STUTZKI ENGINEERING INC. P.C.
241 N Broadway, Suite 302
Milwaukee, Wisconsin 53202
USA
PHONE: (414) 455-4815
MAIL: info@stutzkiengineering.com
WEB: www.stutzkiengineering.com

Preliminary Design Report

For

Glass Lantern

At Renwick Lighthouse

Lighthouse Park, Roosevelt Island, NY

Orig. Issue: April 27, 2020

Prepared for: Thomas A. Fenniman, Architect

Project No. 2019-174

By: MMH, Chk: BJJ

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1. Project Description

The Renwick Lighthouse is located on the northern most tip of Roosevelt Island in New York. The lantern portion of the renovation project will feature an all glass, point-supported enclosure with structural steel framing underneath. The lantern is located at the top of the lighthouse at an elevation of 43'-6" above grade. The lantern in plan is arranged in an octagon and features three main sections: the lower drum, cone, and upper drum. The lower drum features vertical glass panels approximately 7'-0" x 3'-2" in elevation. The cone features sloping glass panels at 70 degrees from the horizontal and are approximately 8'-9' x 3'-5" at the base and 1'-3" at the top. The upper drum features vertical glass panels approximately 2'-6" x 1'-3" in elevation. The support structure underneath the glass consists of a moment frame structure of various steel sections. The Stutzki engineering scope includes recommendations for the sketches and engineering of the structural framing, glass thickness, and glass support options. Anchorage of the steel structure into the main lighthouse structure is also included in the scope for Stutzki. Not included in the engineering scope is thermal analysis of the structure, structural review of building concrete or steel structural systems, review of foundations, door engineering, etc. See Stutzki-Fenniman contract for complete list of exclusions.

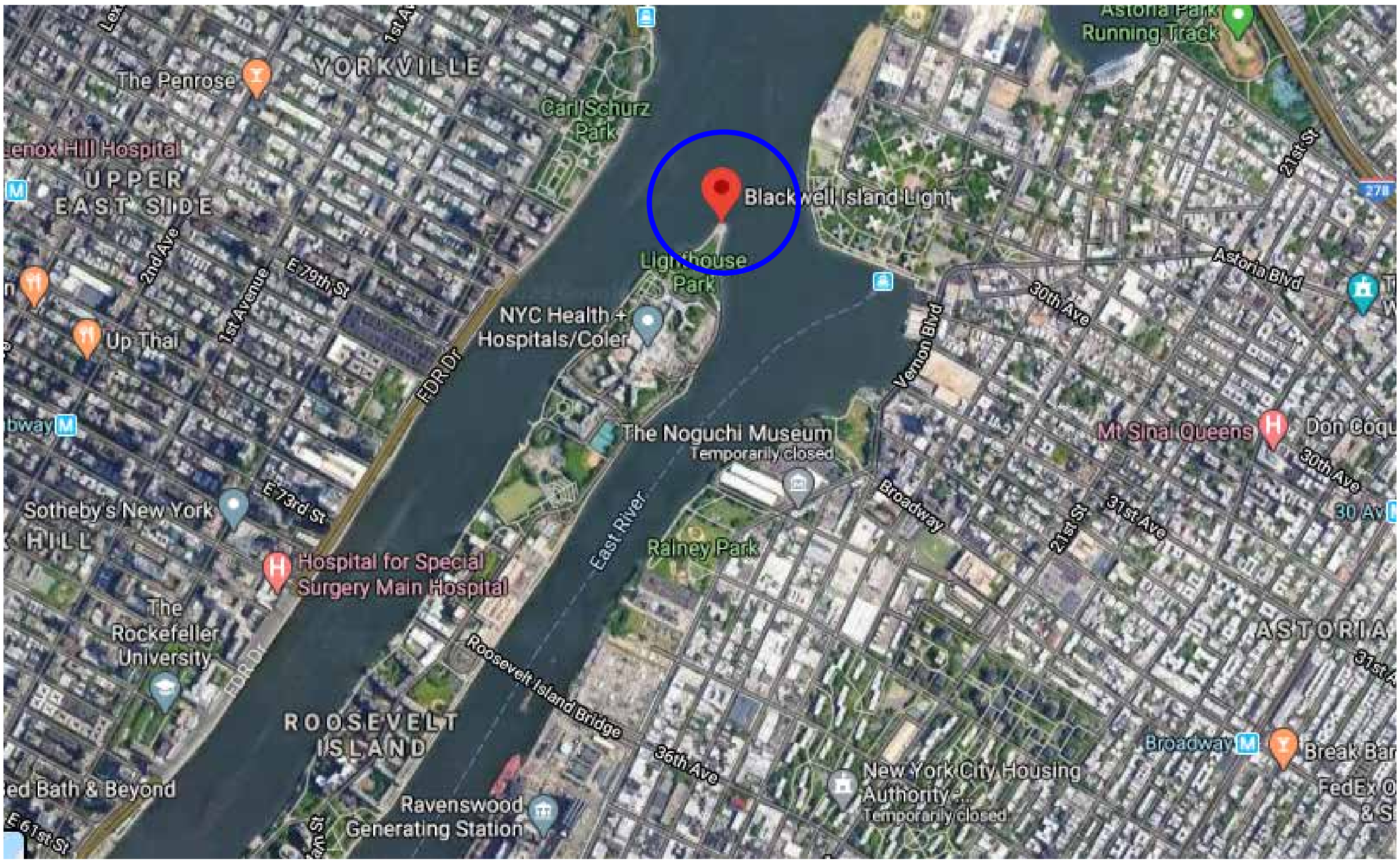


Figure 1: Location of Lighthouse on Roosevelt Island

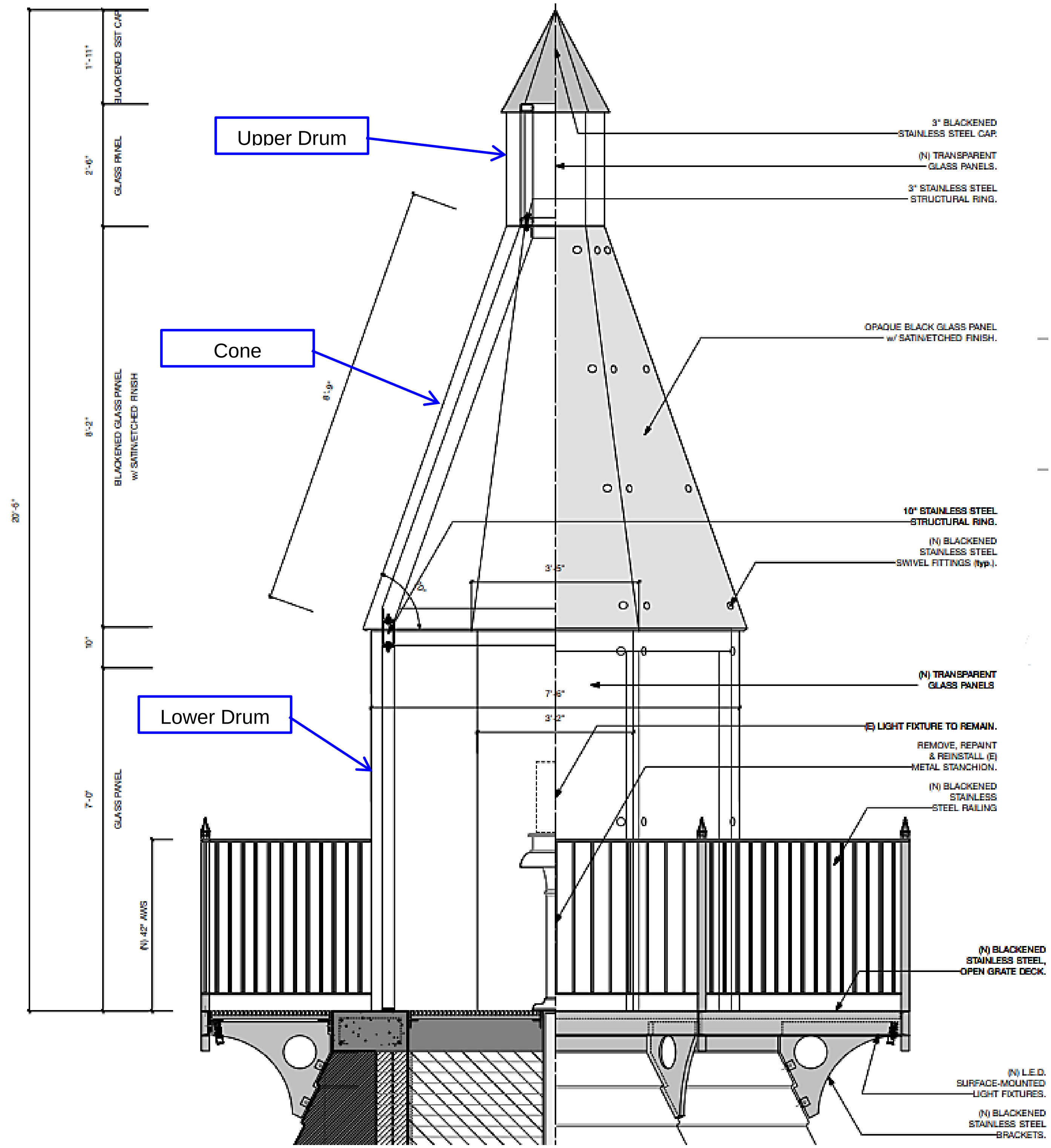


Figure 2: Lantern Partial Plan and Elevation

2. Codes, Standards, and References

1) 2014 New York City Building Code
2) ASCE 7-05 Minimum Design Loads for Buildings and Other Structures (for Snow and Wind)
3) ASCE 7-10 Minimum Design Loads for Buildings and Other Structures (for Seismic)
4) ACI 318-11
5) AISC 360-10 Specification for Structural Steel Buildings and AISC 14th Ed.
6) ASTM E1300-16 Standard Practice for Determining Load Resistance of Glass in Buildings
7) ASTM C1048 Standard Specification for Heat Treated Flat Glass
8) AAMA TIR A9-91 Metal Curtain Wall Fasteners
9) AAMA CW-12-84 Structural Properties of Glass, 2006
10) Architectural drawings, dated 23-Feb-2020, 90% CD set

3. Design Loads

3.1. Dead Load

1. Glass Dead Load = 13.2 psf/in. of glass thickness
2. Steel Dead Load = 490 pcf

3.2. Roof Live Load

Use **20psf** on projected area of sloped panels.

3.3. Snow Load

Not applicable. Due to steep roof angle (70 degrees), snow will not build up on roof structure.

Snow Loads

Design According to NYC Building Code, based on ASCE7-05 Chapter 7

Ground Snow Load	$p_g = 25 \cdot psf$	(Section 1608.2)	
Building Category	Cat := "II"	(Table 1604.5)	
Exposure Factor	$C_e := 1.0$	(Table 1608.3.1)	
Thermal Factor	$C_t := 1.2$	(Table 1608.3.2)	
Snow Importance Factor	$I_s = 1$	(Table 1604.5.2)	
Roof Slope Factor	$C_s := 0$	(Figure 7-2)	
Windward Drift Length	$l_{uw} := 11ft$		
Leeward Drift Length	$l_{ul} := 0ft$		
Parapit Height	$PAR := 0ft$		
Flat Roof Snow Load	$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 21 \cdot psf$		(Eq. 7.3-1)
Minimum Flat Roof Snow Load	$p_{fmin} := \text{if}\left(p_g \le 20psf, I_s \cdot p_g, 20psf \cdot I_s\right) = 20 \cdot psf$		(Sect. 7.3.4)

(pf for Low-Sloped Roofs, See ASCE Section 7.3.4 to see where minimum values on apply)

Rain-On-Snow Surcharge Load	$ROS := \text{if}\left(p_g \le 20psf, 5psf, 0psf\right) = 0 \cdot psf$		(Sect. 7.10)

(ROS for Low-Sloped Roofs, See ASCE Section 7.10 to see where Rain-On-Snow Surchage Loads apply)

Sloped Roof Snow Load	$p_s := C_s \cdot p_f = 0\,psf$		(Sect. 7.4)

3.4. Wind Loads

3.4.1. Check as other structure

Due to the irregular shape of the lantern, ASCE 7-05 provisions for design of wind load for other structures was used.

Wind Loading

Design According to NYC Building Code, based on ASCE7-05 for Section 6.5.15 Design Wind Loads on Other Structures

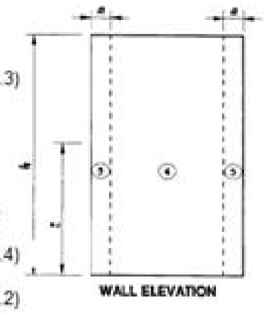
Basic Wind Speed	$V = 98 \cdot mph$	(Section 1609.3)
Mean Roof Height	$h := 64ft$	
Height of Component	$z := 50ft$	
Diameter	$D := 7.5ft$	
Roof angle	$\theta := 70 \cdot deg$	
Building Category	Cat := "II"	(Table 1604.5)
Exposure	Exp := "C"	(Section 1609.4)
Enclosure Classification	Enc := "Enclosed"	(Section 1609.2)
Least Horizontal Dimension	$L := 14ft + 5in$	
Wind Importance Factor	$I_w = 1$	(Table 1604.5.2)
Wind Directionality Factor	$K_d := 0.95$	(Section 6.5.4.4 & Table 6-4)
Velocity Pressure Coefficient	$K_h = 1.152$	(Section 6.5.6.6 & Table 6-3)
	$K_z = 1.094$	
Topographical Factor	$K_{zt} := 1.0$	(Section 6.5.7.2)
Velocity Pressure	$q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I_w = 26.9 \cdot psf$	(Eq. 6-15)
	$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I_w = 25.5 \cdot psf$	
Gust-Effect Factor	$G := 0.85$	(Section 6.5.8)
Force Coefficients	$C_f = 1.251$	(Figure 6-21)
Projected Area Normal to Wind	$A_f := 160 \cdot ft^2$	
Design Wind Force	$F := q_z \cdot G \cdot C_f \cdot A_f \cdot lbf = 4346.89 \cdot lbf$	(Eq. 6-28)
Design Wind Pressure	$P := q_z \cdot G \cdot C_f = 27.168 \cdot psf$	

3.4.2. Check as Components and Cladding

Note: Due to cylindrical shape of light house enclosure, zone 5 wind loads do not occur.

Wind Loading for Components and Cladding

Design According to NYC Building Code, based on ASCE7-05 for Walls w/ h> 60ft



Basic Wind Speed	$V = 98 \cdot mph$	(Section 1609.3)
Mean Roof Height	$h := 64ft$	
Height of Component Above Ground	$z := 50ft$	
Building Category	Cat := "II"	(Table 1604.5)
Exposure	Exp := "C"	(Section 1609.4)
Enclosure Classification	Enc := "Enclosed"	(Section 1609.2)
Least Horizontal Dimension	$L := 14ft + 5in$	
Wind Importance Factor	$I_w = 1$	(Table 1604.5.2)
Wind Directionality Factor	$K_d := 0.95$	(Section 6.5.4.4 & Table 6-4)
Velocity Pressure Coefficient @ h	$K_h = 1.152$	(Section 6.5.6.6 & Table 6-3)
Velocity Pressure Coefficient @ z	$K_z = 1.094$	(Section 6.5.6.6 & Table 6-3)
Topographical Factor	$K_{zt} := 1.0$	(Section 6.5.7.1)
Velocity Pressure at Height, h	$q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I_w = 26.9 \cdot psf$	(Eq. 6-15)
Velocity Pressure at Height, z	$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I_w = 25.5 \cdot psf$	(Eq. 6-15)
Internal Pressure Coefficient	$GC_{pi} = 0.18$	(Figure 6-5)
External Pressure Coefficient, GC_p	Varies based on zone & eff. wind area	(Figure 6-17)
Wind Pressure, P	$P = q\,(GC_p) - q_i\,(GC_{pi})$	(Eq. 6-23)

Note: Wind Pressure Varies by Zone and Effective Wind Area. See Chart Below.

