8/15/2022

Geodesic Dome Structure, Colorado
12210 CO-14
Kremmling, CO 80459
RE: Engineer's Analysis and Opinion Report

To Whom It May Concern,

PURPOSE & SCOPE OF REPORT:

The project goal is to investigate the requisite stability and strength for a geodesic dome structure located at the above-referenced address. The client, Jeff Woodward from Blue Bird Backcountry provided the Dome's geometry. Potential cases of loads are subjected to the Dome while examining the structural strength in compliance with US standards and codes. See Figure 1 for the position of the new Dome.

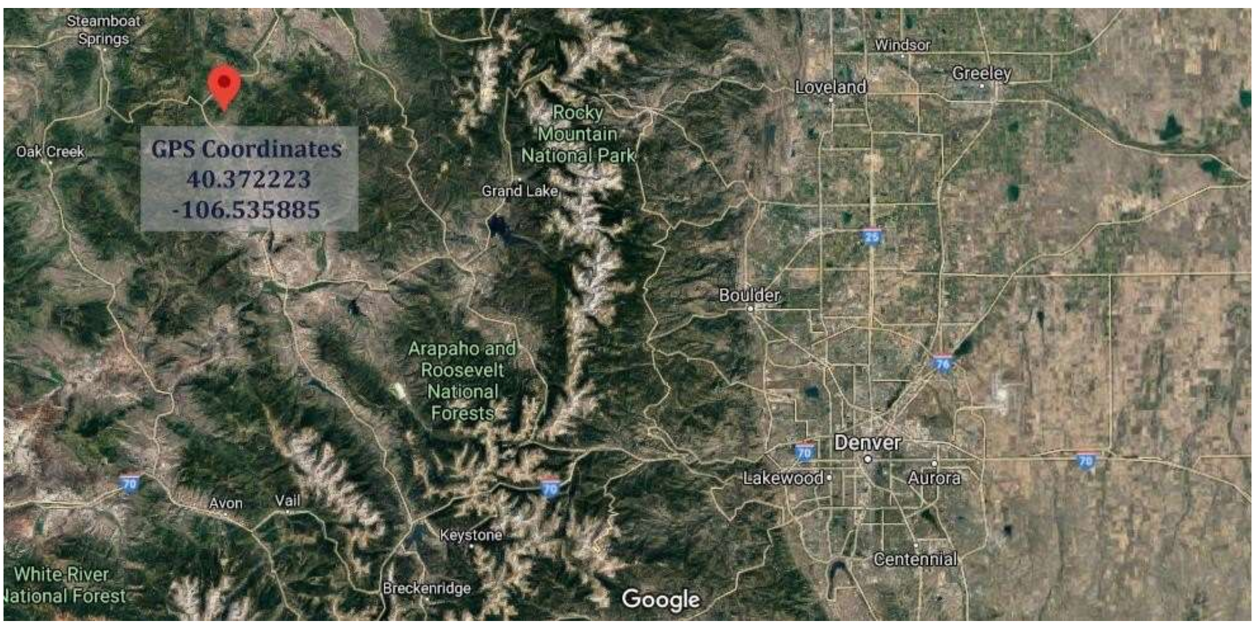


Figure 1 Position of the dome structure

Dome Geometry:

Figures 2 and 3 depict the dome's geometry. As displayed, the dome's diameter is 33 feet (10 meters), and its height is 16.5 feet (5 meters).

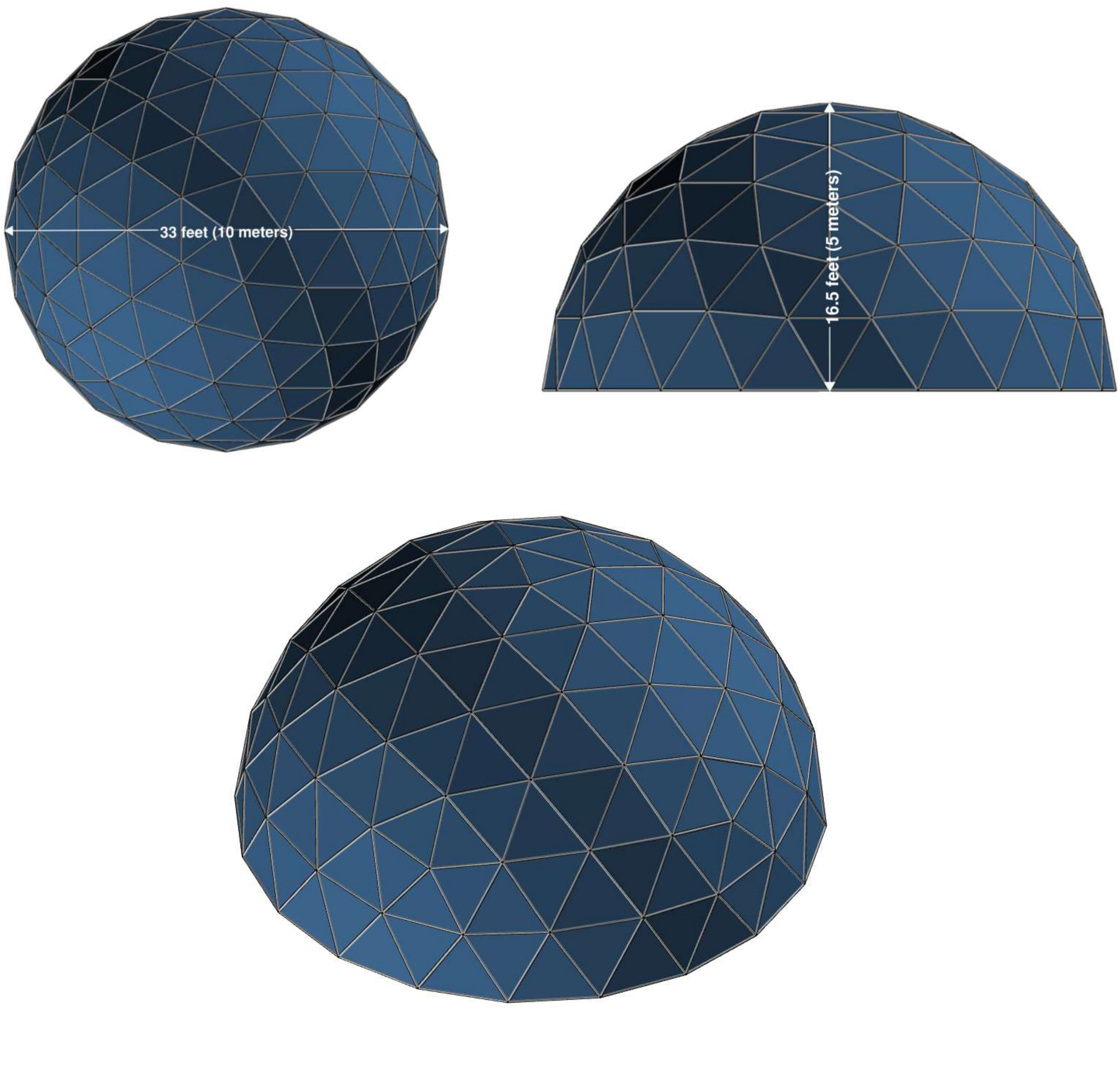


Figure 2 Geodesic Dome Geometry

Design Codes:

The requisite stability and strength of the dome structure were examined in accordance with the following guidelines.

