BTL ENGINEERING

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Structural Calculations

For

Hershkowitz Residence Canopy

6104 84th Avenue SE
Mercer Island, WA 98040

May 19th, 2021

ERIC A. DANN
STATE OF WASHINGTON
33029
REGISTERED
PROFESSIONAL ENGINEER

5.19.2021

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Gravity Analysis

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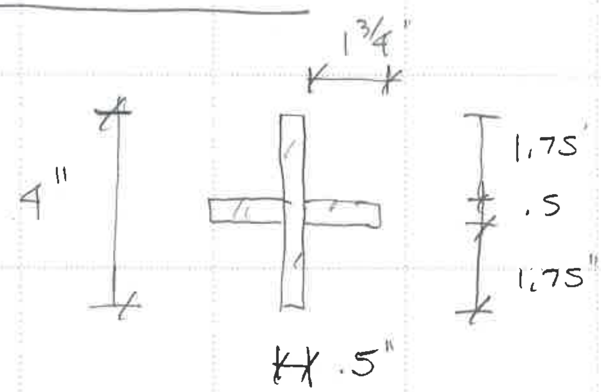
SECTION PROPERTIES OF STEEL COLUMN

$$A = (.5)(4) + (2)(.5)(1.75)$$

$$= 3.75 \text{ in}^2$$



$$S_x = \frac{(.5)(4)^2}{6} + \frac{(4)(.5)^2}{6} - \frac{(.5)(}{6}$$

$$= 1.33 \text{ in}^3 + .167 - .0201 = 1.48 \text{ in}^3$$

$$I_x = \frac{(.5)(4)^3}{12} + \frac{(4)(.5)^3}{12} - \frac{(.5)^3}{12}$$

$$= 2.67 \text{ in}^4 + .042 - .01 = 2.70 \text{ in}^4$$

$$r = \sqrt{\frac{I}{A}} = .848$$

WEIGHT OF COLUMN

$$\frac{3.75 \text{ in}^2}{144}(490 \text{ pcf}) = 13. \text{ PLF}$$

Project: Designed By: Date:

Project Number: Client: Scale: Page: G-1

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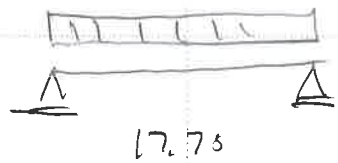
BEAM - WORST CASE

SNOW LOAD — DEAD LOAD

$W = (4/2+5)(25+10)$

$= .245$ KLF



$M = 9.65$ K-ft

$V = 2.2^K$ K — 1.6^K SL, $.6^K$ DL

3 1/2 x 10 1/2 GLB (MINIMUM)

$EI_{TL/180} = 462$ lb·in² x 10⁶

$EI_{LL/240} = 435$ lb·in² x 10⁶

CAPACITY OF 3 1/2 x 10 1/2 GLB (SEE FOLLOWING PAGE)

$M = 14.79$ K-ft

$V = 7.47^K$

$EI = 607$

CHECK COLUMN - SEE C- FOR SECTION PROPERTIES

$K\ell/r = \frac{(1)(7)(12)}{.8485} = 99$

$F_A = 12\pi^2 E / (23)(\frac{K\ell}{r})^2 = 4.85$ ksi

$P_{ALLOW} = (4.855)(A) = (4.85)(3.75) = 18^K > 2.2$ OK

COLUMN OK

DOUGLAS FIR 24F - V4

$F_b =$ 2400 psi
$F_v =$ 265 psi
$E =$ 1800 ksi
$F_{c\perp} =$ 650 psi

b x d	M_{allow} (ft-k) = F_b' * S			V_{allow} (k) = 2/3 * Fv' * b * d			EI	I	S	
(in) (in)	C_D = 1.0	= 1.15	= 1.33	C_D = 1.0	= 1.15	= 1.33	(lb-in^2 x 10^6)	(in^4)	(in^3)	C_V[a]
3 1/2 x 6	4.20	4.83	5.60	3.71	4.27	4.95	113.40	63.00	21.00	1.00
3 1/2 x 7 1/2	6.56	7.55	8.75	4.64	5.33	6.18	221.48	123.05	32.81	1.00
3 1/2 x 9	9.45	10.87	12.60	5.57	6.40	7.42	382.73	212.63	47.25	1.00
3 1/2 x 10 1/2	12.86	14.79	17.15	6.49	7.47	8.66	607.75	337.64	64.31	1.00
3 1/2 x 12	16.80	19.32	22.40	7.42	8.53	9.89	907.20	504.00	84.00	1.00
3 1/2 x 13 1/2	21.26	24.45	28.35	8.35	9.60	11.13	1,291.70	717.61	106.31	1.00
3 1/2 x 15	26.25	30.19	35.00	9.28	10.67	12.37	1,771.88	984.38	131.25	1.00
3 1/2 x 16 1/2	30.84	35.47	41.12	10.20	11.73	13.60	2,358.37	1310.20	158.81	0.97
3 1/2 x 18	36.39	41.85	48.52	11.13	12.80	14.84	3,061.80	1701.00	189.00	0.96
3 1/2 x 19 1/2	42.36	48.72	56.49	12.06	13.87	16.08	3,892.81	2162.67	221.81	0.95
3 1/2 x 21	48.77	56.09	65.03	12.99	14.93	17.31	4,862.03	2701.13	257.25	0.95
3 1/2 x 22 1/2	55.60	63.94	74.13	13.91	16.00	18.55	5,980.08	3322.27	295.31	0.94
3 1/2 x 24	62.85	72.28	83.81	14.84	17.07	19.79	7,257.60	4032.00	336.00	0.94
3 1/2 x 25 1/2	68.53	78.81	91.37	15.77	18.13	21.02	8,705.22	4836.23	379.31	0.90
3 1/2 x 27	76.39	87.85	101.85	16.70	19.20	22.26	10,333.58	5740.88	425.25	0.90
3 1/2 x 28 1/2	84.65	97.35	112.87	17.62	20.27	23.50	12,153.29	6751.83	473.81	0.89
3 1/2 x 30	93.32	107.32	124.43	18.55	21.33	24.73	14,175.00	7875.00	525.00	0.89
3 1/2 x 31 1/2	102.38	117.74	136.51	19.48	22.40	25.97	16,409.33	9116.30	578.81	0.88
3 1/2 x 33	109.38	125.78	145.84	20.41	23.47	27.21	18,866.93	10481.63	635.25	0.86
3 1/2 x 34 1/2	119.02	136.87	158.69	21.33	24.53	28.44	21,558.40	11976.89	694.31	0.86
3 1/2 x 36	129.04	148.40	172.05	22.26	25.60	29.68	24,494.40	13608.00	756.00	0.85

[a]The preceding table assumes the following values. Refer to 2001 NDS 5.3.2 to determine actual C_V.