Effective Wind Area	+ Zone 4 & 5	- Zone 4	- Zone 5
20 sq. ft	$GCp_{posZ45_{20}} = 0.9$	$GCp_{negZ4_{20}} = -0.9$	$GCp_{negZ5_{20}} = -1.8$
50 sq. ft	$GCp_{posZ45_{50}} = 0.815$	$GCp_{negZ4_{50}} = -0.843$	$GCp_{negZ5_{50}} = -1.572$
100 sq. ft	$GCp_{posZ45_{100}} = 0.75$	$GCp_{negZ4_{100}} = -0.8$	$GCp_{negZ5_{100}} = -1.4$
200 sq. ft	$GCp_{posZ45_{200}} = 0.685$	$GCp_{negZ4_{200}} = -0.757$	$GCp_{negZ5_{200}} = -1.228$
300 sq. ft	$GCp_{posZ45_{300}} = 0.648$	$GCp_{negZ4_{300}} = -0.732$	$GCp_{negZ5_{300}} = -1.127$
500 sq. ft	$GCp_{posZ45_{500}} = 0.6$	$GCp_{negZ4_{500}} = -0.7$	$GCp_{negZ5_{500}} = -1$

Effective Wind Area	+ Zone 4 & 5	- Zone 4	- Zone 5
20 sq. ft	$P_{posZ45_{20}} = 27.8 \cdot psf$	$P_{negZ4_{20}} = -29.1 \cdot psf$	$P_{negZ5_{20}} = -53.3 \cdot psf$
50 sq. ft	$P_{posZ45_{50}} = 25.7 \cdot psf$	$P_{negZ4_{50}} = -27.5 \cdot psf$	$P_{negZ5_{50}} = -47.2 \cdot psf$
100 sq. ft	$P_{posZ45_{100}} = 24 \cdot psf$	$P_{negZ4_{100}} = -26.4 \cdot psf$	$P_{negZ5_{100}} = -42.5 \cdot psf$
200 sq. ft	$P_{posZ45_{200}} = 22.4 \cdot psf$	$P_{negZ4_{200}} = -25.2 \cdot psf$	$P_{negZ5_{200}} = -37.9 \cdot psf$
300 sq. ft	$P_{posZ45_{300}} = 21.4 \cdot psf$	$P_{negZ4_{300}} = -24.5 \cdot psf$	$P_{negZ5_{300}} = -35.2 \cdot psf$
500 sq. ft	$P_{posZ45_{500}} = 20.2 \cdot psf$	$P_{negZ4_{500}} = -23.7 \cdot psf$	$P_{negZ5_{500}} = -31.8 \cdot psf$

Corner Zone Dimensions, a $\quad a := \max(0.10 \cdot L, 3ft) = 3 \cdot ft$

since both ways of calculating the wind load yield similar results, use
+/-30psf ASD wind loading.

3.5. Earthquake Load

Design According to NYC Building Code, based on ASCE7-10 Chapter 13

MCE Spectral Response Acceleration at Short Period	$S_s = 0.281 \cdot g$	(Section 1613.5.1)
MCE Spectral Response Acceleration at 1s Period	$S_1 = 0.073 \cdot g$	(Section 1613.5.1)
Building Category	Cat := II	(Table 1604.5)
Site Class	Site := D	
Component Importance Factor	$I_p := 1.0$	(Table 1604.5.2)
Site Coefficient	$F_a = 1.58$	(Table 1613.5.3(1))
Site Coefficient	$F_v = 2.40$	(Table 1613.5.3(2))
Adjusted MCE Spect. Resp. Acc. for Short Periods	$S_{MS} := F_a \cdot S_s = 0.443 \cdot g$	(Eq. 16-47)
Adjusted MCE Spect. Resp. Acc. for Long Periods	$S_{M1} := F_v \cdot S_1 = 0.175 \cdot g$	(Eq. 16-48)
Design Spect. Resp. Acc. for Short Periods	$S_{DS} := \frac{2}{3} \cdot S_{MS} = 0.295 \cdot g$	(Eq. 16-49)
Design Spect. Resp. Acc. for Long Periods	$S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.117 \cdot g$	(Eq. 16-50)

Seismic Design Category based on Tables 1613.5.6(1), 1613.5.6(2) SDC = "B"

Note = "Earthqauke loads are exempt for Architectural Componets in SDC B and Ip=1 per ASCE 13.1.4"

3.6. Temperature Load

Temperature variation determined by Expansion Joints in Buildings: Technical Report no. 65. The following are excerpts from this document:

A. GENERAL

1. The structural analysis of a building should include a determination of the need for thermal expansion joints in view of the potential impact of temperature-produced dimensional changes on structural integrity and building serviceability.*

2. As a minimum, each of the following factors should be examined and taken into account during expansion joint location and design:

   a. Dimensions and configuration of the building.

   b. Design temperature change, which should be computed in accordance with the formula:

$$\Delta t = (T_w - T_m) \text{ or } (T_m - T_c), \qquad (1)$$

whichever is greater, where,

T_m = the mean temperature during the normal construction season in the locality of the building. For the purpose of this report, the normal construction season for a locality is defined as that contiguous period in a year during which the minimum daily temperature equals or exceeds 32 °F. [For example, the normal construction season for Anchorage, Alaska, is 5-1/2 months (April 24-October 8) and for Birmingham, Alabama, is year-round (January-December).]

T_w = the temperature exceeded, on the average, only 1 percent of the time during the summer months of June through September in the locality of the building. (In a normal summer there would be approximately 30 hours at or above this design value.)

T_c = the temperature equaled or exceeded, on the average, 99 percent of the time during the winter months of December, January, and February in the locality of the building. (In a normal winter there would be approximately 22 hours at or below this design value.)

Station	Temperature (°F)		
	T_w	T_m	T_c
New York			
Albany	91	61	-5
Binghampton (CO)	91	67	-2
Buffalo	88	59	3
New York	94	59	11

$\Delta T = (T_W - T_M)$ or $(T_M - T_C)$ = 48ºF, use T=+/- 50 ºF

3.7. Load cases and combinations

Load Cases		
no.	**Description**	**Symbol**
1	Dead Load	D
2	Roof Live Load	Lr
3	Wind in X	W+x
4	Wind at 45 degrees	W 45 deg
5	Wind in Y	W+y
6	Temperature increase	T

Load combo							
no.	LRFD/ASD	D	Lr	W+x	W 45 deg	W+y	T
1	ASD	1.00					
2	ASD	1.00	1.00				
3	ASD	1.00					1.00
4	ASD	1.00					-1.00
5	ASD	1.00	0.75				0.75
6	ASD	1.00	0.75				-0.75
7	ASD	1.00		1.00			
8	ASD	1.00			1.00		
9	ASD	1.00				1.00	
10	ASD	1.00		-1.00			
11	ASD	1.00			-1.00		
12	ASD	1.00				-1.00	
13	ASD	1.00	0.75	0.75			
14	ASD	1.00	0.75		0.75		
15	ASD	1.00	0.75			0.75	
16	ASD	1.00	0.75	-0.75			
17	ASD	1.00	0.75		-0.75		
18	ASD	1.00	0.75			-0.75	
19	ASD	0.60		1.00			
20	ASD	0.60			1.00		
21	ASD	0.60				1.00	
22	ASD	0.60		-1.00			
23	ASD	0.60			-1.00		
24	ASD	0.60				-1.00	
25	LRFD	1.40					
26	LRFD	1.20	0.50				1.20
27	LRFD	1.20	0.50				-1.20
28	LRFD	1.20	1.60				
29	LRFD	1.20	0.50	1.60			
30	LRFD	1.20	0.50		1.60		
31	LRFD	1.20	0.50			1.60	
32	LRFD	1.20	0.50	-1.60			
33	LRFD	1.20	0.50		-0.60		
34	LRFD	1.20	0.50			-1.60	
35	LRFD	0.90		1.60			
36	LRFD	0.90			1.60		
37	LRFD	0.90				1.60	
38	LRFD	0.90		-1.60			
39	LRFD	0.90			-1.60		
40	LRFD	0.90				-1.60	

4. Deflection Limits

Framing members - Dead Load	Δ_{lim} = L/360
Framing members - Wind Load	Δ_{lim} = L/175
Point supported glass panels	Δ_{lim} = L/100

5. Materials

Framing members A500, Grade B (rect.)	Fy = 46 ksi, min	Fu = 58 ksi, min
Framing members, plates A36 (Angle or Plate)	Fy = 36 ksi, min	Fu = 58 ksi, min
Concrete anchors F 1554 Gr. 55	Fy = 55 ksi, min	Fu = 75 ksi, min

6. Structural analysis input and output

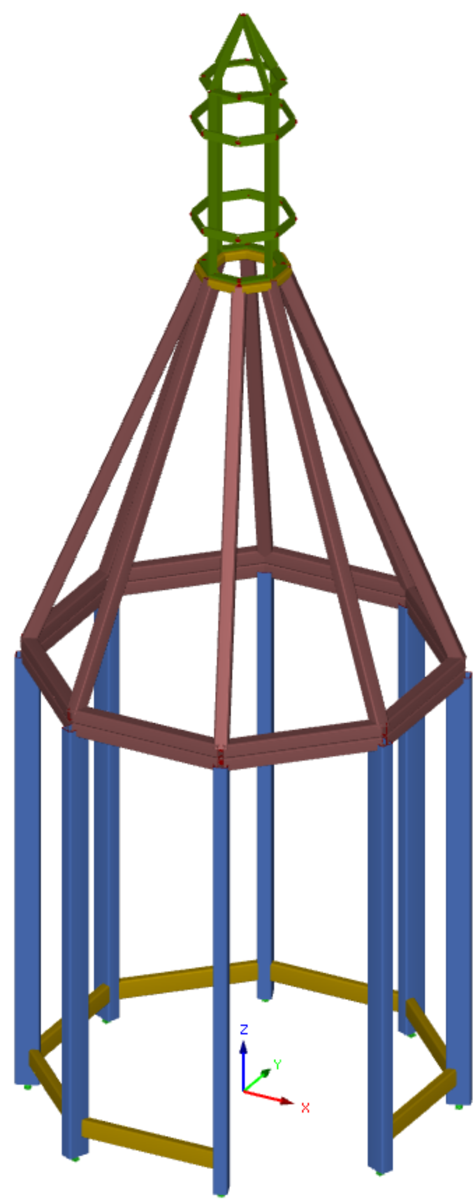


Figure 3 Structural analysis model

STUTZKI ENGINEERING
Stutzki Engineering, Inc.
241 N.Broadway, Suite 302, Milwaukee, WI 53202
(414) 455-4815

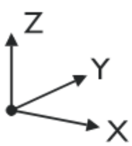
Project: Model: 2020-04-27_2019-174 Lighthouse Frame Date: 4/27/2020

MODEL - GENERAL DATA

	General	Model name	:	2020-04-27_2019-174 Lighthouse Frame
		Type of model	:	3D
		Positive direction of global axis Z	:	Upward
		Classification of load cases and combinations	:	According to Standard: IBC 2015 National Annex: None

1.3 MATERIALS

Matl. No.	Modulus E [ksi]	Modulus G [ksi]	Poisson's Ratio ν [-]	Spec. Weight γ [lbf/ft³]	Coeff. of Th. Exp. α [1/°F]	Partial Factor γ_M [-]	Material Model
1	Steel A500, Grade B (Shapes) | ANSI/AISC 360-05:2005-03						
	29000.0	11200.0	0.295	490.00	6.67E-06	1.00	Isotropic Linear Elastic
2	Steel A36 | ANSI/AISC 360-10:2010						
	29000.0	11200.0	0.295	490.00	6.67E-06	1.00	Isotropic Linear Elastic
3	weightless steel						
	30457.9	11714.6	0.300	1.00E-03	0.00E+00	1.00	Isotropic Linear Elastic
4	Float Glass, Vertical Glazing | TRLV:2006-08						
	10152.6	4127.1	0.230	156.07	5.00E-06	1.00	Isotropic Linear Elastic
5	Rotules						
	29000.0	11200.0	0.295	490.00	6.67E-06	1.00	Isotropic Linear Elastic



1.7 NODAL SUPPORTS

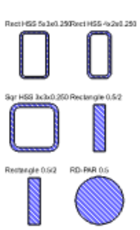
Support No.	Nodes No.	Axis System	Column in Z	Support Conditions $u_{X'}$	$u_{Y'}$	$u_{Z'}$	$\varphi_{X'}$	$\varphi_{Y'}$	$\varphi_{Z'}$
1	8	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐
2	6	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐
3	4	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐
4	2	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐
5	1	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐
6	3	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐
7	5	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐
8	7	User Defined X',Y',Z'	☐	Spring	Spring	☒	☐	☐	☐

1.7.2 NODAL SUPPORTS - SPRINGS

Support No.	Nodes No.	Translation Spring [kip/in] $C_{u,X'}$	$C_{u,Y'}$	$C_{u,Z'}$	Rotation Spring [kipin/°] $C_{\varphi,X'}$	$C_{\varphi,Y'}$	$C_{\varphi,Z'}$
1	8	50.0	50.0	-	-	-	-
2	6	50.0	50.0	-	-	-	-
3	4	50.0	50.0	-	-	-	-
4	2	50.0	50.0	-	-	-	-
5	1	50.0	50.0	-	-	-	-
6	3	50.0	50.0	-	-	-	-
7	5	50.0	50.0	-	-	-	-
8	7	50.0	50.0	-	-	-	-

1.7.10 NODAL SUPPORTS - USER-DEFINED AXIS SYSTEM

Support No.	Direction Type	Sequence	Rotation [°] about X	about Y	about Z	Coordinate System	1st axis	Node 1 No.	Node 2 No.	2nd axis	Referen Node	Member/Line No.
1	Rotated	ZYX	0.00	0.00	22.50							
2	Rotated	ZYX	0.00	0.00	67.50							
3	Rotated	ZYX	0.00	0.00	112.50							
4	Rotated	ZYX	0.00	0.00	157.50							
5	Rotated	ZYX	0.00	0.00	202.50							
6	Rotated	ZYX	0.00	0.00	247.50							
7	Rotated	ZYX	0.00	0.00	292.50							
8	Rotated	ZYX	0.00	0.00	337.50							



1.13 CROSS-SECTIONS

Section No.	Matl. No.	J [in⁴] / A [in²]	I_y [in⁴] / A_y [in²]	I_z [in⁴] / A_z [in²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [in] Width b	Height h
1	Rect HSS 5x3x0.250 | AISC 15							
	1	11.00 3.37	10.70 0.94	4.81 2.02	0.00	0.00	3.000	5.000
2	Rect HSS 4x2x0.250 | AISC 15							
	1	3.82 2.44	4.49 0.56	1.48 1.61	0.00	0.00	2.000	4.000
3	Sqr HSS 3x3x0.250 | AISC 15							
	1	5.08 2.44	3.02 1.22	3.02 1.22	0.00	0.00	3.000	3.000

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Stutzki Engineering, Inc.
241 N.Broadway, Suite 302, Milwaukee, WI 53202
(414) 455-4815

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MODEL

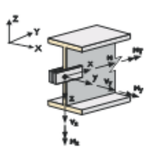
Project: | Model: 2020-04-27_2019-174 Lighthouse Frame | Date: 4/27/2020

1.13 CROSS-SECTIONS

Section No.	Matl. No.	J [in⁴] / A [in²]	I_y [in⁴] / A_y [in²]	I_z [in⁴] / A_z [in²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [in] Width b	Height h
4	Rectangle 0.5/2							
	2	0.07	0.33	0.02	0.00	90.00	0.500	2.000
		1.00	0.83	0.83				
5	Rectangle 0.5/2							
	2	0.07	0.33	0.02	0.00	90.00	0.500	2.000
		1.00	0.83	0.83				
	Spider							
6	RD-PAR 0.5							
	2	0.01	0.00	0.00	0.00	0.00	0.500	0.500
		0.20	0.16	0.16				
	Rotules							
7	Bolts							
	3	5.08	3.02	3.02	0.00	0.00	3.000	3.000
		2.44	1.22	1.22				
8	standoff							
	3	127.23	63.62	63.62	0.00	0.00	6.000	6.000
		28.27	23.75	23.75				

1.13.1 CROSS-SECTIONS - CROSS-SECTION ROTATION

Section No.	Description	Angle α [°]	Mirroring About Axis y	About Axis z
4	Rectangle 0.5/2	90.00	☐	☐
5	Rectangle 0.5/2	90.00	☐	☐

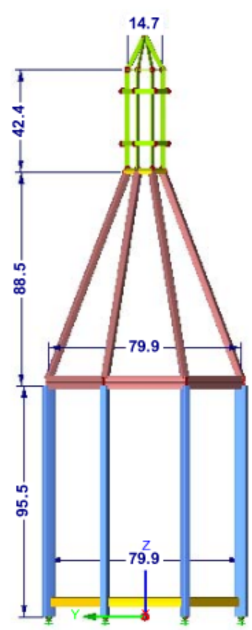


1.14 MEMBER HINGES

Release No.	Reference System	Axial/Shear Release or Spring[kip/in] u_x	u_y	u_z	Moment Release or Spring[kipin/°] φ_x	φ_y	φ_z	Comment
1	Local x,y,z	☐	☐	☐	☒	☒	☒	
2	Local x,y,z	☐	☒	☐	☒	☒	☒	
3	Local x,y,z	☐	☒	☒	☒	☒	☒	
4	Local x,y,z	☐	☐	☐	☐	☒	☒	
5	Local x,y,z	☐	☒	☐	☐	☒	☒	
6	Local x,y,z	☐	☐	☐	☐	☒	☒	