Item	*Standard/Reference*
International Building Code	IBC2018
Minimum Design Loads	ASCE7-16
Steel Structure Design	AISC360-16
2016 Colorado Design Snow Loads	

Units:

The following is a list of the units used in this study.

Item	*Units*
Length	in. (inches), ft. (foot) , mm(millimeter)
Mass	lb. (Pound)
Force	lbf. (Pound Force), Kips (Kilo Pound Force)
Moment	Kip.in , Kip.ft

Materials:

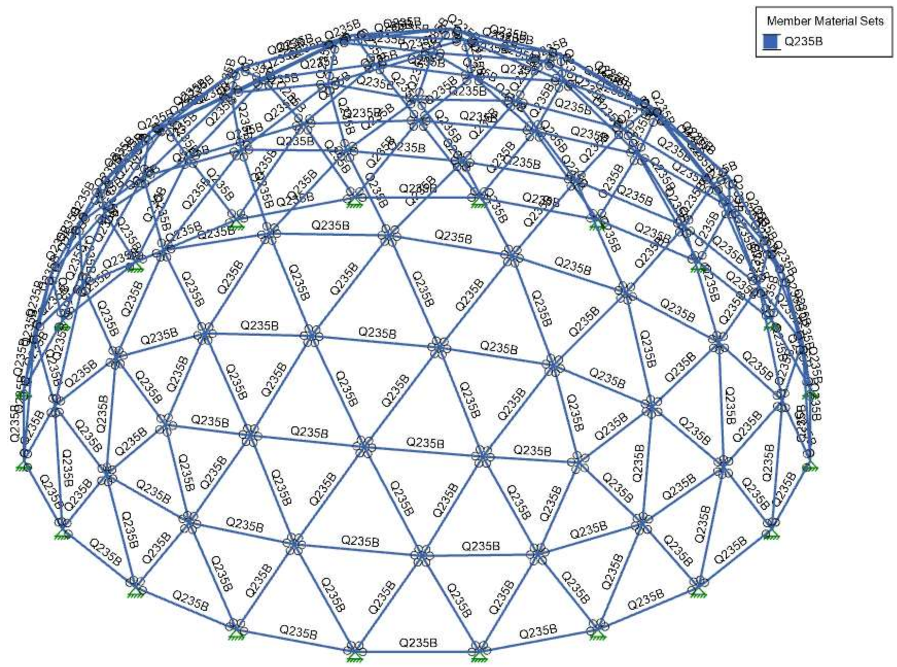
In accordance with Chinese standard GB/T 700, space frames are made of hot-rolled steel pipes that are Q235B-compliant. The table below displays the Q235B material's specifications.

Item	*Chemical composition*	*Yielding stress*	*Ultimate stress*
Q235B	C (0.1%); Mn(0.36%); Si(0.11%); P(0.02%); S(0.021%)	32.6 Ksi	55.8 Ksi

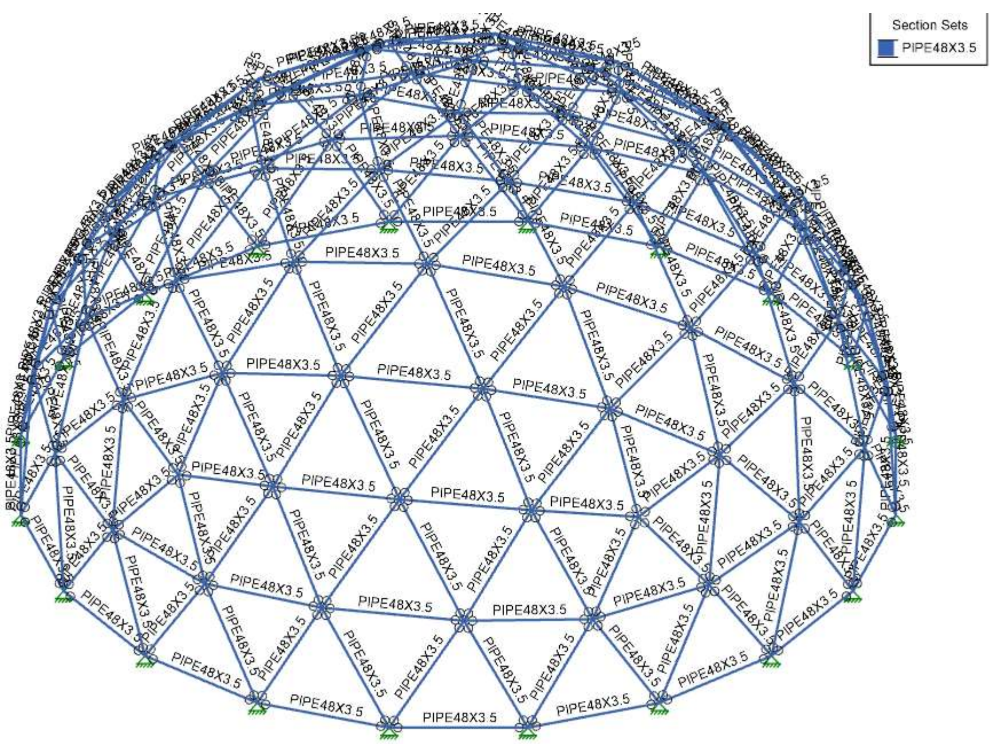
Bolts are made of hot-dip galvanized iron, grade B carbon steel, with a 120ksi ultimate stress. Double PVC-coated polyester and transparent PVC are used to make the roof covering. Base plate yield strength is 32.6 ksi.

Modeling

Risa 3d software was used to model the structure. Pinched frames were used to simulate the space frames, while zero-stiffness shell components were used to model the roof covering. The shell components are not supposed to have a structural response.



