d	1135	1785	2015	2165	*		
2x8	LUS26	18	1 1/8	4 3/4	1 3/4	—		4-10d	4-10d	1165	865	990	1070	Lowest	I7, I27, F6, L5, L17	
	LU26	20	1 1/8	4 3/4	1 1/2	—		6-16d	4-10dx1 1/2	565	835	950	1030	+6%		
	LUS28	18	1 1/8	6 3/8	1 3/4	—		6-10d	4-10d	1165	1100	1255	1360	+23%		
	LU28	20	1 1/8	6 3/8	1 1/2	—		8-16d	6-10dx1 1/2	850	1110	1270	1335	+39%		
	U26	16	1 1/8	4 3/4	2	—		6-16d	4-10dx1 1/2	585	865	980	1055	+43%	I7, F6, L17	
	LUC26Z	18	1 1/8	4 3/4	1 3/4	—		6-16d	4-10dx1 1/2	730	845	965	1040	+160%		
	HU28	14	1 1/8	5 1/4	2 1/4	—		6-16d	4-10dx1 1/2	610	895	1005	1085	+251%		
	HUS26	16	1 1/8	5 3/8	3	—		14-16d	6-16d	1550	2720	3095	3335	+276%		
	HUS28	16	1 1/8	7	3	—		22-16d	8-16d	2000	3965	4120	4220	+409%	I7, I27, F6, L5, L17	
		18	3 1/8	4 7/8	2	—		4-16d	4-16d	1165	1030	1180	1280	Lowest		
DBL 2x8	LUS26-2	18	3 1/8	4 7/8	2	—		4-16d	4-16d	1165	1315	1500	1625	+8%		I7, F6, L17
	LUS28-2	18	3 1/8	7	2	—		6-16d	4-16d	1165	1315	1500	1625	+8%		
	U26-2	16	3 1/8	5	2	—		8-16d	4-10d	740	1150	1305	1410	+65%		
	HUS28-2/HUSC28-2	14	3 1/8	7 1/16	2	—		6-16d	6-16d	1550	1595	1815	1960	+188%		
HU28-2/HUC28-2		14	3 1/8	7	2 1/2	Min		10-16d	4-10d	760	1490	1680	1805	+397%		
	14	3 1/8	7	2 1/2	Min		14-16d	6-10d	1135	2085	2350	2530	+418%			
TPL 2x8	LUS28-3	18	4 5/8	6 1/4	2	—		6-16d	4-16d	1165	1315	1500	1625	*	160	
	U26-3	16	4 5/8	4 1/4	2	—		8-16d	4-10d	740	1150	1305	1410	*		
	HU26-3/HUC26-3	14	4 1 1/16	5 1/2	2 1/2	Min		8-16d	4-10d	760	1190	1345	1445	*		
		14	4 1 1/16	5 1/2	2 1/2	Max		12-16d	6-10d	1135	1785	2015	2165	*		
QUAD 2x8	HU28-4/HUC28-4	14	14	6 1/8	6 1/8	Min		10-16d	4-16d	900	1490	1680	1805	*		
		14	14	6 1/8	6 1/8	Max		14-16d	6-16d	1345	2085	2350	2530	*		
2x10	LUS28	18	1 1/8	6 3/8	1 3/4	—		6-10d	4-10d	1165	1100	1255	1360	Lowest	I7, I27, F6, L5, L17	
	LU28	20	1 1/8	6 3/8	1 1/2	—		8-16d	6-10dx1 1/2	850	1110	1270	1335	+13%		
	LUS210	18	1 1/8	7 13/16	1 3/4	—		8-10d	4-10d	1165	1340	1525	1650	+15%		
	LU210	20	1 1/8	7 13/16	1 1/2	—		10-16d	6-10dx1 1/2	850	1390	1585	1715	+28%		
	U210	16	1 1/8	7 13/16	2	—		10-16d	6-10dx1 1/2	1110	1440	1635	1685	+76%	I7, F6, L17	
	LUC210Z	18	1 1/8	7 3/4	1 3/4	—		10-16d	6-10dx1 1/2	1100	1410	1605	1735	+180%		
	HU210	14	1 1/8	7 1/8	2 1/4	—		8-16d	4-10dx1 1/2	610	1190	1345	1445	+225%		
	HUS210	16	1 1/8	9	3	—		30-16d	10-16d	3000	4255	4445	4575	+450%		
	DBL 2x10	LUS28-2	18	3 1/8	7	2	—		6-16d	4-16d	1165	1315	1500	1625	Lowest	I7, I27, F6, L5, L17
		LUS210-2	18	3 1/8	9	2	—		8-16d	6-16d	1745	1830	2090	2265	+34%	
U210-2		16	3 1/8	8 1/2	2	—		14-16d	6-10d	1110	2015	2285	2465	+88%		
HUS210-2/HUSC210-2		14	3 1/8	9 1/16	2	—		8-16d	8-16d	3295	2125	2420	2615	+217%		
		HU210-2/HUC210-2	14	3 1/8	8 13/16	2 1/2	Min		14-16d	6-10d	1135	2085	2350	2530	+441%	
14			3 1/8	8 13/16	2 1/2	Max		18-16d	10-10d	1895	2680	3020	3250	+467%		
TPL 2x10	HUCQ210-2-SDS	14	2 3/8	9	3	—		12-1/4"x2 1/2" SDS	6-1/4"x2 1/2" SDS	2510	4680	4955	4955	*	F23	
	HHUS210-2	14	3 5/8	9 5/8	3	—		30-16d	10-16d	4000	5635	6380	6880	*	F23	
	LUS28-3	18	4 5/8	6 1/4	2	—		6-16d	4-16d	1165	1315	1500	1625	*	160	
	LUS210-3	18	4 5/8	8 3/8	2	—		8-16d	6-16d	1745	1830	2090	2265	*		
	U210-3	16	4 5/8	7 3/4	2	—		14-16d	6-10d	1110	2015	2285	2465	*		
	HU210-3/HUC210-3	14	4 1 1/16	8 3/8	2 1/2	Min		14-16d	6-10d	1135	2085	2350	2530	*		I7, F6, L17
		14	4 1 1/16	8 3/8	2 1/2	Max		18-16d	10-10d	1895	2680	3020	3250	*		
		HHUS210-3	14	4 1 1/16	8 3/8	3	—		30-16d	10-16d	4000	5635	6380	6880	*	
		HGUS210-3	12	4 15/16	9 1/4	4	—		46-16d	16-16d	4095	9100	9100	9100	*	
	HUCQ210-3-SDS	14	4 5/8	9	3	—		8-1/4"x2 1/2" SDS	4-1/4"x2 1/2" SDS	2510	4680	4955	4955	*	F23	
QUAD 2x10	HU210-4/HUC210-4	14	6 1/8	8 3/8	2 1/2	Min		14-16d	6-16d	1345	2085	2350	2530	*	160	
		14	6 1/8	8 3/8	2 1/2	Max		18-16d	8-16d	1795	2680	3020	3250	*		
	HHUS210-4	14	6 1/8	8 3/8	3	—		30-16d	10-16d	4000	5635	6380	6880	*		F23, 160
	HGUS210-4	12	6 1/8	9 1/4	4	—		46-16d	16-16d	4095	9100	9100	9100	*		
2x12	LUS210	18	1 1/8	7 13/16	1 3/4	—		8-10d	4-10d	1165	1340	1525	1650	Lowest	I7, I27, F6, L5, L17	
	LU210	20	1 1/8	7 13/16	1 1/2	—		10-16d	6-10dx1 1/2	850	1390	1585	1715	+11%		
	U210	16	1 1/8	7 13/16	2	—		10-16d	6-10dx1 1/2	1110	1440	1635	1685	+53%		
	LUC210Z	18	1 1/8	7 3/4	1 3/4	—		10-16d	6-10dx1 1/2	1100	1410	1605	1735	+180%		
	HU212	14	1 1/8	9	2 1/4	—		10-16d	6-10dx1 1/2	1135	1490	1680	1805	+347%	I7, I27, F6, L5, L17	
	HUS210	16	1 1/8	9	3	—		30-16d	10-16d	3000	4255	4445	4575	+378%		
	LUS210-2	18	3 1/8	9	2	—		8-16d	6-16d	1745	1830	2090	2265	Lowest		
	U210-2	16	3 1/8	8 1/2	2	—		14-16d	6-10d	1110	2015	2285	2465	+40%		
	DBL 2x12	LUS214-2	18	3 1/8	10 15/16	2	—		10-16d	6-16d	1745	2110	2410	2610	+56%	I7, F6, L17
		HUS210-2	14	3 1/8	9 1/16	2	—		8-16d	8-16d	3295	2125	2420	2615	*	
HUS212-2/HUSC212-2		14	3 1/8	10 3/4	2	—		10-16d	10-16d	3635	2660	3025	3265	*		
		14	3 1/8	10 3/4	2 1/2	Min		16-16d	6-10d	1135	2380	2685	2890	*		
HU212-2/HUC212-2		14	3 1/8	10 3/4	2 1/2	Max		22-16d	10-10d	1895	3275	3695	3970	+411%		
		HUCQ210-2-SDS	14	2 3/8	9	3	—		12-1/4"x2 1/2" SDS	6-1/4"x2 1/2" SDS	2510	4680	4955	4955	*	F23
			14	2 3/8	9	3	—		12-1/4"x2 1/2" SDS	6-1/4"x2 1/2" SDS	2510	4680	4955	4955	*	

Wood Beam

File = S:\Proj2017\2KZYV1-1SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set: IBC 2015

Material Properties

Analysis Method: Allowable Stress Design
Load Combination IBC 2015

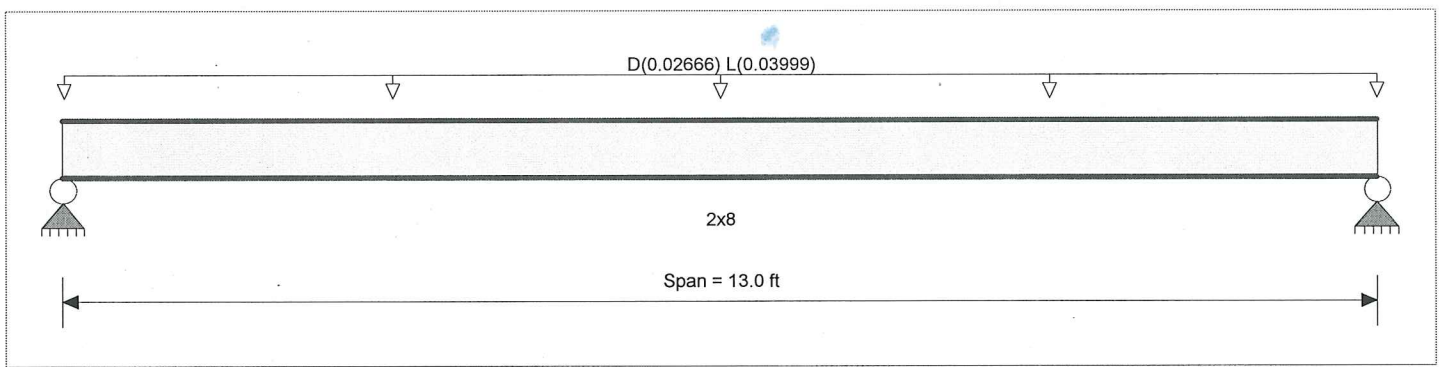
Wood Species: Douglas Fir - Larch
Wood Grade: No.1

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Fb + 1,000.0 psi
Fb - 1,000.0 psi
Fc - Prll 1,500.0 psi
Fc - Perp 625.0 psi
Fv 180.0 psi
Ft 675.0 psi

E: Modulus of Elasticity
Ebend-xx 1,700.0 ksi
Eminbend-xx 620.0 ksi

Density 31.20pcf
Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loads

Uniform Load: D = 0.020, L = 0.030 ksf, Tributary Width = 1.333 ft, (Storage)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.965	1	Maximum Shear Stress Ratio	=	0.314	: 1
Section used for this span		2x8		Section used for this span		2x8	
fb: Actual	=	1,331.22	psi	f _v : Actual	=	56.45	psi
FB: Allowable	=	1,380.00	psi	F _v : Allowable	=	180.00	psi
Load Combination		+D+L+H		Load Combination		+D+L+H	
Location of maximum on span	=	6.500	ft	Location of maximum on span	=	12.431	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.319	in	Ratio =		488	>=360
Max Upward Transient Deflection		0.000	in	Ratio =		0	<360
Max Downward Total Deflection		0.551	in	Ratio =		283	>=240
Max Upward Total Deflection		0.000	in	Ratio =		0	<240.0

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{FN}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	f _b	F' _b	V	f _v	F' _v
+D+H	Length = 13.0 ft	1	0.451	0.147	0.90	1.200	1.00	1.15	1.00	1.00	1.00	0.61	559.76	1242.00	0.17	23.74	162.00
+D+L+H	Length = 13.0 ft	1	0.965	0.314	1.00	1.200	1.00	1.15	1.00	1.00	1.00	1.46	1,331.22	1380.00	0.41	56.45	180.00
+D+Lr+H	Length = 13.0 ft	1	0.324	0.105	1.25	1.200	1.00	1.15	1.00	1.00	1.00	0.61	559.76	1725.00	0.17	23.74	225.00
+D+S+H	Length = 13.0 ft	1	0.353	0.115	1.15	1.200	1.00	1.15	1.00	1.00	1.00	0.61	559.76	1587.00	0.17	23.74	207.00
+D+0.750Lr+0.750L+H	Length = 13.0 ft	1	0.660	0.215	1.25	1.200	1.00	1.15	1.00	1.00	1.00	1.25	1,138.36	1725.00	0.35	48.27	225.00
+D+0.750L+0.750S+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Nelson Engineering
P.O. Box 1599
430 S. Cache St.
Jackson, Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{F/V}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
Length = 13.0 ft	1		0.717	0.233	1.15	1.200	1.00	1.15	1.00	1.00	1.00	1.25	1,138.36	1587.00	0.35	48.27	207.00
+D+0.60W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.254	0.082	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.61	559.76	2208.00	0.17	23.74	288.00
+D+0.70E+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.254	0.082	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.61	559.76	2208.00	0.17	23.74	288.00
+D+0.750Lr+0.750L+0.450W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.516	0.168	1.60	1.200	1.00	1.15	1.00	1.00	1.00	1.25	1,138.36	2208.00	0.35	48.27	288.00
+D+0.750L+0.750S+0.450W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.516	0.168	1.60	1.200	1.00	1.15	1.00	1.00	1.00	1.25	1,138.36	2208.00	0.35	48.27	288.00
+D+0.750L+0.750S+0.5250E+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.516	0.168	1.60	1.200	1.00	1.15	1.00	1.00	1.00	1.25	1,138.36	2208.00	0.35	48.27	288.00
+0.60D+0.60W+0.60H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.152	0.049	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.37	335.86	2208.00	0.10	14.24	288.00
+0.60D+0.70E+0.60H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1		0.152	0.049	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.37	335.86	2208.00	0.10	14.24	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.5508	6.547		0.0000	0.000

Vertical Reactions

Support notation: Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.449	0.449
Overall MINimum	0.113	0.113
+D+H	0.189	0.189
+D+L+H	0.449	0.449
+D+Lr+H	0.189	0.189
+D+S+H	0.189	0.189
+D+0.750Lr+0.750L+H	0.384	0.384
+D+0.750L+0.750S+H	0.384	0.384
+D+0.60W+H	0.189	0.189
+D+0.70E+H	0.189	0.189
+D+0.750Lr+0.750L+0.450W+H	0.384	0.384
+D+0.750L+0.750S+0.450W+H	0.384	0.384
+D+0.750L+0.750S+0.5250E+H	0.384	0.384
+0.60D+0.60W+0.60H	0.113	0.113
+0.60D+0.70E+0.60H	0.113	0.113
D Only	0.189	0.189
Lr Only		
L Only	0.260	0.260
S Only		
W Only		
E Only		
H Only		

Wood Beam

File = S:\Proj2017\2KZYV1-1SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set: IBC 2015

Material Properties

Analysis Method: Allowable Stress Design
Load Combination IBC 2015

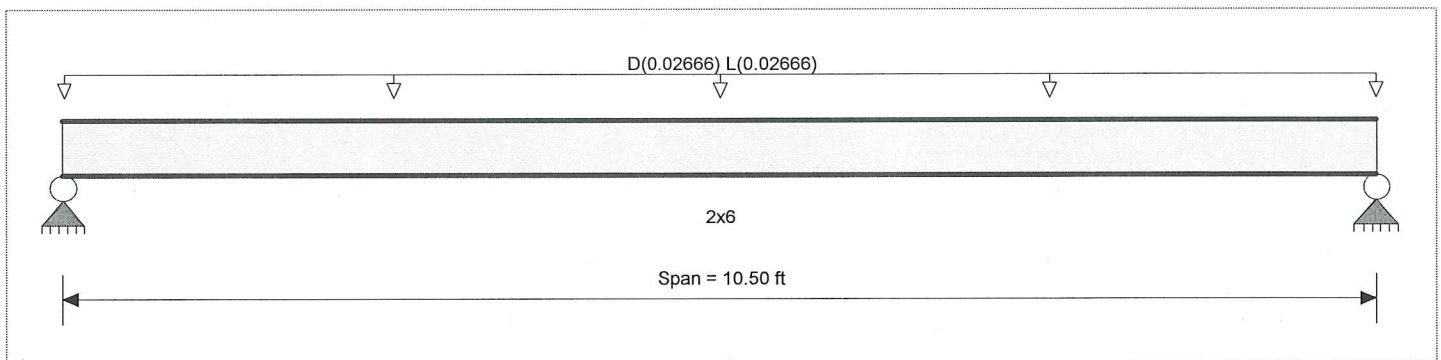
Wood Species: Douglas Fir - Larch
Wood Grade: No.1

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Fb + 1,000.0 psi
Fb - 1,000.0 psi
Fc - Prll 1,500.0 psi
Fc - Perp 625.0 psi
Fv 180.0 psi
Ft 675.0 psi

E: Modulus of Elasticity
Ebend-xx 1,700.0 ksi
Eminbend-xx 620.0 ksi

Density 31.20pcf
Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loads

Uniform Load: D = 0.020, L = 0.020 ksf, Tributary Width = 1.333 ft, (Storage)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.806	1	Maximum Shear Stress Ratio	=	0.269	: 1
Section used for this span		2x6		Section used for this span		2x6	
fb: Actual	=	1,205.08	psi	fv: Actual	=	48.38	psi
FB: Allowable	=	1,495.00	psi	Fv: Allowable	=	180.00	psi
Load Combination		+D+L+H		Load Combination		+D+L+H	
Location of maximum on span	=	5.250	ft	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		0.207	in	Ratio =		607	>=360
Max Upward Transient Deflection		0.000	in	Ratio =		0	<360
Max Downward Total Deflection		0.429	in	Ratio =		293	>=240.
Max Upward Total Deflection		0.000	in	Ratio =		0	<240.0

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
+D+H	Length = 10.50 ft	1	0.462	0.154	0.90	1.300	1.00	1.15	1.00	1.00	1.00	0.39	622.08	1345.50	0.14	24.97	162.00
+D+L+H	Length = 10.50 ft	1	0.806	0.269	1.00	1.300	1.00	1.15	1.00	1.00	1.00	0.76	1,205.08	1495.00	0.27	48.38	180.00
+D+Lr+H	Length = 10.50 ft	1	0.333	0.111	1.25	1.300	1.00	1.15	1.00	1.00	1.00	0.39	622.08	1868.75	0.14	24.97	225.00
+D+S+H	Length = 10.50 ft	1	0.362	0.121	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.39	622.08	1719.25	0.14	24.97	207.00
+D+0.750Lr+0.750L+H	Length = 10.50 ft	1	0.567	0.189	1.25	1.300	1.00	1.15	1.00	1.00	1.00	0.67	1,059.33	1868.75	0.23	42.53	225.00
+D+0.750L+0.750S+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Nelson Engineering
P.O. Box 1599
430 S. Cache St.
Jackson, Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Wood Beam

File = S:\Proj2017\2KZYV1-1SU53VC-6TDLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

Load Combination	Segment Length	Span #	Max Stress Ratios		C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	Moment Values			Shear Values		
			M	V								M	f _b	F' _b	V	f _v	F' _v
Length = 10.50 ft	1		0.616	0.205	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.67	1,059.33	1719.25	0.23	42.53	207.00
+D+0.60W+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.260	0.087	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.39	622.08	2392.00	0.14	24.97	288.00
+D+0.70E+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.260	0.087	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.39	622.08	2392.00	0.14	24.97	288.00
+D+0.750Lr+0.750L+0.450W+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.443	0.148	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.67	1,059.33	2392.00	0.23	42.53	288.00
+D+0.750L+0.750S+0.450W+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.443	0.148	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.67	1,059.33	2392.00	0.23	42.53	288.00
+D+0.750L+0.750S+0.5250E+H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.443	0.148	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.67	1,059.33	2392.00	0.23	42.53	288.00
+0.60D+0.60W+0.60H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.156	0.052	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.24	373.25	2392.00	0.08	14.98	288.00
+0.60D+0.70E+0.60H						1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.156	0.052	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.24	373.25	2392.00	0.08	14.98	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.4288	5.288		0.0000	0.000

Vertical Reactions

Support notation: Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.289	0.289
Overall MINimum	0.090	0.090
+D+H	0.149	0.149
+D+L+H	0.289	0.289
+D+Lr+H	0.149	0.149
+D+S+H	0.149	0.149
+D+0.750Lr+0.750L+H	0.254	0.254
+D+0.750L+0.750S+H	0.254	0.254
+D+0.60W+H	0.149	0.149
+D+0.70E+H	0.149	0.149
+D+0.750Lr+0.750L+0.450W+H	0.254	0.254
+D+0.750L+0.750S+0.450W+H	0.254	0.254
+D+0.750L+0.750S+0.5250E+H	0.254	0.254
+0.60D+0.60W+0.60H	0.090	0.090
+0.60D+0.70E+0.60H	0.090	0.090
D Only	0.149	0.149
Lr Only		
L Only	0.140	0.140
S Only		
W Only		
E Only		
H Only		