$C_V = (21/L)^{1/x} (12/d)^{1/x} (5.125/b)^{1/x} \leq 1.0$

x = 10
$L \leq 20'$ for depths up to 15"
$20' \leq L \leq 30'$ for depths up to 24"
$30' \leq L \leq 40'$ for depths up to 31½"
$40' \leq L \leq 50'$ for depths up to 36"

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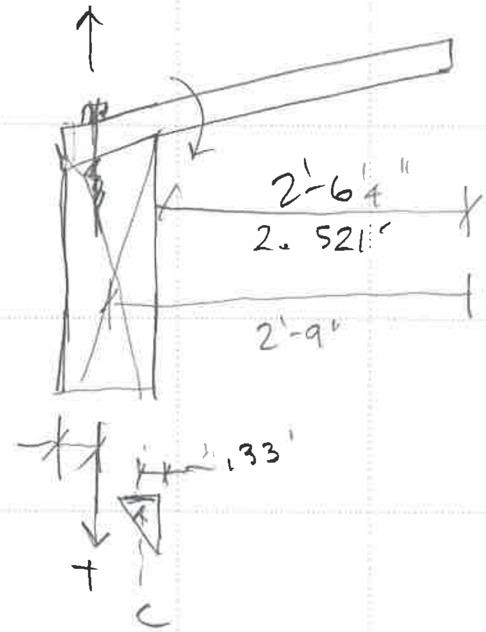
CONNECTION OF
CANTILEVERED T & G DECKING

DL = 5 psf LL = 25 psf (SNOW)



$M = (5+25)\dfrac{(2.521)^2}{2}$

$= 95.3 \text{ lb-ft}$

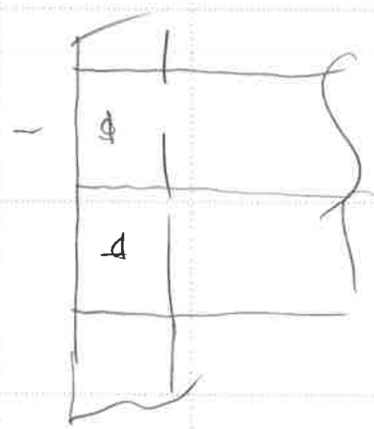
$T = C = \dfrac{(95.3)(12)}{5.5 - 2.33} = 361 \text{ lb/ft}$

USE 1/2" φ x 3" LAG SCREW

@ 5 1/2 o.c. ✓

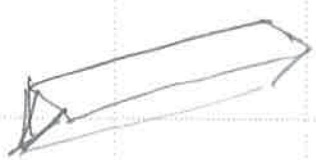
CAPACITY $= 334 \times 1.5 \text{ in} \times \dfrac{12"}{5\frac{1}{2}}$

$= 1000 \text{ lb}$



PLAN

CHECK COMPRESSION IN BEAM



$(x)(1)(12) = 361 \text{ lb}$

$x = 30.1 \text{ psi}$. $\boxed{\text{OK}}$

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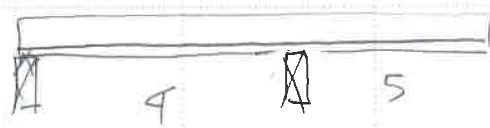
CHECK BENDING OF T/G DECKING

USE: F_b = 1650 PSI
E = 1700 KSI

DEDUCE BENDING CAPACITY FROM DESIGN CHARTS
SEE PAGE 6-4

$M = (.101)(7)^2/8 = .62$ K·'

DL = 5 PSF
LL = 25



$M = (.03)\frac{(5)^2}{2} = .38$ K·' $< .62$ K

$\boxed{\text{2x DECKING OK}}$

TABLE 4

TWO INCH NOMINAL THICKNESS[a]
ALLOWABLE ROOF LOAD LIMITED BY BENDING

Bending Stress, psi	Allowable Uniformly Distributed Total Roof Load[b,c,d,e], psf													
	Simple Span, ft							Controlled Random Layup Span, ft						
	6	7	8	9	10	11	12	6	7	8	9	10	11	12
875	73	54	41	32	26	22	18	61	45	34	27	22	18	15
950	79	58	44	35	28	24	20	66	48	37	29	24	20	16
1000	83	61	47	37	30	25	21	69	51	39	31	25	21	17
1050	88	64	49	39	32	26	22	73	54	41	32	26	22	18
1100	92	67	52	41	33	27	23	76	56	43	34	28	23	19
1150	96	70	54	42	34	28	24	80	59	45	35	29	24	20
1200	100	73	56	44	36	30	25	83	61	47	37	30	25	21
1250	104	76	58	46	38	31	26	87	64	49	39	31	26	22
1300	108	80	61	48	39	32	27	90	66	51	40	32	27	22
1350	112	83	63	50	40	33	28	94	69	53	42	34	28	23
1400	117	86	66	52	42	35	29	97	71	55	43	35	29	24
1450	121	89	68	54	44	36	30	101	74	57	45	36	30	25
1500	125	92	70	56	45	37	31	104	76	58	46	38	31	26
1550	129	95	73	57	46	38	32	108	79	60	48	39	32	27
1600	133	98	75	59	48	40	33	111	82	62	49	40	33	28
1650	138	101	77	61	50	41	34	114	84	64	51	41	34	29
1700	142	104	80	63	51	42	35	118	87	66	52	42	35	30
1750	146	107	82	65	52	43	36	122	89	68	54	44	36	30
1900	158	116	89	70	57	47	40	132	97	74	59	48	39	33
2000	167	122	94	74	60	50	42	139	102	78	62	50	41	35

[a] Based on 1-1/2 in. net thickness. To determine allowable loads for 1-7/16 in. net thickness, multiply tabulated values by 0.918.

[b] To determine allowable uniformly distributed total roof loads for other span conditions, use simple span load values for combination simple span and two-span continuous, and two-span continuous layups; and use controlled random layup road values for cantilevered pieces intermixed layup.

[c] Duration of load, C_D = 1.0 used in this table. For other durations of load, adjust by the appropriate factor.

[d] No increase for size effect has been applied (C_F = 1.00). F_b values have been previously adjusted.

[e] Dry conditions of use.

Table 12.2A Lag Screw Reference Withdrawal Values, W[1]

Tabulated withdrawal design values (W) are in pounds per inch of thread penetration into side grain of wood member. Length of thread penetration in main member shall not include the length of the tapered tip (see 12.2.1.1).

Specific Gravity, G[2]	Lag Screw Diameter, D										
	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
0.73	397	469	538	604	668	789	905	1016	1123	1226	1327
0.71	381	450	516	579	640	757	868	974	1077	1176	1273
0.68	357	422	484	543	600	709	813	913	1009	1103	1193
0.67	349	413	473	531	587	694	796	893	987	1078	1167
0.58	281	332	381	428	473	559	641	719	795	869	940
0.55	260	307	352	395	437	516	592	664	734	802	868
0.51	232	274	314	353	390	461	528	593	656	716	775
0.50	225	266	305	342	378	447	513	576	636	695	752
0.49	218	258	296	332	367	434	498	559	617	674	730
0.47	205	242	278	312	345	408	467	525	580	634	686
0.46	199	235	269	302	334	395	453	508	562	613	664
0.44	186	220	252	283	312	369	423	475	525	574	621
0.43	179	212	243	273	302	357	409	459	508	554	600
0.42	173	205	235	264	291	344	395	443	490	535	579
0.41	167	198	226	254	281	332	381	428	473	516	559
0.40	161	190	218	245	271	320	367	412	455	497	538
0.39	155	183	210	236	261	308	353	397	438	479	518
0.38	149	176	202	227	251	296	340	381	422	461	498
0.37	143	169	194	218	241	285	326	367	405	443	479
0.36	137	163	186	209	231	273	313	352	389	425	460
0.35	132	156	179	200	222	262	300	337	373	407	441
0.31	110	130	149	167	185	218	250	281	311	339	367

1. Tabulated withdrawal design values, W, for lag screw connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Specific gravity, G, shall be determined in accordance with Table 12.3.3A.

adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 1380\ G^{5/2}\ D \qquad (12.2\text{-}3)$$

(b) The nail or spike reference withdrawal design value, W, in lbs/in. of penetration, for a smooth shank stainless steel nail or spike driven into the side grain of a wood member, with the nail or spike axis perpendicular to the wood fibers, shall be determined from Table 12.2D or Equation 12.2-4, within the range of specific gravities, G, and nail or spike diameters, D, given in Table 12.2D. Reference withdrawal design values, W, shall be multiplied by all applicable adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 465\ G^{3/2}\ D \qquad (12.2\text{-}4)$$

(c) For calculation of the fastener reference withdrawal design value in pounds, the unit reference withdrawal design value in lbs/in. of fastener penetration from 12.2.3.1a or 12.2.3.1b shall be multiplied by the length of fastener penetration, p_t, into the wood member.