Materials Modeling



Section Size Modeling

Dead Load

Dead loads comprise the weight of the roof cover and the frames. The software model automatically takes the weight of the framing into account. The roof cover is evenly distributed across the shells in the direction of gravity.

Snow Load

IBC-2018 Figure 1608.2 specifies the ground snow loads Pg for the 50-year mean recurrence interval (MRI). The 2016 *Colorado Design Snow Loads* provide more specific information on the ground snow loads. With reference to Figure 1.1.b, K is calculated to be equal to 18.

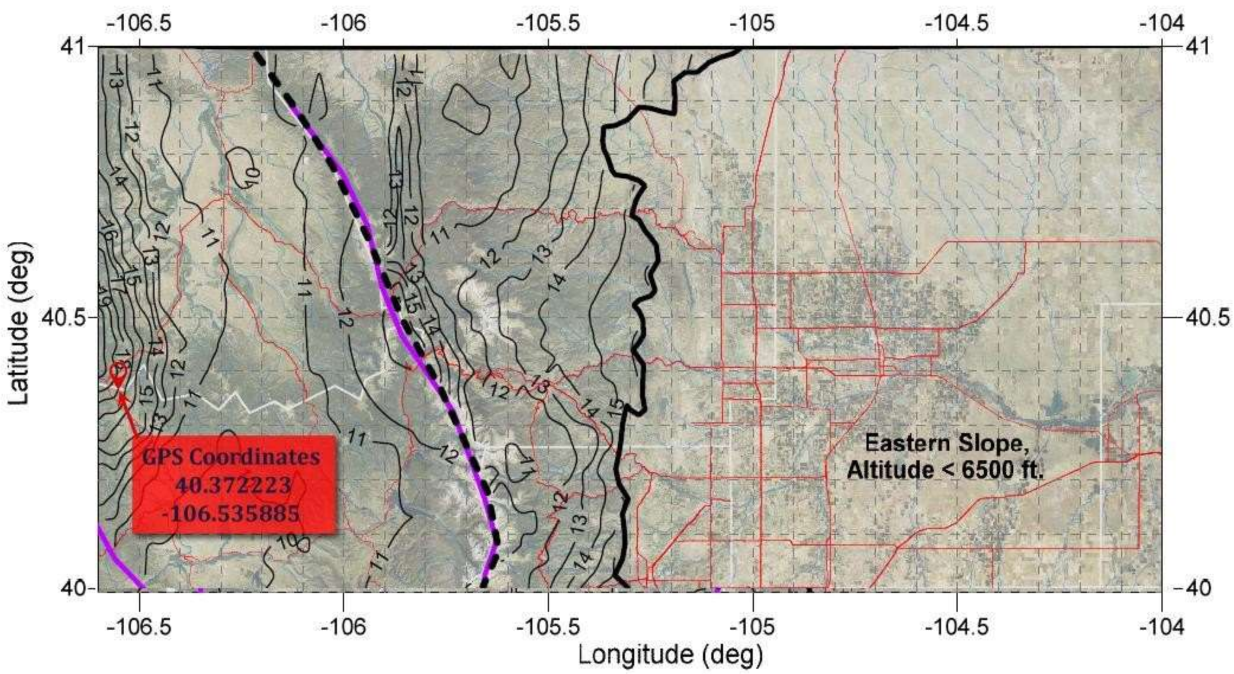


Figure 3 parameter map for calculating ground snow loads in North Central Colorado

The ground snow load is calculated using the equation below, where A represents altitude in thousands of feet, and K is taken directly from the map.

$$Pg = \max\left(\frac{k}{100}xA^3, 25\right)$$

k = 18

A = 8945 feet

Pg = max$\left(\frac{18}{100}x8.945^3, 25\right) = 128.7\ psf$ = 6.17 $\frac{kN}{m^2}$

Sections 7.3 to 7-6 of ASCE7-16 include the guidelines for calculating snow loads on structures. Snow loading may be classified into two categories: balanced and unbalanced, as shown in Figure 7.4-2 ASCE7-16.

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.10

Building details

Roof type	Curved
Width of roof	$b = \mathbf{10.00}$ m
Angle of roof at eaves level	$\alpha_{eaves} = \mathbf{90.00}$ deg
Dist from eaves to where c_s equals 1.0	$L_{cs1} = \mathbf{3.71}$ m
Dist from eaves to 30deg point	$L_{30} = \mathbf{2.50}$ m
Dist from eaves to 70deg point	$L_{70} = \mathbf{0.30}$ m

Ground snow load

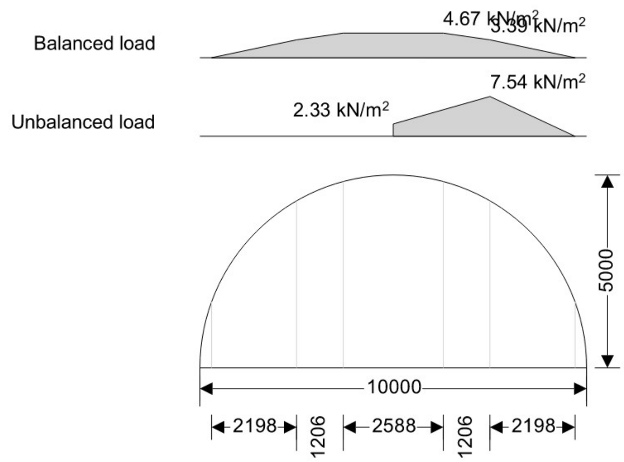
Ground snow load (Figure 7.2-1)	$p_g = \mathbf{6.17}$ kN/m^2
Density of snow	$\gamma = \min(0.426 \times p_g / 1\text{m} + 2.2\text{kN/m}^3, 4.7\text{kN/m}^3) = \mathbf{4.70}$ kN/m^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Fully exposed
Exposure factor (Table 7.3-1)	$C_e = \mathbf{0.90}$
Thermal condition (Table 7.3-2)	Unheated structures
Thermal factor (Table 7.3-2)	$C_t = \mathbf{1.20}$
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	$I_s = \mathbf{1.00}$
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = \mathbf{4.67}$ kN/m^2

Cold roof slope factor (C_t > 1.0)

Roof surface type	Slippery
Ventilation	Ventilated
Thermal resistance (R-value)	$R = \mathbf{4.00}$ °C m^2 / W
Roof slope factor Fig 7.4-1c (dashed line)	$C_s = \mathbf{0.73}$