Wood Beam

File = S:\Proj2017\2KZYV1-1\SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set: IBC 2015

Material Properties

Analysis Method: Allowable Stress Design
Load Combination: IBC 2015

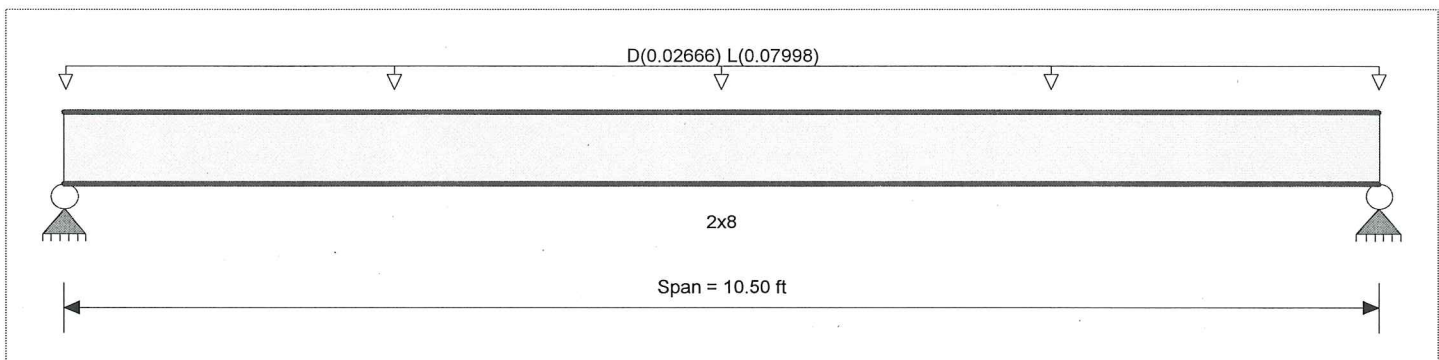
Fb + 1,000.0 psi
Fb - 1,000.0 psi
Fc - Prll 1,500.0 psi
Fc - Perp 625.0 psi
Fv 180.0 psi
Ft 675.0 psi

E: Modulus of Elasticity
Ebend-xx 1,700.0 ksi
Eminbend-xx 620.0 ksi

Wood Species: Douglas Fir - Larch
Wood Grade: No.1

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Density 31.20pcf
Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loads

Uniform Load: D = 0.020, L = 0.060 ksf, Tributary Width = 1.333 ft, (Storage)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.994	1	Maximum Shear Stress Ratio	=	0.390	: 1
Section used for this span		2x8		Section used for this span		2x8	
fb: Actual	=	1,371.72	psi	fv: Actual	=	70.29	psi
FB: Allowable	=	1,380.00	psi	Fv: Allowable	=	180.00	psi
Load Combination		+D+L+H		Load Combination		+D+L+H	
Location of maximum on span	=	5.250	ft	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		0.272	in	Ratio =		463	>=360
Max Upward Transient Deflection		0.000	in	Ratio =		0	<360
Max Downward Total Deflection		0.370	in	Ratio =		340	>=240
Max Upward Total Deflection		0.000	in	Ratio =		0	<240.0

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		Moment Values										Shear Values		
			M	V	C _d	C _{FV}	C _i	C _r	C _m	C _t	C _L	M	fb	F'b	V	fv	F'v
+D+H	Length = 10.50 ft	1	0.294	0.116	0.90	1.200	1.00	1.15	1.00	1.00	1.00	0.40	365.17	1242.00	0.00	0.00	0.00
+D+L+H	Length = 10.50 ft	1	0.994	0.390	1.00	1.200	1.00	1.15	1.00	1.00	1.00	1.50	1,371.72	1380.00	0.00	0.00	0.00
+D+Lr+H	Length = 10.50 ft	1	0.212	0.083	1.25	1.200	1.00	1.15	1.00	1.00	1.00	0.40	365.17	1725.00	0.00	0.00	0.00
+D+S+H	Length = 10.50 ft	1	0.230	0.090	1.15	1.200	1.00	1.15	1.00	1.00	1.00	0.40	365.17	1587.00	0.00	0.00	0.00
+D+0.750Lr+0.750L+H	Length = 10.50 ft	1	0.649	0.255	1.25	1.200	1.00	1.15	1.00	1.00	1.00	1.23	1,120.08	1725.00	0.00	0.00	0.00
+D+0.750L+0.750S+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Nelson Engineering
P.O. Box 1599
430 S. Cache St.
Jackson, Wyoming

Project Title: TOJ Fairground Building
Engineer: WAA
Project Descr:

Project ID: 17-202-01

Wood Beam

File = S:\Proj2017\2KZYV1-1SU53VC-6\TDLLGK-A.EC6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. #: KW-06004539

Licensee: NELSON ENGINEERING

Description: Ceiling joists

Load Combination	Segment Length	Span #	Max Stress Ratios		C_d	C_{FV}	C_i	C_r	C_m	C_t	C_L	Moment Values			Shear Values		
			M	V								M	fb	F'b	V	fv	F'v
Length = 10.50 ft	1		0.706	0.277	1.15	1.200	1.00	1.15	1.00	1.00	1.00	1.23	1,120.08	1587.00	0.42	57.39	207.00
+D+0.60W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.165	0.065	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.40	365.17	2208.00	0.14	18.71	288.00
+D+0.70E+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.165	0.065	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.40	365.17	2208.00	0.14	18.71	288.00
+D+0.750Lr+0.750L+0.450W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.507	0.199	1.60	1.200	1.00	1.15	1.00	1.00	1.00	1.23	1,120.08	2208.00	0.42	57.39	288.00
+D+0.750L+0.750S+0.450W+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.507	0.199	1.60	1.200	1.00	1.15	1.00	1.00	1.00	1.23	1,120.08	2208.00	0.42	57.39	288.00
+D+0.750L+0.750S+0.5250E+H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.507	0.199	1.60	1.200	1.00	1.15	1.00	1.00	1.00	1.23	1,120.08	2208.00	0.42	57.39	288.00
+0.60D+0.60W+0.60H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.099	0.039	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.24	219.10	2208.00	0.08	11.23	288.00
+0.60D+0.70E+0.60H						1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1		0.099	0.039	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.24	219.10	2208.00	0.08	11.23	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.3703	5.288		0.0000	0.000

Vertical Reactions

Support notation: Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.572	0.572
Overall MINimum	0.091	0.091
+D+H	0.152	0.152
+D+L+H	0.572	0.572
+D+Lr+H	0.152	0.152
+D+S+H	0.152	0.152
+D+0.750Lr+0.750L+H	0.467	0.467
+D+0.750L+0.750S+H	0.467	0.467
+D+0.60W+H	0.152	0.152
+D+0.70E+H	0.152	0.152
+D+0.750Lr+0.750L+0.450W+H	0.467	0.467
+D+0.750L+0.750S+0.450W+H	0.467	0.467
+D+0.750L+0.750S+0.5250E+H	0.467	0.467
+0.60D+0.60W+0.60H	0.091	0.091
+0.60D+0.70E+0.60H	0.091	0.091
D Only	0.152	0.152
Lr Only		
L Only	0.420	0.420
S Only		
W Only		
E Only		
H Only		



Design Maps Summary Report

User-Specified Input

Report Title 17-202-01

Wed July 19, 2017 14:35:42 UTC

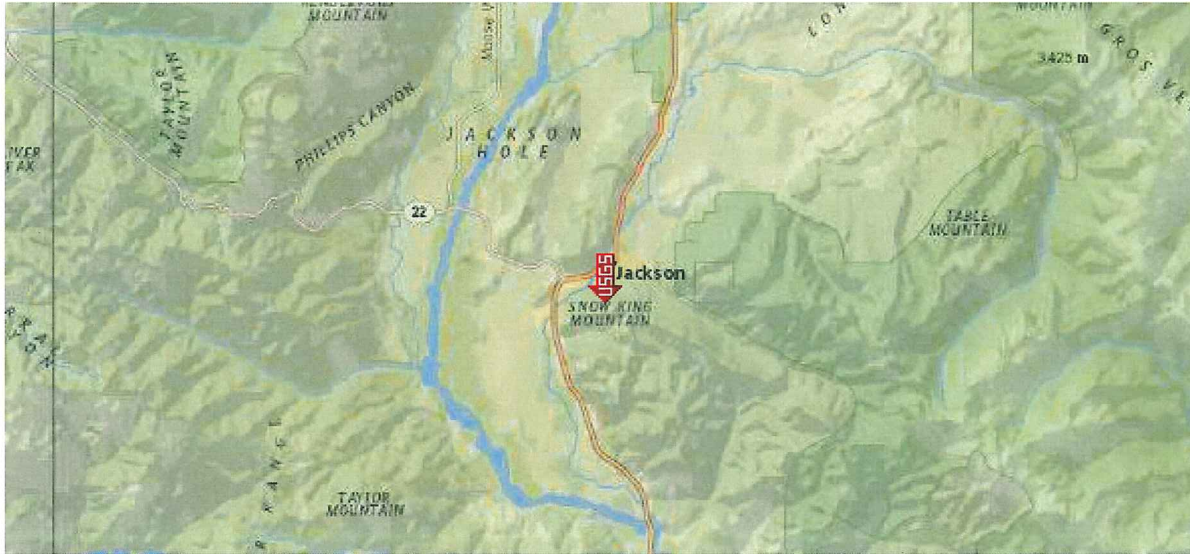
Building Code Reference Document ASCE 7-10 Standard

(which utilizes USGS hazard data available in 2008)

Site Coordinates 43.4741°N, 110.7682°W

Site Soil Classification Site Class D – "Stiff Soil"

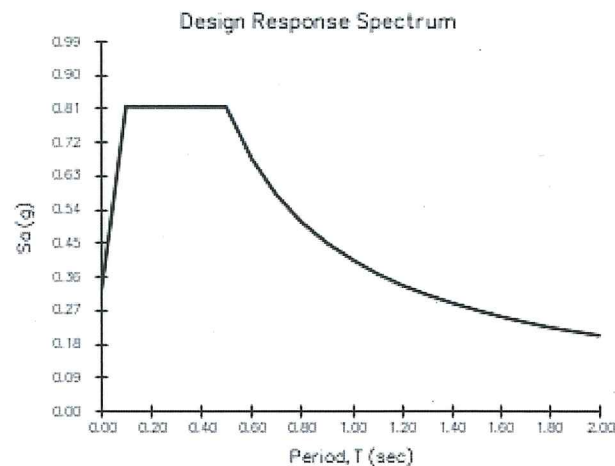
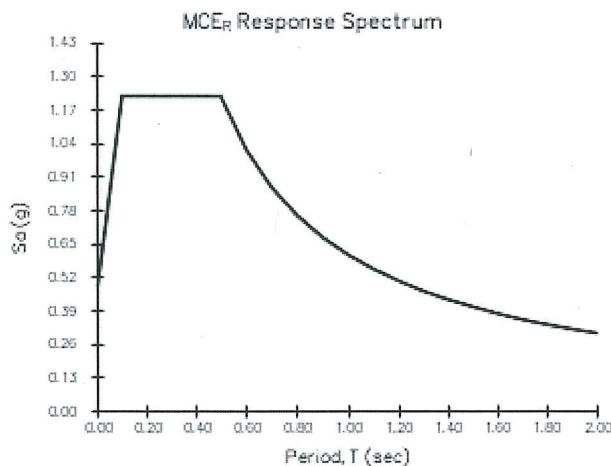
Risk Category IV (e.g. essential facilities) – Firestation



USGS-Provided Output

$S_s = 1.201 \text{ g}$ $S_{MS} = 1.225 \text{ g}$ $S_{DS} = 0.816 \text{ g}$
 $S_1 = 0.364 \text{ g}$ $S_{M1} = 0.609 \text{ g}$ $S_{D1} = 0.406 \text{ g}$

For information on how the S_s and S_1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.



For PGA_M , T_L , C_{RS} , and C_{R1} values, please [view the detailed report](#).

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Summary Report

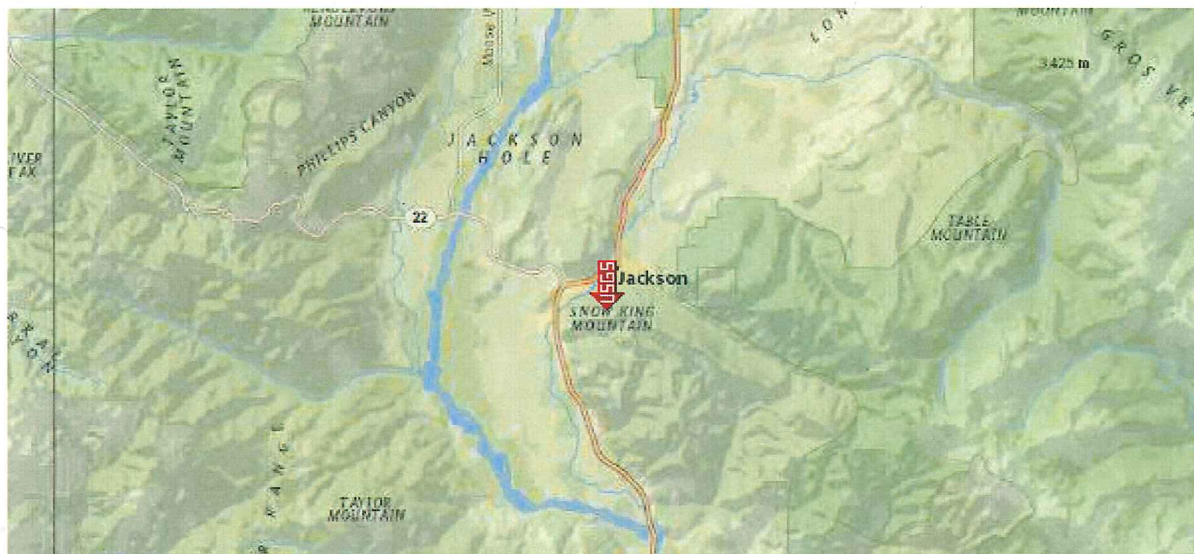
User-Specified Input

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 43.4741°N, 110.7682°W

Site Soil Classification Site Class D – “Stiff Soil”

Risk Category I/II/III

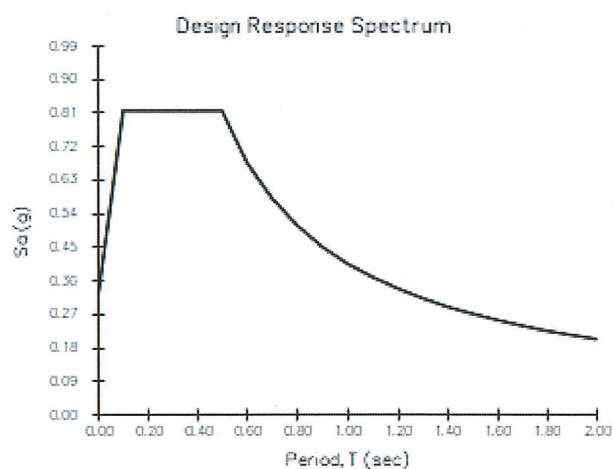
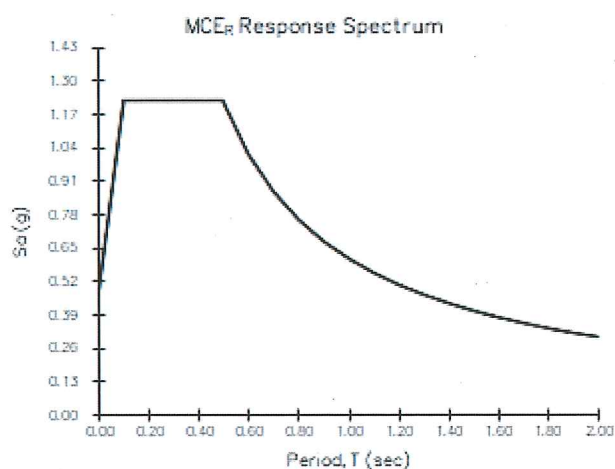


USGS–Provided Output

$S_S = 1.201 \text{ g}$ $S_{MS} = 1.225 \text{ g}$ $S_{DS} = 0.816 \text{ g}$

$S_1 = 0.364 \text{ g}$ $S_{M1} = 0.609 \text{ g}$ $S_{D1} = 0.406 \text{ g}$

For information on how the S_S and S_1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.



For PGA_M , T_L , C_{RS} , and C_{R1} values, please [view the detailed report](#).

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

Seismic

$$SDS = 0.816$$

Steel ordinary moment frame: seismic category D

Note i: see 12.2.5.6 for limitations in structures assigned to seismic cat. D

$$\begin{array}{llll} 12.2.5.6: & \text{permitted up to } h = 65 \text{ ft} & > & h \approx 21.5' \quad \text{OK} \\ & DL \leq 20 \text{ psf} & = & DL = 20 \text{ psf} \quad \text{OK} \\ & \text{wall } DL > 35' \text{ from base} & & \text{n/a} \quad \text{OK} \end{array}$$

$\therefore$ OK to consider steel ordinary moment frame

$$R = 3.5; \quad \gamma = 3; \quad C_d = 3 \quad [\text{Table 12.2-1}]$$

$$C_s = \frac{SDS}{R/I} \quad \text{where } I = 1.50 \quad \checkmark \quad \text{for risk category IV (firehouse) from Table 1.5-2}$$

$$C_s = \frac{0.816}{3.5/1.5} = \underline{0.3497} = C_s$$

Using: 20 psf DL
75 psf SL (Town of Jackson)

$$W_R = DL + 0.2 SL = 20 \text{ psf} + 0.2(75 \text{ psf}) = \underline{35 \text{ psf}} = W_R$$

$$T_R = W_R C_s = 35 \text{ psf} (0.3497) = \underline{12.24 \text{ psf}} = T_R \quad \text{LRFD}$$

w/ 5.7 psf DL (to match original)

$$W_R = DL = 5.7 \text{ psf}$$

$$T_R = 5.7 \text{ psf} (0.3497) = \underline{1.993 \text{ psf}} = T_R$$

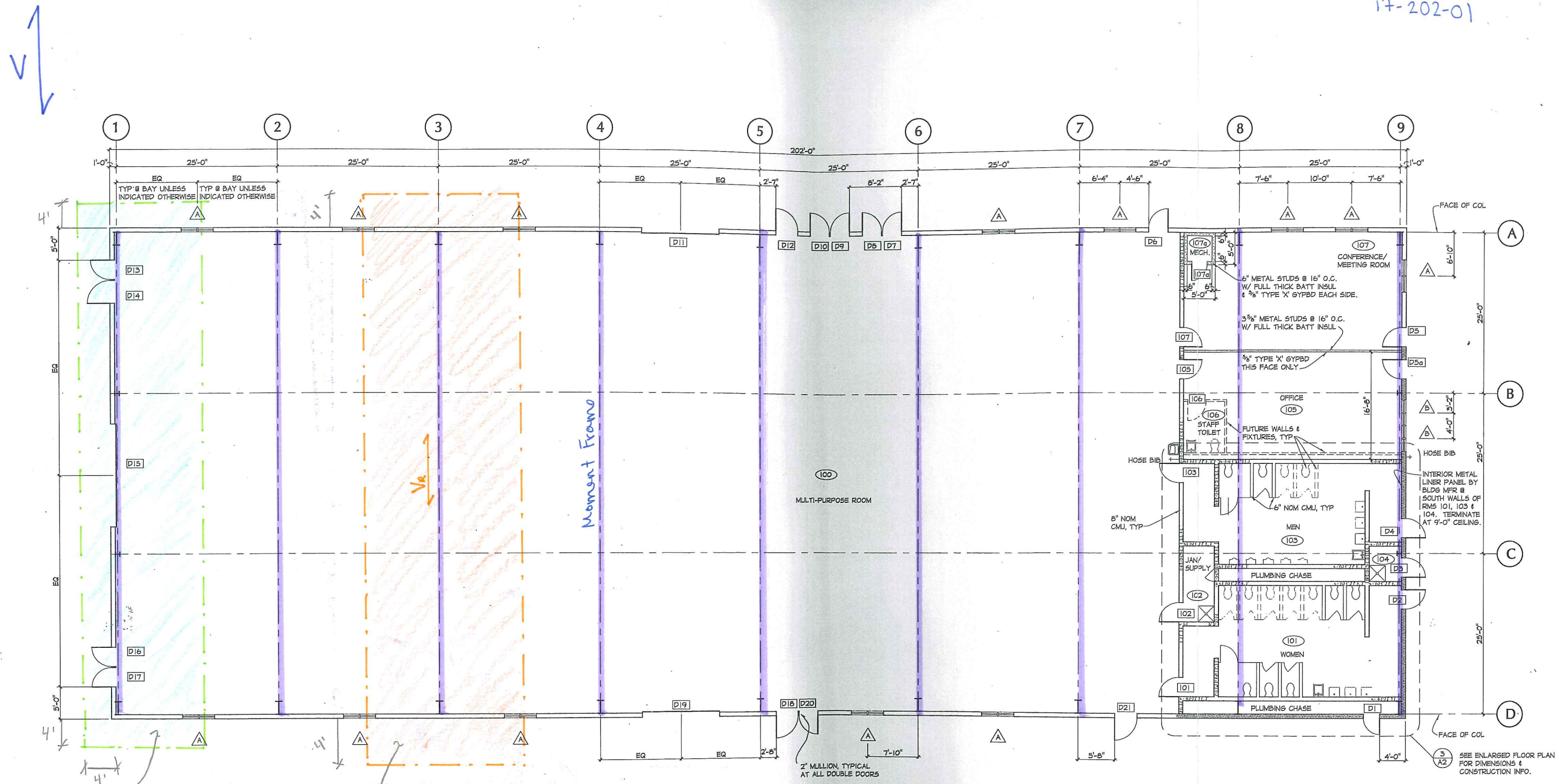
Table 12.2-1 (Continued)

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_o^b	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits ^c				
					Seismic Design Category				
					B	C	D ^d	E ^d	F ^e
19. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
21. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35 ^h	NP ^h	NP ^h
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP ⁱ	NP ⁱ	NP ⁱ
5. Special reinforced concrete moment frames ^a	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame ^p	14.1	3½	3 ^o	3½	35	35	35	35	35
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES									
	12.2.5.1								
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Steel special concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls ^{lm}	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls ^l	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Steel and concrete composite special concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
7. Steel and concrete composite plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Steel and concrete composite special shear walls	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Steel and concrete composite ordinary shear walls	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Steel special plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL

Continued

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

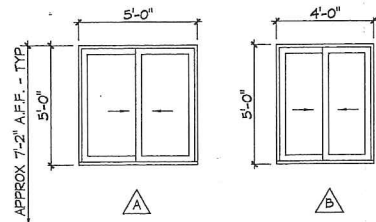
Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_o^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits ^c				
					Seismic Design Category				
					B	C	D ^d	E ^d	F ^e
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls ^{l, m}	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls ^l	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls ^l	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls ^l	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls ^l	14.2	4	2½	4	NL	NL	40 ^k	40 ^k	40 ^k
6. Ordinary precast shear walls ^l	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35 ^j	35 ^j	NP ^j
4. Special reinforced concrete shear walls ^{l, m}	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls ^l	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls ^l	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls ^l	14.2	1½	2½	1½	NL	NP	NP	NP	NP
8. Intermediate precast shear walls ^l	14.2	5	2½	4½	NL	NL	40 ^k	40 ^k	40 ^k
9. Ordinary precast shear walls ^l	14.2	4	2½	4	NL	NP	NP	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2 ½	4	NL	NL	160	160	100
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
18. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP



$A = \left(\frac{25'}{2} + 4'\right) \times (25' + 25' + 25' + 4' + 4') = 16.5' \times 83' = 1369.5 \text{ ft}^2$
 $A_{typ} = \left(\frac{25'}{2} + \frac{25'}{2}\right) \times (25' + 25' + 25' + 4' + 4') = 25' \times 83' = 2075 \text{ ft}^2$
 $V_R = T_R A_R = 12.24 \text{ psf} (2075 \text{ ft}^2) = 25,398 \text{ lb LRFD}$
 $V_R = T_R A_R = 1.993 \text{ psf} (1369.5 \text{ ft}^2) = 2729.414 \text{ lb}$
 $V_R = T_R A_R = 1.993 \text{ psf} (2075 \text{ ft}^2) = 4135.475 \text{ lb LRFD}$
 $x1.33 = 5500.18 \text{ lb LRFD}$

FLOOR PLAN

WINDOW TYPES



NOTE:
 ALL WINDOWS TO BE
 PROVIDED BY METAL BLDG
 MANUFACTURER. GLAZING
 SHALL BE 3/8" INSULATED
 CLEAR GLASS.