12.2.3.2 Deformed shank nails

(a) The reference withdrawal design value, in lbs/in. of ring shank penetration, for a Roof Sheathing Ring Shank nail or Post-Frame Ring Shank nail driven in the side grain of the main member, with the nail axis perpendicular to the wood fibers, shall be determined from Table 12.2E or Equation 12.2-5, within the range of specific gravities and nail diameters given in Table 12.2E. Reference withdrawal design values, W, shall be multiplied by all applicable adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 1800\ G^2\ D \qquad (12.2\text{-}5)$$

6-9

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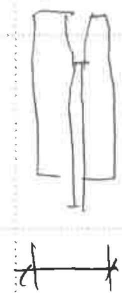
CHECK BOLTED CONNECTION OF BEAM TO STEEL PLATE

BEAM REACTION = 2.2k SEE 6-2



SHEAR CAPACITY OF 1" ϕ BOLT W/ STEEL SIDE PLATE AND 1½" THICK MEMBER

= 480 lb (SEE FOLLOWING PAGE) 6-9

OF BOLTS REQ'D $\frac{2.2}{(2)(.480)} = 2.3$

$\boxed{\text{USE (3) 1" } \phi \text{ BOLTS}}$

Project: Designed By: Date:
Project Number: Client: Scale: Page: 6-8

BOLTS

Table 12B BOLTS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2]

for sawn lumber or SCL main member with 1/4" ASTM A 36 steel side plate

Thickness: Main Member t_m in.	Thickness: Side Member t_s in.	Bolt Diameter D in.	G=0.67 Red Oak		G=0.55 Mixed Maple Southern Pine		G=0.50 Douglas Fir-Larch		G=0.49 Douglas Fir-Larch(N)		G=0.46 Douglas Fir(S) Hem-Fir(N)		G=0.43 Hem-Fir		G=0.42 Spruce-Pine-Fir		G=0.37 Redwood		G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods		G=0.35 Northern Species	
			$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.
1-1/2	1/4	1/2	730	420	620	350	580	310	580	310	550	290	520	280	510	270	470	240	460	240	450	230
		5/8	910	480	780	400	730	360	720	360	690	340	650	320	640	320	590	290	580	280	560	270
		3/4	1090	550	940	450	870	420	860	410	820	390	780	360	770	360	710	320	690	320	680	310
		7/8	1270	600	1090	510	1020	470	1010	450	960	430	910	410	900	400	820	370	810	360	790	350
		1	1460	660	1250	550	1170	510	1150	500	1100	480	1040	450	1030	450	940	400	930	400	900	390
1-3/4	1/4	1/2	810	460	690	370	640	340	630	330	600	310	570	290	560	280	510	250	500	250	490	240
		5/8	1020	520	870	430	800	390	790	380	750	360	710	340	700	330	640	300	630	290	610	280
		3/4	1220	590	1040	480	960	440	950	430	900	410	860	380	840	370	770	330	750	330	730	320
		7/8	1420	650	1210	540	1130	490	1110	480	1050	450	1000	420	980	420	890	380	880	370	850	360
		1	1630	710	1380	580	1290	540	1270	520	1200	500	1140	470	1120	460	1020	410	1000	410	980	400
2-1/2	1/4	1/2	930	600	860	470	830	410	820	400	780	380	740	350	720	340	650	300	640	290	620	280
		5/8	1370	670	1150	530	1050	470	1040	470	980	430	920	400	910	390	810	340	800	330	770	320
		3/4	1640	750	1370	590	1270	530	1250	520	1180	490	1110	450	1090	440	980	380	960	370	930	360
		7/8	1910	820	1600	650	1480	590	1450	570	1370	530	1290	490	1270	480	1140	420	1120	410	1080	400
		1	2190	880	1830	700	1690	640	1660	620	1570	580	1480	540	1450	530	1300	460	1280	450	1240	440
3-1/2	1/4	1/2	930	620	860	550	830	510	820	510	800	480	770	450	770	430	720	370	720	360	710	350
		5/8	1370	860	1260	690	1210	610	1200	600	1160	550	1130	500	1120	490	1060	420	1050	410	1020	400
		3/4	1900	990	1740	760	1670	680	1660	660	1580	610	1480	560	1450	540	1290	460	1260	450	1220	440
		7/8	2530	1070	2170	840	1990	740	1950	710	1840	660	1720	610	1690	590	1510	510	1480	500	1430	470
		1	2980	1150	2480	890	2270	800	2230	770	2100	730	1970	660	1930	650	1720	560	1690	540	1630	530
5-1/4	1/4	5/8	1370	860	1260	760	1210	710	1200	700	1160	670	1130	640	1120	630	1060	580	1050	560	1030	540
		3/4	1900	1140	1740	1000	1670	940	1660	930	1610	860	1560	770	1550	760	1460	640	1450	620	1420	600
		7/8	2530	1460	2320	1190	2220	1050	2200	1010	2140	920	2070	840	2050	820	1940	700	1920	680	1890	640
		1	3260	1660	2980	1270	2860	1130	2840	1080	2750	1010	2670	920	2640	890	2490	750	2450	730	2360	710
5-1/2	1/4	5/8	1370	860	1260	760	1210	710	1200	700	1160	670	1130	640	1120	630	1060	580	1050	570	1030	560
		3/4	1900	1140	1740	1000	1670	940	1660	930	1610	890	1560	810	1550	790	1460	660	1450	640	1420	620
		7/8	2530	1460	2320	1240	2220	1090	2200	1050	2140	960	2070	880	2050	860	1940	730	1920	710	1890	660
		1	3260	1730	2980	1320	2860	1170	2840	1130	2750	1050	2670	950	2640	930	2490	780	2470	760	2420	740
7-1/2	1/4	5/8	1370	860	1260	760	1210	710	1200	700	1160	670	1130	640	1120	630	1060	580	1050	570	1030	560
		3/4	1900	1140	1740	1000	1670	940	1660	930	1610	890	1560	850	1550	840	1460	760	1450	750	1420	740
		7/8	2530	1460	2320	1280	2220	1210	2200	1180	2140	1130	2070	1080	2050	1070	1940	960	1920	930	1890	870
		1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1400	2670	1270	2640	1230	2490	1030	2470	1000	2420	960
9-1/2	1/4	3/4	1900	1140	1740	1000	1670	940	1660	930	1610	890	1560	850	1550	840	1460	760	1450	750	1420	740
		7/8	2530	1460	2320	1280	2220	1210	2200	1180	2140	1130	2070	1080	2050	1070	1940	980	1920	970	1890	930
		1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1420	2670	1350	2640	1330	2490	1220	2470	1200	2420	1180
11-1/2	1/4	7/8	2530	1460	2320	1280	2220	1210	2200	1180	2140	1130	2070	1080	2050	1070	1940	980	1920	970	1890	930
		1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1420	2670	1350	2640	1330	2490	1220	2470	1200	2420	1180
13-1/2	1/4	1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1420	2670	1350	2640	1330	2490	1220	2470	1200	2420	1180

1. Tabulated lateral design values, Z, for bolted connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for "full-body diameter" bolts (see Appendix Table L1) with bolt bending yield strength, F_{yb}, of 45,000 psi and dowel bearing strength, F_e, of 87,000 psi for ASTM A36 steel.

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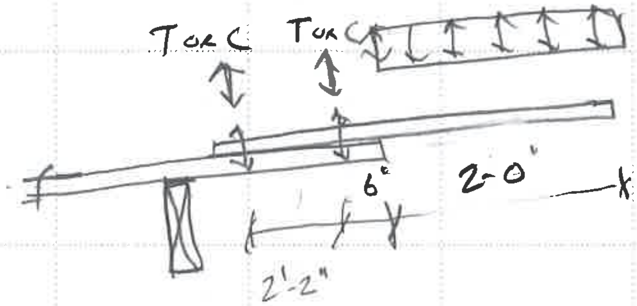
CONNECTION OF GLASS PANEL



GRAVITY: 25 PSF + 2 PSF = 27 PSF = w

NET UPLIFT - 30 PSF ← GOVERNS

SPACING OF BOLTS = 1'-10" O.C (1.833')

$$T_{MAX} = \left[\frac{(30)(2)\left(\frac{2}{2} + 1.583\right)}{2.167}\right](1.833) = 131 \text{ lb}$$

USE 1/4" ϕ LAG SCREW, 1" PENETRATION

CAPACITY = (218)(1.15) = 251 > 131 lb

OK

Table 12.2A Lag Screw Reference Withdrawal Values, W[1]

Tabulated withdrawal design values (W) are in pounds per inch of thread penetration into side grain of wood member. Length of thread penetration in main member shall not include the length of the tapered tip (see 12.2.1.1).