Curved roof loads

Roof slope factor at eaves (Figure 7.4-1)	$C_s^* = \mathbf{0.00}$
Roof slope factor at 30° point (Figure 7.4-1)	$C_s^{**} = \mathbf{0.73}$
Balanced load at crown	$p_{s\_bal\_cwn} = p_f = \mathbf{4.67}$ kN/m^2
Balanced load at eaves	$p_{s\_bal\_eav} = p_f \times C_s^* = \mathbf{0.00}$ kN/m^2
Balanced load at 30° point	$p_{s\_bal\_30} = p_f \times C_s^{**} = \mathbf{3.39}$ kN/m^2
Balanced load at 70° point	$p_{s\_bal\_70} = 0$ lb/ft^2 $= \mathbf{0.00}$ kN/m^2
Unbalanced load at crown	$p_{s\_unbal\_cwn} = 0.5 \times p_f = \mathbf{2.33}$ kN/m^2
Unbalanced load at eaves	$p_{s\_unbal\_eav} = 2 \times p_f \times C_s^* / C_e = \mathbf{0.00}$ kN/m^2
Unbalanced load at 30° point	$p_{s\_unbal\_30} = 2 \times p_f \times C_s^{**} / C_e = \mathbf{7.54}$ kN/m^2
Unbalanced load at 70° point	$p_{s\_unbal\_70} = \mathbf{0.00}$ kN/m^2



Roof elevation

The category of C for surface roughness and a fully exposed roof were taken into consideration while determining the Ce factor. The exposure factor is therefore equal to 0.9., the thermal factor, Ct, was set to 1.2 for the worst circumstances of the dome when it is not occupied and unheated. The structure's importance factor equals one for risk category II. Snow loads were applied directly to framing members as linear loads as shown in the figures below.

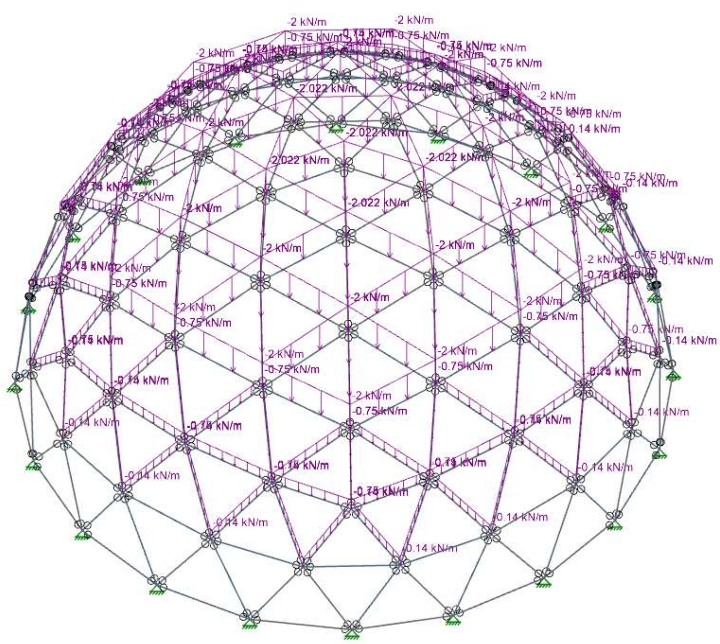


Figure 4 Balanced snow load

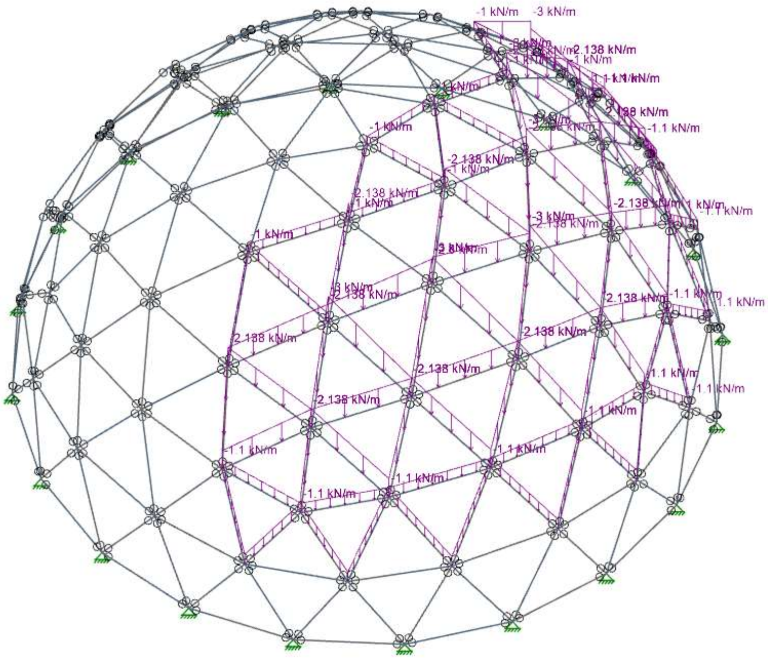


Figure 5 Unbalanced snow load

Wind Load

Due to the geometrical shape of the dome and the nature of the required load combinations in the ASCE 7-16, wind loads were not considered to check the local strength of the dome's structure, as balanced and unbalanced snow loads would control this check. However, wind loads were considered in checking the global stability of the dome using global calculations.

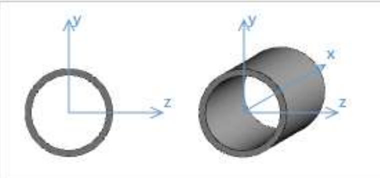
Load Combinations

LRFD load combinations were used to perform this analysis. These combinations are summarized in the table below:

Load Case	Dead Load	Balanced Snow Load	Unbalanced Snow Load
Load Combinations			
Deflection 1	1	-	-
Deflection 2	-	1	-
Deflection 3	-	-	1
Deflection 4	1	1	-
Deflection 5	1	-	1
IBC 16-1	1.4	-	-
IBC 16-3 (c)	1.2	1.6	-
IBC 16-3 (c)	1.2	-	1.6

Analysis Results

Each frame's portion is Pipe48x3.5mm and has the following properties.