■ = steel frame locations

NOTES

1. # INDICATES DOOR NUMBER - SEE SHEET A2 FOR DOOR SCHEDULE.
2. # INDICATES ROOM NUMBER - SEE SHEET A2 FOR FINISH SCHEDULE.
3. A INDICATES WINDOW NUMBER - SEE WINDOW TYPES THIS SHEET.
4. PROVIDE VINYL FACED BUILDING INSULATION (BY METAL BLDG MFR) FROM SLAB TO TOP OF LOFT FLOOR WHERE INDICATED BY .
5. PROVIDE FULL THICK FIBERGLASS BATT INSULATION AND 6 MIL VISQUEEN VAPOR BARRIER IN CEILING OF ROOMS 101, 102, 103, 104, 105 & 107a. SEE STRUCTURAL DWGS FOR SIZE & SPACING OF CEILING FRAMING.
6. SEE SHEET A2 FOR ADDITIONAL NOTES.
7. SEE STRUCTURAL DWGS FOR ADDITIONAL INFORMATION ON METAL BLDG.

$E_h :$

p_{Eh} Roof Bracing

$$Q_o = 2.0$$

$$R = 3.25$$

$$C_d = 3.25$$

$Q_o E_h$ Wall Bracing

$$\sim 5.7 \text{ psf DL}$$

$$+ 3 \text{ psf C}$$

$$8.7 \text{ psf}$$

$$x 75' \times 202' = 131,805 \#$$

$$S_{ps} = 0.816$$

$$R = 3.25$$

$$I = 1.5$$

$$\frac{0.816}{3.25/1.5} = 0.3497$$

+ Endwall

$$131,805 \# \times 0.3497 = 46,094 \#$$

$$p = 2 - \frac{20}{\frac{1}{3} \sqrt{A_{exw}}}$$

$$\frac{1 \text{ bay}}{3 \text{ bays}}$$

$$p_{Eh} = 59,922$$

$$Q_o E_h = 92,194$$

$$p_{Eh} \div \text{Width} = 59,922 \# \div 75' = 798.96 \text{ lb/ft} \times 18.75' = 14,980 \#$$

$$\div 2 \text{ bays}$$

$$= 7,490 \#$$

$$75' \text{ wide} \div 2 = 37.5' \div 2 = 18.75'$$

$$Q_o E_h = 92,194 \# \text{ Total} \div 2 \text{ bays} = 46,097 \# / \text{ Bay}$$

$$\div 2 \text{ walls} = 23,048 \# / \text{ wall}$$



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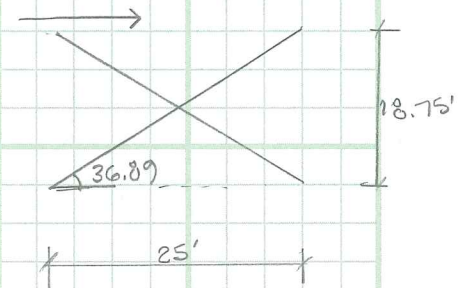
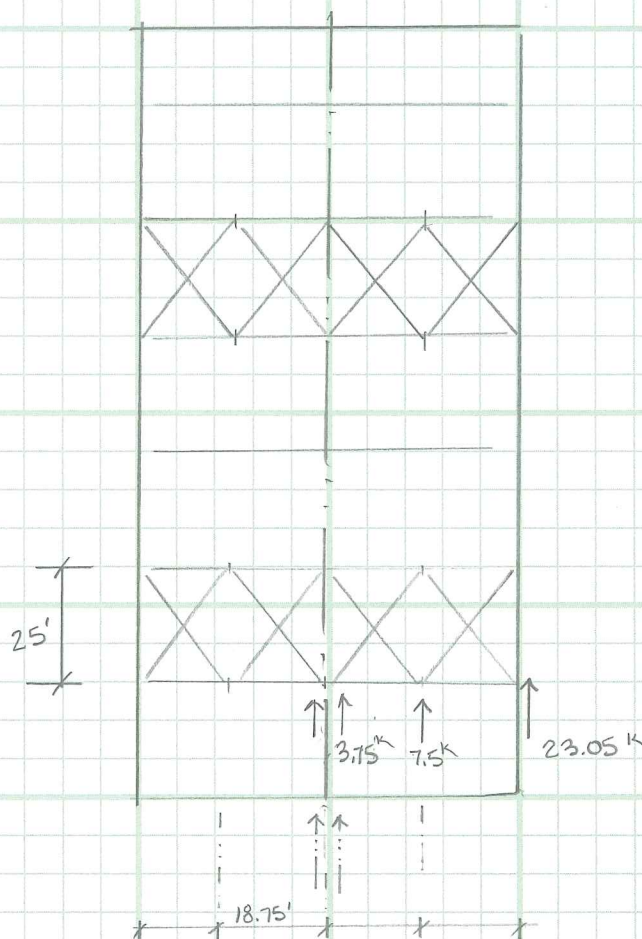
Project Temporary Fire Station Job no. 17-202-01
Cross Bracing sheet no. _____ of _____
By CH Date _____

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$\frac{3}{8}$ " Extra High Strength Cable

$$\text{Max Tension} = \frac{\text{Seismic Load} / x\text{-braces}}{\cos \phi}$$

$$\text{Cable Strength} = \frac{15.40 \text{ k}}{2} = 7.7 \text{ k}$$



$$\cos \theta = \frac{a}{h}$$

$$\cos 36.89 = 0.8$$

$$3.75 \text{ k} \div 0.8 = 4.6875 \text{ k}$$

$$7.5 \text{ k} \div 0.8 = 9.375 \text{ k}$$

$$> 7.7 \text{ k}$$

⇒ Increase Bracing

$$23.05 \text{ k}$$

$$h \approx 12.67' / 25 = 0.50 \quad \theta = 26.87^\circ$$

$$\cos \theta = 0.892$$

$$23.05 \text{ k} \div 0.892 = 25.84 \text{ k}$$

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE-RESISTING SYSTEMS

Seismic Force-Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit ^c				
					Seismic Design Category				
					B	C	D ^d	E ^d	F ^e
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	2½	4	NL	NL	40 ^k	40 ^k	40 ^k
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	6½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	2½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.1.4.2, and 14.5	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	14.1	8	2	4	NL	NL	160	160	100
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	14.1	7	2	4	NL	NL	160	160	100
3. Special steel concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
4. Ordinary steel concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35 ^j	35 ^j	NP ^j
5. Special reinforced concrete shear walls	14.2 and 14.2.3.6	6	2½	5	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	5	2½	4½	NL	NL	NP	NP	NP
7. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP	NP	NP
8. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP	NP	NP
9. Intermediate precast shear walls	14.2 and 14.2.3.5	5	2½	4½	NL	NL	40 ^k	40 ^k	40 ^k
10. Ordinary precast shear walls	14.2 and 14.2.3.3	4	2½	4	NL	NP	NP	NP	NP
11. Composite steel and concrete eccentrically braced frames	14.3	8	2	4	NL	NL	160	160	100
12. Composite steel and concrete concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
13. Ordinary composite steel and concrete braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
14. Composite steel plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
15. Special composite reinforced concrete shear walls with steel elements	14.3	6	2½	5	NL	NL	160	160	100
16. Ordinary composite reinforced concrete shear walls with steel elements	14.3	5	2½	4½	NL	NL	NP	NP	NP
17. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
18. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
19. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
20. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
21. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE-RESISTING SYSTEMS (continued)

Seismic Force-Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit ^c				
					Seismic Design Category				
					B	C	D ^d	E ^d	F ^e
22. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	7	2½	4½	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	14.1	7	2	5½	NL	NL	160	160	100
26. Buckling-restrained braced frames, moment-resisting beam-column connections	14.1	8	2½	5	NL	NL	160	160	100
27. Special steel plate shear wall	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Special steel moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35 ^{h,i}	NP ^h	NP ⁱ
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP ^h	NP ^h	NP ⁱ
5. Special reinforced concrete moment frames	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Ordinary composite moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES									
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Special steel concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Composite steel and concrete eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Composite steel and concrete concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
7. Composite steel plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Special composite reinforced concrete shear walls with steel elements	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Ordinary composite reinforced concrete shear walls with steel elements	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Buckling-restrained braced frame	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Special steel plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL

Wall Cross Bracing: Existing is 1" ϕ Rod.

$$\text{Max } P_{Eh} = 60,000 \# \div 2 \text{ walls} = 30,000 \#$$

$$30,000 \# \div 175' = 171.42 \text{ pcf} \times 12' = 2057 \#$$

$\Rightarrow$ 15/32 Plywood on stud walls w/
8d nails @ 6" o.c.

$$\text{Allow} = 520 \text{ pcf} \div 2.0 = 260 \text{ pcf Allow.}$$

$\Rightarrow$ 8d nailing @ 6" o.c.

ok to assist w/ Lateral loading.

Uplift:

$$30,000 \# \div 7 \text{ Bays} = 4286 \#/\text{Bay} \div 25' = 171 \text{ pcf}$$

(not accounting for doors - but ok)

$$171 \text{ pcf} \times 12' = 2057 \#$$

Hilti KWIK Bolt TZ Expansion Anchor

$$\text{Allow} = 2767 \quad \alpha_s \phi N_n > 2057 \# \text{ -ok-}$$

(See tall stud design)

→ Try HS20 Truck Loading

Axle Load = 32K

Wheel Spacing = 6 ft = 72 in

see previous calc pages { $MR = 569 \text{ psi}$
 $WS = 345 \text{ psi}$ (SF=1.5 ; JF=1.1)

Slab Thickness

Slab Stress = $\frac{345}{32} = 10.78 \text{ psi}$

Interior Loading → 9"
Edge Loading → 12"

w/ SF=1.4

WS = 369 psi

Slab Stress = $\frac{369}{32} = 11.5 \text{ psi}$

Interior Loading → 9"
Edge Loading → 11"

→ Try 29K Load

(SF=1.5) Slab Stress = $\frac{345}{29} = 11.9 \text{ psi}$

Interior Loading → 9"
Edge Loading → 11"

(SF=1.4) Slab Stress = $\frac{369}{29} = 12.7 \text{ psi}$

Interior Loading → 9"
Edge Loading → 10"

* Use 9" min thick slab w/ 12" thickened slab edge

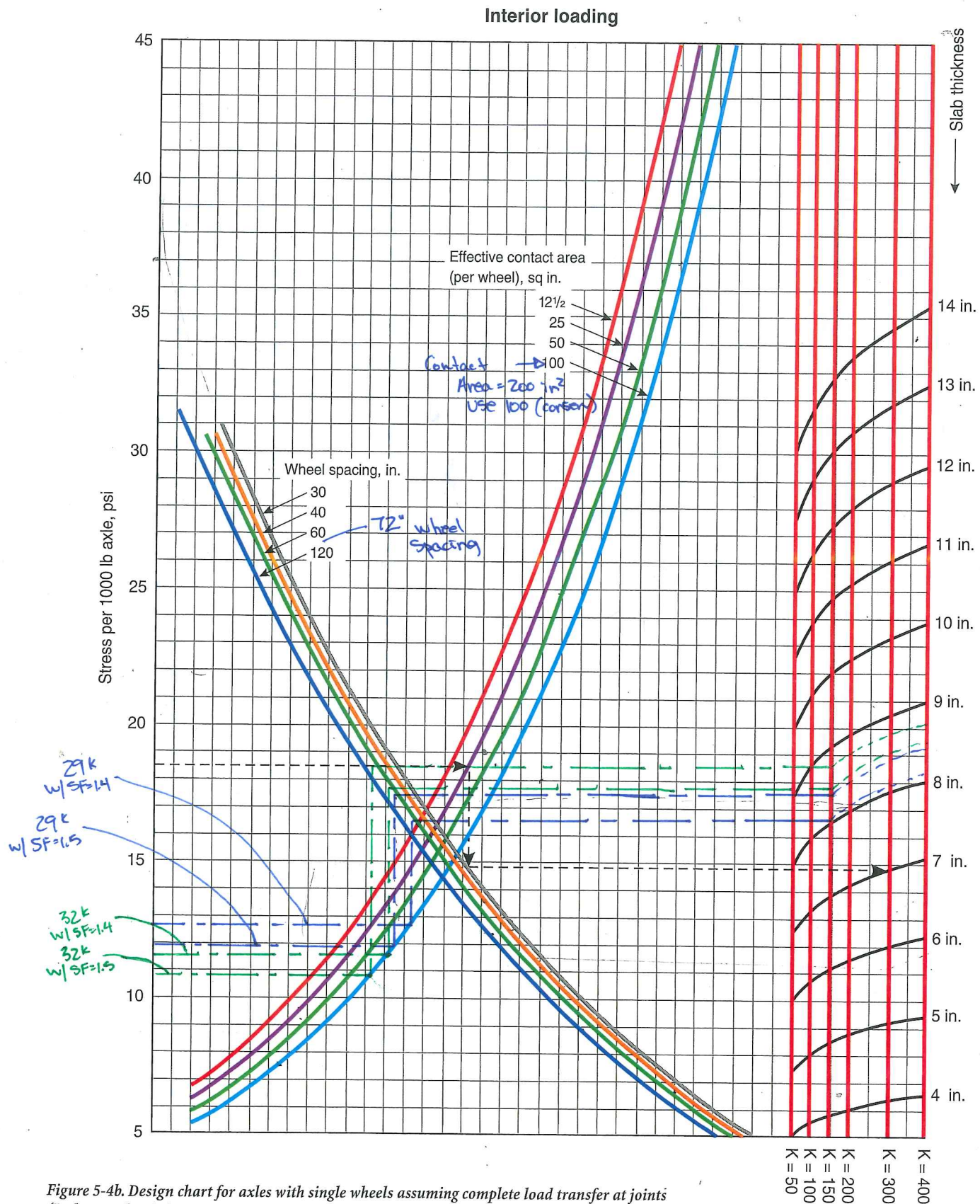


Figure 5-4b. Design chart for axles with single wheels assuming complete load transfer at joints (Inch-pound units). K—psi/in.

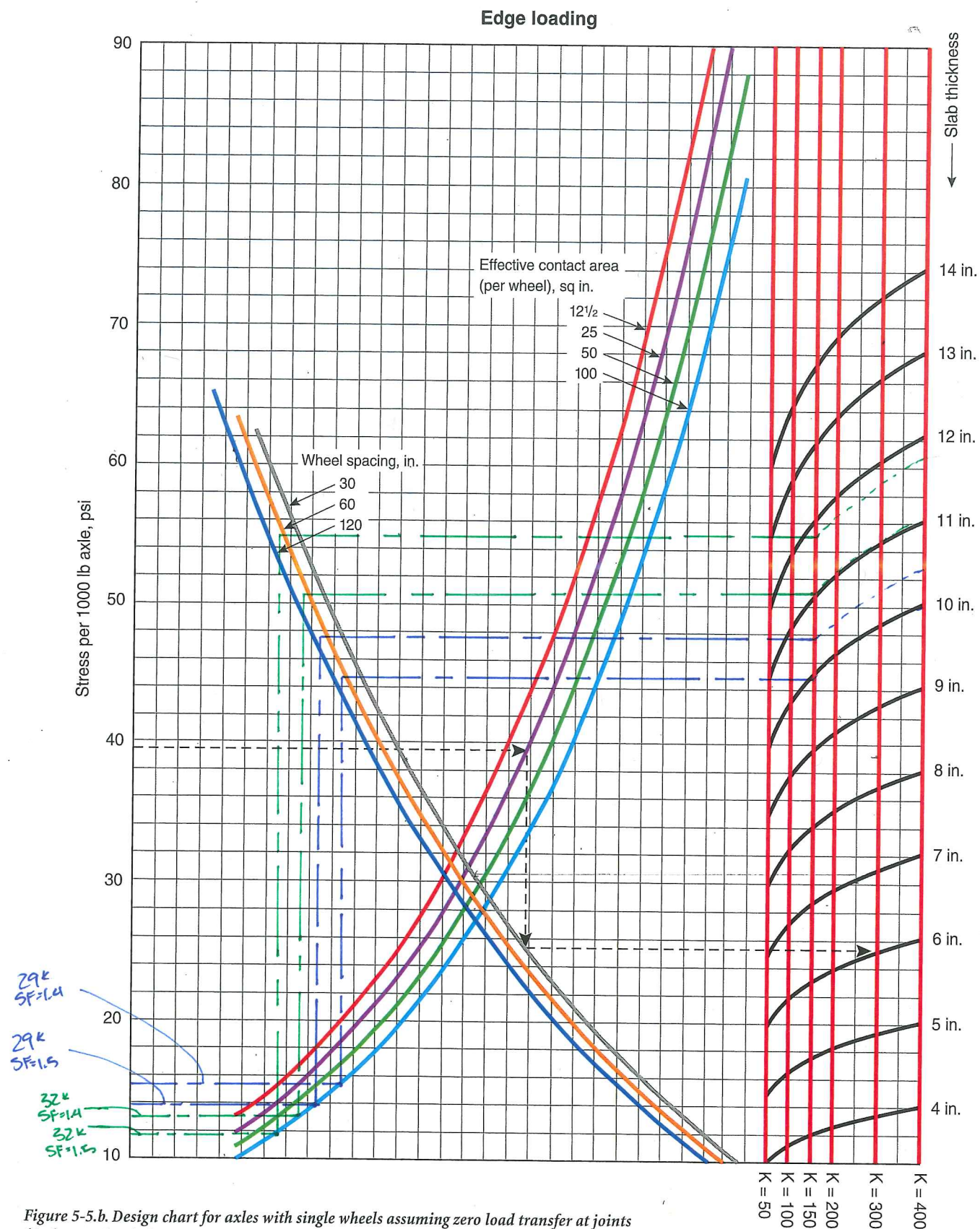
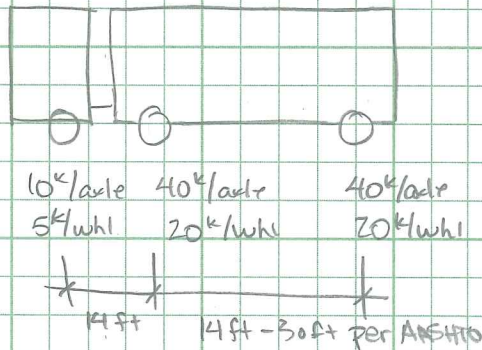


Figure 5-5.b. Design chart for axles with single wheels assuming zero load transfer at joints (Inch-pound units). K-psi/in.