Specific Gravity, G[2]	Lag Screw Diameter, D										
	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
0.73	397	469	538	604	668	789	905	1016	1123	1226	1327
0.71	381	450	516	579	640	757	868	974	1077	1176	1273
0.68	357	422	484	543	600	709	813	913	1009	1103	1193
0.67	349	413	473	531	587	694	796	893	987	1078	1167
0.58	281	332	381	428	473	559	641	719	795	869	940
0.55	260	307	352	395	437	516	592	664	734	802	868
0.51	232	274	314	353	390	461	528	593	656	716	775
0.50	225	266	305	342	378	447	513	576	636	695	752
0.49	218	258	296	332	367	434	498	559	617	674	730
0.47	205	242	278	312	345	408	467	525	580	634	686
0.46	199	235	269	302	334	395	453	508	562	613	664
0.44	186	220	252	283	312	369	423	475	525	574	621
0.43	179	212	243	273	302	357	409	459	508	554	600
0.42	173	205	235	264	291	344	395	443	490	535	579
0.41	167	198	226	254	281	332	381	428	473	516	559
0.40	161	190	218	245	271	320	367	412	455	497	538
0.39	155	183	210	236	261	308	353	397	438	479	518
0.38	149	176	202	227	251	296	340	381	422	461	498
0.37	143	169	194	218	241	285	326	367	405	443	479
0.36	137	163	186	209	231	273	313	352	389	425	460
0.35	132	156	179	200	222	262	300	337	373	407	441
0.31	110	130	149	167	185	218	250	281	311	339	367

1. Tabulated withdrawal design values, W, for lag screw connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Specific gravity, G, shall be determined in accordance with Table 12.3.3A.

adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 1380\ G^{5/2}\ D \qquad (12.2\text{-}3)$$

(b) The nail or spike reference withdrawal design value, W, in lbs/in. of penetration, for a smooth shank stainless steel nail or spike driven into the side grain of a wood member, with the nail or spike axis perpendicular to the wood fibers, shall be determined from Table 12.2D or Equation 12.2-4, within the range of specific gravities, G, and nail or spike diameters, D, given in Table 12.2D. Reference withdrawal design values, W, shall be multiplied by all applicable adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 465\ G^{3/2}\ D \qquad (12.2\text{-}4)$$

(c) For calculation of the fastener reference withdrawal design value in pounds, the unit reference withdrawal design value in lbs/in. of fastener penetration from 12.2.3.1a or 12.2.3.1b shall be multiplied by the length of fastener penetration, p_t, into the wood member.

12.2.3.2 Deformed shank nails

(a) The reference withdrawal design value, in lbs/in. of ring shank penetration, for a Roof Sheathing Ring Shank nail or Post-Frame Ring Shank nail driven in the side grain of the main member, with the nail axis perpendicular to the wood fibers, shall be determined from Table 12.2E or Equation 12.2-5, within the range of specific gravities and nail diameters given in Table 12.2E. Reference withdrawal design values, W, shall be multiplied by all applicable adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 1800\ G^2\ D \qquad (12.2\text{-}5)$$

DOWEL-TYPE FASTENERS

12

6-11

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Wind

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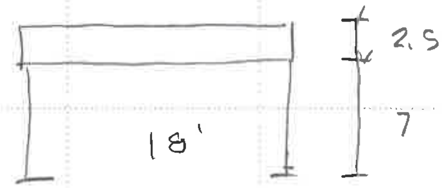
WIND DESIGN (ASCE 7-16, CHAPTER 29)

MODEL/ANALYZE THIS STRUCTURE
AS A SIGN/FENCE/BILLBOARD
WITH THIS PROFILE →



BASIC WIND SPEED, $V = 100$ MPH
(FIGURE 26.5-1B)

DIRECTIONALITY FACTOR, $K_d = 0.85$ TABLE 26.10.1

GROUND ELEVATION FACTOR, $K_e = 1.0$ TABLE 26.9-1

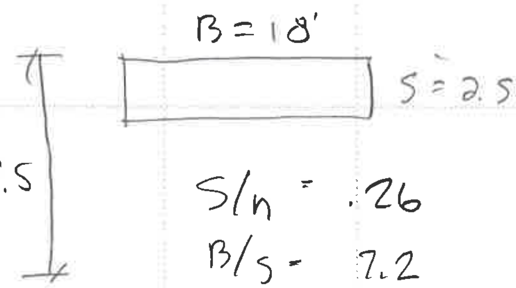
$q_h = .00256\, K_z K_d K_e K_{zt} V^2 = 19.6$ PSF (ULTIMATE)

$F = q_h G C_F A_s$

$= (19.6)(.85)(1.8)(18 \times 2.5)$

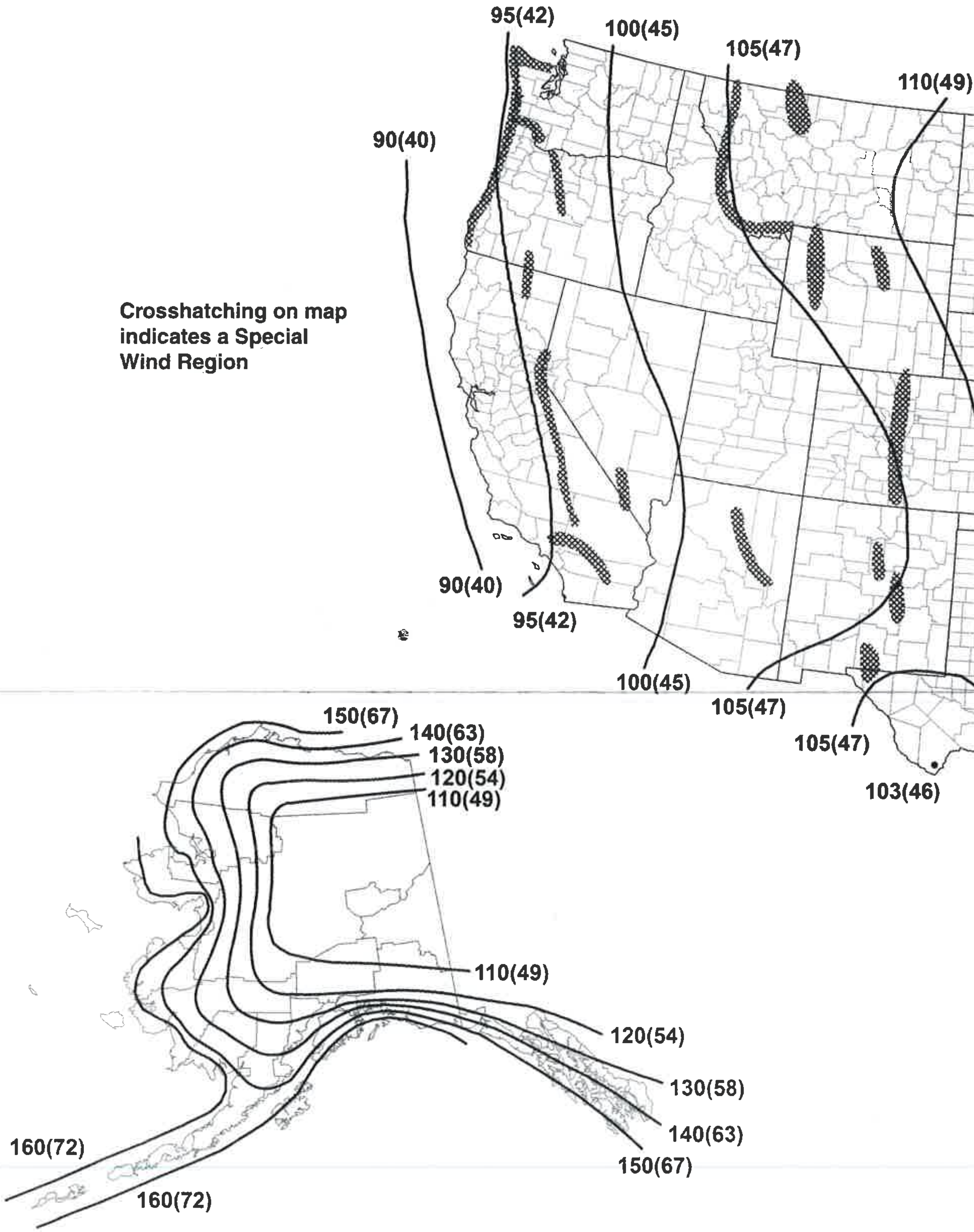
$F = 1.35$ K (ULTIMATE)