Input Data:

Shape:	PIPE48X3.5	I Node:	N102
Member Type:	Beam	J Node:	N160
Length (m):	1.564	I Release:	BenPIN
Material Type:	Hot Rolled Steel	J Release:	BenPIN
Design Rule:	R1	I Offset (mm):	N/A
Number of Internal Sections:	97	J Offset (mm):	N/A

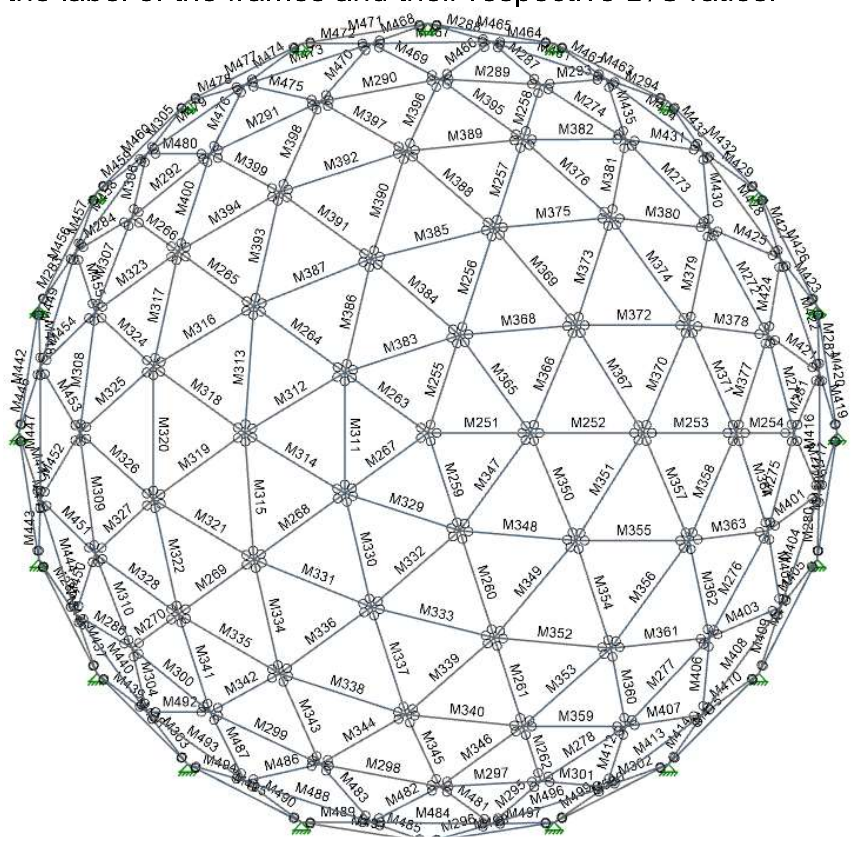
Material Properties:

Material:	Q235B	Therm. Coeff. ($1e^{5}$°C^{-1}):	1.17	R_y:	1.5
E (MPa):	1.999e+5	Density (kg/m^3):	7849.047	F_u (MPa):	400
G (MPa):	76904.123	F_y (MPa):	250	R_t:	1.2
Nu:	0.3				

Shape Properties:

d (mm):	48	Area (mm^2):	489.303	I_{yy} (mm^4):	7.437
t (mm):	3.5	J (mm^4):	2.437e+5	I_{zz} (mm^4):	1.219e+5
Z (mm^3):	6945.167				

The frame design and verification followed the AISC360-16 standard. The figures below display the label of the frames and their respective D/C ratios.