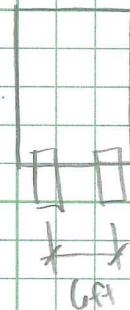
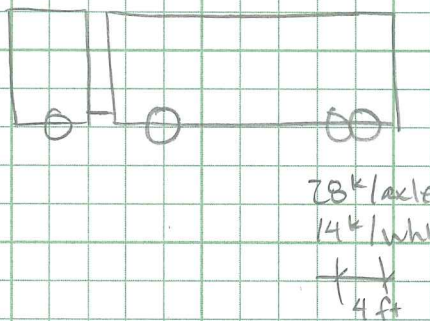
Slab Design

→ Truck Loading

H25



Tandem



→ Tire Contact Area (AASHTO Section 3.6.1.2.5)

width = 20 in
length = 10 in

⇒ $A = 200 \text{ in}^2$

→ Wheel Spacing

40k load → 14 ft = 168 in

→ 6 ft = 72 in

28k tandem load → 4 ft = 48 in



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sheet no. _____ of _____

By WAA

Date 8-8-17

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Chapter 5 - Floor Thickness Design

- $JF = 1.1$ or 1.2 for low shrinkage concrete mixtures

→ Use 1.1

- Stress Ratio for load repetitions

→ Building will be used temporarily as
Fire Station ~ 3 to 5 years

→ # of times a day fire trucks will move
in & out of fire station ~ 1 to 4 times/day

	3 yrs = 1095 days	SR	5 yrs = 1825 days	SR
1x	1095	0.72	1825	0.70
2x	2190	0.69	3650	0.68
4x	4380	0.67	7300	0.65

SR = 0.65 to 0.72

1.54 to 1.39 → Use 1.5 safety factor
SR = 0.67

- Subgrade modulus $k = 150 \text{ psi/in}$
- Concrete compressive strength $f'_c = 4000 \text{ psi}$
- Concrete flexural strength $MR = 9 \times \sqrt{f'_c}$
 $= 9 \times \sqrt{4000} = 569 \text{ psi}$ ✓

- Concrete Working Stress $WS = \left(\frac{MR}{SF \times TF} \right)$

$$WS = \left(\frac{569 \text{ psi}}{1.5 \times 1.1} \right) = 345 \text{ psi} \checkmark$$

- Slab Stress per 1,000 # of axle load $= \left(\frac{WS}{\text{axle load}} \right)$

$$\left(\frac{345}{40} \right) = 8.625 \text{ psi} \checkmark$$

$$\left(\frac{345}{28} \right) = 12.32 \text{ psi} \checkmark$$

Interior Loading

CASE 1: 40 k load spaced @ 14 ft / 6 ft

→ 10 in slab **≡ Governs**

CASE 2: 28 k load spaced @ 4 ft

→ 9 in slab

Joint Spacing per Table 6-1

for 10" slab → 20 ft joint spacing
9" → 18 ft
8" → 16 ft

* Use max 20 ft joint spacing

• Reinforcement $A_s = \frac{L C_f W h}{K f_s}$

L = distance b/w joints

C_f = coefficient of subgrade resistance to movement
(1.5 is commonly used)

W = concrete density

h = slab thickness

K = 24 for English

f_s = allowable tensile stress of steel

$$A_s = \frac{(20 \text{ ft})(1.5)(150 \text{ pcf})(10 \text{ in})}{24 (40,000 \text{ psi})} = 0.046875 \text{ in}^2/\text{ft}$$

→ Min temperature / shrinkage steel ratio per ACI

$$\rho = 0.0018 = \frac{A_s}{A_c}$$

Governs → $A_s = 0.0018 A_c \Rightarrow 0.0018 (10 \text{ in})(12 \text{ in}) = 0.216 \text{ in}^2/\text{ft}$

#4 bars @ 10" oc $A_s = 0.2356 \text{ in}^2 \rightarrow 0.2827 \text{ in}^2/\text{ft}$

#5 bars @ 12" oc $A_s = 0.30679 \text{ in}^2$

#5 bars @ 16" oc $A_s = 0.2300 \text{ in}^2 \rightarrow 0.1726 \text{ in}^2/\text{ft}$ NG

Use 10" thick slab w/ #4 Bars @ 10" oc
w/ max joint spacing @ 20 ft

9" Slab: $0.0018 \times 9" \times 12" = 0.1944 \text{ in}^2$

#4 @ 12" = 0.1963 in^2

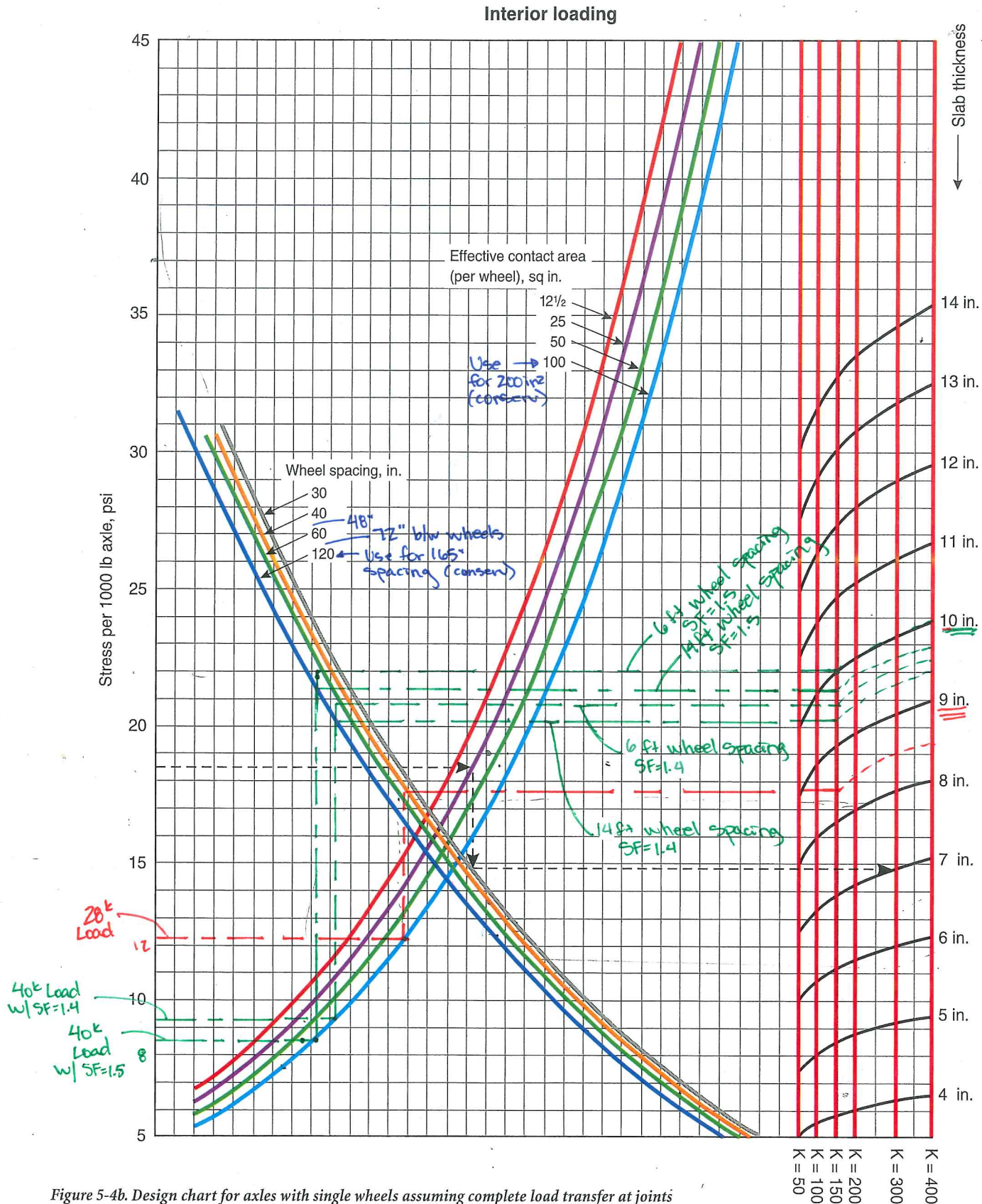


Figure 5-4b. Design chart for axles with single wheels assuming complete load transfer at joints (Inch-pound units). K—psi/in.

Say SF = 1.4 (ie 3yrs 1x day usage)

$$WS = \frac{569}{1.4 \times 1.1} = 369 \text{ psi}$$

$$\text{Slab Stress} = \frac{369}{40} = 9.23$$

⇒ Still use 10" slab

Say JF = 1.2

$$WS = \frac{569}{1.4 \times 1.2} = 338 \text{ psi}$$

$$\text{Slab Stress} = \frac{338}{40} = 8.45 \approx 8.625$$

⇒ Still use 10" slab



Carla Hansen

From: Matt Redwine <mredwine@tetonwyo.org>
Sent: Friday, August 11, 2017 9:34 AM
To: Kevin Meagher; Carla Hansen
Subject: RE: UPDATE

Good Morning,
The following GVWR are for our three heaviest trucks.

Engine 11 – 31 feet in Length
Front 14,600
Rear 26,000
Gross 40,600

Truck 16 – 46 feet in length
Front 23,000
Rear 1 – 29,000
Rear 2 - 29,000
Gross – 81,000

Tender 17 32 feet in length
Front – 7,257
Rear - 18,144
Gross – 25,401

The other vehicles are much lighter,, the heaviest single axle weight appears to be the dual axles on Truck 16 @ 29,000 pounds
Because of the lengths we cannot park all three of these in a single row.
Hope this helps.

Matt

From: Kevin Meagher [mailto:kmeagher@jacksonwy.gov]
Sent: Friday, August 11, 2017 8:36 AM
To: Carla Hansen <chansen@nelsonengineering.net>
Cc: Matt Redwine <mredwine@tetonwyo.org>
Subject: RE: UPDATE

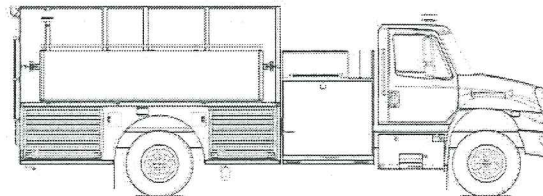
Carla,
I sent your email to our budget estimator to see what type of wall is the most cost effective.
For the concrete equipment slab, less is better, so if we can get by with thinner slab that would be preferable.
I'll ask Matt if the heavy trucks can be parked in one area so we might be able to thicken a slab area but not all of it?
Thanks,
Kevin

From: Carla Hansen [mailto:chansen@nelsonengineering.net]
Sent: Wednesday, August 09, 2017 3:26 PM
To: Kevin Meagher <kmeagher@jacksonwy.gov>
Cc: Kaitlyn Mangis <kmangis@tetonwyo.org>
Subject: UPDATE

Appendix A

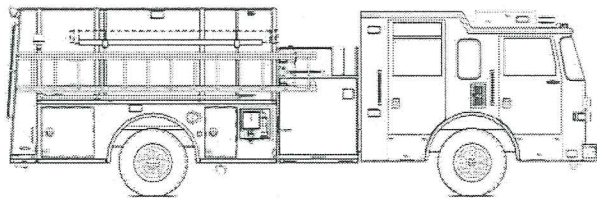
Typical Fire Apparatus Configurations

Commercial Chassis Pumper – Single Rear Axle



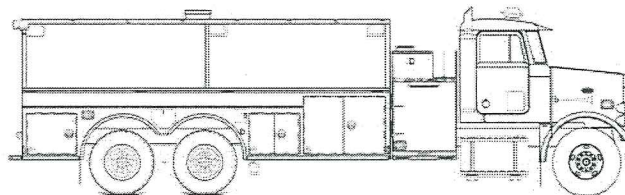
	<u>Min</u>	<u>Max</u>
Front GAWR	12,000	18,000
Rear GAWR	21,000	31,000
Width (in.)	98	100
Height (ft.)	9	12
Length (ft.)	24	35

Custom Chassis Pumper – Single Rear Axle



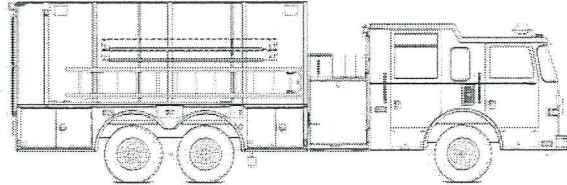
	<u>Min</u>	<u>Max</u>
Front GAWR	18,000	24,000
Rear GAWR	24,000	31,000 - 32 ^k /axle
Width (in.)	98	100
Height (ft.)	9	12
Length (ft.)	30	34

Commercial Chassis Tanker – Tandem Rear Axle



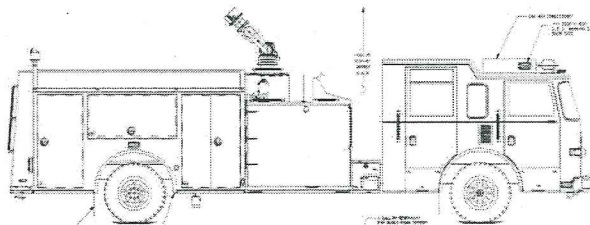
	<u>Min</u>	<u>Max</u>
Front GAWR	12,000	18,000
Rear GAWR	34,000	56,000
Width (in.)	98	100
Height (ft.)	10	12
Length (ft.)	30	40

Custom Chassis Tanker – Tandem Rear Axle



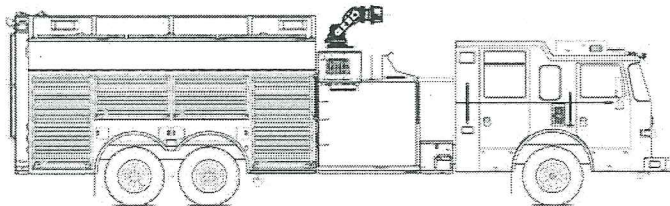
	<u>Min</u>	<u>Max</u>
Front GAWR	18,740	22,800
Rear GAWR	40,000	56,000
Width (in.)	98	100
Height (ft.)	10	12
Length (ft.)	34	40

Industrial Foam Pumper – Single Rear Axle



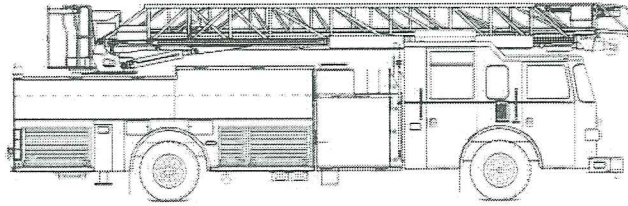
	<u>Min</u>	<u>Max</u>
Front GAWR	20,000	24,000
Rear GAWR	24,000	31,000 - 32 1/2 axle
Width (in.)	98	100
Height (ft.)	10	12
Length (ft.)	30	36

Industrial Foam Pumper – Tandem Rear Axle



	<u>Min</u>	<u>Max</u>
Front GAWR	20,000	24,000
Rear GAWR	40,000	56,000
Width (in.)	98	100
Height (ft.)	10	12
Length (ft.)	36	40

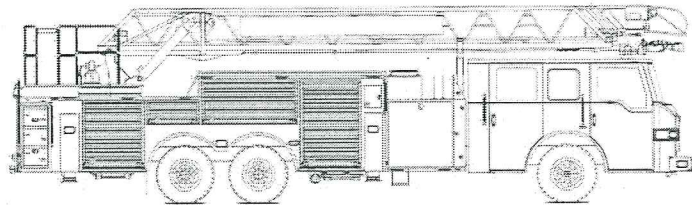
Aerial Ladder – Single Rear Axle



	<u>Min</u>	<u>Max</u>
Front GAWR	20,000	22,800
Rear GAWR	24,000	33,500
Width (in.)	98	100
Height (ft.)	11	12.5
Length (ft.)	36	43

— 32^k HS 20
40 HS 25

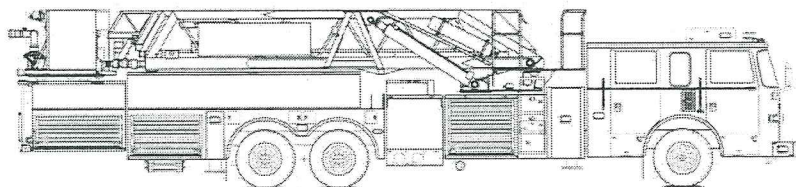
Aerial Ladder – Tandem Rear Axle



	<u>Min</u>	<u>Max</u>
Front GAWR	20,000	22,800
Rear GAWR	34,000	<u>54,000</u>
Width (in.)	98	100
Height (ft.)	11	12.5
Length (ft.)	39	43

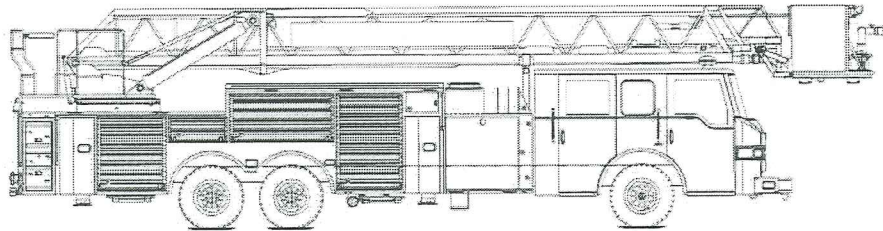
1. — 56^k

Aerial Platform Mid Mount – Tandem Rear Axle



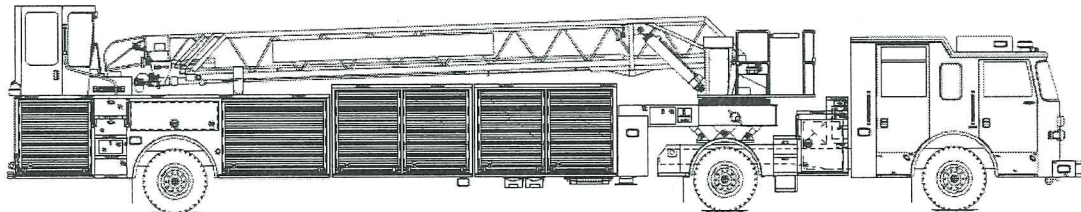
	<u>Min</u>	<u>Max</u>
Front GAWR	21,500	24,000
Rear GAWR	40,000	62,000
Width (in.)	98	100
Height (ft.)	9.5	11.5
Length (ft.)	46	51

Aerial Platform Rear Mount – Tandem Rear Axle



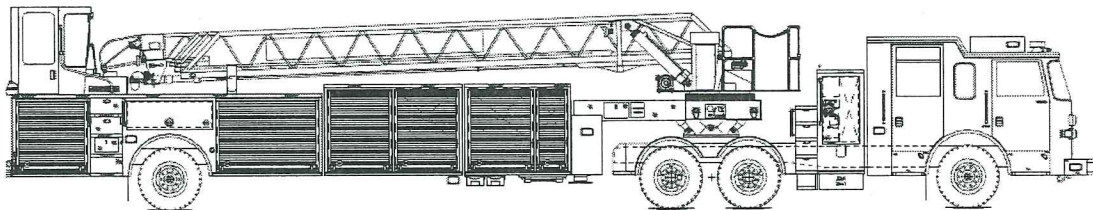
	Min	Max
Front GAWR	21,500	24,000
Rear GAWR	46,000	62,000
Width (in.)	98	100
Height (ft.)	11.5	13
Length (ft.)	46	48

Aerial Ladder – Tiller Single Rear TractorAxle



	Min	Max
Front GAWR	21,500	24,000
Rear GAWR	27,000	31,000
Tiller GAWR	21,500	24,000
Width (in.)	98	100
Height (ft.)	10.5	11.5
Length (ft.)	56	63

Aerial Ladder – Tiller Tractor Tandem Rear Axle



	Min	Max
Front GAWR	21,500	24,000
Rear GAWR	34,000	44,000
Tiller GAWR	21,500	24,000
Width (in.)	98	100
Height (ft.)	10.5	11.5
Length (ft.)	56	63

JOINT LAYOUT

When developing the joint layout in a facility, the designer should consider minimizing the number of joints exposed to hard-wheeled traffic. While floor loading (rack positions, and placement of products) should be taken into consideration, joints are most often located on column lines and at an appropriate intermediate spacing to minimize random cracking. A sample joint layout is illustrated in Figure 6-10. All three types of joints— isolation, contraction, and construction—will likely be used in a slab installation. Isolation joints are provided around the perimeter of the floor where it abuts the walls and around all fixed elements that restrain movement of the slab in a horizontal plane. This includes columns and machinery bases that pass through the floor slab. With the slab isolated from other building elements, the remaining task is to locate and correctly space contraction joints to minimize random cracking. Joints should create panels as nearly square as possible, with no length to width ratio greater than $1\frac{1}{2}$ to 1. In addition contraction joints should

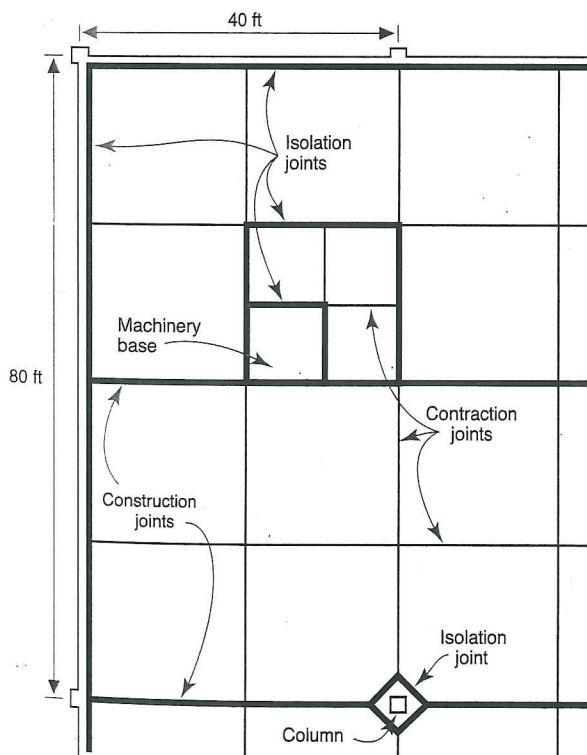


Figure 6-10. This sample joint layout indicates typical locations for various joint types. Note, due to the isolated machinery base the full adjoining slab panel should be bordered with an isolation joint and load transfer devices (plates or dowels) should be provided, or the adjoining panel should be bordered with contraction joints, the interior contraction joints should be eliminated and steel reinforcement provided at the re-entrant corner to hold the inevitable random crack tightly closed.

always be terminated at slab edges or isolated joints (avoid T intersections in contraction joint layouts) to minimize the potential for random cracking to continue into adjoining slab panels. Proper planning of construction joints allows them to coincide with contraction joint placement. This allows breaking the job into placements that can be built in a single day and yet minimizes the number of joints in the floor.