$= .81$ K (ALLOWABLE)



$s/h = .26$
$B/s = 7.2$

$C_f = 1.8$ (FIGURE 29.3-1)



Notes

1. Values are nominal design 3-s gust wind speeds in mi/h (m/s) at 33 ft (10 m) above ground for Exposure Category C.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 years).
6. Location-specific basic wind speeds shall be permitted to be determined using www.atcouncil.org/windspeed.

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

W-2

continues

conditions and locations of buildings and other structures meet all of the following conditions:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature ($100H$) or 2 mi (3.22 km), whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mi (3.22-km) radius in any quadrant by a factor of 2 or more.
3. The building or other structure is located as shown in Fig. 26.8-1 in the upper one-half of a hill or ridge or near the crest of an escarpment.
4. $H/L_h \geq 0.2$.
5. H is greater than or equal to 15 ft (4.5 m) for Exposure C and D and 60 ft (18 m) for Exposure B.

26.8.2 Topographic Factor. The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \qquad (26.8\text{-}1)$$

where K_1, K_2, and K_3 are given in Fig. 26.8-1.

If site conditions and locations of buildings and other structures do not meet all the conditions specified in Section 26.8.1, then $K_{zt} = 1.0$.

26.9 GROUND ELEVATION FACTOR

The ground elevation factor to adjust for air density, K_e, shall be determined in accordance with Table 26.9-1. It is permitted to take $K_e = 1$ for all elevations.

26.10 VELOCITY PRESSURE

26.10.1 Velocity Pressure Exposure Coefficient. Based on the exposure category determined in Section 26.7.3, a velocity pressure exposure coefficient, K_z or K_h, as applicable, shall be determined from Table 26.10-1. For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z or K_h, between those shown in Table 26.10-1 are permitted provided that they are determined by a rational analysis method defined in the recognized literature.

Table 26.9-1 Ground Elevation Factor, K_e

Ground Elevation above Sea Level		Ground Elevation Factor
ft	m	K_e
<0	<0	See note 2
0	0	1.00
1,000	305	0.96
2,000	610	0.93
3,000	914	0.90
4,000	1,219	0.86
5,000	1,524	0.83
6,000	1,829	0.80
>6,000	>1,829	See note 2

Notes
1. The conservative approximation $K_e = 1.00$ is permitted in all cases.
2. The factor K_e shall be determined from the above table using interpolation or from the following formula for all elevations:
   $K_e = e^{-0.0000362 z_g}$ (z_g = ground elevation above sea level in ft).
   $K_e = e^{-0.000119 z_g}$ (z_g = ground elevation above sea level in m).
3. K_e is permitted to be take as 1.00 in all cases.

Table 26.10-1 Velocity Pressure Exposure Coefficients, K_h and K_z

Height above Ground Level, z		Exposure		
ft	m	B	C	D
0–15	0–4.6	0.57 (0.70)[a]	0.85	1.03
20	6.1	0.62 (0.70)[a]	0.90	1.08
25	7.6	0.66 (0.70)[a]	0.94	1.12
30	9.1	0.70	0.98	1.16
40	12.2	0.76	1.04	1.22
50	15.2	0.81	1.09	1.27
60	18.0	0.85	1.13	1.31
70	21.3	0.89	1.17	1.34
80	24.4	0.93	1.21	1.38
90	27.4	0.96	1.24	1.40
100	30.5	0.99	1.26	1.43
120	36.6	1.04	1.31	1.48
140	42.7	1.09	1.36	1.52
160	48.8	1.13	1.39	1.55
180	54.9	1.17	1.43	1.58
200	61.0	1.20	1.46	1.61
250	76.2	1.28	1.53	1.68
300	91.4	1.35	1.59	1.73
350	106.7	1.41	1.64	1.78
400	121.9	1.47	1.69	1.82
450	137.2	1.52	1.73	1.86
500	152.4	1.56	1.77	1.89

[a]Use 0.70 in Chapter 28, Exposure B, when $z < 30$ ft (9.1 m).

Notes
1. The velocity pressure exposure coefficient K_z may be determined from the following formula:
   For 15 ft (4.6 m) $\leq z \leq z_g$ $\quad K_z = 2.01(z/z_g)^{2/\alpha}$
   For $z < 15$ ft (4.6 m) $\quad K_z = 2.01(15/z_g)^{2/\alpha}$
2. α and z_g are tabulated in Table 26.11-1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 26.7.

26.10.2 Velocity Pressure. Velocity pressure, q_z, evaluated at height z above ground shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \ (\text{lb/ft}^2);\ V \text{ in mi/h} \qquad (26.10\text{-}1)$$

$$q_z = 0.613 K_z K_{zt} K_d K_e V^2 \ (\text{N/m}^2);\ V \text{ in m/s} \qquad (26.10\text{-}1.\text{si})$$

where

K_z = velocity pressure exposure coefficient, see Section 26.10.1.
K_{zt} = topographic factor, see Section 26.8.2.
K_d = wind directionality factor, see Section 26.6.
K_e = ground elevation factor, see Section 26.9.
V = basic wind speed, see Section 26.5.
q_z = velocity pressure at height z.

The velocity pressure at mean roof height is computed as $q_h = q_z$ evaluated from Eq. (26.10-1) using K_z at mean roof height h.

The basic wind speed, V, used in determination of design wind loads on rooftop structures, rooftop equipment, and other

W-3

conditions and locations of buildings and other structures meet all of the following conditions:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature ($100H$) or 2 mi (3.22 km), whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mi (3.22-km) radius in any quadrant by a factor of 2 or more.
3. The building or other structure is located as shown in Fig. 26.8-1 in the upper one-half of a hill or ridge or near the crest of an escarpment.
4. $H/L_h \geq 0.2$.
5. H is greater than or equal to 15 ft (4.5 m) for Exposure C and D and 60 ft (18 m) for Exposure B.

26.8.2 Topographic Factor. The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \qquad (26.8\text{-}1)$$

where K_1, K_2, and K_3 are given in Fig. 26.8-1.

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Table 26.9-1 Ground Elevation Factor, K_e

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5,000	1,524	0.83
6,000	1,829	0.80
>6,000	>1,829	See note 2

Notes
1. The conservative approximation $K_e = 1.00$ is permitted in all cases.
2. The factor K_e shall be determined from the above table using interpolation or from the following formula for all elevations:
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50	15.2	0.81	1.09	1.27
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80	24.4	0.93	1.21	1.38
90	27.4	0.96	1.24	1.40
100	30.5	0.99	1.26	1.43
120	36.6	1.04	1.31	1.48
140	42.7	1.09	1.36	1.52
160	48.8	1.13	1.39	1.55
180	54.9	1.17	1.43	1.58
200	61.0	1.20	1.46	1.61
250	76.2	1.28	1.53	1.68
300	91.4	1.35	1.59	1.73
350	106.7	1.41	1.64	1.78
400	121.9	1.47	1.69	1.82
450	137.2	1.52	1.73	1.86
500	152.4	1.56	1.77	1.89

[a]Use 0.70 in Chapter 28, Exposure B, when $z < 30$ ft (9.1 m).