MODEL

In X-direction



53.868 in

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MODEL

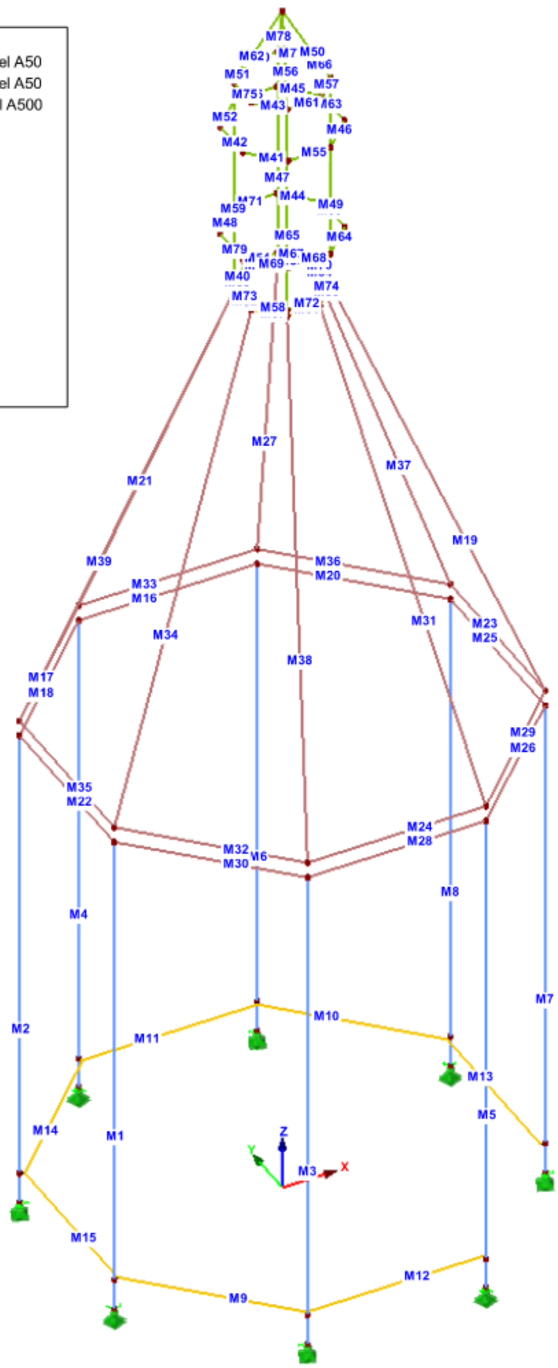
Project: | Model: 2020-04-27_2019-174 Lighthouse Frame | Date: 4/27/2020

MODEL

Member Numbering

Isometric

Cross-Sections
1: Rect HSS 5x3x0.250 | AISC 15; Steel A50
2: Rect HSS 4x2x0.250 | AISC 15; Steel A50
3: Sqr HSS 3x3x0.250 | AISC 15; Steel A500
4: Rectangle 0.5/2; Steel A36; 90.0 °



21.26 in
M 1:27

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MODEL

Project:

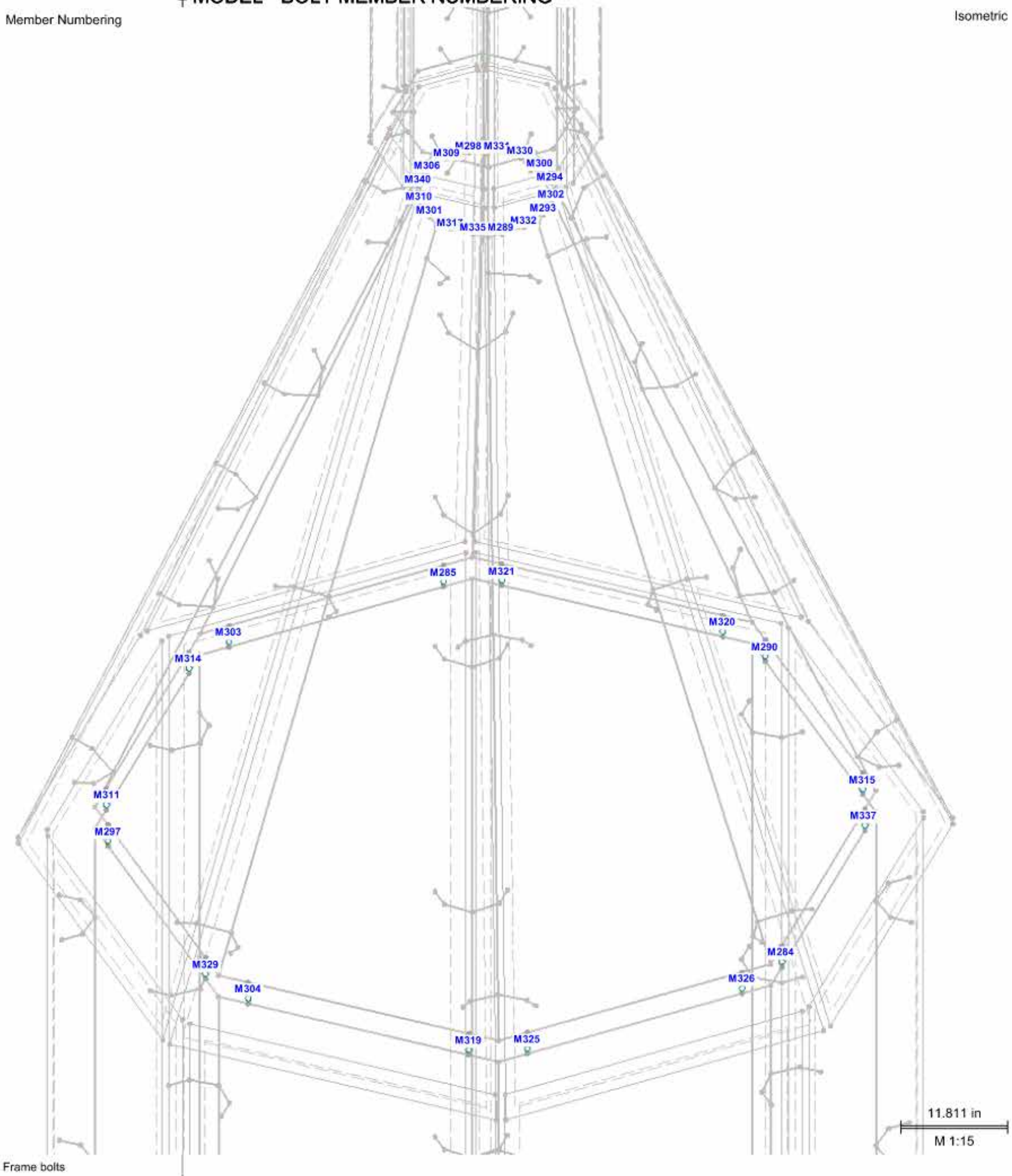
Model: 2020-04-27_2019-174 Lighthouse Frame

Date: 4/27/2020

MODEL - BOLT MEMBER NUMBERING

Member Numbering

Isometric



Frame bolts

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LOADS

Project: Model: 2020-04-27_2019-174 Lighthouse Frame Date: 4/27/2020

2.1 LOAD CASES

Load Case	Load Case Description	IBC 2015 Action Category	Active	Self-Weight - Factor in Direction X	Y	Z
LC1	D	Dead	☒	0.000	0.000	-1.000
LC2	Lr	Roof Live	☐			
LC3	W+x	Snow	☐			
LC4	W 45deg	Wind	☐			
LC5	W+y	Wind	☐			
LC6	T	Self-Straining Force	☐			

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		1.0D	1	1.00	LC1	D
CO2		1.0D + 1.0Lr	1	1.00	LC1	D
			2	1.00	LC2	Lr
CO3		1.0D + 1.0T	1	1.00	LC1	D
			2	1.00	LC6	T
CO4		1.0D - 1.0T	1	1.00	LC1	D
			2	-1.00	LC6	T
CO5		1.0D + 0.8Lr + 0.8T	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	0.75	LC6	T
CO6		1.0D + 0.8Lr - 0.8T	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	-0.75	LC6	T
CO7		1.0D + 1.0W+x	1	1.00	LC1	D
			2	1.00	LC3	W+x
CO8		1.0D + 1.0W 45 deg	1	1.00	LC1	D
			2	1.00	LC4	W 45deg
CO9		1.0D + 1.0W+y	1	1.00	LC1	D
			2	1.00	LC5	W+y
CO10		1.0D - 1.0W+x	1	1.00	LC1	D
			2	-1.00	LC3	W+x
CO11		1.0D - 1.0W 45 deg	1	1.00	LC1	D
			2	-1.00	LC4	W 45deg
CO12		1.0D - 1.0W+y	1	1.00	LC1	D
			2	-1.00	LC5	W+y
CO13		1.0D + 0.8Lr + 0.8W+x	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	0.75	LC3	W+x
CO14		1.0D + 0.8Lr + 0.8W 45 deg	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	0.75	LC4	W 45deg
CO15		1.0D + 0.8Lr + 0.8W+y	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	0.75	LC5	W+y
CO16		1.0D + 0.8Lr - 0.8W+x	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	-0.75	LC3	W+x
CO17		1.0D + 0.8Lr - 0.8W 45 deg	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	-0.75	LC4	W 45deg
CO18		1.0D + 0.8Lr - 0.8W+y	1	1.00	LC1	D
			2	0.75	LC2	Lr
			3	-0.75	LC5	W+y
CO19		0.6D + 1.0W+x	1	0.60	LC1	D
			2	1.00	LC3	W+x
CO20		0.6D + 1.0W 45 deg	1	0.60	LC1	D
			2	1.00	LC4	W 45deg
CO21		0.6D + 1.0W+y	1	0.60	LC1	D
			2	1.00	LC5	W+y
CO22		0.6D - 1.0W+x	1	0.60	LC1	D
			2	-1.00	LC3	W+x
CO23		0.6D - 1.0W 45 deg	1	0.60	LC1	D
			2	-1.00	LC4	W 45deg
CO24		0.6D - 1.0W+y	1	0.60	LC1	D
			2	-1.00	LC5	W+y
CO25		1.4D	1	1.40	LC1	D
CO26		1.2D + 0.5Lr + 1.2T	1	1.20	LC1	D
			2	0.50	LC2	Lr
			3	1.20	LC6	T
CO27		1.2D + 0.5Lr - 1.2T	1	1.20	LC1	D
			2	0.50	LC2	Lr
			3	-1.20	LC6	T
CO28		1.2D + 1.6Lr	1	1.20	LC1	D
			2	1.60	LC2	Lr
CO29		1.2D + 0.5Lr + 1.6W+x	1	1.20	LC1	D
			2	0.50	LC2	Lr
			3	1.60	LC3	W+x
CO30		1.2D + 0.5Lr + 1.6W 45 deg	1	1.20	LC1	D
			2	0.50	LC2	Lr
			3	1.60	LC4	W 45deg
CO31		1.2D + 0.5Lr + 1.6W+y	1	1.20	LC1	D
			2	0.50	LC2	Lr
			3	1.60	LC5	W+y
CO32		1.2D + 0.5Lr - 1.6W+x	1	1.20	LC1	D

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
			2	0.50	LC2	Lr
			3	-1.60	LC3	W+x
CO33		1.2D + 0.5Lr - 0.6W 45 deg	1	1.20	LC1	D
			2	0.50	LC2	Lr
			3	-0.60	LC4	W 45deg
CO34		1.2D + 0.5Lr - 1.6W+y	1	1.20	LC1	D
			2	0.50	LC2	Lr
			3	-1.60	LC5	W+y
CO35		0.9D + 1.6W+x	1	0.90	LC1	D
			2	1.60	LC3	W+x
CO36		0.9D + 1.6W 45 deg	1	0.90	LC1	D
			2	1.60	LC4	W 45deg
CO37		0.9D + 1.6W+y	1	0.90	LC1	D
			2	1.60	LC5	W+y
CO38		0.9D - 1.6W+x	1	0.90	LC1	D
			2	-1.60	LC3	W+x
CO39		0.9D - 1.6W 45 deg	1	0.90	LC1	D
			2	-1.60	LC4	W 45deg
CO40		0.9D - 1.6W+y	1	0.90	LC1	D
			2	-1.60	LC5	W+y

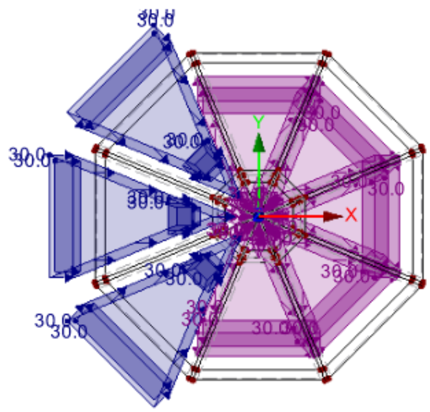
2.7 RESULT COMBINATIONS

Result Combin	Description	Loading
RC1	ASD	CO1 or to CO24
RC2	LRFD	CO25 or to CO40

LC3: W+X

LC3: W+x
Loads [psf]

Against Z-direction

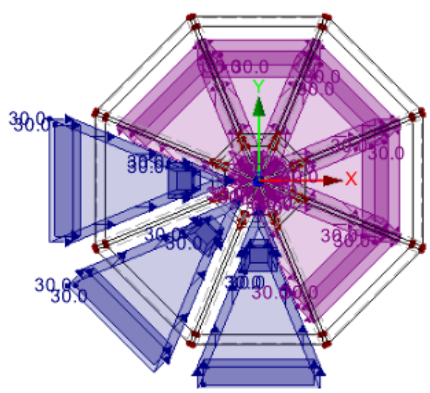


38.058 in

LC4: W 45DEG

LC4: W 45deg
Loads [psf]

Against Z-direction

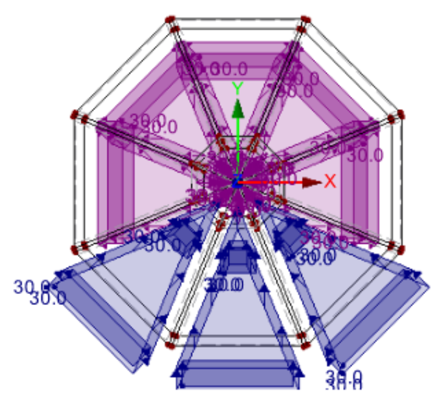


38.058 in

LC5: W+Y

LC5: W+y
Loads [psf]

Against Z-direction

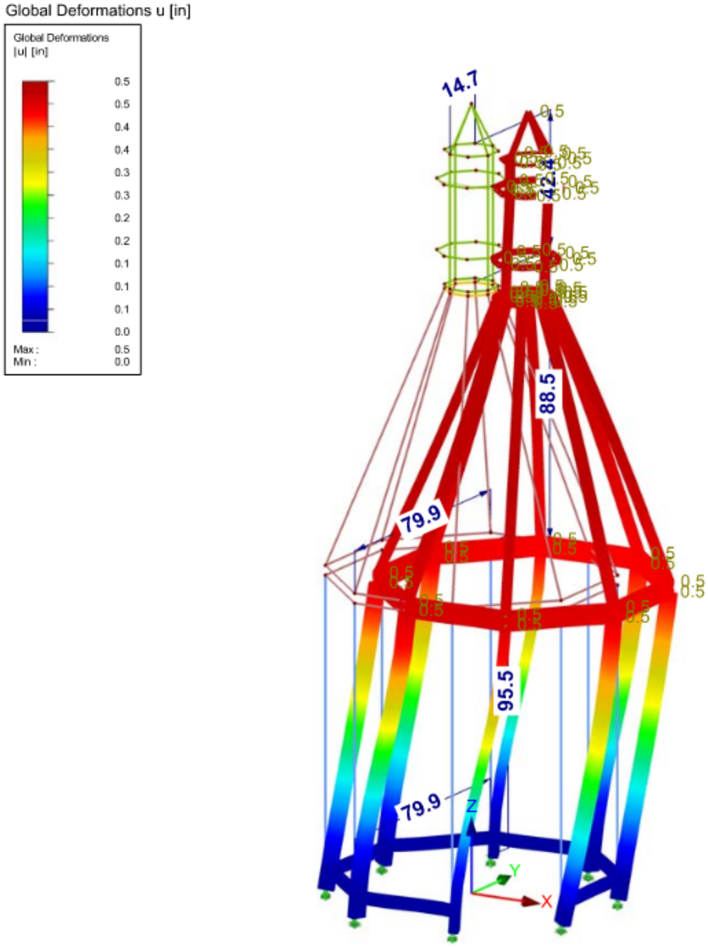


38.058 in

GLOBAL DEFORMATIONS u

LC3: W+x
Global Deformations u [in]