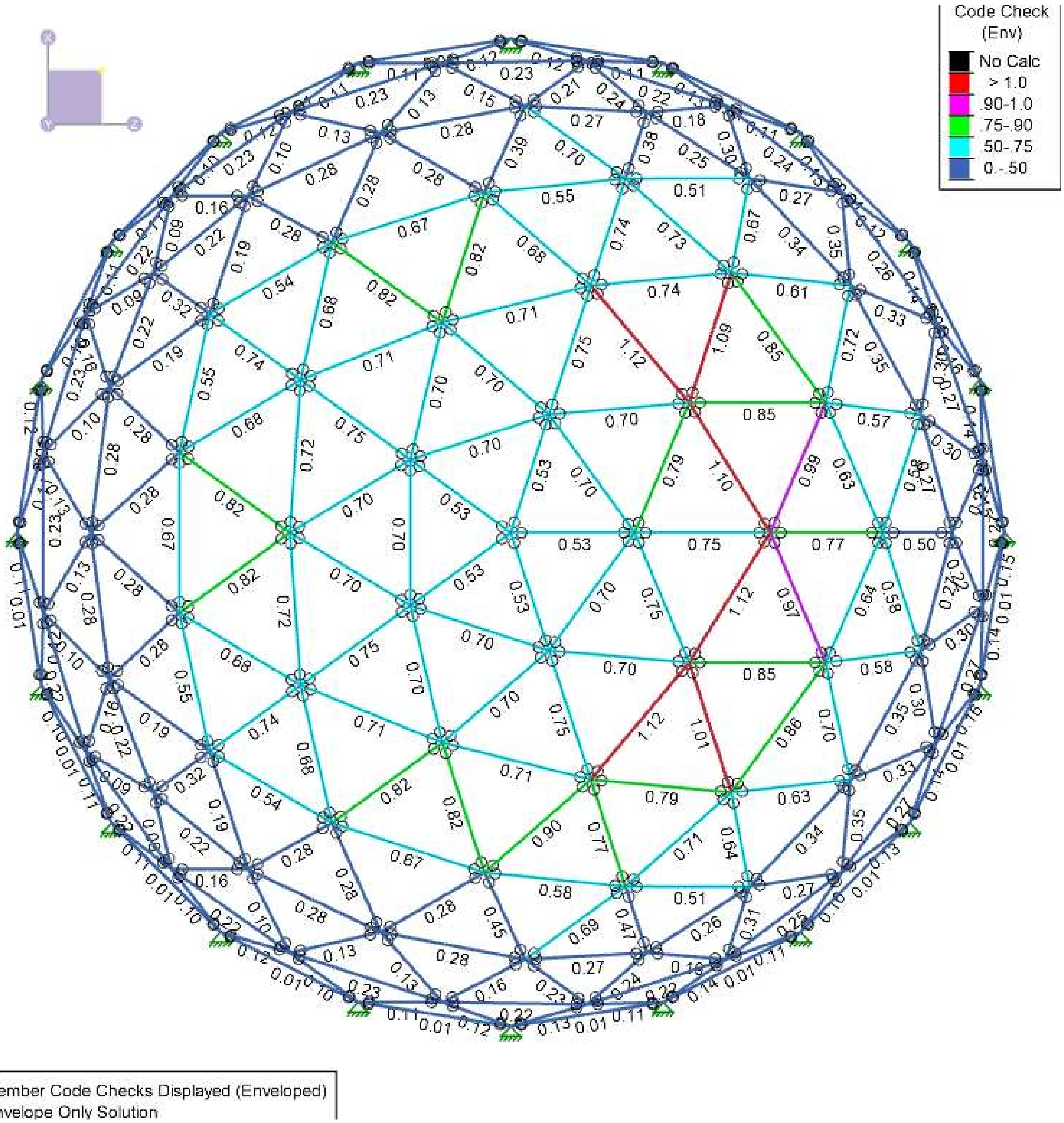


Figure 7 Envelope bending code check

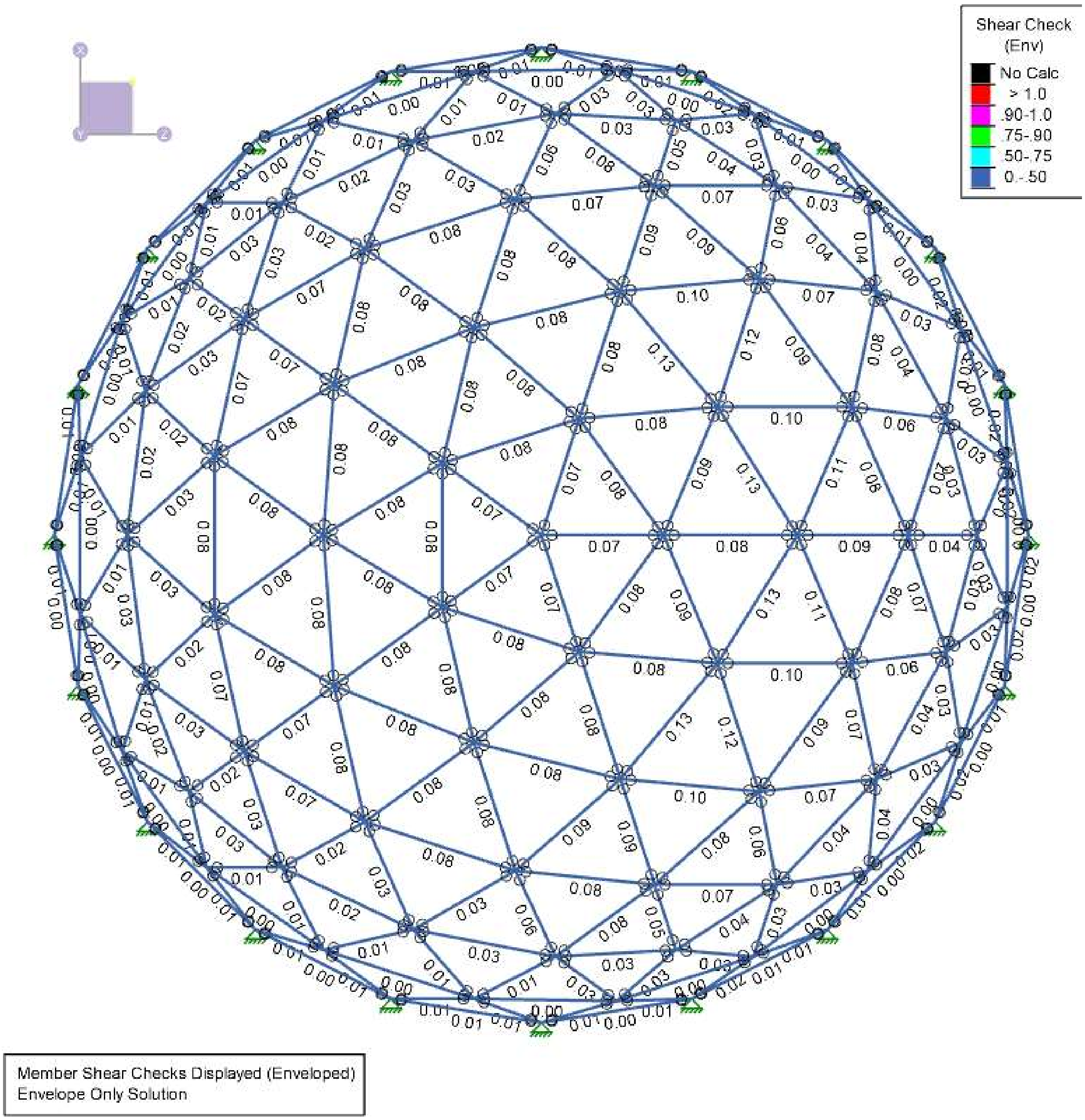


Figure 8 Envelope shear code check

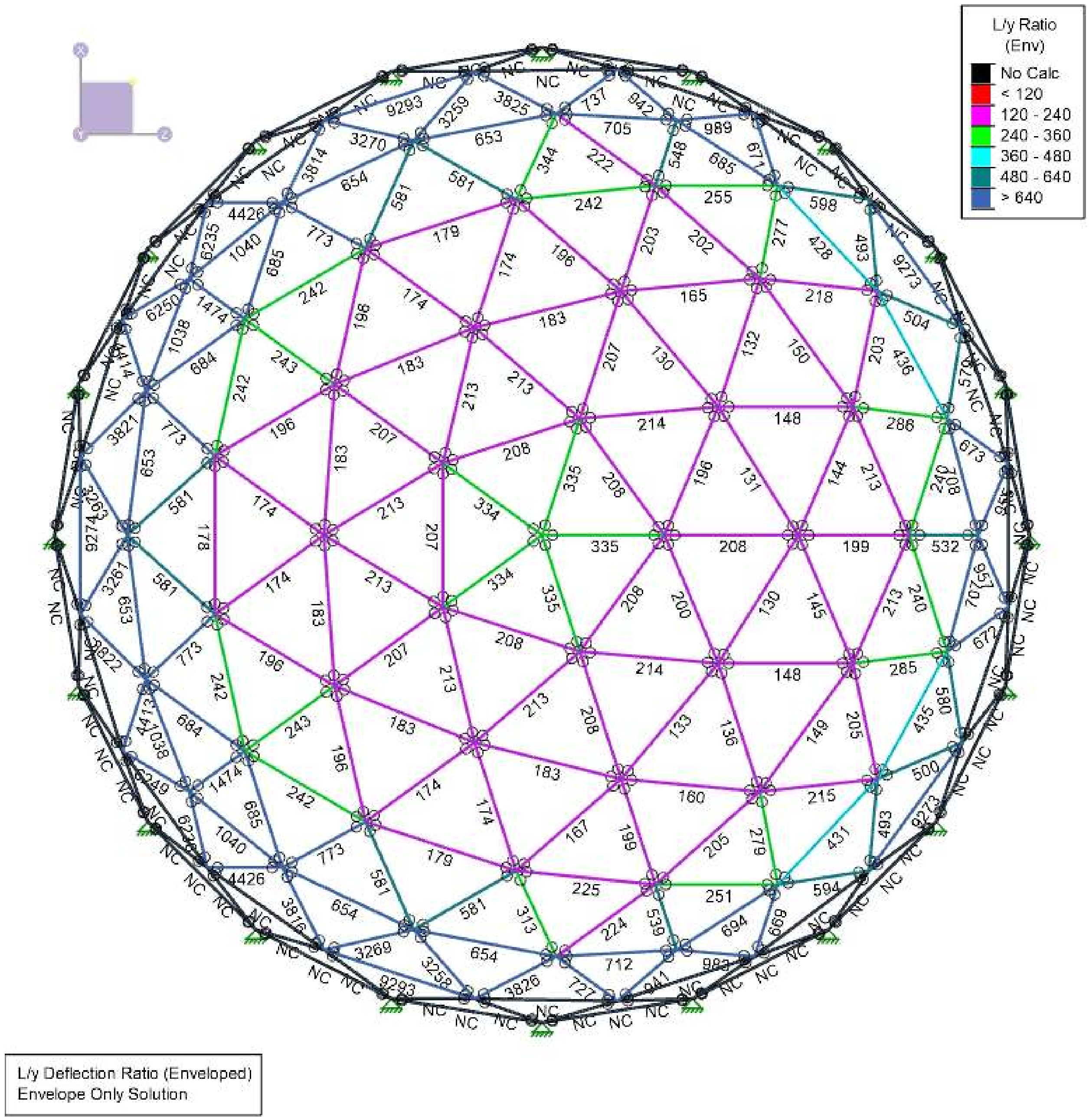


Figure 9 Envelope deflection code check

Frames Connection

The connection between frames is conducted using screw bolts as shown in the below figures.

Figure 10 Typical pin connection

The maximum shear force acting on the crucial bolt was found from the analysis to be 29 kips due to circumferential tensile stress. Section J3.6 of AISC360-16 states that the shear strength of bolts is determined as follows.

$$\Phi V_n = \Phi . A_b . F_{nv}$$

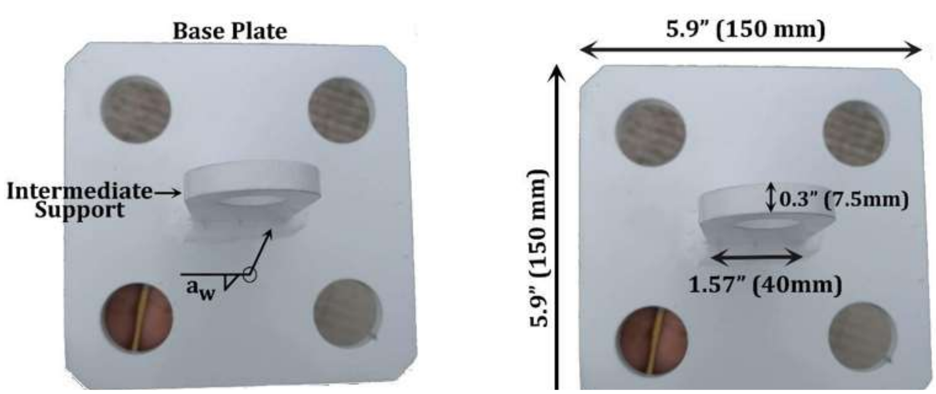
$F_{nv} = 54\ ksi$

$A_b = \frac{\pi}{4} x (1.0")^2 = 0.79\ in.^2$

$\Phi V_n = 0.75\ x\ 42.66\ kips = 32\ kips > 29\ kips \rightarrow Satisfactory$

Base Plate Connection to Ground

The support loads are transmitted to the ground using 20 base plates. The dimensions of the figure are provided below.



COLUMN BASE PLATE DESIGN

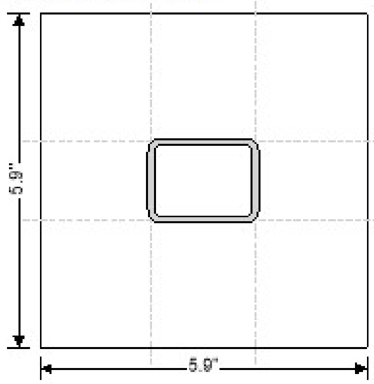
In accordance with AISC Steel Design Guide 1 and AISC 360-16

Tedds calculation version 2.1.07

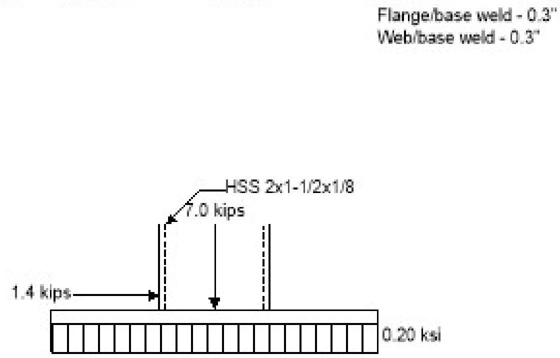
Design summary

Overall design status PASS

Description	Unit	Capacity	Applied	Utilization	Result
Plate thickness	(in)	0.249	0.250	0.997	PASS