Notes
1. The velocity pressure exposure coefficient K_z may be determined from the following formula:
For 15 ft (4.6 m) $\leq z \leq z_g$ $\quad K_z = 2.01(z/z_g)^{2/\alpha}$
For $z <$ 15 ft (4.6 m) $\quad K_z = 2.01(15/z_g)^{2/\alpha}$
2. α and z_g are tabulated in Table 26.11-1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 26.7.

26.10.2 Velocity Pressure. Velocity pressure, q_z, evaluated at height z above ground shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \text{ (lb/ft}^2\text{); } V \text{ in mi/h} \qquad (26.10\text{-}1)$$

$$q_z = 0.613 K_z K_{zt} K_d K_e V^2 \text{ (N/m}^2\text{); } V \text{ in m/s} \qquad (26.10\text{-}1.\text{si})$$

where

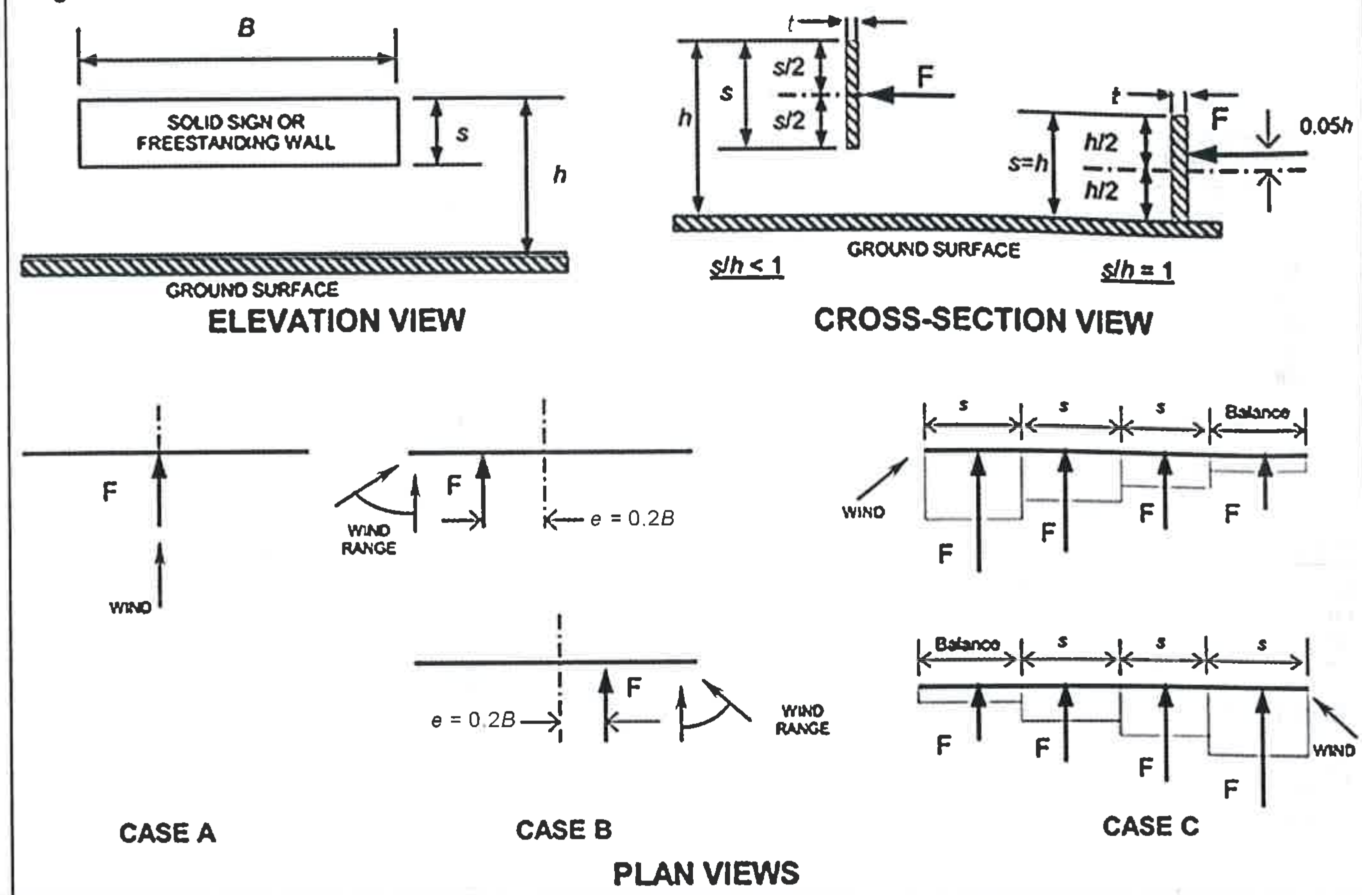
K_z = velocity pressure exposure coefficient, see Section 26.10.1.
K_{zt} = topographic factor, see Section 26.8.2.
K_d = wind directionality factor, see Section 26.6.
K_e = ground elevation factor, see Section 26.9.
V = basic wind speed, see Section 26.5.
q_z = velocity pressure at height z.

The velocity pressure at mean roof height is computed as $q_h = q_z$ evaluated from Eq. (26.10-1) using K_z at mean roof height h.

The basic wind speed, V, used in determination of design wind loads on rooftop structures, rooftop equipment, and other

W-4

Diagrams



Notation

B = Horizontal dimension of sign, in ft (m)
e = Eccentricity of force, in ft (m)
F = Design wind force for other structures, in lb (N)
h = Height of the sign, in ft (m)
L_r = Horizontal dimension of return corner, in ft (m)
R_{min} = t/min (B and s)
R_{max} = t/max (B and s)
s = Vertical dimension of the sign, in ft (m)
t = Thickness of the sign in ft (m)
ε = Ratio of solid area to gross area

Force Coefficients, C_f, for Case A and Case B

Clearance Ratio, s/h	Aspect Ratio, B/s											
	≤0.05	0.1	0.2	0.5	1	2	4	5	10	20	30	≥45
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35	1.35	1.30	1.30	1.30	1.30
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45	1.45	1.40	1.40	1.40	1.40
0.7	1.90	1.85	1.75	1.70	1.65	1.60	1.60	1.55	1.55	1.55	1.55	1.55
0.5	1.95	1.85	1.80	1.75	1.75	1.70	1.70	1.70	1.70	1.70	1.70	1.75
0.3	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.80	1.85	1.85	1.85
0.2	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.85	1.90	1.90	1.95
≤0.16	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95

FIGURE 29.3-1 Design Wind Loads (All Heights): Force Coefficients, C_f, for Other Structures—Solid Freestanding Walls and Solid Freestanding Signs

continues

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Seismic

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Hershkowitz Canopy
21-353-01

Revision Date: 3/31/2021

Criteria

Code:	2018 IBC		
	Allowable Stress Design (ASD)		
Seismic Design:	**ASCE 7-16: 12.8 Equivalent Lateral Force Procedure**		
Wind Design:	**ASCE 7-16: Ch. 28 Envelope Procedure, Low Rise**		
Risk Category:	II - Other Structures		*Table 1.5-1*
Snow Importance Factor	I_S = 1.00	*Table 1.5-2*	
Ice Importance Factor - Thickness	I_i = 1.00	*Table 1.5-2*	
Ice Importance Factor - Wind	I_w = 1.00	*Table 1.5-2*	
Seismic Importance Factor	I_e = 1.00	*Table 1.5-2*	
Spectral Response, Short Period	S_S = 1.462	(Mapped)	
Spectral Response, 1-s Period	S_1 = 0.507	(Mapped)	
Site Class assumed, no Geotechnical Report			
Site Class:	D		*Table 20.3-1*
Site Coefficient	F_a = 1.20	*Table 11.4-1*	
Site Coefficient	F_v = 1.79	*Table 11.4-2*	