Isometric

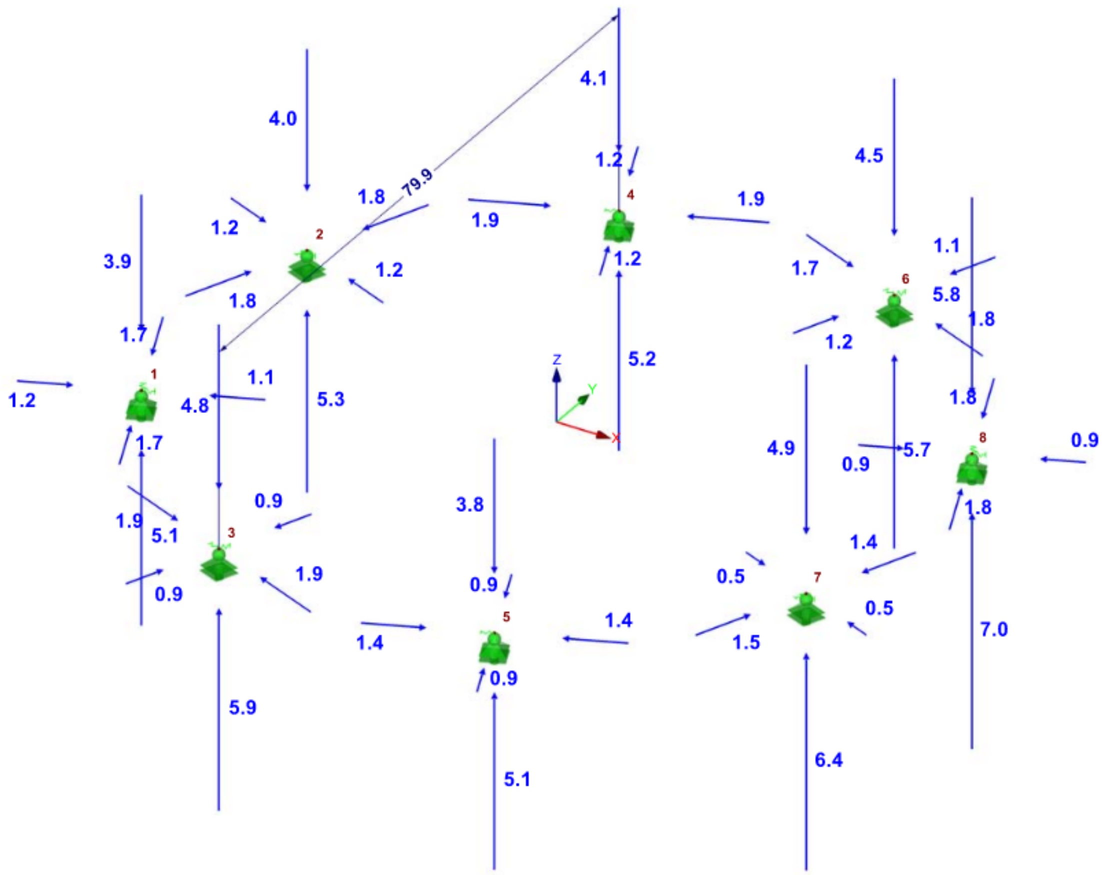


Max u: 0.5, Min u: 0.0 [in]
Factor of deformations: 38.00

SUPPORT REACTIONS

RC2: LRFD
Support Reactions[kip]
Result Combinations: Max and Min Values

Isometric



Max P-X': 1.2, Min P-X': -1.2 kip
Max P-Y': 1.9, Min P-Y': -1.9 kip
Max P-Z': 5.8, Min P-Z': -7.0 kip

4.1 NODES - SUPPORT FORCES

Result Combinations

Node No.	RC		Support Forces [kip] $P_{X'}$	$P_{Y'}$	$P_{Z'}$	Support Moments [kipin] $M_{X'}$	$M_{Y'}$	$M_{Z'}$	
1*	RC1	Max	1.0	1.1	2.4	0.0	0.0	0.0	ASD
		Min	-1.0	-1.1	-3.3	0.0	0.0	0.0	ASD
	RC2	Max	1.2	1.7	3.9	0.0	0.0	0.0	LRFD
		Min	-1.1	-1.7	-5.1	0.0	0.0	0.0	LRFD
2*	RC1	Max	1.0	1.1	2.5	0.0	0.0	0.0	ASD
		Min	-1.0	-1.1	-3.4	0.0	0.0	0.0	ASD
	RC2	Max	1.2	1.8	4.0	0.0	0.0	0.0	LRFD
		Min	-1.2	-1.8	-5.3	0.0	0.0	0.0	LRFD
3*	RC1	Max	0.7	1.2	2.9	0.0	0.0	0.0	ASD
		Min	-0.7	-1.2	-3.9	0.0	0.0	0.0	ASD
	RC2	Max	0.9	1.9	4.8	0.0	0.0	0.0	LRFD
		Min	-0.9	-1.9	-5.9	0.0	0.0	0.0	LRFD
4*	RC1	Max	1.0	1.2	2.5	0.0	0.0	0.0	ASD
		Min	-1.0	-1.2	-3.5	0.0	0.0	0.0	ASD
	RC2	Max	1.2	1.9	4.1	0.0	0.0	0.0	LRFD

■ 4.1 NODES - SUPPORT FORCES

Result Combinations

Node No.	RC		Support Forces [kip] P_X	P_Y	P_Z	Support Moments [kipin] M_X	M_Y	M_Z	
4*		Min	-1.2	-1.9	-5.2	0.0	0.0	0.0	LRFD
5*	RC1	Max	0.6	0.9	2.4	0.0	0.0	0.0	ASD
		Min	-0.6	-0.9	-3.3	0.0	0.0	0.0	ASD
	RC2	Max	0.9	1.4	3.8	0.0	0.0	0.0	LRFD
		Min	-0.9	-1.4	-5.1	0.0	0.0	0.0	LRFD
6*	RC1	Max	1.0	1.1	2.8	0.0	0.0	0.0	ASD
		Min	-1.0	-1.1	-3.7	0.0	0.0	0.0	ASD
	RC2	Max	1.1	1.7	4.5	0.0	0.0	0.0	LRFD
		Min	-1.2	-1.8	-5.7	0.0	0.0	0.0	LRFD
7*	RC1	Max	0.4	0.9	3.0	0.0	0.0	0.0	ASD
		Min	-0.4	-0.9	-4.0	0.0	0.0	0.0	ASD
	RC2	Max	0.5	1.4	4.9	0.0	0.0	0.0	LRFD
		Min	-0.5	-1.5	-6.4	0.0	0.0	0.0	LRFD
8*	RC1	Max	0.7	1.1	3.6	0.0	0.0	0.0	ASD
		Min	-0.7	-1.1	-4.5	0.0	0.0	0.0	ASD
	RC2	Max	0.9	1.8	5.8	0.0	0.0	0.0	LRFD
		Min	-0.9	-1.8	-7.0	0.0	0.0	0.0	LRFD

■ 4.6 MEMBERS - INTERNAL FORCES

Result Combinations

Member No.	RC	Node No.	Location x [in]		Forces [kip] N	V_y	V_z	Moments [kipin] M_T	M_y	M_z	Corresponding Load Cases
284	RC2	424	3.000	Max N	2.5	0.4	0.4	0.0	0.0	0.0	CO 35
		425	0.000	Min N	-2.8	-0.4	-0.4	0.0	1.2	-1.1	CO 32
285	RC2	381	3.000	Max N	2.3	-0.3	0.8	0.0	0.0	0.0	CO 38
		382	0.000	Min N	-2.6	0.3	-0.8	-0.0	2.3	1.0	CO 29
290	RC2	478	3.000	Max N	2.1	-0.3	-0.7	0.0	0.0	0.0	CO 40
		479	0.000	Min N	-2.4	0.3	0.6	0.0	-1.9	0.9	CO 31
297	RC2	59	3.000	Max N	2.0	-0.4	0.7	0.0	0.0	0.0	CO 40
		60	0.000	Min N	-2.2	0.4	-0.6	-0.0	1.9	1.1	CO 31
303	RC2	156	3.000	Max N	2.6	-0.1	-1.0	0.0	0.0	0.0	CO 35
		157	0.000	Min N	-3.0	0.0	0.9	0.0	-2.8	0.1	CO 32
304	RC2	71	3.000	Max N	1.5	-0.2	0.9	0.0	0.0	0.0	CO 35
		72	0.000	Min N	-1.8	0.3	-0.8	-0.0	2.4	0.8	CO 32
311	RC2	73	3.000	Max N	2.6	-0.0	-1.0	0.0	0.0	0.0	CO 36
		74	0.000	Min N	-2.9	0.0	0.9	0.0	-2.8	0.1	CO 39
314	RC2	111	3.000	Max N	2.2	-0.4	0.8	0.0	0.0	0.0	CO 39
		112	0.000	Min N	-2.5	0.3	-0.7	-0.0	2.2	1.0	CO 30
315	RC2	476	3.000	Max N	2.6	0.1	0.9	0.0	0.0	0.0	CO 37
		477	0.000	Min N	-3.0	-0.1	-0.9	-0.0	2.6	-0.4	CO 34
319	RC2	109	3.000	Max N	1.8	-0.2	-0.7	0.0	0.0	0.0	CO 38
		110	0.000	Min N	-2.1	0.2	0.7	-0.0	-2.1	0.6	CO 29
320	RC2	464	3.000	Max N	2.5	0.3	0.7	0.0	0.0	0.0	CO 37
		465	0.000	Min N	-2.9	-0.3	-0.6	0.0	1.9	-1.0	CO 34
321	RC2	426	3.000	Max N	2.5	0.3	-0.7	0.0	0.0	0.0	CO 35
		427	0.000	Min N	-2.8	-0.3	0.6	-0.0	-1.9	-0.8	CO 32
325	RC2	154	3.000	Max N	2.8	0.1	1.1	0.0	0.0	0.0	CO 35
		155	0.000	Min N	-3.1	-0.1	-1.0	0.0	3.1	-0.2	CO 32
326	RC2	379	3.000	Max N	2.5	-0.3	-0.7	0.0	0.0	0.0	CO 38
		380	0.000	Min N	-2.8	0.3	0.6	0.0	-1.9	1.0	CO 29
329	RC2	57	3.000	Max N	2.4	0.0	-0.9	0.0	0.0	0.0	CO 37
		58	0.000	Min N	-2.8	-0.0	0.8	0.0	-2.5	-0.0	CO 34
337	RC2	462	3.000	Max N	1.7	-0.5	-0.5	0.0	0.0	0.0	CO 39
		463	0.000	Min N	-2.0	0.5	0.4	0.0	-1.1	1.4	CO 30

■ 2.2 DESIGN BY CROSS-SECTION

Sect. No.	Member No.	Location x [in]	LC/CO/ RC	Design		Equation No.	Description
1	**Rect HSS 5x3x0.250 | AISC 15**						
	7	48.375	RC2	0.01	≤ 1	100)	Negligible internal forces
	8	6.000	RC2	0.06	≤ 1	101)	Chapter D - Tensile strength acc. to D2
	8	6.000	RC2	0.18	≤ 1	111)	Chapter F - Yielding - Bending about y-axis acc. to F2 - F12
	3	6.000	RC2	0.58	≤ 1	112)	Chapter F - Yielding - Bending about z-axis acc. to F2 - F12
	8	6.000	RC2	0.18	≤ 1	115)	Chapter F - Lateral-torsional buckling
	1	6.000	RC2	0.00	≤ 1	126)	Chapter F - Local buckling of HSS section does not apply - acc. to F7
	1	6.000	RC2	0.00	≤ 1	147)	Chapter F - Local buckling of HSS section does not apply - acc. to F7
	2	0.000	RC2	0.04	≤ 1	160)	Chapter G - Nominal shear strength in z-axis - acc. to G2 (G3,G4) - Unstiffened cross-section
	3	6.000	RC2	0.12	≤ 1	161)	Chapter G - Nominal shear strength in y-axis - acc. to G6 (G3,G4) - Unstiffened cross-section
	8	0.000	RC2	0.09	≤ 1	301)	Chapter E - Flexural buckling about y-axis acc. to E3
	8	0.000	RC2	0.12	≤ 1	311)	Chapter E - Flexural buckling about z-axis acc. to E3
	8	0.000	RC2	0.08	≤ 1	321)	Chapter E - Torsional or flexural-torsional buckling acc. to E4
	3	6.000	RC2	0.65	≤ 1	331)	Chapter H - Biaxial bending without axial forces - acc. to H1.1
	3	6.000	RC2	0.67	≤ 1	332)	Chapter H - Single axis /or biaxial/ flexure with axial c

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Stutzki Engineering, Inc.
241 N.Broadway, Suite 302, Milwaukee, WI 53202
(414) 455-4815

2.2 DESIGN BY CROSS-SECTION

Sect. No.	Member No.	Location x [in]	LC/CO/ RC	Design		Equation No.	Description
							compression force - acc. to H1.1
	3	6.000	RC2	0.65	≤ 1	333)	Chapter H - Single axis /or biaxial/ flexure with tensile force - acc. to H1.2
	5	36.834	RC2	0.11	≤ 1	334)	Chapter H - Compression force with single/major axis bending - acc. to H1.3
	7	65.688	RC2	0.21	≤ 1	335)	Chapter H - Compression force with single/minor axis bending - acc. to H1.3
	3	95.494	RC2	0.02	≤ 1	336)	Chapter H - Torsional strength of round and rectangular HSS - acc. to H3.1
	4	6.000	RC2	0.62	≤ 1	337)	Chapter H - HSS subject to combined torsion, shear, flexure and axial force, if Tr < 0.2 Tc - acc. to H3.2
2	**Rect HSS 4x2x0.250 | AISC 15**						
	83	0.000	RC2	0.01	≤ 1	100)	Negligible internal forces
	14	0.000	RC2	0.03	≤ 1	101)	Chapter D - Tensile strength acc. to D2
	12	0.000	RC2	0.78	≤ 1	111)	Chapter F - Yielding - Bending about y-axis acc. to F2 - F12
	84	6.080	RC2	0.10	≤ 1	112)	Chapter F - Yielding - Bending about z-axis acc. to F2 - F12
	12	0.000	RC2	0.78	≤ 1	115)	Chapter F - Lateral-torsional buckling
	9	0.000	RC2	0.00	≤ 1	126)	Chapter F - Local buckling of HSS section does not apply - acc. to F7
	9	0.000	RC2	0.00	≤ 1	147)	Chapter F - Local buckling of HSS section does not apply - acc. to F7
	12	33.084	RC2	0.14	≤ 1	160)	Chapter G - Nominal shear strength in z-axis - acc. to G2 (G3,G4) - Unstiffened cross-section
	80	6.080	RC2	0.15	≤ 1	161)	Chapter G - Nominal shear strength in y-axis - acc. to G6 (G3,G4) - Unstiffened cross-section
	14	0.000	RC2	0.03	≤ 1	301)	Chapter E - Flexural buckling about y-axis acc. to E3
	14	0.000	RC2	0.03	≤ 1	311)	Chapter E - Flexural buckling about z-axis acc. to E3
	14	0.000	RC2	0.03	≤ 1	321)	Chapter E - Torsional or flexural-torsional buckling acc. to E4
	12	0.000	RC2	0.80	≤ 1	331)	Chapter H - Biaxial bending without axial forces - acc. to H1.1
	10	33.084	RC2	0.64	≤ 1	332)	Chapter H - Single axis /or biaxial/ flexure with axial compression force - acc. to H1.1
	10	33.084	RC2	0.66	≤ 1	333)	Chapter H - Single axis /or biaxial/ flexure with tensile force - acc. to H1.2
	10	0.000	RC2	0.54	≤ 1	334)	Chapter H - Compression force with single/major axis bending - acc. to H1.3
	80	6.080	RC2	0.05	≤ 1	335)	Chapter H - Compression force with single/minor axis bending - acc. to H1.3
	11	33.084	RC2	0.10	≤ 1	336)	Chapter H - Torsional strength of round and rectangular HSS - acc. to H3.1
	12	0.000	RC2	0.80	≤ 1	337)	Chapter H - HSS subject to combined torsion, shear, flexure and axial force, if Tr < 0.2 Tc - acc. to H3.2
3	**Sqr HSS 3x3x0.250 | AISC 15**						
	31	5.281	RC2	0.01	≤ 1	100)	Negligible internal forces
	23	3.500	RC2	0.04	≤ 1	101)	Chapter D - Tensile strength acc. to D2
	28	33.084	RC2	0.60	≤ 1	111)	Chapter F - Yielding - Bending about y-axis acc. to F2 - F12
	27	92.540	RC2	0.07	≤ 1	112)	Chapter F - Yielding - Bending about z-axis acc. to F2 - F12
	28	33.084	RC2	0.60	≤ 1	115)	Chapter F - Lateral-torsional buckling
	16	0.000	RC2	0.00	≤ 1	126)	Chapter F - Local buckling of HSS section does not apply - acc. to F7
	16	0.000	RC2	0.00	≤ 1	147)	Chapter F - Local buckling of HSS section does not apply - acc. to F7
	28	33.084	RC2	0.30	≤ 1	160)	Chapter G - Nominal shear strength in z-axis - acc. to G2 (G3,G4) - Unstiffened cross-section
	25	29.584	RC2	0.04	≤ 1	161)	Chapter G - Nominal shear strength in y-axis - acc. to G6 (G3,G4) - Unstiffened cross-section
	27	0.000	RC2	0.05	≤ 1	301)	Chapter E - Flexural buckling about y-axis acc. to E3
	27	0.000	RC2	0.05	≤ 1	311)	Chapter E - Flexural buckling about z-axis acc. to E3
	23	29.584	RC2	0.04	≤ 1	321)	Chapter E - Torsional or flexural-torsional buckling acc. to E4
	28	33.084	RC2	0.63	≤ 1	331)	Chapter H - Biaxial bending without axial forces - acc. to H1.1
	28	0.000	RC2	0.60	≤ 1	332)	Chapter H - Single axis /or biaxial/ flexure with axial compression force - acc. to H1.1
	28	0.000	RC2	0.58	≤ 1	333)	Chapter H - Single axis /or biaxial/ flexure with tensile force - acc. to H1.2
	28	0.000	RC2	0.58	≤ 1	334)	Chapter H - Compression force with single/major axis bending - acc. to H1.3
	21	0.000	RC2	0.06	≤ 1	335)	Chapter H - Compression force with single/minor axis bending - acc. to H1.3
	28	33.084	RC2	0.09	≤ 1	336)	Chapter H - Torsional strength of round and rectangular HSS - acc. to H3.1
	28	33.084	RC2	0.63	≤ 1	337)	Chapter H - HSS subject to combined torsion, shear, flexure and axial force, if Tr < 0.2 Tc - acc. to H3.2
4	**Rectangle 0.5/2**						
	43	3.040	RC2	0.01	≤ 1	100)	Negligible internal forces
	57	3.040	RC2	0.42	≤ 1	101)	Chapter D - Tensile strength acc. to D2
	56	0.000	RC2	0.45	≤ 1	111)	Chapter F - Yielding - Bending about y-axis acc. to F2 - F12