JOINT SPACING

Closely spaced joints help to reduce the number of random cracks between joints. They also typically decrease the amount of warping uplift that occurs at the joint. However, the joint spacing should not be too tight or there is a risk that only every other, or perhaps even every third joint, will activate (that is, crack below the sawcut). If this occurs or if joints activate at different ages, undesired cracking may occur or some joints may open wider than others causing localized load transfer efficiency (LTE) issues.

For unreinforced concrete slabs, the maximum recommended spacing between joints depends primarily on:

- slab thickness,
- cooling contraction and drying shrinkage potential, and
- curing environment.

Maximum contraction joint spacings recommended in Table 6-1 will typically reduce the risk for random cracking when concrete mixtures are optimized for minimal shrinkage. Experience with local materials and job practices may indicate that different spacings than those shown can be used

Table 6-1. Maximum Spacing of Contraction Joints in Meters (Feet)*

Slab thickness mm (in.)	Maximum-size aggregate less than 19 mm (¾ in.)	Maximum-size aggregate 19 mm (¾ in.) and larger
125 (5)	3.0 (10)	3.75 (13)
150 (6)	3.75 (12)	4.5 (15)
175 (7)	4.25 (14)	5.25 (18)**
200 (8)	5.0 (16)**	6.0 (20)**
225 (9)	5.5 (18)**	6.75 (23)**
250 (10)	6.0 (20)**	7.5 (25)**

* If concrete cools at an early age, shorter spacings may be needed to control random cracking. A temperature difference of only 6 °C (10°F) may be critical. For slump less than 100 mm (4 in.), joint spacing can be increased by 20%.

**When spacings exceed 4.5 m (15 ft), load transfer by aggregate interlock decreases markedly. If shrinkage is high or unknown, joints should not exceed 4.5 m (15 ft).

Maximum Joint Spacing:
 ~ 24 to 36 x Slab t : 4" : ~ 96" - 144" ~ 8' to 12'
 6" : ~ 144" - 216" ~ 12' to 18'



Figure 5-16. Distributed loads on a floor surface.

Two criteria control floor design where distributed loads are concerned:

1. preventing cracks in the aiseways due to excessive negative moment (tension in top of slab)
2. avoiding objectionable settlement of the slab

Distributed loads can result in cracking in an unjointed aisle, although distributed loads placed on floors are seldom great enough to cause excessive settlement of properly prepared and compacted subgrades. Although making the slab thicker can control cracking, it does not prevent slab settlement. For very heavy distributed loads on compressible subgrades, the possibility of soil consolidation should be examined using soil foundation engineering techniques.

Heavy distributed loads on portions of the slab may cause differential settlement and slab deformation. The slab bending stresses due to slab deformations may be additive to the negative bending stresses in aisles. Design of slabs subjected to differential slab settlement is beyond the scope of this publication. A geotechnical engineer should be consulted in these cases.

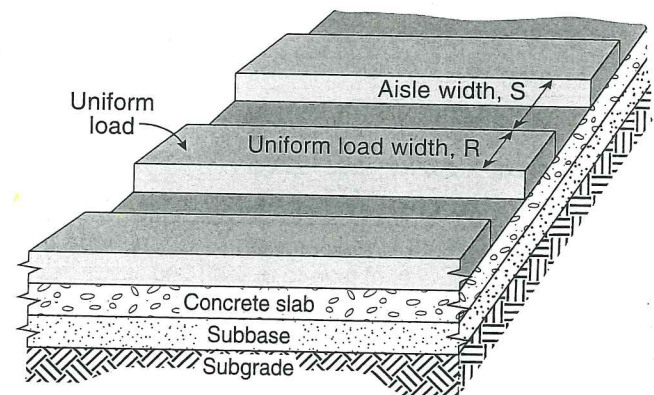


Figure 5-17. Uniform or "strip" loading on a concrete floor.

Allowable Loads to Prevent Cracking in an Unjointed Aisleway

In an unjointed aisleway between distributed load areas, the maximum negative bending moment in the slab may be up to twice as great as the bending moment in the slab beneath the loaded area. As a result, one design objective is to limit maximum negative moment stresses in the aisleway so that cracking will not occur.

This publication presents two methods to determine the magnitude of allowable distributed loads based on this design objective: one uses tables and the other involves figures. The use of the tables is discussed first.

Table 5-3 is used if the aisle and storage layout is variable and may be changed during the service life of the floor. If

Table 5-3a. Allowable Distributed Loads, Unjointed Aisle, Metric (Nonuniform Loading, Variable Layout)

Slab thickness, mm	Subgrade k ,* MPa/m	Allowable load, kPa**			
		Concrete flexural strength, MPa			
		3.8	4.1	4.5	4.8
125	13.5	25.6	28.0	30.4	32.8
	27	36.4	36.4	43.1	46.2
	54	51.5	56.3	60.8	65.6
150	13.5	28.0	30.6	33.3	35.9
	27	39.7	43.3	46.9	50.5
	54	56.3	61.3	66.6	71.6
200	13.5	32.6	35.4	38.3	41.4
	27	46.0	50.0	54.3	58.4
	54	64.9	70.9	76.8	82.6
250	13.5	36.4	39.7	42.9	46.2
	27	51.2	56.0	60.6	65.4
	54	72.5	79.2	85.7	92.4
300	13.5	39.7	43.3	46.9	50.5
	27	56.3	61.3	66.6	71.6
	54	79.5	86.7	94.1	101.5
350	13.5	42.9	46.9	50.8	54.6
	27	60.8	66.3	71.8	77.3
	54	85.9	93.8	101.5	109.4

* k of subgrade; disregard increase in k due to subbase.

** For allowable stress equal to $\frac{1}{2}$ flexural strength. Based on aisle and load widths giving maximum stress.

Table 5-3b. Allowable Distributed Loads, Unjointed Aisle, Inch-Pounds (Nonuniform Loading, Variable Layout)

Slab thickness, in.	Subgrade k ,* psi/in.	Allowable load, psf**			
		Concrete flexural strength, psi			
		550	600	650	700
5	50	535	585	635	685
	100	760	760	900	965
	200	1075	1175	1270	1370
6	50	585	640	695	750
	100	830	905	980	1055
	200	1175	1280	1390	1495
8	50	680	740	800	865
	100	960	1045	1135	1220
	200	1355	1480	1603	1725
10	50	760	830	895	965
	100	1070	1170	1265	1365
	200	1515	1655	1790	1930
12	50	830	905	980	1055
	100	1175	1280	1390	1495
	200	1660	1810	1965	2115
14	50	895	980	1060	1140
	100	1270	1385	1500	1615
	200	1795	1960	2120	2285

* k of subgrade; disregard increase in k due to subbase.

** For allowable stress equal to $\frac{1}{2}$ flexural strength. Based on aisle and load widths giving maximum stress.

$$W = 18.53 \text{ psf}$$

$$\text{factor} = \frac{28.2}{18.53} = 1.52$$

OLD LOADING

D+W (original loading)

	x	y
Reactions	-8.52	-15.62
Haunch M	-88.34	
Ridge M	-2.95	

$$* 1.52 = 134.4 < 143.025$$

Conservative; Continue analysis w/ adjusted moments below

D+W (to match original)

	x	y
Reactions	-10.993	9.579
Haunch M	-76.098	
Ridge M	12.066	

adjustment factor for models acting differently
% difference of originally loading models
1.160872 $\Rightarrow 88.34 / 76.098$
0.244489 $\Rightarrow 2.95 / 12.066$

$$W \approx 28.2 \text{ psf}$$

NEW LOADING

Adjusted Moments

D+W (uniform loading)

	x	y
Reactions	-17.287	-16.357
Haunch M	-123.205	
Ridge M	18.92	

$$\frac{143.025}{88.34} = 1.62 > 1.52$$

$\therefore$ conserv.

includes above factor
-143.025 $\Rightarrow 123.205 * 1.16$
4.626 $\Rightarrow 18.92 * 0.244$

D+W (point @ haunch)

	x	y
Reactions	-17.107	-16.948
Haunch M	-144.598	-167.860
Ridge M	19.42	4.748

D+W (point @ haunch)

	x	y
Reactions	-17.187	-16.652
Haunch M	-133.78	-155.301
Ridge M	19.088	4.667

Wind port
Load @
Haunch &
mid-span

Max moment Haunch was designed for $457.58 \text{ k-ft} > 167.9 \text{ k-ft}$
(D+C+S) OK

Max moment Ridge was designed for 7.64 k-ft
(D+C+S)

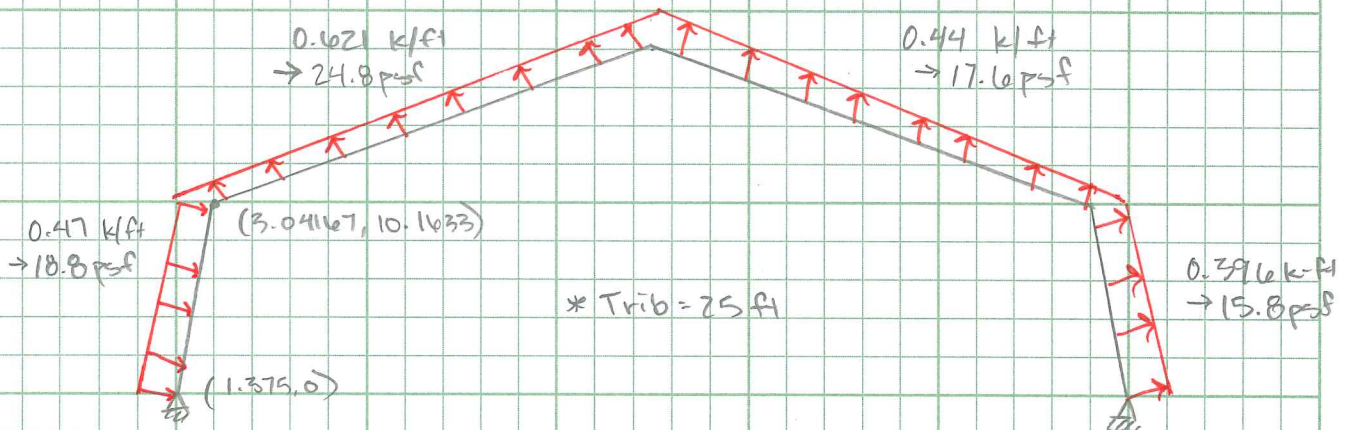
$\hookrightarrow$ (4) $\frac{5}{8}$ " Bolted Ridge Connection $M_{au} = 124.5 \text{ k-ft} > 19.42 \text{ k-ft}$
OK

Moment @ Mid-span of Column $\sim M = 64.919 * 1.16 = 75.3 \text{ k-ft}$

$M_{au} = 257.04 \text{ k-ft}$ OK

Wind Loading

* Uniformly Distributed



① * Point Load @ Haunch

conserv. since half of wind Load will be distributed to base connection

$$18.8 \text{ psf} * 25 \text{ ft} * 10.1633 = 4777 \#$$

$$15.8 \text{ psf} * 25 \text{ ft} * 6.1633 = 4015 \#$$

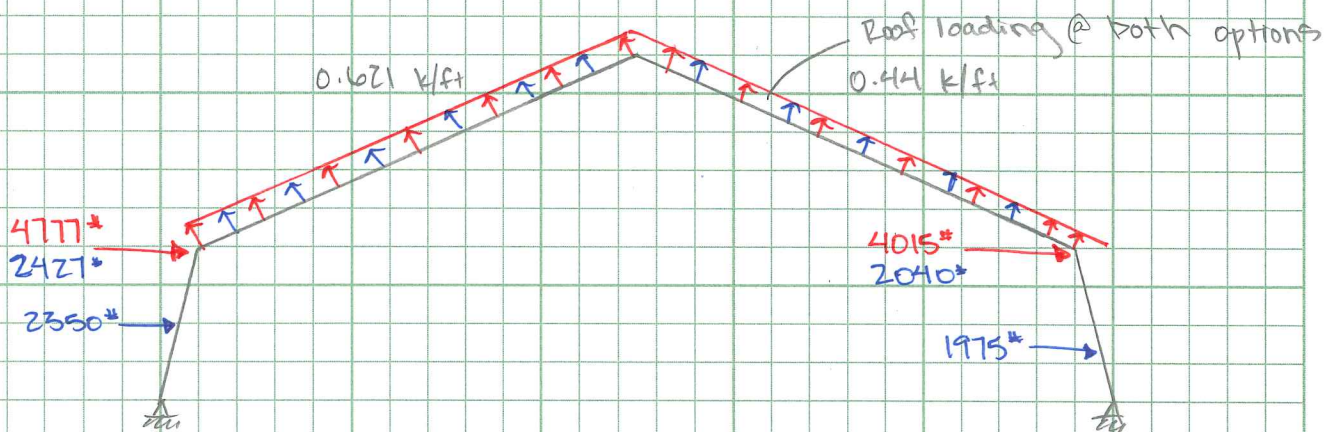
② * Point Load @ Mid + Haunch

$$\text{@ } y=5 \quad 18.8 \text{ psf} * 25 \text{ ft} * 5 \text{ ft} = 2350 \#$$

$$15.8 \text{ psf} * 25 \text{ ft} * 5 \text{ ft} = 1975 \#$$

$$\text{@ Haunch} \quad 4777 - 2350 = 2427 \#$$

$$4015 - 1975 = 2040 \#$$



Original Loading - to compare to original calcs

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	1.375	0	0
2	N2	3.04167	10.1633	0
3	N3	15.6708	13.715	0
4	N4	38.1667	20.1467	0
5	N5	60.6625	13.715	0
6	N6	73.2917	10.1633	0
7	N7	74.9583	0	0

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]	Footing
1	N1	Reaction	Reaction		
2	N7	Reaction	Reaction		
3	N2				
4	N3				
5	N4				
6	N5				
7	N6				

Member Primary Data

	Label	I Joint	J Joint	Rotate(...)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2		W21x55	Beam	Wide Flange	A36 Gr.36	Typical
2	M2	N2	N3		W21x93	Beam	Wide Flange	A36 Gr.36	Typical
3	M3	N3	N4		W21x68	Beam	Wide Flange	A36 Gr.36	Typical
4	M4	N4	N5		W21x68	Beam	Wide Flange	A36 Gr.36	Typical
5	M5	N5	N6		W21x93	Beam	Wide Flange	A36 Gr.36	Typical
6	M6	N6	N7		W21x55	Beam	Wide Flange	A36 Gr.36	Typical

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	DL	DL					4
2	SL	SL					4
3	WL,R	WL					6
4	WL,L	WL					6
5	QE,R	EL			2		
6	QE,L	EL			2		
7	C	DL			5		4
8	unbalanced snow	None					2

Load Combinations

	Description	Solve	PDelta	SRSS	BLC	Fa... B...	Fa... B...	Fa... B...	Fa... B...	Fa... B...	Fa... B...	Fa... B...	Fa... B...	Fa...
1	DL+SL		Y		1	1	2	1						
2	DL + 0.6WL,R		Y		1	1	3	.6						
3	DL + 0.6WL,L		Y		1	1	4	.6						
4	DL + 0.75(0.6WL,R) + 0.75SL		Y		1	1	3	.45	2	.75				
5	DL + 0.75(0.6WL,L) + 0.75SL		Y		1	1	4	.45	2	.75				
6	0.6DL + 0.6WL,R		Y		1	.6	3	.6						
7	0.6DL + 0.6WL,L		Y		1	.6	4	.6						
8			Y											
9	(1.0+0.14SDS)DL + 0.7*RHO*...		Y		1	1.1...	5	.7						
10	(1.0+0.14SDS)DL + 0.7*RHO*...		Y		1	1.1...	6	.7						

Member Distributed Loads (BLC 8 : unbalanced snow)

	Member Label	Direction	Start Magnitude	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M4	Y	-2.345	-2.345	0	0
2	M5	Y	-2.345	-2.345	0	0

Joint Reactions

	LC	Joint Label	X [k]	Y [k]	MZ [k-ft]
1	17	N1	-10.993	-9.579	0
2	17	N7	4.789	-7.111	0
3	17	Totals:	-6.204	-16.691	
4	17	COG (ft):	X: 34.605	Y: 15.139	

Member Section Forces

	LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
1	17	M1	1	-11.232	9.273	0
2			2	-11.163	8.854	-10.372
3			3	-11.095	8.436	-20.264
4			4	-11.026	8.017	-29.678
5			5	-10.958	7.598	-38.613
6			6	-10.889	7.18	-47.068
7			7	-10.82	6.761	-55.045
8			8	-10.751	6.342	-62.542
9			9	-10.683	5.923	-69.559
10			10	-10.614	5.504	-76.098
11	17	M2	1	-9.5	-7.263	-76.098
12			2	-9.391	-6.877	-65.792
13			3	-9.283	-6.491	-56.049
14			4	-9.174	-6.106	-46.868
15			5	-9.066	-5.72	-38.249
16			6	-8.957	-5.335	-30.192
17			7	-8.849	-4.949	-22.697
18			8	-8.741	-4.564	-15.763
19			9	-8.632	-4.179	-9.392
20			10	-8.524	-3.793	-3.581
21	17	M3	1	-8.54	-3.762	-3.581
22			2	-8.344	-3.075	5.305
23			3	-8.147	-2.387	12.404
24			4	-7.951	-1.7	17.717
25			5	-7.754	-1.012	21.243
26			6	-7.558	-.325	22.981
27			7	-7.361	.362	22.933
28			8	-7.165	1.05	21.097
29			9	-6.968	1.737	17.475
30			10	-6.772	2.424	12.066
31	17	M4	1	-7.024	-1.541	12.066
32			2	-7.154	-1.085	15.479
33			3	-7.284	-.63	17.709
34			4	-7.415	-.174	18.754
35			5	-7.545	.281	18.614
36			6	-7.675	.737	17.29
37			7	-7.806	1.193	14.782
38			8	-7.936	1.648	11.089
39			9	-8.066	2.104	6.212
40			10	-8.196	2.559	.151
41	17	M5	1	-8.185	2.589	.151
42			2	-8.257	2.844	-3.809

Updated Wind Loading

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	1.375	0	0
2	N2	3.04167	10.1633	0
3	N3	15.6708	13.715	0
4	N4	38.1667	20.1467	0
5	N5	60.6625	13.715	0
6	N6	73.2917	10.1633	0
7	N7	74.9583	0	0

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]	Footing
1	N1	Reaction	Reaction		
2	N7	Reaction	Reaction		
3	N2				
4	N3				
5	N4				
6	N5				
7	N6				

Member Primary Data

	Label	I Joint	J Joint	Rotate(...)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2		W21x55	Beam	Wide Flange	A572 Gr.50	Typical
2	M2	N2	N3		W21x93	Beam	Wide Flange	A572 Gr.50	Typical
3	M3	N3	N4		W21x68	Beam	Wide Flange	A572 Gr.50	Typical
4	M4	N4	N5		W21x68	Beam	Wide Flange	A572 Gr.50	Typical
5	M5	N5	N6		W21x93	Beam	Wide Flange	A572 Gr.50	Typical
6	M6	N6	N7		W21x55	Beam	Wide Flange	A572 Gr.50	Typical

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	DL	DL					4
2	SL	SL					4
3	WL,R	WL					6
4	WL,L	WL					6
5	QE,R	EL			2		
6	QE,L	EL			2		
7	C	DL					4
8	SL2 (w/ l=1.1)	SL					4
9	W,R (point @ hauch)	WL			2		4
10	W,R (point @ mid & haunch)	WL			2	2	4

Load Combinations

	Description	Solve	PDelta	SRSS	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...
1	DL+SL		Y		1	1	2	1									
2	DL + 0.6WL,R		Y		1	1	3	.6									
3	DL + 0.6WL,L		Y		1	1	4	.6									
4	DL + 0.75(0.6WL,R) + 0.75SL		Y		1	1	3	.45	2	.75							
5	DL + 0.75(0.6WL,L) + 0.75SL		Y		1	1	4	.45	2	.75							
6	0.6DL + 0.6WL,R		Y		1	.6	3	.6									
7	0.6DL + 0.6WL,L		Y		1	.6	4	.6									
8			Y														

Member Distributed Loads (BLC 2 : SL)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M2	Y	-1.875	-1.875	0	0
2	M5	Y	-1.875	-1.875	0	0
3	M3	Y	-1.875	-1.875	0	0
4	M4	Y	-1.875	-1.875	0	0

Member Distributed Loads (BLC 3 : WL,R)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M1	X	.47	.47	0	0
2	M2	Y	.621	.621	0	0
3	M3	Y	.621	.621	0	0
4	M5	Y	.44	.44	0	0
5	M4	Y	.44	.44	0	0
6	M6	X	.396	.396	0	0

Member Distributed Loads (BLC 4 : WL,L)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M6	X	-.47	-.47	0	0
2	M5	Y	.621	.621	0	0
3	M4	Y	.621	.621	0	0
4	M2	Y	.44	.44	0	0
5	M3	Y	.44	.44	0	0
6	M1	X	-.396	-.396	0	0