Bearing strength	(kips)	7.00	115.40	0.061	PASS
Flange weld strength	(kips/in)	0.000	10.441	0.000	PASS
Shear weld strength	(kips/in)	0.382	6.961	0.055	PASS
Frictional shear resistance	(kips)	1.35	2.10	0.643	PASS



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force P_u = **7.0** kips (Compression)

Bending moment M_u = **0.0** kip_in

Shear force F_v = **1.4** kips

Column details

Depth d = **2.000** in

Breadth b_f = **1.500** in

Thickness t = **0.116** in

Baseplate details

Depth N = **5.900** in

Breadth B = **5.900** in

Thickness t_p = **0.250** in

Design strength F_y = **36.0** ksi

Strength reduction factors

Compression	ϕ_c = **0.65**
Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**

Plate cantilever dimensions

Minimum distance to edge of concrete	$l_{min} = \min(\min(x_{ce1}, x_{ce2}) - N / 2, \min(y_{ce1}, y_{ce2}) - B / 2)$ = **27.050** in
Area of base plate	$A_1 = B \times N$ = **34.810** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min})^2 \times (B / N)$ = **3600.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **177.5** kips
Bending line cantilever distance m	$m = (N - 0.95 \times d) / 2$ = **2.000** in
Bending line cantilever distance n	$n = (B - 0.95 \times b_f) / 2$ = **2.238** in
Maximum bending line cantilever	$l = \max(m, n)$ = **2.238** in