STUTZKI ENGINEERING

Stutzki Engineering, Inc.
241 N.Broadway, Suite 302, Milwaukee, WI 53202
(414) 455-4815

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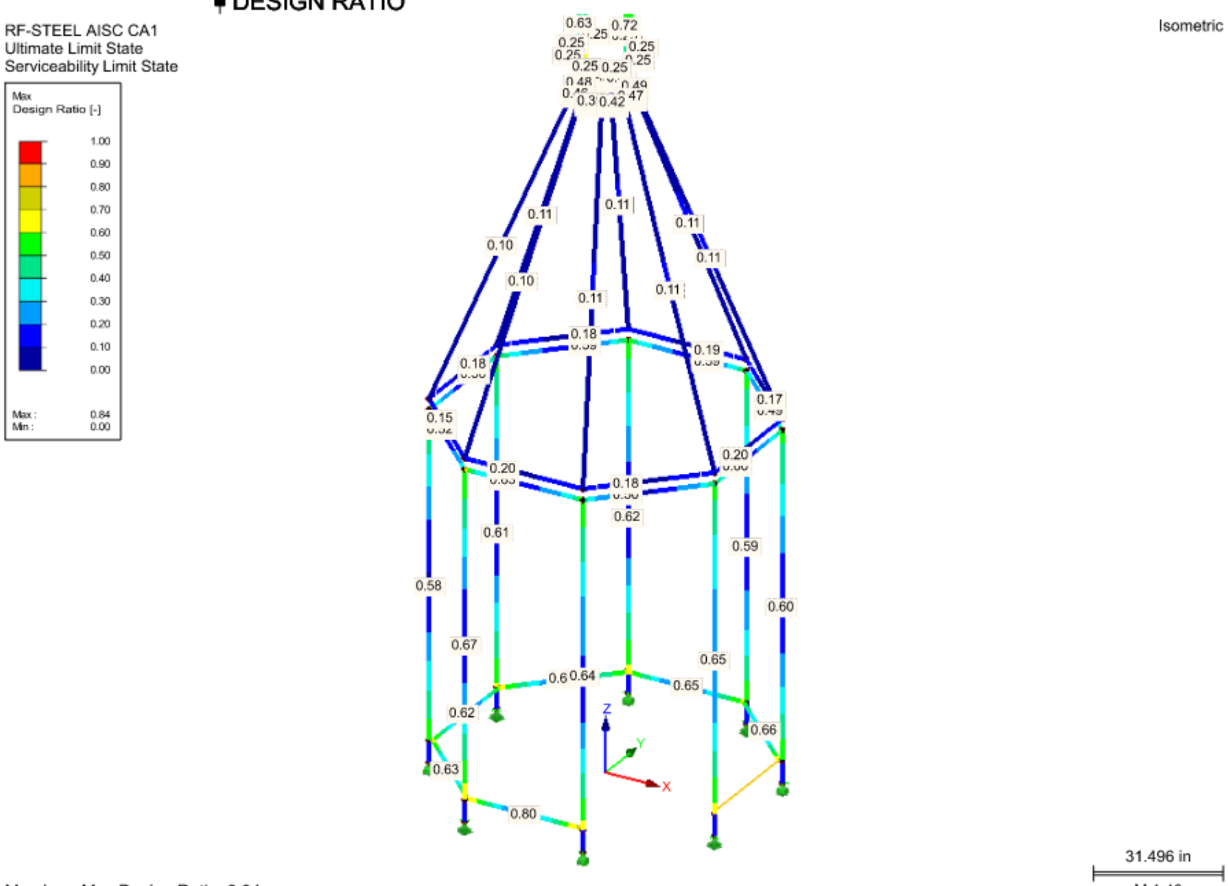
RF-STEEL AISC

Project: Model: 2020-04-27_2019-174 Lighthouse Frame Date: 4/27/2020

2.2 DESIGN BY CROSS-SECTION

Sect. No.	Member No.	Location x [in]	LC/CO/ RC	Design		Equation No.	Description
	61	6.080	RC2	0.38	≤ 1	112)	Chapter F - Yielding - Bending about z-axis acc. to F2 - F12
	56	0.000	RC2	0.45	≤ 1	115)	Chapter F - Lateral-torsional buckling
	56	0.000	RC2	0.04	≤ 1	160)	Chapter G - Nominal shear strength in z-axis - acc. to G2 (G3,G4) - Unstiffened cross-section
	67	1.678	RC2	0.07	≤ 1	161)	Chapter G - Nominal shear strength in y-axis - acc. to G6 (G3,G4) - Unstiffened cross-section
	67	1.678	RC2	0.49	≤ 1	216)	Design Guide No. 9 - Check of shear stress due to torsion and shear stresses
	57	0.000	RC2	0.42	≤ 1	301)	Chapter E - Flexural buckling about y-axis acc. to E3
	47	0.000	RC2	0.60	≤ 1	311)	Chapter E - Flexural buckling about z-axis acc. to E3
	57	0.000	RC2	0.43	≤ 1	321)	Chapter E - Torsional or flexural-torsional buckling acc. to E4
	49	0.000	RC2	0.34	≤ 1	331)	Chapter H - Biaxial bending without axial forces - acc. to H1.1
	50	0.000	RC2	0.84	≤ 1	332)	Chapter H - Single axis /or biaxial/ flexure with axial compression force - acc. to H1.1
	61	6.080	RC2	0.78	≤ 1	333)	Chapter H - Single axis /or biaxial/ flexure with tensile force - acc. to H1.2
	50	0.000	RC2	0.77	≤ 1	334)	Chapter H - Compression force with single/major axis bending - acc. to H1.3
	57	3.040	RC2	0.55	≤ 1	335)	Chapter H - Compression force with single/minor axis bending - acc. to H1.3

DESIGN RATIO



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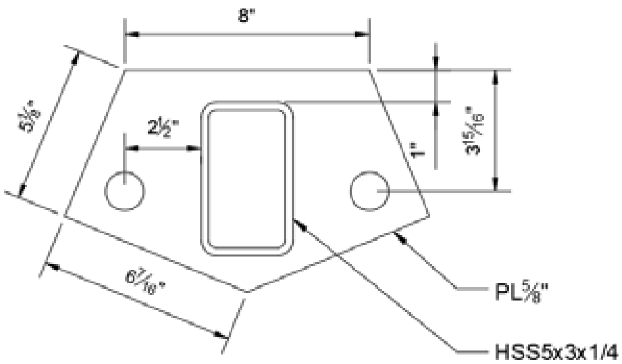
Members Max Design Ratio: 0.84

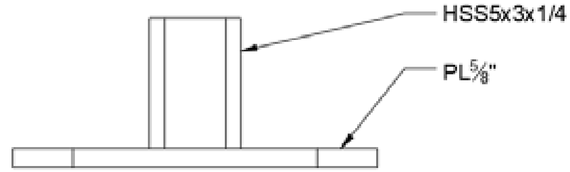
7. Structure Support Design

7.1. Reaction Summary

Max LRFD Reactions
Px = ±1.2k
Py = ±1.9k
Pz = 5.8k (tension), 7.0k (compression)

7.2. Base Plate Bending Check





#1 Base Plate Bending Check

5/8" Thick A36 Plate

Loads (LRFD)

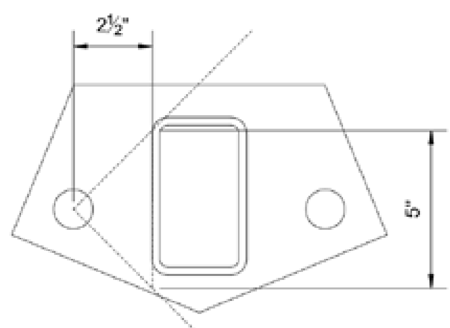
$P_{u1} := 5.8kip$

Properties:

$t_{pl} := 0.625in$ $F_y := 36 \cdot ksi$ $F_u := 58ksi$

Whitmore Section $w_{pl} := 5in$ $ecc := 2.5in$

$Z_y := \frac{w_{pl} \cdot t_{pl}^2}{4} = 0.49 \cdot in^3$ $S_y := \frac{w_{pl} \cdot t_{pl}^2}{6} = 0.33 \cdot in^3$



Demand

Tension in Bolt $T_{ub} := \frac{P_{u1}}{2} = 2.9\,kip$

Bending Demand $M_u := T_{ub} \cdot ecc = 7.25 \cdot in \cdot kip$

Capacity

Flexural Yielding $\phi M_n := 0.9\,min(F_y \cdot Z_y, 1.6 \cdot F_y \cdot S_y) = 15.82 \cdot in \cdot kip$

Interaction

$IR1 := \left(\frac{M_u}{\phi M_n}\right) = 0.46$ Check1 = "OK"

7.3. Hilti PROFIS Anchor Checks

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Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	4/27/2020
E-Mail:			

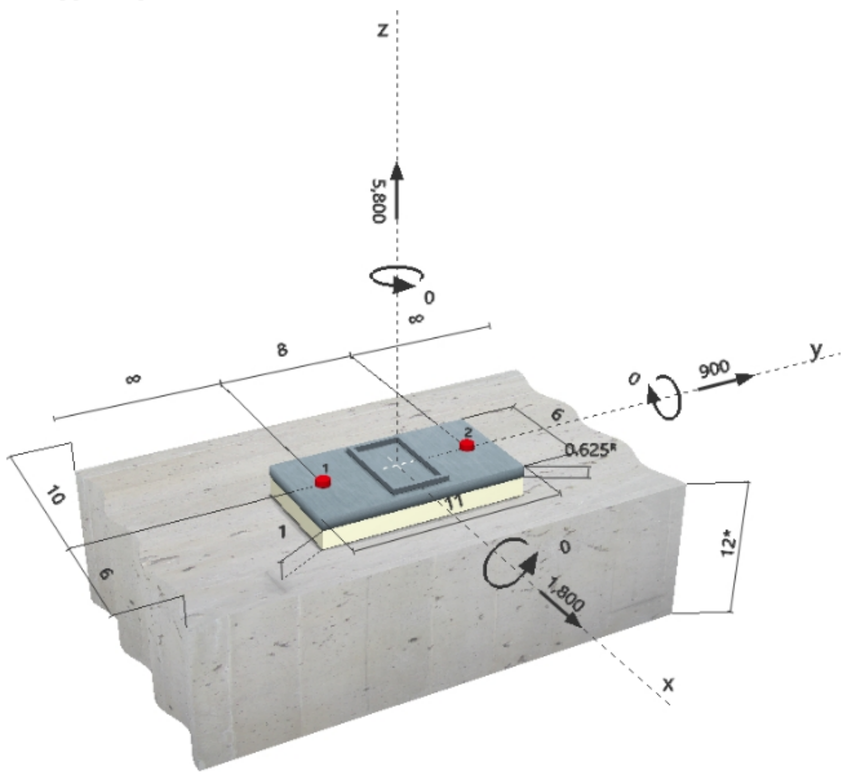
Specifier's comments:

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-11 / CIP
Stand-off installation:	without clamping (anchor); restraint level (anchor plate): 2.00; e_b = 1.000 in.; t = 0.625 in.
	Hilti Grout: CB-G MG (50), multipurpose, $f_{c,Grout}$ = 6,962 psi
Anchor plate:	l_x x l_y x t = 6.000 in. x 11.000 in. x 0.625 in.; (Recommended plate thickness: not calculated
Profile:	Rectangular HSS (AISC), HSS5X3X.250; (L x W x T) = 5.000 in. x 3.000 in. x 0.250 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar

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

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



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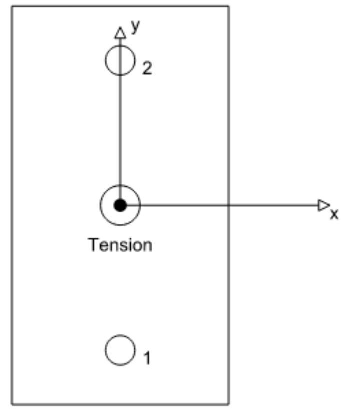
Phone I Fax:	|	Date:	4/27/2020
E-Mail:			

2 Load case/Resulting anchor forces



Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2,900	1,006	900	450
2	2,900	1,006	900	450

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 5,800 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2,900	18,787	16	OK
Pullout Strength*	2,900	20,406	15	OK
Concrete Breakout Strength**	5,800	16,917	35	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-11 Eq. (D-2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [lb]
25,050

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
25,050	0.750	18,787	2,900

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3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-11 Eq. (D-13)

$N_p = 8 A_{brg} f'_c$ ACI 318-11 Eq. (D-14)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,000

Calculations

N_p [lb]
29,152

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
29,152	0.700	20,406	2,900

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-11 Eq. (D-4)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

A_{Nc} see ACI 318-11, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-11 Eq. (D-5)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-11 Eq. (D-8)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-11 Eq. (D-10)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-12)

$N_b = k_c\ \lambda_a \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-11 Eq. (D-6)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
6.000	0.000	0.000	6.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
390.00	324.00	1.000	1.000	0.900	1.000	22,308

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
24,167	0.700	16,917	5,800

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E-Mail:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1,006	7,816	13	OK
Steel failure (with lever arm)*	1,006	1,521	67	OK
Pryout Strength**	2,012	33,834	6	OK
Concrete edge failure in direction x+**	2,012	8,459	24	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-11 Eq. (D-29)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [lb]
15,030

Results

V_{sa} [lb]	ϕ_{steel}	ϕ_{eb}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
15,030	0.650	0.800	7,816	1,006