Member Distributed Loads (BLC 7 : C)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M2	Y	-.075	-.075	0	0
2	M5	Y	-.075	-.075	0	0
3	M3	Y	-.075	-.075	0	0
4	M4	Y	-.075	-.075	0	0

Member Distributed Loads (BLC 8 : SL2 (w/ l=1.1))

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M2	Y	-2.063	-2.063	0	0
2	M3	Y	-2.063	-2.063	0	0
3	M4	Y	-2.063	-2.063	0	0
4	M5	Y	-2.063	-2.063	0	0

Member Distributed Loads (BLC 9 : W,R (point @ hauch))

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M2	Y	.621	.621	0	0
2	M3	Y	.621	.621	0	0
3	M4	Y	.44	.44	0	0
4	M5	Y	.44	.44	0	0

Member Distributed Loads (BLC 10 : W,R (point @ mid & haunch))

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft. %]	End Location[ft. %]
1	M4	Y	.44	.44	0	0
2	M5	Y	.44	.44	0	0
3	M2	Y	.621	.621	0	0
4	M3	Y	.621	.621	0	0

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
30			10	-11.66	4.524	18.92
31	17	M4	1	-12.274	-2.379	18.92
32			2	-12.487	-1.635	24.138
33			3	-12.699	-.892	27.423
34			4	-12.912	-.148	28.775
35			5	-13.124	.595	28.193
36			6	-13.337	1.339	25.679
37			7	-13.55	2.083	21.231
38			8	-13.762	2.826	14.85
39			9	-13.975	3.57	6.536
40			10	-14.187	4.314	-3.711
41	17	M5	1	-14.169	4.364	-3.711
42			2	-14.286	4.781	-10.376
43			3	-14.403	5.198	-17.65
44			4	-14.521	5.616	-25.532
45			5	-14.638	6.033	-34.022
46			6	-14.756	6.451	-43.121
47			7	-14.873	6.868	-52.829
48			8	-14.99	7.286	-63.145
49			9	-15.108	7.703	-74.07
50			10	-15.225	8.121	-85.603
51	17	M6	1	-13.836	-10.324	-85.603
52			2	-13.763	-9.877	-74.045
53			3	-13.69	-9.43	-62.998
54			4	-13.616	-8.983	-52.463
55			5	-13.543	-8.535	-42.44
56			6	-13.47	-8.088	-32.928
57			7	-13.396	-7.641	-23.929
58			8	-13.323	-7.194	-15.441
59			9	-13.25	-6.747	-7.465
60			10	-13.176	-6.299	0
61	26	M1	1	-19.493	14.04	0
62			2	-19.493	14.04	-16.066
63			3	-19.493	14.04	-32.133
64			4	-19.493	14.04	-48.199
65			5	-19.493	14.04	-64.266
66			6	-19.493	14.04	-80.332
67			7	-19.493	14.04	-96.399
68			8	-19.493	14.04	-112.465
69			9	-19.493	14.04	-128.531
70			10	-19.493	14.04	-144.598
71	26	M2	1	-16.458	-12.936	-144.598
72			2	-16.269	-12.264	-126.231
73			3	-16.08	-11.593	-108.843
74			4	-15.891	-10.921	-92.434
75			5	-15.702	-10.25	-77.004
76			6	-15.513	-9.578	-62.552
77			7	-15.325	-8.907	-49.08
78			8	-15.136	-8.236	-36.586
79			9	-14.947	-7.564	-25.07
80			10	-14.758	-6.893	-14.534
81	26	M3	1	-14.788	-6.833	-14.534
82			2	-14.446	-5.637	1.676
83			3	-14.104	-4.441	14.777
84			4	-13.762	-3.245	24.768
85			5	-13.42	-2.049	31.65
86			6	-13.078	-.853	35.423

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
144			4	-13.755	-2.943	29.15
145			5	-13.413	-1.747	35.246
146			6	-13.071	-.551	38.233
147			7	-12.729	.645	38.111
148			8	-12.387	1.841	34.879
149			9	-12.045	3.037	28.538
150			10	-11.703	4.233	19.088
151	27	M4	1	-12.155	-2.648	19.088
152			2	-12.368	-1.905	25.006
153			3	-12.58	-1.161	28.991
154			4	-12.793	-.417	31.043
155			5	-13.005	.326	31.162
156			6	-13.218	1.07	29.348
157			7	-13.431	1.813	25.6
158			8	-13.643	2.557	19.92
159			9	-13.856	3.301	12.306
160			10	-14.068	4.044	2.759
161	27	M5	1	-14.051	4.086	2.759
162			2	-14.168	4.503	-3.501
163			3	-14.285	4.921	-10.369
164			4	-14.403	5.338	-17.846
165			5	-14.52	5.755	-25.931
166			6	-14.638	6.173	-34.625
167			7	-14.755	6.59	-43.928
168			8	-14.872	7.008	-53.838
169			9	-14.99	7.425	-64.358
170			10	-15.107	7.843	-75.486
171	27	M6	1	-13.209	-8.332	-75.486
172			2	-13.209	-8.332	-65.951
173			3	-13.209	-8.332	-56.416
174			4	-13.209	-8.332	-46.881
175			5	-13.209	-8.332	-37.347
176			6	-12.889	-6.383	-29.218
177			7	-12.889	-6.383	-21.914
178			8	-12.889	-6.383	-14.609
179			9	-12.889	-6.383	-7.305
180			10	-12.889	-6.383	0

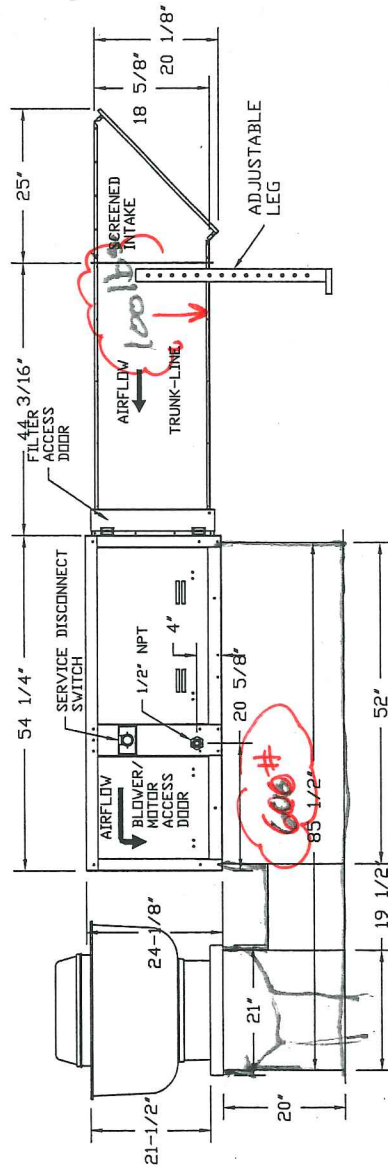
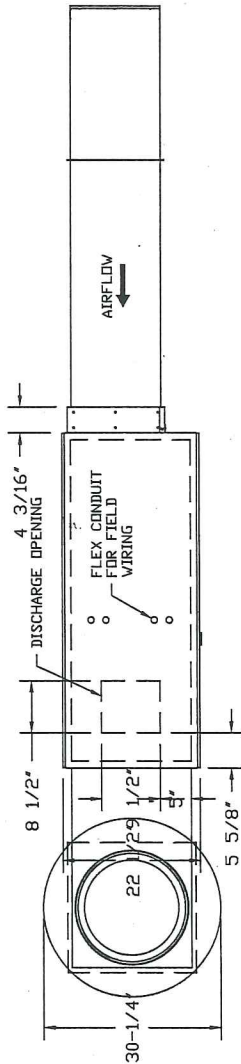
Envelope Member Section Forces

	Member	Sec		Axial[k]	LC	Shear[k]	LC	Moment[k-ft]	LC
1	M1	1	max	-18.939	17	14.351	17	0	17
2			min	-19.493	26	14.04	26	0	17
3		2	max	-18.852	17	14.183	27	-16.066	26
4			min	-19.493	26	13.82	17	-16.23	27
5		3	max	-18.765	17	14.183	27	-31.63	17
6			min	-19.493	26	13.29	17	-32.46	27
7		4	max	-18.678	17	14.183	27	-46.534	17
8			min	-19.493	26	12.759	17	-48.689	27
9		5	max	-18.59	17	14.183	27	-60.831	17
10			min	-19.493	26	12.228	17	-64.919	27
11		6	max	-18.503	17	14.04	26	-74.521	17
12			min	-19.493	26	11.697	17	-80.332	26
13		7	max	-18.416	17	14.04	26	-87.603	17
14			min	-19.493	26	11.167	17	-96.399	26
15		8	max	-18.329	17	14.04	26	-100.077	17

Envelope Member Section Forces (Continued)

Member	Sec		Axial[k]	LC	Shear[k]	LC	Moment[k-ft]	LC
73	7	max	-13.275	26	2.083	17	29.984	26
74		min	-13.55	17	1.554	26	21.231	17
75	8	max	-13.487	26	2.826	17	24.978	26
76		min	-13.762	17	2.297	26	14.85	17
77	9	max	-13.7	26	3.57	17	18.04	26
78		min	-13.975	17	3.041	26	6.536	17
79	10	max	-13.913	26	4.314	17	9.168	26
80		min	-14.187	17	3.784	26	-3.711	17
81	M5	1	max	-13.896	26	4.364	9.168	26
82		min	-14.169	17	3.817	26	-3.711	17
83	2	max	-14.013	26	4.781	17	3.299	26
84		min	-14.286	17	4.235	26	-10.376	17
85	3	max	-14.131	26	5.198	17	-3.178	26
86		min	-14.403	17	4.652	26	-17.65	17
87	4	max	-14.248	26	5.616	17	-10.264	26
88		min	-14.521	17	5.07	26	-25.532	17
89	5	max	-14.366	26	6.033	17	-17.958	26
90		min	-14.638	17	5.487	26	-34.022	17
91	6	max	-14.483	26	6.451	17	-26.26	26
92		min	-14.756	17	5.905	26	-43.121	17
93	7	max	-14.6	26	6.868	17	-35.172	26
94		min	-14.873	17	6.322	26	-52.829	17
95	8	max	-14.718	26	7.286	17	-44.692	26
96		min	-14.99	17	6.74	26	-63.145	17
97	9	max	-14.835	26	7.703	17	-54.82	26
98		min	-15.108	17	7.157	26	-74.07	17
99	10	max	-14.953	26	8.121	17	-65.557	26
100		min	-15.225	17	7.574	26	-85.603	17
101	M6	1	max	-12.584	26	-6.365	-65.557	26
102		min	-13.836	17	-10.324	17	-85.603	17
103	2	max	-12.584	26	-6.365	26	-58.273	26
104		min	-13.763	17	-9.877	17	-74.045	17
105	3	max	-12.584	26	-6.365	26	-50.989	26
106		min	-13.69	17	-9.43	17	-62.998	17
107	4	max	-12.584	26	-6.365	26	-43.705	26
108		min	-13.616	17	-8.983	17	-52.463	17
109	5	max	-12.584	26	-6.365	26	-36.42	26
110		min	-13.543	17	-8.535	17	-42.44	17
111	6	max	-12.584	26	-6.365	26	-29.136	26
112		min	-13.47	17	-8.088	17	-32.928	17
113	7	max	-12.584	26	-6.365	26	-21.852	26
114		min	-13.396	17	-7.641	17	-23.929	17
115	8	max	-12.584	26	-6.365	26	-14.568	26
116		min	-13.323	17	-7.194	17	-15.441	17
117	9	max	-12.584	26	-6.365	26	-7.284	26
118		min	-13.25	17	-6.747	17	-7.465	17
119	10	max	-12.584	26	-6.299	17	0	17
120		min	-13.176	17	-6.383	27	0	17

- FAN #1 NRTA-E76-EADJ50H - COMBINATION UNIT W/ HEATER
 1. COMBINATION UNIT WITH A HIGH SPEED DIRECT DRIVE UPBLAST CENTRIFUGAL EXHAUST FAN WITH A 13.75" WHEEL AND A LOW CFM DIRECT-FIRED MAKE-UP AIR UNIT
 2. TRUNKLINE W/INTAKE HOOD W/EZ FILTERS
 3. DOWN DISCHARGE - AIR FLOW RIGHT -> LEFT
 4. GREASE CUP FOR KITCHEN-DUTY CENTRIFUGAL EXHAUST FANS,
 BOX DIMENSIONS 17-1/8 L X 5-1/16 W X 3-3/4 H (18 GA) (INCLUDES DOWN SPOUT)
 5. COOLING INTERLOCK RELAY, 120V CONTACTS, LOCKS OUT BURNER CIRCUIT WHEN AC IS ENERGIZED.
 6. GAS PRESSURE GAUGE, 0-35", 2.5" DIAMETER, 1/4" THREAD SIZE
 7. GAS PRESSURE GAUGE, -5 TO +15 INCHES WC, 2.5" DIAMETER, 1/4" THREAD SIZE
 8. HIGH PROFILE LID OPTION FOR D76, REQUIRED WITH BELT DRIVE, SINGLE PHASE MOTORS.



SUPPLY SIDE HEATER INFORMATION

WINTER TEMPERATURE = 20°F. TEMP. RISE = 55°F.
 BTUS CALCULATED OFF STANDARD AIR DENSITY
 OUTPUT BTUS AT ALTITUDE OF 0.0 ft. = 47520
 INPUT BTUS AT ALTITUDE OF 0.0 ft. = 51652



Direct Fired (DF) Profile Plate Assembly

Direct Fired Profile Plate Specifications

Direct fired burners shall have patented US Patent No. US629523B2, self-adjusting profile plates designed to ensure proper air velocity and pressure drop across the burner. Profile plates shall be constructed of 304 stainless steel and shall be welded to the burner. Profile plates shall be constructed of 304 stainless steel and shall be welded to the burner.

Application: Burner profile plates are designed to automatically react to the demands of a burner at extreme, without the need for any motors or actuators to mechanically adjust them. With this feature, all DF units are designed for demand control ventilation (DCV) requirements.

Qualifications: Assemblies shall be designed to the 3F units, ETL listing and comply with combined safety standards ANSI Z83.4 and CSA 3.7 non-reticulating DF heaters and ANSI Z83.4 (recirculating DF heaters).

General Construction

Profile plates shall be formed from 600 galvanized steel.
 Profile plates shall vary in size per unit.
 Profile plates shall be mounted along the same plane as the discharge of the burner.
 Profile plates shall be constructed of 304 stainless steel and shall be welded to the burner.
 Spring hinges shall be made from pitted steel.



JOB	Jackson Fire Station Temp Kitcher
LOCATION	SALT LAKE CITY, UT,
DATE	8/16/2017
JOB #	3117501
DWG #	7
DRAWN BY	TT-72
REV.	SCALE 3/8" = 1'-0"

Floor Joist Span Tables

10 psf Dead Load and 50 psf Live Load

Member	Fy (ksi)	Live Load Deflection L/360						Live Load Deflection L/480					
		Single Span Spacing (in) o.c.			Two Equal Spans Spacing (in) o.c.			Single Span Spacing (in)			Two Equal Spans Spacing (in)		
		12	16	24	12	16	24	12	16	24	12	16	24
600S162-33	33	11' 3" e	9' 9" e	8' 0" e	11' 3" e	9' 5" e	7' 1" e	10' 7" e	9' 7" e	8' 0" e	11' 3" e	9' 5" e	7' 1" e
600S200-33	33	11' 8" e	10' 1" e	8' 3" e	11' 8" e	9' 8" e	7' 2" e	11' 1" e	10' 0" e	8' 3" e	11' 8" e	9' 8" e	7' 2" e
600S162-43	33	12' 8" e	11' 6" e	9' 8" e	13' 7" e	11' 9" e	9' 8" e	11' 6" e	10' 5" e	9' 1" e	12' 11" e	11' 9" e	9' 8" e
600S200-43	33	13' 3" e	12' 0" e	9' 9" e	13' 10" e	12' 0" e	9' 9" e	12' 1" e	11' 0" e	9' 7" e	13' 7" e	12' 0" e	9' 9" e
600S250-43	33	13' 11" e	12' 4" e	10' 0" e	14' 2" e	12' 4" e	10' 0" e	12' 8" e	11' 6" e	10' 0" e	14' 2" e	12' 4" e	10' 0" e
600S162-54	50	13' 7" e	12' 4" e	10' 9" e	15' 3" e	13' 10" e	12' 1" e	12' 4" e	11' 2" e	9' 9" e	13' 10" e	12' 7" e	11' 0" e
600S200-54	50	14' 3" e	13' 0" e	11' 4" e	16' 0" e	14' 7" e	12' 9" e	13' 0" e	11' 9" e	10' 3" e	14' 7" e	13' 3" e	11' 7" e
600S250-54	50	14' 11" e	13' 6" e	11' 10" e	16' 8" e	15' 2" e	13' 3" e	13' 6" e	12' 3" e	10' 9" e	15' 2" e	13' 9" e	12' 1" e
600S162-68	50	14' 7" e	13' 3" e	11' 7" e	16' 4" e	14' 10" e	13' 0" e	13' 3" e	12' 0" e	10' 6" e	14' 10" e	13' 6" e	11' 9" e
600S200-68	50	15' 4" e	13' 11" e	12' 2" e	17' 2" e	15' 7" e	13' 8" e	13' 11" e	12' 8" e	11' 0" e	15' 7" e	14' 2" e	12' 5" e
600S250-68	50	16' 1" e	14' 7" e	12' 9" e	18' 0" e	16' 4" e	14' 4" e	14' 7" e	13' 3" e	11' 7" e	16' 4" e	14' 10" e	13' 0" e
600S162-97	50	16' 2" e	14' 8" e	12' 10" e	18' 1" e	16' 5" e	14' 4" e	14' 8" e	13' 4" e	11' 8" e	16' 5" e	14' 11" e	13' 1" e
600S200-97	50	17' 0" e	15' 5" e	13' 6" e	19' 1" e	17' 4" e	15' 2" e	15' 5" e	14' 0" e	12' 3" e	17' 4" e	15' 9" e	13' 9" e
600S250-97	50	17' 10" e	16' 3" e	14' 2" e	20' 0" e	18' 2" e	15' 11" e	16' 3" e	14' 9" e	12' 10" e	18' 2" e	16' 6" e	14' 5" e
600S162-118	50	17' 0" e	15' 6" e	13' 6" e	19' 1" e	17' 4" e	15' 2" e	15' 6" e	14' 1" e	12' 3" e	17' 4" e	15' 9" e	13' 9" e
600S200-118	50	18' 0" e	16' 4" e	14' 3" e	20' 2" e	18' 4" e	16' 0" e	16' 4" e	14' 10" e	13' 0" e	18' 4" e	16' 8" e	14' 7" e
600S250-118	50	18' 11" e	17' 2" e	15' 0" e	21' 3" e	19' 3" e	16' 10" e	17' 2" e	15' 7" e	13' 7" e	19' 3" e	17' 6" e	15' 4" e
800S162-33	33	12' 6" e	10' 10" e	7' 11" e	10' 9" e	8' 8" e	6' 3" e	12' 6" e	10' 10" e	7' 11" e	10' 9" e	8' 8" e	6' 3" e
800S200-33	33	13' 5" e	11' 7" e	7' 11" e	11' 2" e	8' 11" e	6' 4" e	13' 5" e	11' 7" e	7' 11" e	11' 2" e	8' 11" e	6' 4" e
800S162-43	33	15' 0" e	12' 11" e	10' 7" e	15' 0" e	12' 11" e	10' 3" e	14' 4" e	12' 11" e	10' 7" e	15' 0" e	12' 11" e	10' 3" e
800S200-43	33	16' 8" e	14' 7" e	11' 11" e	16' 10" e	14' 7" e	11' 0" e	15' 2" e	13' 9" e	11' 11" e	16' 10" e	14' 7" e	11' 0" e
800S250-43	33	17' 0" e	14' 8" e	12' 0" e	17' 0" e	14' 8" e	11' 1" e	15' 10" e	14' 4" e	12' 0" e	17' 0" e	14' 8" e	11' 1" e
800S162-54	50	17' 0" e	15' 5" e	13' 6" e	19' 1" e	17' 4" e	14' 4" e	15' 5" e	14' 0" e	12' 3" e	17' 4" e	15' 9" e	13' 9" e
800S200-54	50	17' 11" e	16' 3" e	14' 3" e	20' 1" e	18' 3" e	15' 9" e	16' 3" e	14' 9" e	12' 11" e	18' 3" e	16' 7" e	14' 6" e
800S250-54	50	18' 7" e	16' 11" e	14' 9" e	20' 11" e	19' 0" e	15' 11" e	16' 11" e	15' 4" e	13' 5" e	19' 0" e	17' 3" e	15' 1" e
800S162-68	50	18' 4" e	16' 8" e	14' 7" e	20' 7" e	18' 9" e	16' 4" e	16' 8" e	15' 2" e	13' 3" e	18' 9" e	17' 0" e	14' 10" e
800S200-68	50	19' 3" e	17' 6" e	15' 3" e	21' 7" e	19' 7" e	17' 2" e	17' 6" e	15' 11" e	13' 10" e	19' 7" e	17' 10" e	15' 7" e
800S250-68	50	20' 1" e	18' 3" e	15' 11" e	22' 6" e	20' 6" e	17' 11" e	18' 3" e	16' 7" e	14' 6" e	20' 6" e	18' 7" e	16' 3" e
800S162-97	50	20' 5" e	18' 6" e	16' 2" e	22' 11" e	20' 10" e	18' 2" e	18' 6" e	16' 10" e	14' 9" e	20' 10" e	18' 11" e	16' 6" e
800S200-97	50	21' 5" e	19' 5" e	17' 0" e	24' 0" e	21' 10" e	19' 1" e	19' 5" e	17' 8" e	15' 5" e	21' 10" e	19' 10" e	17' 4" e
800S250-97	50	22' 4" e	20' 4" e	17' 9" e	25' 1" e	22' 10" e	19' 11" e	20' 4" e	18' 5" e	16' 1" e	22' 10" e	20' 9" e	18' 1" e
800S162-118	50	21' 7" e	19' 7" e	17' 2" e	24' 3" e	22' 0" e	19' 3" e	19' 7" e	17' 10" e	15' 7" e	22' 0" e	20' 0" e	17' 6" e
800S200-118	50	22' 8" e	20' 7" e	18' 0" e	25' 5" e	23' 1" e	20' 2" e	20' 7" e	18' 8" e	16' 4" e	23' 1" e	21' 0" e	18' 4" e
800S250-118	50	23' 8" e	21' 6" e	18' 10" e	26' 7" e	24' 2" e	21' 2" e	21' 6" e	19' 7" e	17' 1" e	24' 2" e	22' 0" e	19' 2" e
1000S162-43	33	16' 11" e	14' 8" e	11' 11" e	16' 5" e	13' 4" e	9' 11" e	16' 11" e	14' 8" e	11' 11" e	16' 5" e	13' 4" e	9' 11" e
1000S200-43	33	18' 0" e	15' 7" e	12' 8" e	17' 0" e	13' 10" e	10' 2" e	17' 10" e	15' 7" e	12' 8" e	17' 0" e	13' 10" e	10' 2" e
1000S250-43	33	18' 10" e	16' 4" e	13' 4" e	17' 6" e	14' 2" e	10' 5" e	18' 10" e	16' 4" e	13' 4" e	17' 6" e	14' 2" e	10' 5" e
1000S162-54	50	20' 2" e	18' 4" e	16' 0" e	22' 8" e	19' 10" e	15' 11" e	18' 4" e	16' 8" e	14' 7" e	20' 7" e	18' 8" e	15' 11" e
1000S200-54	50	21' 1" e	19' 2" e	16' 9" e	23' 8" e	20' 8" e	16' 4" e	19' 2" e	17' 5" e	15' 3" e	21' 6" e	19' 7" e	16' 4" e
1000S250-54	50	22' 3" e	20' 3" e	17' 8" e	25' 0" e	21' 8" e	16' 10" e	20' 3" e	18' 5" e	16' 1" e	22' 9" e	20' 8" e	16' 10" e
1000S162-68	50	21' 11" e	19' 11" e	17' 4" e	24' 7" e	22' 4" e	18' 11" e	19' 11" e	18' 1" e	15' 9" e	22' 4" e	20' 3" e	17' 9" e
1000S200-68	50	22' 10" e	20' 9" e	18' 2" e	25' 8" e	23' 4" e	20' 1" e	20' 9" e	18' 10" e	16' 6" e	23' 4" e	21' 2" e	18' 6" e
1000S250-68	50	24' 0" e	21' 9" e	19' 0" e	26' 11" e	24' 5" e	21' 4" e	21' 9" e	19' 9" e	17' 3" e	24' 5" e	22' 3" e	19' 5" e
1000S162-97	50	24' 7" e	22' 4" e	19' 6" e	27' 7" e	25' 1" e	21' 11" e	22' 4" e	20' 3" e	17' 9" e	25' 1" e	22' 9" e	19' 11" e
1000S200-97	50	25' 8" e	23' 4" e	20' 4" e	28' 10" e	26' 2" e	22' 10" e	23' 4" e	21' 2" e	18' 6" e	26' 2" e	23' 9" e	20' 9" e
1000S250-97	50	26' 9" e	24' 3" e	21' 2" e	30' 0" e	27' 3" e	23' 10" e	24' 3" e	22' 1" e	19' 3" e	27' 3" e	24' 9" e	21' 8" e
1000S162-118	50	26' 0" e	23' 8" e	20' 8" e	29' 3" e	26' 7" e	23' 2" e	23' 8" e	21' 6" e	18' 9" e	26' 7" e	24' 1" e	21' 1" e
1000S200-118	50	27' 3" e	24' 9" e	21' 7" e	30' 7" e	27' 9" e	24' 3" e	24' 9" e	22' 6" e	19' 7" e	27' 9" e	25' 3" e	22' 0" e
1000S250-118	50	28' 4" e	25' 9" e	22' 6" e	31' 10" e	28' 11" e	25' 3" e	25' 9" e	23' 5" e	20' 5" e	28' 11" e	26' 3" e	22' 11" e