Structural Systems:

non building structure - inverted pendulum

All other structural systems	T_L = 6	*(Figs. 22-14 thru 22-17)*
Response Modification Coefficient	R = 2	*Table 12.2-1*
Overstrength Factor	Ω_0 = 2	*Table 12.2-1*
Deflection Amplification Factor	C_d = 2	*Table 12.2-1*

Basic Wind Speed:	100 mph	
Exposure to Wind:	Exposure B	*Section 26.7.3*
Topographical Factor	K_{ZT} = 1.00	

ATC Hazards by Location

Search Information

Address:	6104 84th Ave SE, Mercer Island, WA 98040, USA
Coordinates:	47.5484196, -122.2267079
Elevation:	287 ft
Timestamp:	2021-03-31T21:30:45.025Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

Basic Parameters

Name	Value	Description
S_S	1.462	MCE_R ground motion (period=0.2s)
S_1	0.507	MCE_R ground motion (period=1.0s)
S_{MS}	1.754	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.17	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.898	Coefficient of risk (1.0s)
PGA	0.626	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.751	Site modified peak ground acceleration

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T_L	6	Long-period transition period (s)
SsRT	1.462	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.621	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	4.256	Factored deterministic acceleration value (0.2s)
S1RT	0.507	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.564	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.643	Factored deterministic acceleration value (1.0s)
PGAd	1.421	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

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Table 15.4-2 Seismic Coefficients for Nonbuilding Structures Not Similar to Buildings

Nonbuilding Structure Type	Detailing Requirements[c]	R	Ω_0	C_d	Structural System and Structural Height, h_n, Limits (ft)[a,b]				
					Seismic Design Category				
					B	C	D	E	F
Elevated tanks, vessels, bins, or hoppers:									
On symmetrically braced legs (not similar to buildings)	Sec. 15.7.10	3	2[d]	2.5	NL	NL	160	100	100
On unbraced legs or asymmetrically braced legs (not similar to buildings)	Sec. 15.7.10	2	2[d]	2.5	NL	NL	100	60	60
Horizontal, saddle-supported welded steel vessels	Sec. 15.7.14	3	2[d]	2.5	NL	NL	NL	NL	NL
Flat-bottom ground-supported tanks:	Sec. 15.7								
Steel or fiber-reinforced plastic:									
Mechanically anchored		3	2[d]	2.5	NL	NL	NL	NL	NL
Self-anchored		2.5	2[d]	2	NL	NL	NL	NL	NL
Reinforced or prestressed concrete:									
Reinforced nonsliding base		2	2[d]	2	NL	NL	NL	NL	NL
Anchored flexible base		3.25	2[d]	2	NL	NL	NL	NL	NL
Unanchored and unconstrained flexible base		1.5	1.5[d]	1.5	NL	NL	NL	NL	NL
All other		1.5	1.5[d]	1.5	NL	NL	NL	NL	NL
Cast-in-place concrete silos that have walls continuous to the foundation	Sec. 15.6.2	3	1.75	3	NL	NL	NL	NL	NL
All other reinforced masonry structures not similar to buildings detailed as intermediate reinforced masonry shear walls	Sec. 14.4.1[e]	3	2	2.5	NL	NL	50	50	50
All other reinforced masonry structures not similar to buildings detailed as ordinary reinforced masonry shear walls	Sec. 14.4.1	2	2.5	1.75	NL	160	NP	NP	NP
All other nonreinforced masonry structures not similar to buildings	Sec. 14.4.1	1.25	2	1.5	NL	NP	NP	NP	NP
Concrete chimneys and stacks	Sec. 15.6.2 and ACI 307	2	1.5	2.0	NL	NL	NL	NL	NL
Steel chimneys and stacks	15.6.2 and ASME STS-1	2	2	2	NL	NL	NL	NL	NL
All steel and reinforced concrete distributed mass cantilever structures not otherwise covered herein, including stacks, chimneys, silos, skirt-supported vertical vessels; single-pedestal or skirt-supported	Sec. 15.6.2								
Welded steel	Sec. 15.7.10	2	2[d]	2	NL	NL	NL	NL	NL
Welded steel with special detailing[f]	Secs. 15.7.10 and 15.7.10.5 a and b	3	2[d]	2	NL	NL	NL	NL	NL
Prestressed or reinforced concrete	Sec. 15.7.10	2	2[d]	2	NL	NL	NL	NL	NL
Prestressed or reinforced concrete with special detailing	Secs. 15.7.10 and ACI 318, Chapter 18, Secs. 18.2 and 18.10	3	2[d]	2	NL	NL	NL	NL	NL
Trussed towers (freestanding or guyed), guyed stacks, and chimneys	Sec. 15.6.2	3	2	2.5	NL	NL	NL	NL	NL
Steel tubular support structures for onshore wind turbine generator systems	Sec. 15.6.7	1.5	1.5	1.5	NL	NL	NL	NL	NL
Cooling towers:									
Concrete or steel		3.5	1.75	3	NL	NL	NL	NL	NL
Wood frames		3.5	3	3	NL	NL	NL	50	50
Telecommunication towers:	Sec. 15.6.6								
Truss: Steel		3	1.5	3	NL	NL	NL	NL	NL
Pole: Steel		1.5	1.5	1.5	NL	NL	NL	NL	NL
Wood		1.5	1.5	1.5	NL	NL	NL	NL	NL
Concrete		1.5	1.5	1.5	NL	NL	NL	NL	NL
Frame: Steel		3	1.5	1.5	NL	NL	NL	NL	NL
Wood		1.5	1.5	1.5	NL	NL	NL	NL	NL
Concrete		2	1.5	1.5	NL	NL	NL	NL	NL
Amusement structures and monuments	Sec. 15.6.3	2	2	2	NL	NL	NL	NL	NL
Inverted pendulum type structures (except elevated tanks, vessels, bins, and hoppers)	Sec. 12.2.5.3	2	2	2	NL	NL	NL	NL	NL
Ground-supported cantilever walls or fences	Sec. 15.6.8	1.25	2	2.5	NL	NL	NL	NL	NL
Signs and billboards		3.0	1.75	3	NL	NL	NL	NL	NL
All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are not similar to buildings		1.25	2	2.5	NL	NL	50	50	50

[a]NL = no limit and NP = not permitted.
[b]For the purpose of height limit determination, the height of the structure shall be taken as the height to the top of the structural frame making up the primary seismic force-resisting system.
[c]If a section is not indicated in the detailing requirements column, no specific detailing requirements apply.
[d]See Section 15.7.3.a for the application of the overstrength factors, Ω_0, for tanks and vessels.
[e]Detailed with an essentially complete vertical load-carrying frame.
[f]Sections 15.7.10.5.a and 15.7.10.5.b shall be applied for any risk category.