4.2 Steel failure (with lever arm)

$V_s^M = \frac{\alpha_M \cdot M_s}{L_b}$ bending equation for stand-off

$M_s = M_s^0 \left(1 - \frac{N_{ua}}{\phi\ N_{sa}}\right)$ resultant flexural resistance of anchor

$M_s^0 = (1.2)\ (S)\ (f_{u,min})$ characteristic flexural resistance of anchor

$\left(1 - \frac{N_{ua}}{\phi\ N_{sa}}\right)$ reduction for tensile force acting simultaneously with a shear force on the anchor

$S = \frac{\pi (d)^3}{32}$ elastic section modulus of anchor bolt at concrete surface

$L_b = z + (n)(d_0)$ internal lever arm adjusted for spalling of the surface concrete

$\phi\ V_s^M \geq V_{ua}$ ACI 318-11 Table D.4.1.1

Variables

α_M	$f_{u,min}$ [psi]	N_{ua} [lb]	$\phi\ N_{sa}$ [lb]	z [in.]	n	d_0 [in.]
2.00	75,000	2,900	18,787	1.313	0.500	0.750

Calculations

M_s^0 [in.lb]	$\left(1 - \frac{N_{ua}}{\phi\ N_{sa}}\right)$	M_s [in.lb]	L_b [in.]
2,334.754	0.846	1,974.366	1.688

Results

V_s^M [lb]	ϕ_{steel}	$\phi\ V_s^M$ [lb]	V_{ua} [lb]
2,340	0.650	1,521	1,006

4.3 Pryout Strength

$V_{cpg} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ec,N}\,\psi_{ed,N}\,\psi_{c,N}\,\psi_{cp,N}\,N_b\right]$	ACI 318-11 Eq. (D-41)
$\phi\ V_{cpg} \geq V_{ua}$	ACI 318-11 Table D.4.1.1
A_{Nc} see ACI 318-11, Part D.5.2.1, Fig. RD.5.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-11 Eq. (D-5)
$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-11 Eq. (D-8)
$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-11 Eq. (D-10)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-11 Eq. (D-12)
$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$	ACI 318-11 Eq. (D-6)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
390.00	324.00	1.000	1.000	0.900	1.000	22,308

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
48,335	0.700	33,834	2,012

4.4 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\, \psi_{ed,V}\, \psi_{c,V}\, \psi_{h,V}\, \psi_{parallel,V}\, V_b$	ACI 318-11 Eq. (D-31)
$\phi\ V_{cbg} \geq V_{ua}$	ACI 318-11 Table D.4.1.1
A_{Vc} see ACI 318-11, Part D.6.2.1, Fig. RD.6.2.1(b)	
$A_{Vc0} = 4.5\, c_{a1}^2$	ACI 318-11 Eq. (D-32)
$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$	ACI 318-11 Eq. (D-36)
$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-11 Eq. (D-38)
$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-11 Eq. (D-39)
$V_b = 9\, \lambda_a \sqrt{f_c'}\, c_{a1}^{1.5}$	ACI 318-11 Eq. (D-34)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
6.000	-	0.000	1.000	12.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
234.00	162.00	1.000	1.000	1.000	8,366

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
12,084	0.700	8,459	2,012

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.343	0.662	5/3	68	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- ACI 318 does not specifically address anchor bending when a stand-off condition exists. PROFIS Anchor calculates a shear load corresponding to anchor bending when stand-off exists and includes the results as a shear Design Strength!
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

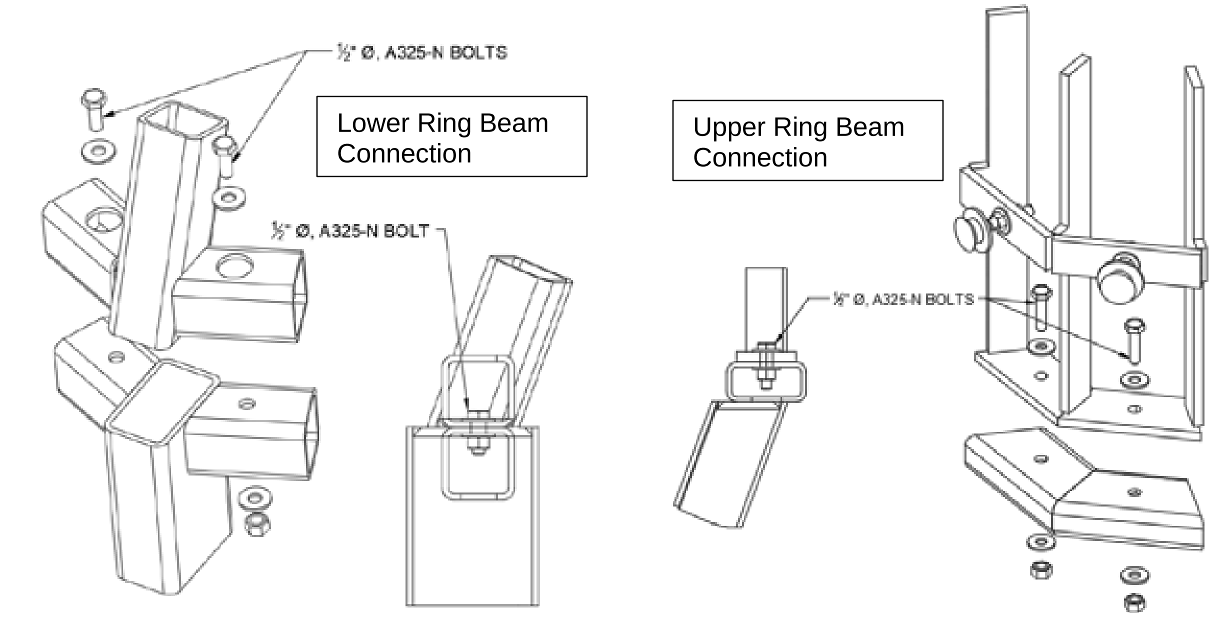
8. Structure Connection Design

8.1. Member end force summary (if applicable)

Max LRFD Bolt Reactions

N = 2.8k (tension), 3.1 kip (compression)
Vy = ±0.5k
Vz = ±1.1k

8.2. Ring Bolt Tension and Shear Connection Check



#2 Ring Bolt Tension and Shear Check

1/2" Diameter, A325-N Bolt

Loads (LRFD)

$P_{u2} := 2.8 kip$

$V_{uy2} := 0.5 kip$ $\quad V_{uz2} := 1.1 kip$ $\quad V_{u2} := \sqrt{V_{uy2}^2 + V_{uz2}^2} = 1.21\,kip$

Properties:

$F_{nt} := 90 \cdot ksi$ $\quad F_{nv} := 54 ksi$

$d_b := 0.5 in$ $\quad d_h := d_b + \frac{1}{16} \cdot in = 0.56\,in$

$A_b := \pi \cdot \frac{d_b^2}{4} = 0.2\,in^2$

Demand

Tension $\quad T_u := P_{u2} = 2800\,lbf$

Shear $\quad V_u := V_{u2} = 1208.3\,lbf$

Capacity

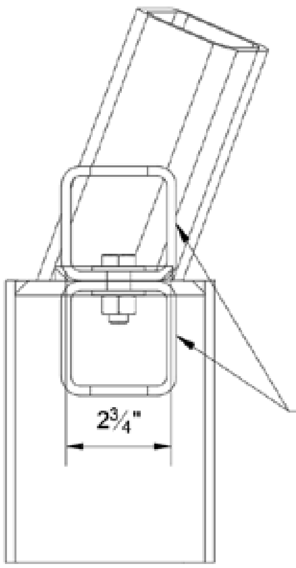
Tension $\quad \phi R_{nt} := 0.75 \cdot F_{nt} \cdot A_b = 13253.59\,lbf$

Shear $\quad \phi R_{nv} := 0.75 \cdot F_{nv} \cdot A_b = 7952.16\,lbf$

Interaction

$IR2_t := \frac{T_u}{\phi R_{nt}} = 0.21$ $\quad Check2_t = \text{"OK"}$ $\quad IR2_v := \frac{V_u}{\phi R_{nv}} = 0.15$ $\quad Check2_v = \text{"OK"}$

8.3. Ring Beam (HSS) Wall Bending Check



#3 HSS Wall Bending Check

1/4" Thick HSS Wall, A500 Grade B or better

Loads (LRFD)

$P_{u3} := 2.8kip$

$V_{uy3} := 0.5kip$ $V_{uz3} := 1.1kip$ $V_{u3} := \sqrt{V_{uy2}^2 + V_{uz2}^2} = 1.21\,kip$

Properties:

$t_{hss} := 0.233in$ $F_{y3} := 46 \cdot ksi$ $F_{u3} := 58ksi$

$b := 2.75in$ $d_h = 0.5625\,in$ $w_{hss} := b - d_h = 2.1875\,in$

$Z_{y3} := \frac{w_{hss} \cdot t_{hss}^2}{4} = 0.03 \cdot in^3$ $S_{y3} := \frac{w_{hss} \cdot t_{hss}^2}{6} = 0.02 \cdot in^3$

Demand

Bending Demand $M_{u3} := \frac{P_{u3} \cdot b}{8} = 0.96 \cdot in \cdot kip$

Capacity

Flexural Yielding $\phi M_{n3} := 0.9\,\min\left(F_{y3} \cdot Z_{y3}, 1.6 \cdot F_{y3} \cdot S_{y3}\right) = 1.23 \cdot in \cdot kip$

Interaction

$IR3 := \left(\frac{M_{u3}}{\phi M_{n3}}\right) = 0.78$ Check3 = "OK"

9. Glass and Cladding Design

9.1. General design criteria

Young's Modulus: 10,400,000 psi
Poisson Ratio: 0.22
Coefficient of Thermal Expansion = 4.9 x 10^{-6} in/in/°F

Glass Allowable Stress per ASTM E1300 Appendix X4 thru X7

		Allowable Edge Stress of Glass POB 1/1000 (psi)				
Load Duration	Load Type	Annealed (A) [Clean Cut]	Annealed (A) [Seamed]	Annealed (A) [Polished]	Heat Strengthed (HS) [Seamed & Polished]	Fully Tempered (FT) [Seamed & Polished]
3s	Wind, EQ	1783	1969	2155	3938	7876
10s		1654	1826	1999	3793	7681
60s		1479	1633	1787	3586	7399
10min	Live, Conc	1280	1414	1548	3337	7053
60min	Live, Dist	1145	1264	1384	3155	6794
12h		980	1082	1185	2920	6452
24h		939	1036	1134	2857	6359
1week		831	918	1004	2689	6107
1month	Snow	759	838	917	2569	5924
1year		649	717	784	2376	5624
beyond 1 year	Dead	553	611	669	2193	5332

The glass is analyzed with the Minimum Thickness per Table 4 in ASTM E1300:

Dimensional Tolerances for Transparent Flat Glass per ASTM C1036

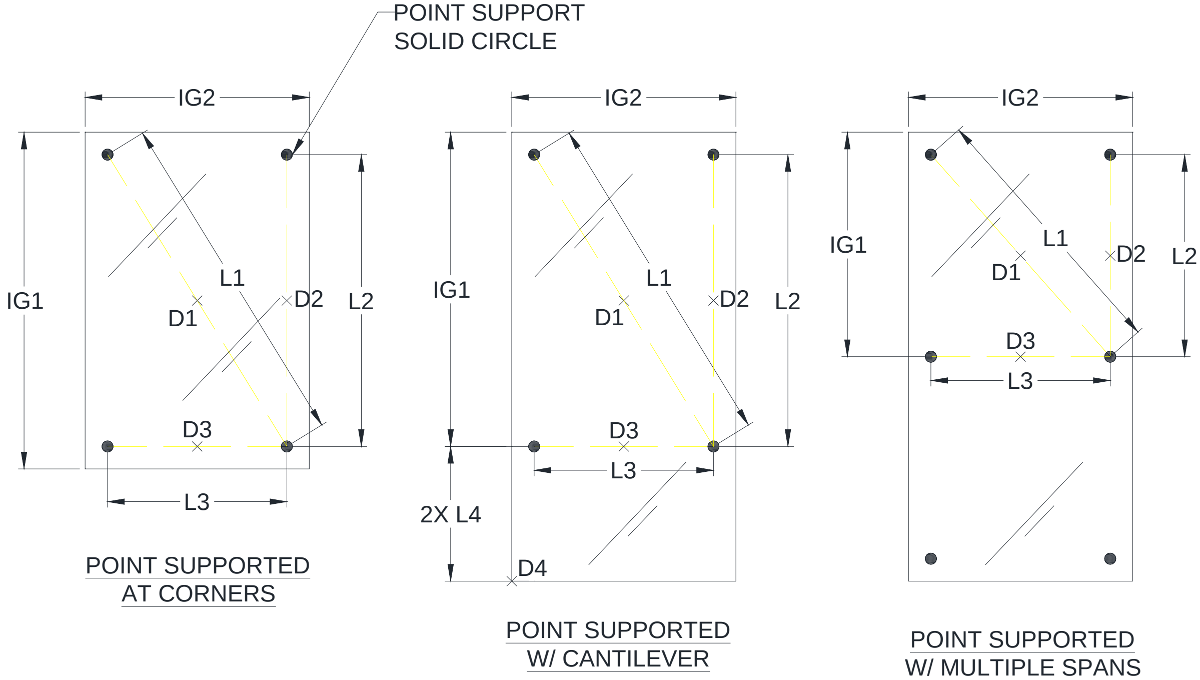
Nominal Designation		**Thickness Range**	
Imperial (US)	SI (Metric)	Minimum *(mm)*	Maximum *(mm)*
5/32"	4 mm	0.149" *(3.78)*	0.165" *(4.19)*
3/16"	5 mm	0.180" *(4.57)*	0.199" *(5.05)*
1/4"	6 mm	0.219" *(5.56)*	0.244" *(6.20)*
5/16"	8 mm	0.292" *(7.42)*	0.332" *(8.43)*
3/8"	10 mm	0.355" *(9.02)*	0.406" *(10.31)*
1/2"	12 mm	0.469" *(11.91)*	0.531" *(13.49)*
5/8"	16 mm	0.595" *(15.09)*	0.656" *(16.66)*
3/4"	19 mm	0.719" *(18.26)*	0.781" *(19.84)*

Note: Table for reference only

Analyzing Glass for Deflection/Serviceability:

Deflection Limit assumed to be L/100 between applicable point supports. See figure below noting applicable spans.

The following diagrams show the span used in the deflection analysis based on glass configuration.



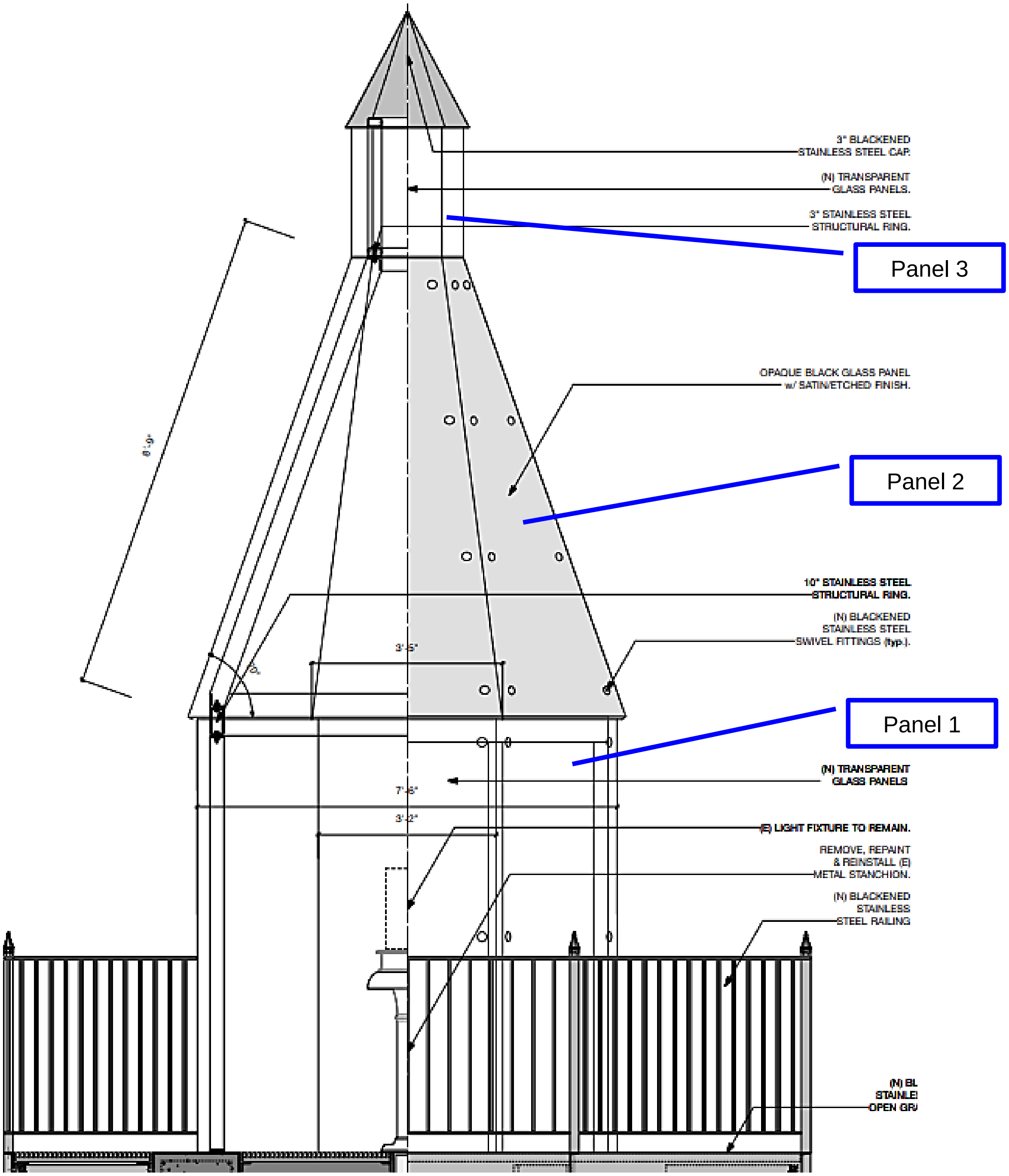
Note: For glass cantilevers, use 2 x Length for determining deflection limits.