MECH UNIT:

$$600\# + 100\# = 700\#$$

$$\div 2 \text{ Purlins}$$

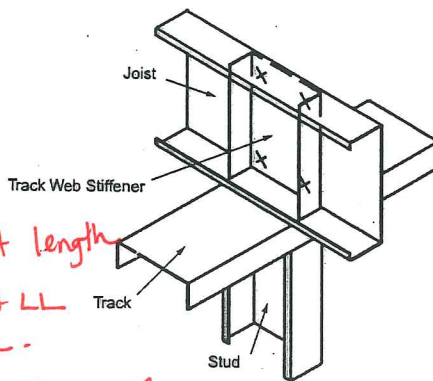
$$350\#/\text{purlin}$$

$$\div \sim 7' = 50\text{psf}$$

along unit length

⇒ 50psf LL

OK.



Web Stiffener:
Track Inside Joist

"e" Requires web stiffeners at end supports
Web stiffeners required at all interior supports
for 2-span conditions.

See page 34 for Allowable Floor Joist Span Table Notes.

-Use (2) Cee Purlins below unit
for 25' span

FLEXOSPAN - CEE AND ZEE LOAD TABLES

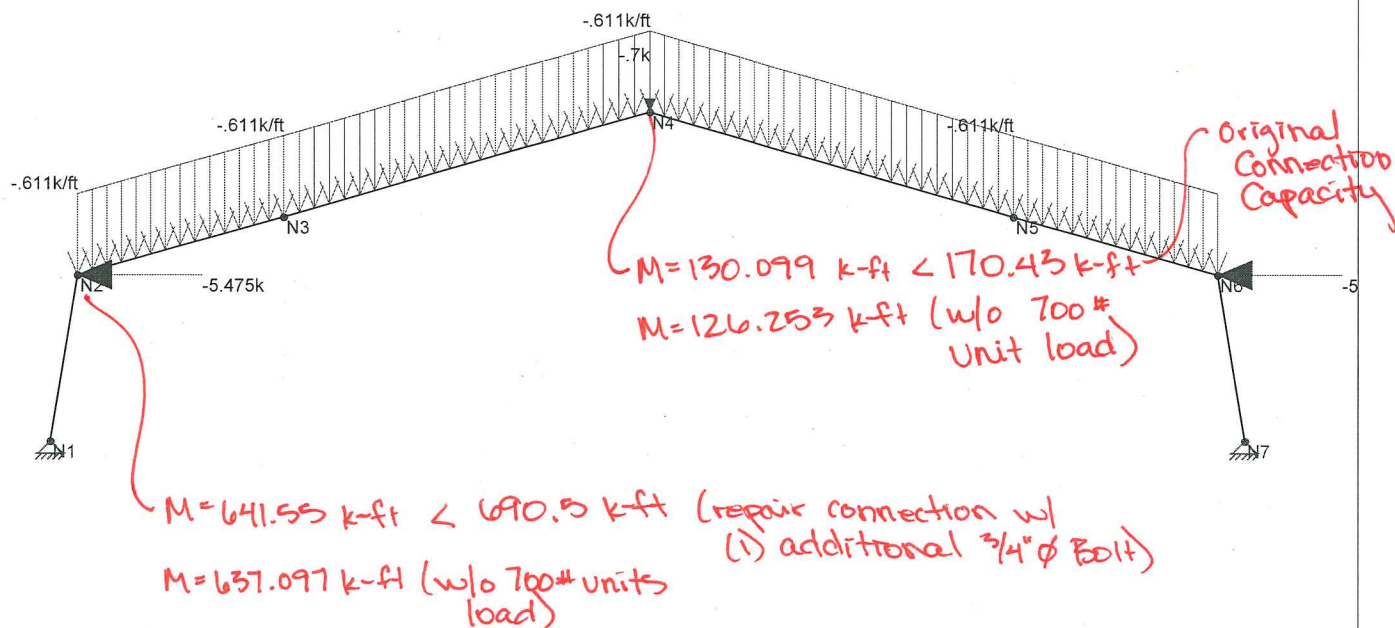
Allowable Uniform Loads in Pounds Per Lineal Foot

CEE		Simple Span						ZEE		Simple Span						3 or More Spans, Std. Lap					
Section	Bay	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	Section	Bay	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.
6" Web	10 ft	251	-	331	-	480	-	6" Web	10 ft	254	-	331	-	499	-	-	-	-	-	-	-
	12 ft	174	-	230	-	333	-		12 ft	126	-	230	-	346	-	-	-	-	-	-	-
	14 ft	128	-	169	-	244	-		14 ft	129	-	169	-	254	-	-	-	-	-	-	-
	15 ft	111	-	147	-	213	-		15 ft	113	-	147	-	221	-	-	-	-	-	-	-
	18 ft	77	-	102	-	148	-		18 ft	78	-	102	-	154	-	176	-	235	-	357	-
	20 ft	62	-	82	-	120	-		20 ft	63	-	82	-	124	-	118	-	157	-	238	-
	22 ft	51	-	68	-	99	-		22 ft	52	-	68	-	103	-	94	-	124	-	188	-
	24 ft	43	-	57	-	83	-		24 ft	44	-	57	-	86	-	76	-	101	-	153	-
	25 ft	40	-	53	-	76	-		25 ft	40	-	53	-	79	-	63	-	84	-	126	-
	28 ft	32	-	42	-	61	-		28 ft	32	-	42	-	63	-	58	-	76	-	116	-
8" Web	12 ft	260	269	341	365	493	545	8" Web	12 ft	260	265	340	364	510	545	-	-	-	-	-	-
	14 ft	191	198	250	268	362	400		14 ft	191	195	250	267	374	401	-	-	-	-	-	-
	15 ft	166	172	218	233	315	349		15 ft	166	169	218	233	326	349	-	-	-	-	-	-
	18 ft	115	119	151	162	219	242		18 ft	115	117	151	161	226	242	156	158	222	235	345	368
	20 ft	93	97	122	131	177	196		20 ft	93	95	122	131	183	196	127	129	178	188	275	293
	22 ft	77	80	101	108	146	162		22 ft	77	78	101	108	151	162	105	107	145	154	223	238
	24 ft	65	67	85	91	123	136		24 ft	65	66	85	91	127	136	88	90	121	129	185	198
	25 ft	60	62	78	84	113	125		25 ft	59	61	78	83	117	125	81	83	111	118	170	181
	28 ft	47	49	62	67	90	100		28 ft	47	48	62	66	93	100	65	66	87	93	133	142
	30 ft	41	43	54	58	78	87		30 ft	41	42	54	58	81	87	56	57	76	81	115	123
10" Web	20 ft	115	-	168	173	243	266	10" Web	20 ft	115	119	168	-	250	266	131	133	217	-	365	387
	22 ft	95	-	139	143	200	220		22 ft	80	82	116	-	173	185	96	97	154	-	248	263
	24 ft	80	-	116	120	168	185		24 ft	74	76	107	-	160	170	89	91	142	-	228	242
	25 ft	74	-	107	111	155	170		28 ft	59	60	85	-	127	136	72	74	114	-	179	191
	28 ft	59	-	85	88	124	136		30 ft	51	52	74	-	111	118	64	65	99	-	155	165
	30 ft	51	-	74	77	108	118		32 ft	45	46	65	-	97	104	56	58	87	-	135	144
	32 ft	45	-	65	67	94	104		35 ft	37	38	54	-	81	87	48	49	73	-	112	119
	34 ft	40	-	58	60	84	92		38 ft	32	33	46	-	69	73	41	42	61	-	94	101
	35 ft	37	-	54	56	79	87		20 ft	-	-	183	203	301	345	-	210	220	418	463	511
	38 ft	32	-	46	48	67	73		24 ft	-	-	127	141	209	239	-	153	162	289	324	368
12" Web	20 ft	-	-	185	206	293	344	12" Web	25 ft	-	-	117	130	192	220	-	142	151	265	299	344
	24 ft	-	-	128	143	203	239		28 ft	-	-	93	103	153	176	-	116	124	210	238	277
	25 ft	-	-	118	132	187	220		30 ft	-	-	81	90	133	153	-	102	109	183	207	242
	28 ft	-	-	94	105	149	175		32 ft	-	-	71	79	117	134	-	90	97	160	181	216
	30 ft	-	-	82	91	130	153		35 ft	-	-	59	66	98	112	-	76	82	133	151	176
	32 ft	-	-	72	80	114	134		37 ft	-	-	53	59	88	100	-	68	74	119	135	160
	34 ft	-	-	64	71	101	119		40 ft	-	-	45	50	75	86	-	59	64	101	115	135
	35 ft	-	-	60	67	95	112														
	38 ft	-	-	51	57	81	95														
	40 ft	-	-	46	51	73	86														

Notes: 1. The weight of the section has not been subtracted from these values. 2. Both flanges of member must be fully braced. 3. These loads are based on the transfer of the support loads directly to the web of the section by the use of clips or plates. For flanges bearing directly on structural, contact factory for section selection. 4. See back page for weights per lineal foot of members shown here. 5. Loads shown are stress governing. When deflection limits are specified, contact factory. 6. These sample calculations are very basic. Many different variables can affect loading. For instance; drift loading, building height, geographic location, etc. Please consult Flexspan if special conditions exist. 7. The selection of sections for your application is subject to final approval by your design professional. 8. Capacity values have been calculated in accordance with the AISI 2001 design manual. 9. Values shown in the load tables for three or more spans are based on uniform bay spacings. If non-uniform bay spacings exist, contact factory. UNCONTROLLED COPY



Main Frame $I=1.1$ w/
700# Unit DL @ ridge



Haunch Connection: (8) bolt $\frac{3}{4}" \phi$ T/B = 690.5 k-ft
Ridge Connection: (4) bolt $\frac{3}{4}" \phi$ T/B = 170.43 k-ft

Loads: LC 22, D + 0.25S + QE, L
Results for LC 23, D+C+S2

Nelson Engineering

MDL

17-202-01

Fairgrounds Moment Frame

SK - 1

Aug 29, 2017 at 8:24 AM

Fairgrounds Truss with redundancy...

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	1.375	0	0
2	N2	3.04167	10.1633	0
3	N3	15.6708	13.715	0
4	N4	38.1667	20.1467	0
5	N5	60.6625	13.715	0
6	N6	73.2917	10.1633	0
7	N7	74.9583	0	0

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]	Footing
1	N1	Reaction	Reaction		
2	N7	Reaction	Reaction		
3	N2				
4	N3				
5	N4				
6	N5				
7	N6				

Member Primary Data

	Label	I Joint	J Joint	Rotate(...)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2		W21x55	Beam	Wide Flange	A572 Gr.50	Typical
2	M2	N2	N3		W21x93	Beam	Wide Flange	A572 Gr.50	Typical
3	M3	N3	N4		W21x68	Beam	Wide Flange	A572 Gr.50	Typical
4	M4	N4	N5		W21x68	Beam	Wide Flange	A572 Gr.50	Typical
5	M5	N5	N6		W21x93	Beam	Wide Flange	A572 Gr.50	Typical
6	M6	N6	N7		W21x55	Beam	Wide Flange	A572 Gr.50	Typical

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	DL	DL			1		4
2	SL	SL					4
3	WL,R	WL					6
4	WL,L	WL					6
5	QE,R	EL			2		
6	QE,L	EL			2		
7	C	DL					4
8	SL2 (w/ I=1.1)	SL					4
9	W,R (point @ hauch)	WL			2		4
10	W,R (point @ mid & haunch)	WL			2	2	4

Load Combinations

	Description	Solve	PDelta	SRSS	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...
1	DL+SL		Y		1	1	2	1											
2	DL + 0.6WL,R		Y		1	1	3	.6											
3	DL + 0.6WL,L		Y		1	1	4	.6											
4	DL + 0.75(0.6WL,R) + 0.75SL		Y		1	1	3	.45	2	.75									
5	DL + 0.75(0.6WL,L) + 0.75SL		Y		1	1	4	.45	2	.75									
6	0.6DL + 0.6WL,R		Y		1	.6	3	.6											
7	0.6DL + 0.6WL,L		Y		1	.6	4	.6											
8			Y																

Load Combinations (Continued)

	Description	Solve	PDelta	SRSS	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
9	(1.0+0.14SDS)DL + 0.7*RHO*		Y		1	1.1...	5	.7										
10	(1.0+0.14SDS)DL + 0.7*RHO*		Y		1	1.1...	6	.7										
11	(1.0+0.10SDS)DL + 0.525*RH...		Y		1	1.0...	5	.525	2	.75								
12	(1.0+0.10SDS)DL + 0.525*RH...		Y		1	1.0...	6	.525	2	.75								
13	(0.6-0.14SDS)DL + 0.7*RHO*		Y		1	.486	5	.7										
14	(0.6-0.14SDS)DL + 0.7*RHO*		Y		1	.486	6	.7										
15			Y															
16	D + C + S		Y		1	1	7	1	2	1								
17	D + W,R (uniform wind)		Y		1	1	3	1										
18	D + W,L (uniform wind)		Y		1	1	4	1										
19	D + QE,R		Y		1	1	5	1										
20	D + QE,L		Y		1	1	6	1										
21	D + 0.25S + QE,R		Y		1	1	2	.25	5	1								
22	D + 0.25S + QE,L		Y		1	1	2	.25	6	1								
23	D+C+S2	Yes	Y		1	1	7	1	8	1								
24	D+0.25S2+QE,R	Yes	Y		1	1	8	.25	5	1								
25			Y															
26	D+W,R (point load @ haunch)		Y		1	1	9	1										
27	D+W,R (point load @ mid & h...		Y		1	1	10	1										

Joint Loads and Enforced Displacements (BLC 1 : DL)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s...
1	N4	L	Y	-.7

Joint Loads and Enforced Displacements (BLC 5 : QE,R)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s...
1	N2	L	X	5.47
2	N6	L	X	5.47

Joint Loads and Enforced Displacements (BLC 6 : QE,L)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s...
1	N6	L	X	-5.475
2	N2	L	X	-5.475

Joint Loads and Enforced Displacements (BLC 9 : W,R (point @ hauch))

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s...
1	N2	L	X	4.777
2	N6	L	X	4.015

Joint Loads and Enforced Displacements (BLC 10 : W,R (point @ mid & haunch))

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s...
1	N2	L	X	2.427
2	N6	L	X	2.04

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft, %]	End Location[ft, %]
1	M2	Y	-.142	-.142	0	0
2	M5	Y	-.142	-.142	0	0
3	M3	Y	-.142	-.142	0	0
4	M4	Y	-.142	-.142	0	0

Member Distributed Loads (BLC 2 : SL)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M2	Y	-1.875	-1.875	0	0
2	M5	Y	-1.875	-1.875	0	0
3	M3	Y	-1.875	-1.875	0	0
4	M4	Y	-1.875	-1.875	0	0

Member Distributed Loads (BLC 3 : WL,R)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M1	X	.47	.47	0	0
2	M2	Y	.621	.621	0	0
3	M3	Y	.621	.621	0	0
4	M5	Y	.44	.44	0	0
5	M4	Y	.44	.44	0	0
6	M6	X	.396	.396	0	0

Member Distributed Loads (BLC 4 : WL,L)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M6	X	-.47	-.47	0	0
2	M5	Y	.621	.621	0	0
3	M4	Y	.621	.621	0	0
4	M2	Y	.44	.44	0	0
5	M3	Y	.44	.44	0	0
6	M1	X	-.396	-.396	0	0

Member Distributed Loads (BLC 7 : C)

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M2	Y	-.075	-.075	0	0
2	M5	Y	-.075	-.075	0	0
3	M3	Y	-.075	-.075	0	0
4	M4	Y	-.075	-.075	0	0

Member Distributed Loads (BLC 8 : SL2 (w/ l=1.1))

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M2	Y	-2.063	-2.063	0	0
2	M3	Y	-2.063	-2.063	0	0
3	M4	Y	-2.063	-2.063	0	0
4	M5	Y	-2.063	-2.063	0	0

Member Distributed Loads (BLC 9 : W,R (point @ hauch))

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M2	Y	.621	.621	0	0
2	M3	Y	.621	.621	0	0
3	M4	Y	.44	.44	0	0
4	M5	Y	.44	.44	0	0

Member Distributed Loads (BLC 10 : W,R (point @ mid & haunch))

	Member Label	Direction	Start Magnitud...	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M4	Y	.44	.44	0	0
2	M5	Y	.44	.44	0	0
3	M2	Y	.621	.621	0	0
4	M3	Y	.621	.621	0	0

Joint Reactions

	LC	Joint Label	X [k]	Y [k]	MZ [k-ft]
1	23	N1	75.421	83.644	0
2	23	N7	-75.421	83.644	0
3	23	Totals:	0	167.287	
4	23	COG (ft):	X: 38.167	Y: 15.158	
5	24	N1	16.434	22.844	0
6	24	N7	-27.374	25.939	0
7	24	Totals:	-10.94	48.783	
8	24	COG (ft):	X: 38.167	Y: 15.209	

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Moment [k-ft]	LC
1	N1	max	75.421	23	83.644	23	0	23
2		min	16.434	24	22.844	24	0	23
3	N7	max	-27.374	24	83.644	23	0	23
4		min	-75.421	23	25.939	24	0	23
5	Totals:	max	0	23	167.287	23		
6		min	-10.94	24	48.783	24		