Plate thickness

Required plate thickness	$t_{p,req} = l \times \sqrt{((2 \times P_u) / (\phi_b \times F_y \times B \times N))}$ = **0.249** in
Specified plate thickness	t_p = **0.250** in

PASS - Thickness of plate exceeds required thickness

Design bearing strength (AISC 360-05-J8)

Design bearing strength	P_p = **177.53** kips
Factored bearing strength	$\phi_c P_p$ = **115.40** kips

PASS - Allowable bearing stress exceeds applied bearing stress

Frictional shear resistance

Steel / concrete friction coefficient	μ = **0.4**
Frictional shear resistance	$\phi V_n = \min(\phi_v \times \mu \times P_u, \phi_v \times 0.2 \times f'_c \times A_1, \phi_v \times 800psi \times A_1)$ = **2.10** kips

PASS - Frictional shear resistance exceeds applied shear

Flange weld

Flange weld leg length	t_{wf} = **0.3125** in
Tension capacity of flange	$P_{tf} = b_f \times t \times F_{yCol}$ = **8.7** kips
Force in tension flange	$F_{tf} = M_u / (d - t) - P_u \times (b_f \times t) / A_{col}$ = **-1.7** kips
Critical force in flange	$F_f = \min(P_{tf}, \max(F_{tf}, 0kips))$ = **0.0** kips
Flange weld force per in	$R_{wf} = F_f / b_f$ = **0.0** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5})$ = **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_{nw} \times t_{wf} / \sqrt{(2)}$ = **10.4** kips/in

PASS - Available strength of flange weld exceeds force in flange weld

Shear weld

Shear web weld leg length	t_{ww} = **0.3125** in
Shear web weld force per in	$R_{wl} = F_v / (2 \times (d - 2 \times t))$ = **0.382** kips/in

Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(0deg))^{1.5})$ = **31.500**ksi
Design strength of weld per in	$\phi R_{nl} = \phi F_{nw} \times t_{ww} / \sqrt{(2)}$ = **7.0** kips/in
	PASS - Available strength of shear weld exceeds force in shear weld

Ground Anchors

The base plates are secured to the earth using duckbill earth anchors. Each base plate shall be anchored to the earth using four Duckbill anchors. Each anchor shall have an uplift force of at least 155 lbs. The working load capacity of Duckbill anchors in typical soils is displayed in the table below.

Duckbill Model	Recommended Working Load in Normal Soils	Wire Rope Capacity	Standard Installation Depth
40	300 lbs	480 lbs	20 in
68	1100 lbs	1700 lbs	30 in

OPINIONS AND Recommendations:

In our opinion and based on the analysis results presented in this report, framing members of the structure located directly below the peak unbalanced snow load were over-stressed in bending and as shown in figure 7. These members are located at nearly ±2.5 meters from the center of the dome. Below is a list of our recommended solutions for the over-stressed members:

1) Keeping the structure heated when it is not occupied during a severe snow event so that the Thermal factor Ct can be kept close to 1.0. This would bring the D/C ratio to 0.9 (max.) as shown in the figure 12. It is the building owner's responsibility to satisfy this requirement.

2) If the structure cannot be kept heated during a severe snow event, we recommend using a pipe size of O.D.= 48 mm and thickness = 4 mm directly under the peak unbalanced snow load which is located within a circle with a radius of ±2.5 m from the center of the dome.

3) We recommend using a 1" Φ bolt with Fy = 54 ksi between all connected pipes of the dome.

4) Per section 7.2 of the ASCE 7-16, the extreme value of snow load is based on a 2% annual probability of occurrence. Therefore, if the building owner wants to use the dome in an unheated condition, there is a 2% chance that the over-stress in dome members in figure 7 would occur.

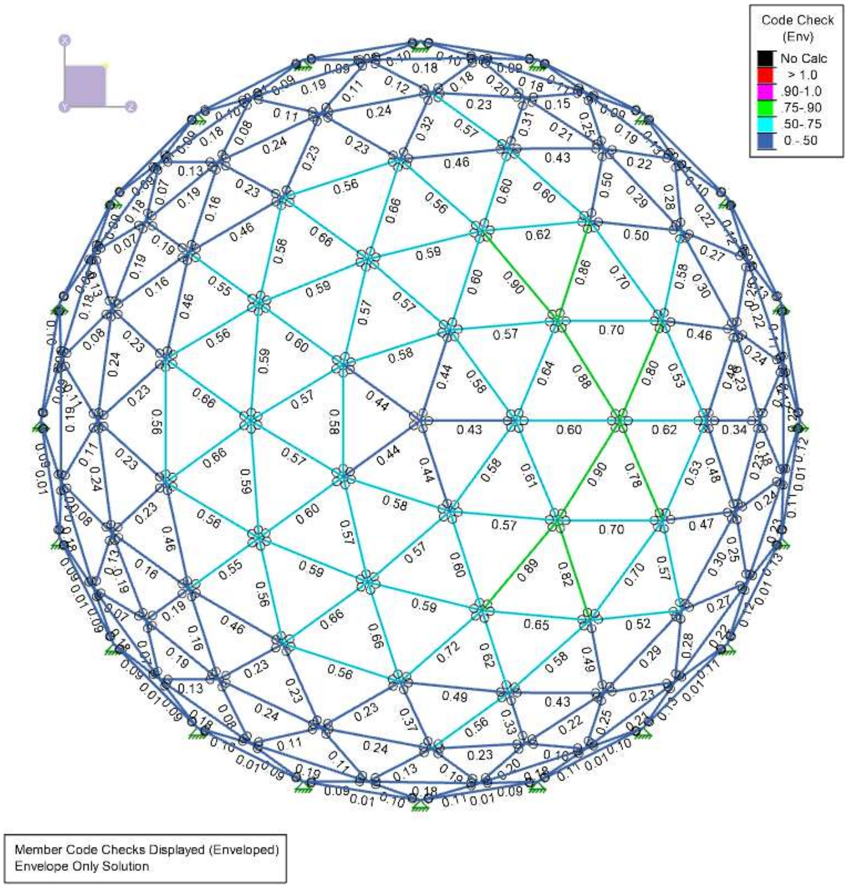


Figure 12 D/C ratio of structure with Ct = 1.0

If you have any questions or concerns regarding our findings, please contact us.

Sincerely,

Mustafa H.F. Alsaid, P.E.