9.2. SGP Interlayer (Sentry Glass)

Thickness = 0.060"

Temp (C)	Temp (F)	Load Duration	Load Type	Young's Modulus (psi)	Poisson's Ratio
10	50	3 sec	Wind	98,771	0.443
10	50	1 month	Snow	72,374	0.458
24	75	3 sec	Wind*	81,502	0.453
24	75	1 month	Snow*	31,473	0.480
40	104	3 sec	Wind	27,122	0.484
40	104	10 min	Live, conc	6,382	0.493
40	104	1 hour	Live, dist	4,032	0.498
40	104	10 years	Dead	1,282	0.499

9.3. Overall location, geometry

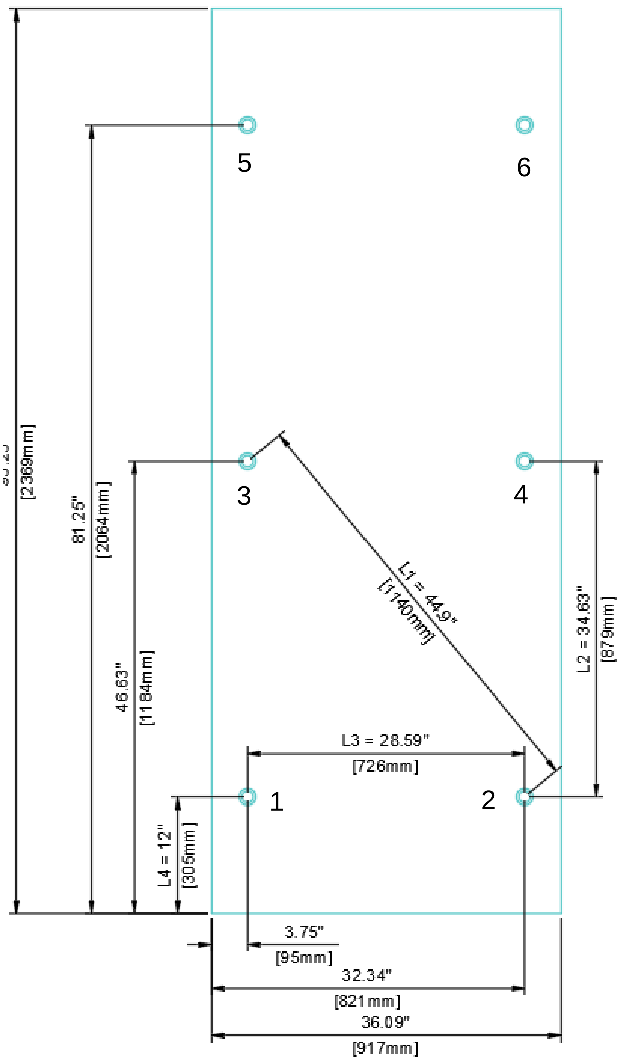


9.4. Glass Make-up

1. 3/8" Fully Tempered Glass
2. 0.06" ~~PVB~~ Interlayer
3. 1/4" Fully Tempered Glass

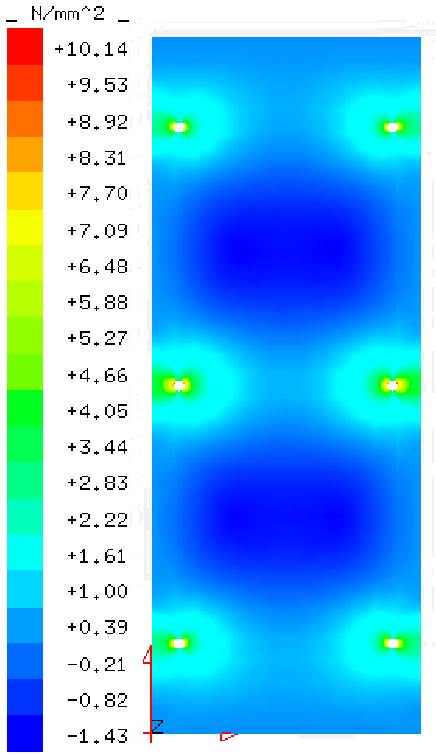
9.5. Glass Panel 1: Lower Drum

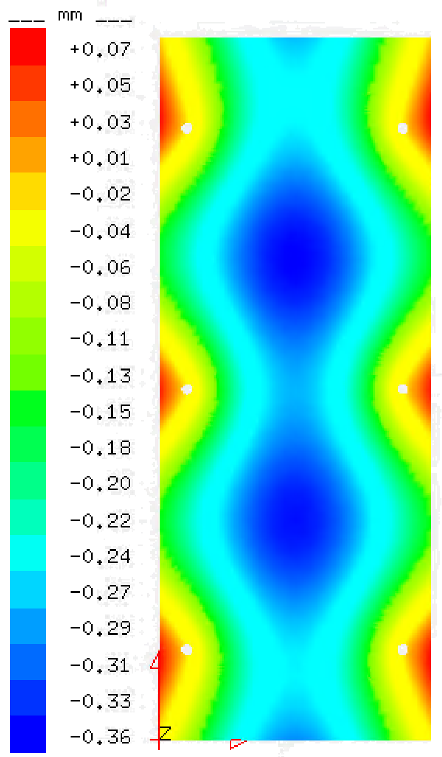
9.5.1. Geometry and Point Support Locations



L1 Deflection Limit = 1140 mm/100 = 11.40 mm
L2 Deflection Limit = 879 mm/100 = 8.79 mm
L3 Deflection Limit = 726 mm/100 = 7.26 mm
L4 Deflection Limit = 2*305 mm/100 = 6.10 mm

9.5.2. Stress and Deflection Results from Load Case 2 – D + Wp





Stress: 10.14 MPa < 7876 psi = 54.3 MPa, OK

Deflection: 0.36 mm < L1 = 11.40 mm, OK

9.5.3. Support Reactions

Axial Rotule Reactions from LC 2, Axial Rotule Reactions from LC3
(Axial Load on Rotule)

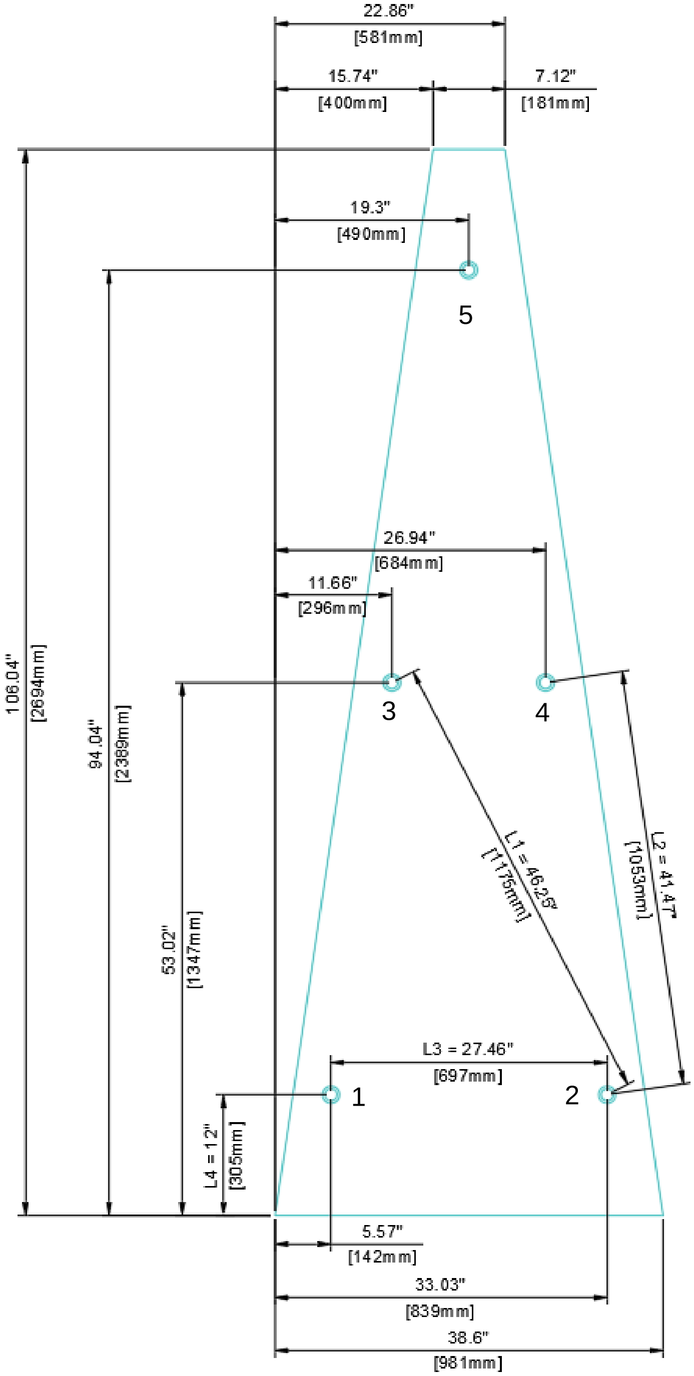
1. 480 N 475 N
2. 481 N 476 N
3. 598 N 600 N
4. 599 N 601 N
5. 476 N 480 N
6. 477 N 481 N

Vertical Panel Point Support Reactions from LC 2, Vertical Panel Point Support Reactions from LC3
(Shear load on Rotule)

1. 97 N 91 N
2. 97 N 91 N
3. 92 N 92 N
4. 92 N 92 N
5. 92 N 96 N
6. 92 N 96 N

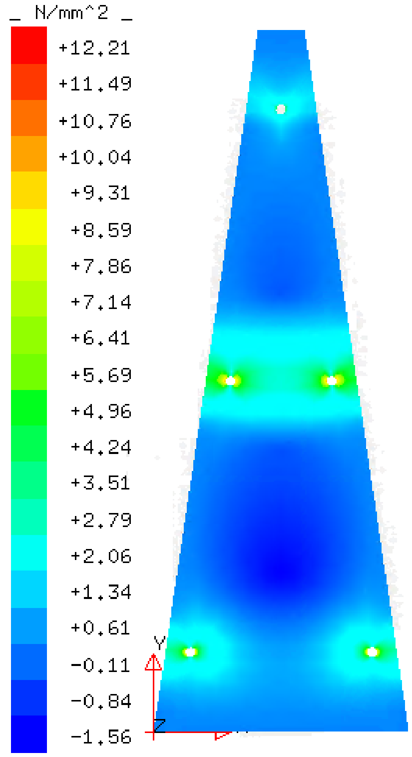
9.6. Glass panel 2: Cone

9.6.1. Geometry and Point Support Locations

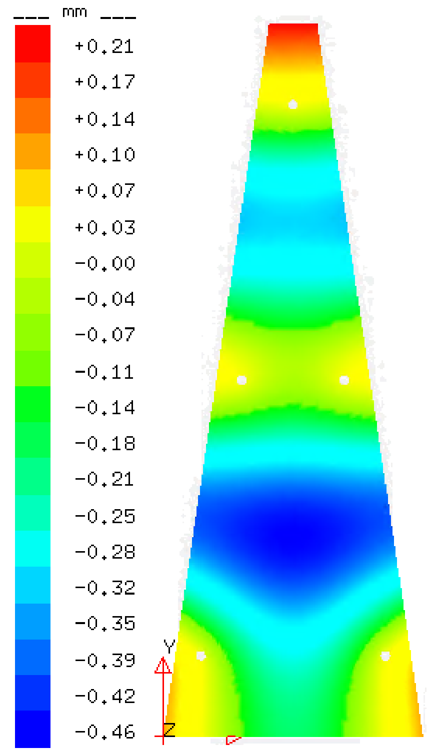


L1 Deflection Limit = 1175 mm/100 = 11.75 mm
L2 Deflection Limit = 1053 mm/100 = 10.53 mm
L3 Deflection Limit = 697 mm/100 = 6.97 mm
L4 Deflection Limit = 2*305 mm/100 = 6.10 mm

9.6.2. Stress and Deflection Results from Load Case 2 – D + Wp



Stress: 12.21 MPa < 7876 psi = 54.3 MPa, OK



Deflection: 0.46 mm < L1 = 11.75 mm, OK

9.6.3. Support Reactions

Axial Rotule Reactions from LC 2, Axial Rotule Reactions from LC3
(Axial Load on Rotule)

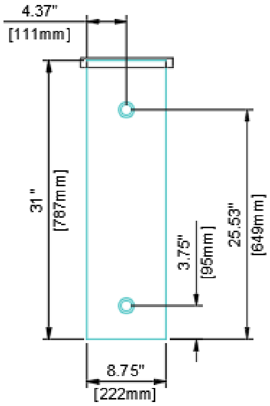
1.	530 N	439 N
2.	530 N	439 N
3.	524 N	439 N
4.	528 N	472 N
5.	330 N	278 N

Vertical Panel Point Support Reactions from LC 2, Vertical Panel Point Support Reactions from LC3
(Shear load on Rotule)

1.	76 N	65 N
2.	76 N	65 N
3.	67 N	73 N
4.	67 N	73 N
5.	66 N	81 N

9.7. Glass panel 3: Upper Drum

9.7.1. Geometry and Point Support Locations



9.7.2. Stress and Deflection Results

Due to size of glass panel, panel will not govern over panel 1 or 2's stresses, deflections, or reactions.