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Redundancy, ρ 1.0 *(Section 12.3.4)*

Design Base Shear

$S_{MS} = F_a S_S$ *(Eq. 11.4-1)*
$= 1.75$

$S_{DS} = ⅔ S_{MS}$ *(Eq. 11.4-3)*
$= 1.17$

$S_{M1} = F_v S_1$ *(Eq. 11.4-2)*
$= 0.91$

$S_{D1} = ⅔ S_{M1}$ *(Eq. 11.4-4)*
$= 0.61$

Seismic Design Category:

Short Period -- D

1-Second Period -- D

Structure Period and Weight:

$C_t = 0.020$ *Table 12.8-2*

$x = 0.75$

Building Height (Mean Roof), $h_n = 22$ ft

Approximate Fundamental Period, $T_a = C_t (h_n)^x$ *(Eq. 12.8-7)*

$T = T_a = 0.20$

$T_L = 6$ *(Figs. 22-14 thru 22-17)*

Calculated design base shear:

$V = C_s W$ (Eq. 12.8-1)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$ (Eq. 12.8-2)

$C_s = 0.58$

The total design base shear need not exceed:

(Eq. 12.8-3)

for T O T_L $$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

$C_s = 1.49$

(Eq. 12.8-4)

for $T > T_L$ $$C_s = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)}$$

$C_s = 44.05$

$C_s = 1.49$ *$T \leq TL$*

$C_s = 2.24$ *1.5 times Cs in accordance with 11.4.8*

The total design base shear shall not be less than:

$C_s = 0.044 S_{DS} I_e \geq 0.01$ (Eq. 12.8-5)

$C_s = 0.05$

nor where $S_1 \geq 0.6g$:

$C_s = 0.5 S_1/(R/I_e)$ (Eq. 12.8-6)

$C_s = 0.00$

$C_s = 0.58$

$V = 0.58\ W$

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SEISMIC MASS

T/G

(3.33 + 6.5)(18)(4.0 PSF) = 708 lb

TRANSLUCENT PANEL

(2.75)(18)(1.5) = 75 lb

BEAMS

3½ x 15 GLB x 18' x 3 x 12.8 PLF = 252 lb

WEIGHT OF COLUMN

13 x 10 x 4 = 520 lb

SEE 6-1

Σ = 1555 lb

V = .58 W = .902 K ULTIMATE

(.7)(.902) = .631 K ALLOWABLE

Project: ______ Designed By: ______ Date: ______

Project Number: ______ Client: ______ Scale: ______ Page: E-6

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Lateral Analysis

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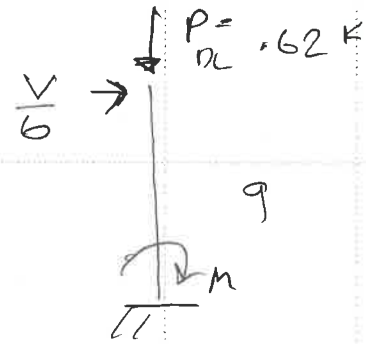
CHECK BENDING IN COLUMN



$V_E = .63^K$ (SEE E-6

$V_W = .81^K$ (SEE W-1)

GOVERNS

$M = (.81/6)(9) = 1.215$ K-'

$f_b = M/S = \frac{(1.215)(12)}{1.48} = 9.85$ KSI OK

SEE G-1

DEFLECTION - $\frac{(.81/6)(9)^3(1728)}{(3)(29{,}000)(2.7)} = .72^{IN}$

SEE G-1

$C_d\Delta = (2)(.72) = 1.44'' < .025h_x = 2.7$ IN

$\boxed{\text{DEFLECTION OK}}$

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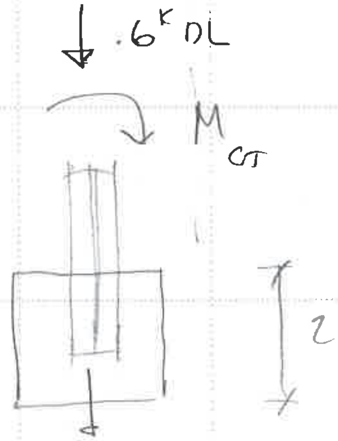
SIZE FOOTING

$M_{OT} = (.81/6)(9-2) = 1.48 \text{ k'}$



$W_F = (2.5 \times 2.5 \times 2) \times .15$
$= 1.87$

$M_R = (.6 + 1.88)(2.5/2) = 3.1 \text{ k'}$

REACTION POINT $= \dfrac{3.1 - 1.48}{2.48} = .653$

SOIL PRESSURE $= \dfrac{(2)(2.48)}{(3)(.653)(2.5)} = 1012$ PSF

USE 30" × 30" × 18"
W/ (3) #5 EA. WAY

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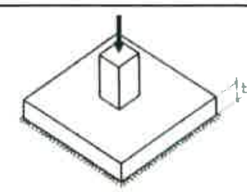
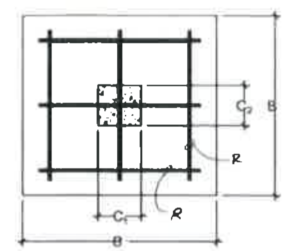
Project: Typical Footing
Footing: 30" x 30" x 8" thick



Isolated footing



Footing B = 2.50 ft
t = 8 in

Reinforcement R = (3) #4
A_{s1} = 0.60 in²
d = 4.25 in Cover: 3 in

Column C_1 = 3.50 in C_2 = 3.50 in

Materials f'_c = 2500 psi Normalweight λ = 1.00
f_y = 40000 psi Uncoated ψ_e = 1.00

Net Footing Weight
P_{FTG} = 0.17 k

Soil Pressure:
$P_{ASD} = q_a B^2 - P_{FTG} =$

One-way shear: ϕ = 0.75
$V_c = 2\lambda\sqrt{f'_c}Bd =$ 12.75 k
$V_u \leq \phi V_c$ $\phi V_c =$ 9.56 k

$$V_u = q_u B\left(\frac{B-C_2}{2}-d\right) \rightarrow q_u = \frac{\phi V_c}{B\left(\frac{B-C_2}{2}-d\right)}$$
$$V_u = q_u B\left(\frac{B-C_1}{2}-d\right) \rightarrow q_u = \frac{\phi V_c}{B\left(\frac{B-C_1}{2}-d\right)}$$

$q_u =$ 3974 psf or 3974 psf $P_u = q_u B^2 =$ 24838 #

Two-way shear: ϕ = 0.75

[22.6.5.2(a)] $v_c = 4\lambda\sqrt{f'_c} =$ 200 psi ⇦

[22.6.5.2(b)] $v_c = \left(2+\frac{4}{\beta}\right)\lambda\sqrt{f'_c} =$ 300 psi

[22.6.5.2(c)] $v_c = \left(2+\frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f'_c} =$ 374 psi

β = 1.00
α_x = 40
b_0 = 2(C1+d)+2(C2+d)
31

$V_u \leq \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 19.76 k

$$V_u = q_u[B^2-(C_1+d)(C_2+d)] \rightarrow q_u = \frac{\phi V_c}{[B^2-(C_1+d)(C_2+d)]}$$

$q_u =$ 3388 psf $P_u = q_u B^2 =$ 21176 #

Moment: ϕ = 0.90
$M_n = A_s f_y(d - {}^{a}/_{2}) =$ 8.1 k-ft
$a = A_s f_y/(0.85 f'_c B) =$ 0.38 in
$M_u \leq \phi M_n$ $\phi M_n =$ 7.3 k-ft

$$M_u = \frac{q_u B\left(\frac{B-C_2}{2}\right)^2}{2} \rightarrow q_u = \frac{2\phi M_n}{B\left((B-C_2)/2\right)^2}$$
$$M_u = \frac{q_u B\left(\frac{B-C_1}{2}\right)^2}{2} \rightarrow q_u = \frac{2\phi M_n}{B\left((B-C_1)/2\right)^2}$$

$q_u =$ 4797 psf or 4797 psf $P_u = q_u B^2 =$ 29984 #

Development of Reinforcement:

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b =$$ 10 in ...10 in available OK

Soil Bearing Pressure	1500 psf	2000 psf	2500 psf	3000 psf	3500 psf	4000 psf
Max Load (lbs), Soil	9208	12333	15458	18583	21708	24833
Max Load (lbs), One-Way Shear	15524	15524	15524	15524	15524	15524
Max Load (lbs), Two-Way Shear	13235	13235	13235	13235	13235	13235
Max Load (lbs), Moment	18740	18740	18740	18740	18740	18740
Max Load (ASD)	**9208**	**12333**	**13235**	**13235**	**13235**	**13235**
Max Load (Factored)	**14733**	**19733**	**21176**	**21176**	**21176**	**21176**

Date: 3/19/2018