Member Section Forces

	LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
1	23	M1	1	94.746	-62.292	0
2			2	94.746	-62.292	71.284
3			3	94.746	-62.292	142.567
4			4	94.746	-62.292	213.851
5			5	94.746	-62.292	285.135
6			6	94.746	-62.292	356.418
7			7	94.746	-62.292	427.702
8			8	94.746	-62.292	498.985
9			9	94.746	-62.292	570.269
10			10	94.746	-62.292	641.553
11	23	M2	1	95.249	61.815	641.553
12			2	94.349	58.614	553.78
13			3	93.449	55.413	470.673
14			4	92.548	52.212	392.231
15			5	91.648	49.012	318.455
16			6	90.748	45.811	249.345
17			7	89.848	42.61	184.901
18			8	88.948	39.409	125.122
19			9	88.048	36.209	70.009
20			10	87.147	33.008	19.561
21	23	M3	1	87.282	32.053	19.561
22			2	85.652	26.352	-56.356
23			3	84.022	20.65	-117.451
24			4	82.392	14.949	-163.724
25			5	80.762	9.247	-195.175
26			6	79.132	3.546	-211.804
27			7	77.502	-2.156	-213.611
28			8	75.871	-7.857	-200.595
29			9	74.241	-13.559	-172.758
30			10	72.611	-19.26	-130.099
31	23	M4	1	72.611	19.26	-130.099
32			2	74.241	13.559	-172.758
33			3	75.871	7.857	-200.594

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
34			4	77.502	2.156	-213.609
35			5	79.132	-3.546	-211.803
36			6	80.762	-9.247	-195.174
37			7	82.392	-14.949	-163.723
38			8	84.022	-20.65	-117.45
39			9	85.652	-26.352	-56.355
40			10	87.282	-32.053	19.561
41	23	M5	1	87.147	-33.008	19.561
42			2	88.048	-36.209	70.009
43			3	88.948	-39.41	125.123
44			4	89.848	-42.61	184.902
45			5	90.748	-45.811	249.347
46			6	91.648	-49.012	318.458
47			7	92.548	-52.213	392.235
48			8	93.448	-55.414	470.677
49			9	94.349	-58.614	553.785
50			10	95.249	-61.815	641.559
51	23	M6	1	94.746	62.293	641.559
52			2	94.746	62.293	570.274
53			3	94.746	62.293	498.99
54			4	94.746	62.293	427.706
55			5	94.746	62.293	356.421
56			6	94.746	62.293	285.137
57			7	94.746	62.293	213.853
58			8	94.746	62.293	142.569
59			9	94.746	62.293	71.284
60			10	94.746	62.293	0
61	24	M1	1	25.202	-12.499	0
62			2	25.202	-12.499	14.304
63			3	25.202	-12.499	28.607
64			4	25.202	-12.499	42.911
65			5	25.202	-12.499	57.214
66			6	25.202	-12.499	71.518
67			7	25.202	-12.499	85.822
68			8	25.202	-12.499	100.125
69			9	25.202	-12.499	114.429
70			10	25.202	-12.499	128.732
71	24	M2	1	27.27	16.248	128.732
72			2	27.011	15.324	105.721
73			3	26.751	14.4	84.057
74			4	26.491	13.476	63.739
75			5	26.231	12.553	44.768
76			6	25.971	11.629	27.144
77			7	25.712	10.705	10.867
78			8	25.452	9.781	-4.064
79			9	25.192	8.857	-17.649
80			10	24.932	7.933	-29.886
81	24	M3	1	24.965	7.699	-29.886
82			2	24.495	6.054	-47.763
83			3	24.024	4.408	-61.361
84			4	23.554	2.762	-70.681
85			5	23.083	1.117	-75.723
86			6	22.613	-0.529	-76.487
87			7	22.142	-2.175	-72.973
88			8	21.672	-3.82	-65.181
89			9	21.201	-5.466	-53.11
90			10	20.731	-7.112	-36.761

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
91	24	M4	1	21.582	4.067	-36.761
92			2	22.052	2.422	-45.196
93			3	22.523	.776	-49.352
94			4	22.993	-.87	-49.23
95			5	23.464	-2.515	-44.83
96			6	23.934	-4.161	-36.152
97			7	24.405	-5.807	-23.196
98			8	24.875	-7.452	-5.962
99			9	25.346	-9.098	15.551
100			10	25.816	-10.743	41.341
101	24	M5	1	25.77	-10.822	41.341
102			2	26.03	-11.746	57.79
103			3	26.29	-12.67	75.585
104			4	26.55	-13.594	94.727
105			5	26.809	-14.518	115.215
106			6	27.069	-15.441	137.051
107			7	27.329	-16.365	160.233
108			8	27.589	-17.289	184.761
109			9	27.849	-18.213	210.637
110			10	28.109	-19.137	237.859
111	24	M6	1	30.027	23.095	237.859
112			2	30.027	23.095	211.43
113			3	30.027	23.095	185.001
114			4	30.027	23.095	158.573
115			5	30.027	23.095	132.144
116			6	30.027	23.095	105.715
117			7	30.027	23.095	79.286
118			8	30.027	23.095	52.858
119			9	30.027	23.095	26.429
120			10	30.027	23.095	0

Envelope Member Section Forces

Member	Sec		Axial[k]	LC	Shear[k]	LC	Moment[k-ft]	LC
1	M1	1	max	94.746	23	-12.499	24	0
2			min	25.202	24	-62.292	23	0
3		2	max	94.746	23	-12.499	24	71.284
4			min	25.202	24	-62.292	23	14.304
5		3	max	94.746	23	-12.499	24	142.567
6			min	25.202	24	-62.292	23	28.607
7		4	max	94.746	23	-12.499	24	213.851
8			min	25.202	24	-62.292	23	42.911
9		5	max	94.746	23	-12.499	24	285.135
10			min	25.202	24	-62.292	23	57.214
11		6	max	94.746	23	-12.499	24	356.418
12			min	25.202	24	-62.292	23	71.518
13		7	max	94.746	23	-12.499	24	427.702
14			min	25.202	24	-62.292	23	85.822
15		8	max	94.746	23	-12.499	24	498.985
16			min	25.202	24	-62.292	23	100.125
17		9	max	94.746	23	-12.499	24	570.269
18			min	25.202	24	-62.292	23	114.429
19		10	max	94.746	23	-12.499	24	641.553
20			min	25.202	24	-62.292	23	128.732
21	M2	1	max	95.249	23	61.815	23	641.553
22			min	27.27	24	16.248	24	128.732

Envelope Member Section Forces (Continued)

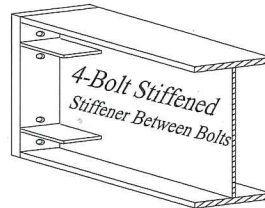
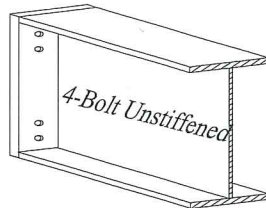
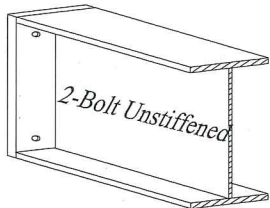
	Member	Sec		Axial[k]	LC	Shear[k]	LC	Moment[k-ft]	LC
23		2	max	94.349	23	58.614	23	553.78	23
24			min	27.011	24	15.324	24	105.721	24
25		3	max	93.449	23	55.413	23	470.673	23
26			min	26.751	24	14.4	24	84.057	24
27		4	max	92.548	23	52.212	23	392.231	23
28			min	26.491	24	13.476	24	63.739	24
29		5	max	91.648	23	49.012	23	318.455	23
30			min	26.231	24	12.553	24	44.768	24
31		6	max	90.748	23	45.811	23	249.345	23
32			min	25.971	24	11.629	24	27.144	24
33		7	max	89.848	23	42.61	23	184.901	23
34			min	25.712	24	10.705	24	10.867	24
35		8	max	88.948	23	39.409	23	125.122	23
36			min	25.452	24	9.781	24	-4.064	24
37		9	max	88.048	23	36.209	23	70.009	23
38			min	25.192	24	8.857	24	-17.649	24
39		10	max	87.147	23	33.008	23	19.561	23
40			min	24.932	24	7.933	24	-29.886	24
41	M3	1	max	87.282	23	32.053	23	19.561	23
42			min	24.965	24	7.699	24	-29.886	24
43		2	max	85.652	23	26.352	23	-47.763	24
44			min	24.495	24	6.054	24	-56.356	23
45		3	max	84.022	23	20.65	23	-61.361	24
46			min	24.024	24	4.408	24	-117.451	23
47		4	max	82.392	23	14.949	23	-70.681	24
48			min	23.554	24	2.762	24	-163.724	23
49		5	max	80.762	23	9.247	23	-75.723	24
50			min	23.083	24	1.117	24	-195.175	23
51		6	max	79.132	23	3.546	23	-76.487	24
52			min	22.613	24	-529	24	-211.804	23
53		7	max	77.502	23	-2.156	23	-72.973	24
54			min	22.142	24	-2.175	24	-213.611	23
55		8	max	75.871	23	-3.82	24	-65.181	24
56			min	21.672	24	-7.857	23	-200.595	23
57		9	max	74.241	23	-5.466	24	-53.11	24
58			min	21.201	24	-13.559	23	-172.758	23
59		10	max	72.611	23	-7.112	24	-36.761	24
60			min	20.731	24	-19.26	23	-130.099	23
61	M4	1	max	72.611	23	19.26	23	-36.761	24
62			min	21.582	24	4.067	24	-130.099	23
63		2	max	74.241	23	13.559	23	-45.196	24
64			min	22.052	24	2.422	24	-172.758	23
65		3	max	75.871	23	7.857	23	-49.352	24
66			min	22.523	24	.776	24	-200.594	23
67		4	max	77.502	23	2.156	23	-49.23	24
68			min	22.993	24	-.87	24	-213.609	23
69		5	max	79.132	23	-2.515	24	-44.83	24
70			min	23.464	24	-3.546	23	-211.803	23
71		6	max	80.762	23	-4.161	24	-36.152	24
72			min	23.934	24	-9.247	23	-195.174	23
73		7	max	82.392	23	-5.807	24	-23.196	24
74			min	24.405	24	-14.949	23	-163.723	23
75		8	max	84.022	23	-7.452	24	-5.962	24
76			min	24.875	24	-20.65	23	-117.45	23
77		9	max	85.652	23	-9.098	24	15.551	24
78			min	25.346	24	-26.352	23	-56.355	23
79		10	max	87.282	23	-10.743	24	41.341	24

Envelope Member Section Forces (Continued)

Member	Sec		Axial[k]	LC	Shear[k]	LC	Moment[k-ft]	LC
80		min	25.816	24	-32.053	23	19.561	23
81	M5	max	87.147	23	-10.822	24	41.341	24
82		min	25.77	24	-33.008	23	19.561	23
83	2	max	88.048	23	-11.746	24	70.009	23
84		min	26.03	24	-36.209	23	57.79	24
85	3	max	88.948	23	-12.67	24	125.123	23
86		min	26.29	24	-39.41	23	75.585	24
87	4	max	89.848	23	-13.594	24	184.902	23
88		min	26.55	24	-42.61	23	94.727	24
89	5	max	90.748	23	-14.518	24	249.347	23
90		min	26.809	24	-45.811	23	115.215	24
91	6	max	91.648	23	-15.441	24	318.458	23
92		min	27.069	24	-49.012	23	137.051	24
93	7	max	92.548	23	-16.365	24	392.235	23
94		min	27.329	24	-52.213	23	160.233	24
95	8	max	93.448	23	-17.289	24	470.677	23
96		min	27.589	24	-55.414	23	184.761	24
97	9	max	94.349	23	-18.213	24	553.785	23
98		min	27.849	24	-58.614	23	210.637	24
99	10	max	95.249	23	-19.137	24	641.559	23
100		min	28.109	24	-61.815	23	237.859	24
101	M6	max	94.746	23	62.293	23	641.559	23
102		min	30.027	24	23.095	24	237.859	24
103	2	max	94.746	23	62.293	23	570.274	23
104		min	30.027	24	23.095	24	211.43	24
105	3	max	94.746	23	62.293	23	498.99	23
106		min	30.027	24	23.095	24	185.001	24
107	4	max	94.746	23	62.293	23	427.706	23
108		min	30.027	24	23.095	24	158.573	24
109	5	max	94.746	23	62.293	23	356.421	23
110		min	30.027	24	23.095	24	132.144	24
111	6	max	94.746	23	62.293	23	285.137	23
112		min	30.027	24	23.095	24	105.715	24
113	7	max	94.746	23	62.293	23	213.853	23
114		min	30.027	24	23.095	24	79.286	24
115	8	max	94.746	23	62.293	23	142.569	23
116		min	30.027	24	23.095	24	52.858	24
117	9	max	94.746	23	62.293	23	71.284	23
118		min	30.027	24	23.095	24	26.429	24
119	10	max	94.746	23	62.293	23	0	23
120		min	30.027	24	23.095	24	0	23

HAUNCH

- connection calculation to match / compare to original design calculations



☐ 2-Bolt Unstiffened

☒ 4-Bolt Unstiffened

☐ 4-Bolt Stiffener Between Bolts

☐ 4-Bolt Stiffener Inside

Service Load Moment $M_w = 650.20$ kip-ft

$M_a = 361.59$ kip-ft

$M_a(\text{original}) = 650.2$

$F_{py} = 36$ psi

End Plate $t_p = 1/2$ "

Bolt Dia. = 3/4" A325

$b_p = 8.000$ in

$t_f = 0.313$ in

$g = 3.250$ in

0.375

1.375

$h = 72.000$ in

$p_f = 3.688$ in

$p_b = 3.000$ in

1.500

Thin Plate Behavior W/ Prying Action Control !

Connection Not OK, Please Use Thicker Endplate or Larger Bolts!

$r_f = 1.25$

$s = 2.55$ in

$h_1 = 68.00$ in

$h_2 = 65.00$ in

$d_1 = 67.84$ in

$d_2 = 64.84$ in

$Y = 543.11$ in

$d_s = 61.84$

$\Sigma d_n = 132.69$ in

End-Plate Yielding: $\phi_b M_{pl} = \phi_b F_{py} t_p^2 Y = 366.60$ k-ft

Bolt Rupture: $P_t = (\pi d_b^2 / 4) F_t = 39.76$ kips

(No Prying Action) $\phi M_{np} = \phi (2 P_t (\Sigma d_n)) = 659.47$ k-ft

Thin Plate Behavior (w/prying action)

$Q_{max,i} = 7.06$ kips

$T_b = 14.00$ kips

$\phi M_q = 542.39$ k-ft

194.53

966.81

÷ 1.5

644.5

644.5

644.5

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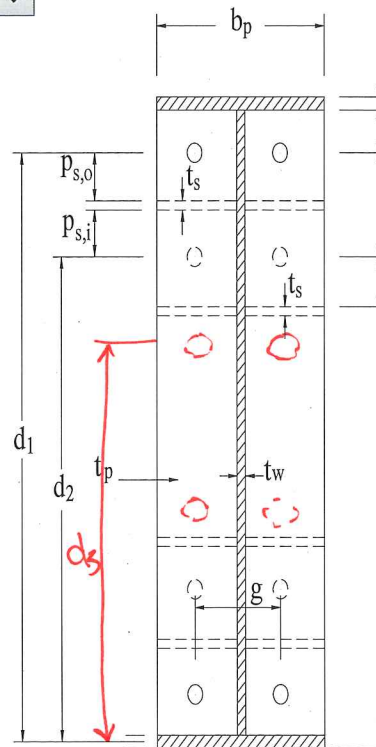
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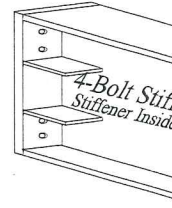
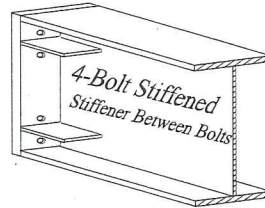
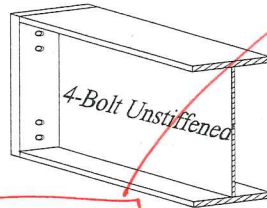
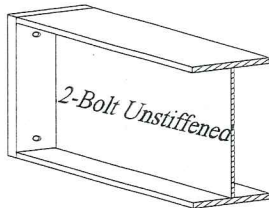


ignores prying action calculation

ignores prying action

Haunch
I=1.1

add (1) additional
3/4" Ø Bolt FS/TB



8-Bolt

* Note = Stiffened / Unstiffened are equal b/c prying action control

☐ 2-Bolt Unstiffened

☒ 4-Bolt Unstiffened

☒ 4-Bolt Stiffener Between Bolts

☐ 4-Bolt Stiffener Inside

1.1 factor to original → 693.2 k-ft

↓ RISA Model

690.5 k-ft

Service Load Moment $M_w = 637.10$ kip-ft > $M_a = 361.59$ kip-ft

$F_{py} = 36$ psi End Plate $t_p = 1/2$ " Bolt Dia. = 3/4" A325

$b_p = 8.000$ in

$t_f = 0.313$ in

$g = 3.250$ in

$h = 72.000$ in

$p_f = 3.688$ in

$p_b = 3.000$ in

Thin Plate Behavior W/ Prying Action Control !
Connection Not OK, Please Use Thicker Endplate or Larger Bolts!

$r_f = 1.25$

$s = 2.55$ in

$d_4 = 58.84$

$h_1 = 68.00$ in

$h_2 = 65.00$ in

$d_1 = 67.84$ in

$d_2 = 64.84$ in

$\Sigma d_n = 253.4$

$Y = 930.69$ in

$d_3 = 61.84$

End-Plate Yielding: $\phi_b M_{pl} = \phi_b F_{py} t^2 Y = 628.22$ k-ft

Bolt Rupture: $P_t = (\pi d_b^2 / 4) F_t = 39.76$ kips

(No Prying Action) $\phi M_{np} = \phi (2 P_t (\Sigma d_n)) = 659.47$ k-ft

Thin Plate Behavior (w/prying action)

$Q_{max,i} = 7.06$ kips

$T_b = 14.00$ kips

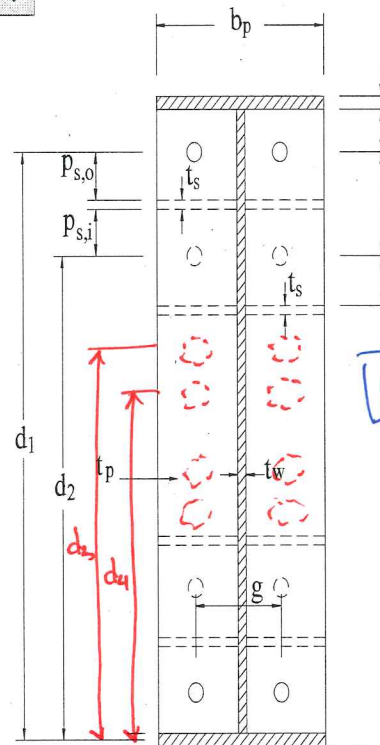
$\phi M_q = 542.39$ k-ft

$(E34 - B28) F_{31} = (39.76 - 7.06) 253.4 = 8286.18$

$B39 * F_{31} = 14.0 (253.4) = 3547.6$

$\phi M_g = 0.75 * 2 * 8286.18 / 12 = 1035.77$

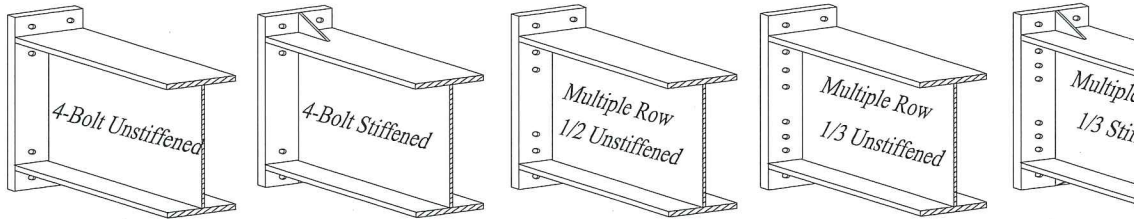
$M_a = \frac{1035.77}{1.5} = 690.5$ k-ft



8-Bolt

RIDGE

-connection calculation to match / compare to original design calculations.



☒ 4-Bolt Unstiffened ☐ 4-Bolt Stiffened ☐ Mul-Row 1/2 Unstiff.. ☐ Mul-Row 1/3 Unstiff. ☐ Mul-F

1.1 Factor to original = 14.66 $\checkmark$ RISA Model (w/o 700* Unit Load)

Service Load Moment $M_w = 126.25$ kip-ft $< M_a = 170.43$ kip-ft

M_a (original) = 115.3 k-ft calcs.

$F_{py} = 36$ psi End Plate $t_p = 1/2$ " Bolt Dia. = 3/4" A325

$b_p = 6.000$ in $h = 29.250$ in
 $t_f = 0.250$ in $p_{f,o} = 1.500$ in
 $g = 3.000$ in $p_{f,i} = 1.500$ in
 $p_{ext} = 3.000$ in 3.000

Thin Plate Behavior W/ Prying Action Control !
Connection Capacity OK !

$r_f = 1.00$ $s = 2.12$ in $d_e = 1.50$ in
 $h_o = 30.75$ in $h_1 = 27.50$ in
 $d_o = 30.63$ in $d_1 = 27.38$ in
 $\Sigma d_n = 58.00$ in
 $Y = 220.28$ in

End-Plate Yielding: $\phi_b M_{pl} = \phi_b F_{py} t^2 Y = 148.69$ k-ft
 Bolt Rupture: $P_t = (\pi d_b^2 / 4) F_t = 39.76$ kips
 (No Prying Action) $\phi M_{np} = \phi (2 P_t (\Sigma d_n)) = 288.27$ k-ft

Thin Plate Behavior(w/prying action)

$T_b = 14.00$ kips
 $Q_{max,i} = 4.50$ kips
 $Q_{max,o} = 4.50$ kips
 $\phi M_q = 255.65$ k-ft