10. Glazing Support/Connection Design

10.1. General Design Criteria

10.1.1. Carbon Steel

A500 Gr. B Steel (Rectangular)	Fy = 46 ksi, min	Fu = 58 ksi, min
A36 (Angle or Plate)	Fy = 36 ksi, min	Fu = 58 ksi, min

10.1.2. Stainless Steel

316 Stainless Steel	Fy = 30 ksi, min	Fu = 70 ksi, min

10.1.3. Fasteners

M14 Stainless Steel Rotule Bolt	Fy = 30 ksi, min	Fu = 75 ksi, min

10.1.4. Point Support Hardware

Countersunk Rotule — Axial Capacity = 2500 N (560 lbf), min
Radial Capacity = 1200 N (270 lbf), min

10.2. Rotule Capacity Check

Loads (ASD)

$P := 601N = 135.1\,lbf$ Max Axial Load from Load Case 3, Lower Drum Panel

$Q := 92N = 20.68\,lbf$ Matching Shear Load from Load Case 3, Lower Drum Panel

#1 Check Rotule Capacity

Countersunk Rotule w/ M14x2, 316 Stainless Steel

Properties:

$d_{b1} := 14 \cdot mm$ $F_{y1} := 30 \cdot ksi$ $F_{u1} := 70ksi$ $N_1 := \frac{1}{2} \cdot \frac{1}{mm} = 12.7 \frac{1}{in}$

$F_{nt1} := \min(0.75 \cdot F_{y1}, 0.40 \cdot F_{u1}) = 22.5 \cdot ksi$ $F_{nv1} := \min(0.75 \cdot F_{y1} \div \sqrt{3}, 0.40 \cdot F_{u1} \div \sqrt{3}) = 12.99 \cdot ksi$

$A_{R1} := 0.7854 \cdot \left(d_{b1} - \frac{1.2269}{N_1}\right)^2 = 0.162\,in^2$ $A_{S1} := 0.7854 \cdot \left(d_{b1} - \frac{0.9743}{N_1}\right)^2 = 0.177\,in^2$

Demand

Bolt Tension $T_1 := P = 135 \cdot lbf$

Bolt Shear $V_1 := Q = 20.68\,lbf$

Capacity

Bolt Axial Capacity $T_{n1} := F_{nt1} \cdot A_{S1} = 3978 \cdot lbf$

Rotule Axial Capacity $T_{pn1} := 2500N = 562 \cdot lbf$

Rotule Radial Capacity $V_{n1} := 1200N = 270 \cdot lbf$

Note: Minimum rotule axial and radial capacities are shown, rotule with larger capacities is OK

Interaction

$$IR1_1 := \frac{T_1}{\min(T_{n1}, T_{pn1})} + \frac{V_1}{V_{n1}} = 0.32$$

CheckASD1 = "OK"

10.3. Plate Bending Check

Loads (ASD)

$P_3 := 601N = 135.1\,lbf$ Max Axial Load from Load Case 3, Lower Drum Panel

$Q_3 := 92N = 20.68\,lbf$ Matching Shear Load from Load Case 3, Lower Drum Panel

#2 Plate Bending Check

1/4" Thick A36 Plate

Properties:

$F_y := 36\cdot ksi$ $F_u := 58ksi$

$t_{pl} := 0.25in$ $h_{pl} := 2in$ $e_2 := 1.75in$

$S_x := \frac{t_{pl}\cdot h_{pl}^2}{6} = 0.17\,in^3$ $S_y := \frac{h_{pl}\cdot t_{pl}^2}{6} = 0.02\,in^3$

$Z_x := \frac{t_{pl}\cdot h_{pl}^2}{4} = 0.25\,in^3$ $Z_y := \frac{h_{pl}\cdot t_{pl}^2}{4} = 0.03\,in^3$

Demand

Strong Axis Bending $M_x := Q_3\cdot e_2 = 0.04\,in\cdot kip$

Weak Axis Bending $M_y := P_3\cdot e_2 = 0.24\,in\cdot kip$

Capacity

Flexural Yielding $M_{nx} := \frac{\min(F_y\cdot Z_x, 1.6\cdot F_y\cdot S_x)}{1.67} = 5.39\,in\cdot kip$

$M_{ny} := \frac{\min(F_y\cdot Z_y, 1.6\cdot F_y\cdot S_y)}{1.67} = 0.67\,in\cdot kip$

Interaction

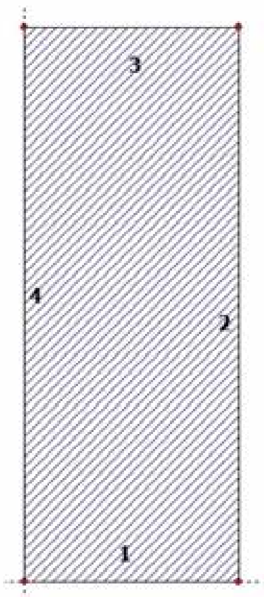
$IR1_3 := \left(\frac{M_x}{M_{nx}}\right) = 0.01$ CheckASD2a = "OK"

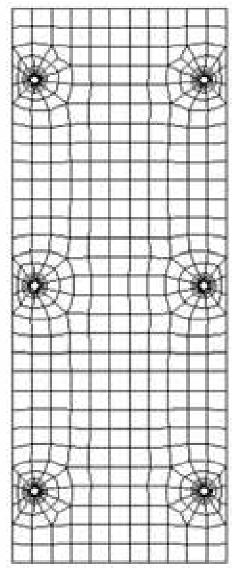
$IR2_3 := \left(\frac{M_y}{M_{ny}}\right) = 0.35$ CheckASD2b = "OK"

11. Appendix

11.1. Mepla FE output

SJ MEPLA Calculation protocol:





Geometry:

Edge	Borderpoint mm	mm	Arccenter mm	mm	Direction of rotation +/-
1	0.00	0.00			
2	917.00	0.00			
3	917.00	2369.00			
4	0.00	2369.00			

Supports:

Spring supports:

Package	Layer	x mm	y mm	z mm	C_x N/mm	C_y N/mm	C_z N/mm	C_φ Nmm	C_θ Nmm
1	1	0.0	0.0	0.0	1.000e+000	1.000e+003	0.000e+000	0.00e+000	0.00e+000
1	1	917.0	0.0	0.0	0.000e+000	1.000e+003	0.000e+000	0.00e+000	0.00e+000

Point fixings:

Position of point fixings:

Reference	------ Position -------- x mm	y mm	z mm	-- Spring rigidities at base point of the point fixing --- C_x N/mm	C_y N/mm	C_z N/mm	C_φ Nmm/rad	C_θ Nmm/rad
1	821.00	2064.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	95.00	2064.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	821.00	1184.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	95.00	1184.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	821.00	305.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	95.00	305.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000

Kind of point fixing:

Reference	Type	Radius Bush mm	Radius Disk mm	E-modul Shim N/mm²	E-modul Bush N/mm²	Thickn. Shim mm	Thickn. Bush mm	Height Cone mm	Radius Cone mm
1	1	18.00	29.50	60.00	500.00	3.00	2.00	5.00	23.00

Point fixing reference:

Reference	Manufactor/Name
1	CountersunkFixing

Force transmittance mechanisms at the bore hole rim:
Point fixing type 1,8: Only the countersunk head is lying in direct contact with the bore hole rim (chamfer)

Contact settings:
All point fixings with contact approaches between bush-glass : tolerance 1.000e-003 [mm]
All point fixings with contact approaches between shim-glass : tolerance 1.000e-003 [mm]

Layers:

Layer order:

Package	Layer	Description
1	3	Fully Tempered Glass
1	2	SGP 40 C° - 3 sec
1	1	Fully Tempered Glass

Mechanical properties:

Package	Layer	E-mod. N/mm²	ν	Thickness mm	Density kg/m³	αt 1/K	ΔT K
1	3	71700.00	0.23	5.56	2550.00	1.0000e-005	0.00
1	2	187.00	0.48	1.52	950.00	1.5000e-005	0.00
1	1	71700.00	0.23	9.02	2550.00	1.0000e-005	0.00

Loads:

Face loads:
- constant distributed:

Package	pressure N/mm²
1	-1.43640e-003

Dead weight:
Inclination of pane: 90.00° degree
Direction vector of gravity acceleration [9.81 m/s²]:

Vx	Vy	Vz
0.00000	-1.00000	0.00000

Calculation approaches:
large deflections, non-linear, (transversal to the plane surface)
static calculation

Characteristics of the finite element mesh:
Element size : 80.0 mm
Number of elements : 917
Number of nodes : 2957 (per package)

Number of unknown : 26589

Calculation results:

Minimum and maximum displacements w:

Package	- Position- x mm	y mm	Displacement w mm
1	458.42	1632.78	-0.36 (min)
	917.00	2123.93	0.07 (max)

Maximum principal stress:

Package	Layer		x mm	y mm	σ N/mm²	σ (max) N/mm²
1	3	(top)	77.37	1189.76	10.14	10.14
		(bottom)	859.58	1140.15	0.59	
1	1	(top)	839.56	1184.08	5.31	5.31
		(bottom)	458.27	1184.34	2.39	

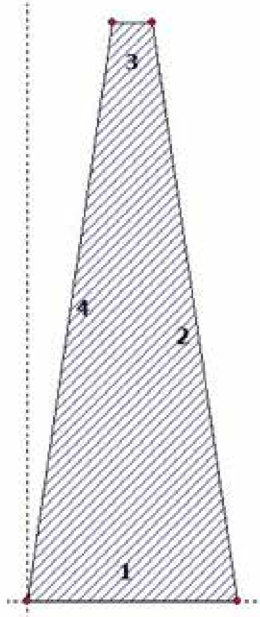
Spring forces and deformations at base point of point fixing:

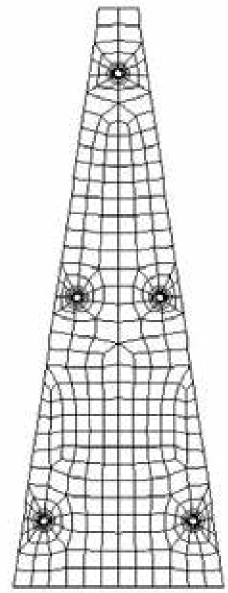
u mm	v mm	w mm	φ rad	θ rad	Fx N	Fy N	Fz N	M_φ Nmm	M_θ Nmm
(x: 821.00 y: 2064.00)									
0.02	-0.09	-0.00	-1.005e-003	-6.080e-004	0.00	-92.63	-477.16	-0.00	-0.00
(x: 95.00 y: 2064.00)									
-0.01	-0.09	-0.00	1.008e-003	-6.078e-004	-0.00	-92.48	-475.86	0.00	-0.00
(x: 821.00 y: 1184.00)									
0.02	-0.09	-0.00	-1.057e-003	-3.536e-004	0.00	-92.14	-599.43	-0.00	-0.00
(x: 95.00 y: 1184.00)									
-0.01	-0.09	-0.00	1.060e-003	-3.531e-004	-0.00	-92.00	-597.75	0.00	-0.00
(x: 821.00 y: 305.00)									
0.02	-0.10	-0.00	-1.017e-003	-2.007e-004	0.00	-96.85	-481.37	-0.00	-0.00
(x: 95.00 y: 305.00)									
-0.01	-0.10	-0.00	1.020e-003	-1.998e-004	-0.00	-96.71	-480.05	0.00	-0.00

Springs:

Package (x / y)	Layer	u mm	v mm	w mm	φ rad	θ rad	Fx N	Fy N	Fz N	M_φ Nmm	M_θ Nmm
(0.00 / 0.00)											
1	1	0.00	-0.13	-0.02	0.0009	-0.0004	0.00	-128.85	-0.00	0.00	-0.00
(917.00 / 0.00)											
1	1	0.01	-0.13	-0.02	-0.0009	-0.0004	0.00	-129.12	-0.00	-0.00	-0.00

SJ MEPLA Calculation protocol:





Geometry:

Edge	Borderpoint mm	mm	Arccenter mm	mm	Direction of rotation +/-
1	0.00	0.00			
2	981.00	0.00			
3	581.00	2694.00			
4	400.00	2694.00			

Supports:

Spring supports:

Package	Layer	x mm	y mm	z mm	C_x N/mm	C_y N/mm	C_z N/mm	C_φ Nmm	C_θ Nmm
1	1	0.0	0.0	0.0	1.000e+000	1.000e+003	0.000e+000	0.00e+000	0.00e+000
1	1	981.0	0.0	0.0	0.000e+000	1.000e+003	0.000e+000	0.00e+000	0.00e+000

Point fixings:

Position of point fixings:

	------ Position --------			-- Spring rigidities at base point of the point fixing ---				
Reference	x mm	y mm	z mm	C_x N/mm	C_y N/mm	C_z N/mm	C_φ Nmm/rad	C_θ Nmm/rad
1	490.00	2389.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	684.00	1347.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	296.00	1347.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	839.00	305.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000
1	142.00	305.00	-5.00	0.000e+000	1.000e+003	1.000e+006	0.000e+000	0.000e+000

Kind of point fixing:

Reference	Type	Radius Bush mm	Radius Disk mm	E-modul Shim N/mm²	E-modul Bush N/mm²	Thickn. Shim mm	Thickn. Bush mm	Height Cone mm	Radius Cone mm
1	1	18.00	29.50	60.00	500.00	3.00	2.00	5.00	23.00

Point fixing reference:

Reference	Manufactor/Name
1	CountersunkFixing

Force transmittance mechanisms at the bore hole rim:
Point fixing type 1,8: Only the countersunk head is lying in direct contact with the bore hole rim (chamfer)

Contact settings:
All point fixings with contact approaches between bush-glass : tolerance 1.000e-003 [mm]
All point fixings with contact approaches between shim-glass : tolerance 1.000e-003 [mm]

Layers:

Layer order:

Package	Layer	Description
1	3	Fully Tempered Glass
1	2	SGP 40 C° - 3 sec
1	1	Fully Tempered Glass

Mechanical properties:

Package	Layer	E-mod. N/mm²	ν	Thickness mm	Density kg/m³	αt 1/K	ΔT K
1	3	71700.00	0.23	5.56	2550.00	1.0000e-005	0.00
1	2	187.00	0.48	1.52	950.00	1.5000e-005	0.00
1	1	71700.00	0.23	9.02	2550.00	1.0000e-005	0.00

Loads:

Face loads:

- constant distributed:

Package	pressure N/mm²
1	-1.43640e-003

Dead weight:

Inclination of pane: 70.00° degree

Direction vector of gravity acceleration [9.81 m/s²]:

Vx	Vy	Vz
0.00000	-0.93974	-0.34190

Calculation approaches:

large deflections, non-linear, (transversal to the plane surface)
static calculation

Characteristics of the finite element mesh:

Element size	: 80.0 mm
Number of elements	: 704
Number of nodes	: 2305 (per package)
Number of unknown	: 20725

Calculation results:

Minimum and maximum displacements w:

Package	- Position- x mm	y mm	Displacement w mm
1	490.49	762.23	-0.46 (min)
	400.00	2694.00	0.21 (max)

Maximum principal stress:

Package	Layer		x mm	y mm	σ N/mm²	σ (max) N/mm²
1	3	(top)	702.48	1348.61	12.21	12.21
		(bottom)	277.52	1348.63	1.91	
1	1	(top)	701.28	1353.79	4.78	4.78
		(bottom)	125.13	800.68	2.59	

Spring forces and deformations at base point of point fixing:

u mm	v mm	w mm	φ rad	θ rad	Fx N	Fy N	Fz N	M_φ Nmm	M_θ Nmm
(x: 490.00 y: 2389.00)									
0.00	-0.07	-0.00	1.200e-005	-1.237e-003	0.00	-65.98	-330.42	0.00	-0.00
(x: 684.00 y: 1347.00)									
0.01	-0.07	-0.00	-3.273e-004	-5.701e-004	0.00	-67.08	-527.71	-0.00	-0.00
(x: 296.00 y: 1347.00)									
-0.00	-0.07	-0.00	3.310e-004	-5.724e-004	-0.00	-67.09	-524.09	0.00	-0.00
(x: 839.00 y: 305.00)									
0.02	-0.08	-0.00	-9.817e-004	2.458e-004	0.00	-76.19	-530.41	-0.00	0.00
(x: 142.00 y: 305.00)									
-0.01	-0.08	-0.00	9.817e-004	2.463e-004	-0.00	-76.11	-530.44	0.00	0.00

Springs:

Package (x / y)	Layer	u mm	v mm	w mm	φ rad	θ rad	Fx N	Fy N	Fz N	M_φ Nmm	M_θ Nmm
(0.00 / 0.00)											
1	1	0.00	-0.10	0.11	0.0009	-0.0001	0.00	-101.44	0.00	0.00	-0.00
(981.00 / 0.00)											
1	1	0.01	-0.10	0.11	-0.0009	-0.0001	0.00	-101.59	0.00	-0.00	-0.00

12. Disclaimers

1.) This report contains only preliminary engineering for other's use in performing a feasibility study for the lighthouse structure and glazing. The full responsibility for the design, engineering and fulfillment of the performance specifications will be with the glazing contractor per delegated design. The glazing contractor's engineering report for the steel structure, anchors, and glass has to be written, signed and sealed by a professional engineer licensed in the State of New York.

2.) This report includes a review of the structural performance of the framing system only, which includes strength and deflection calculations per applicable building codes. Stutzki Engineering is not responsible for final engineering, air and water testing, noise, vibrations, warranties, etc.

3.) All dissimilar metals should be separated and/or protected from water and humidity. Detailing responsibility for this condition is by others.

4.) It is critical that the existing structure will be upgraded to such quality that it can support the lantern structure.

5.) All laminated glass and fully tempered monolithic glass must be considered Safety Glazing per applicable building codes. Compliance with this requirement is to be done by the glass manufacture.

6.) U-value and condensation resistance is not within the scope of this document. If it is within the scope of Stutzki Engineering, Inc., it will be submitted separately.

7.) Testing, certification, and/or calculations for air and water penetration were not conducted nor reviewed by Stutzki Engineering, Inc. If required by the project specification, air and water penetration tests are to be coordinated by others.