Calculation Report
Geodesic Dome Structure, Colorado

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1. Definition

1.1 General Information

The objective of this project is the investigation of required strength and stability for a geodesic dome structure located in the hillside of Bear Mountain, Jackson County, Colorado. The geometry of dome has been determined by client. The dome is loaded under probable load types and the structural requirements are investigated accordance the US standards/codes. Figures 1 shows the location of this new dome structure.

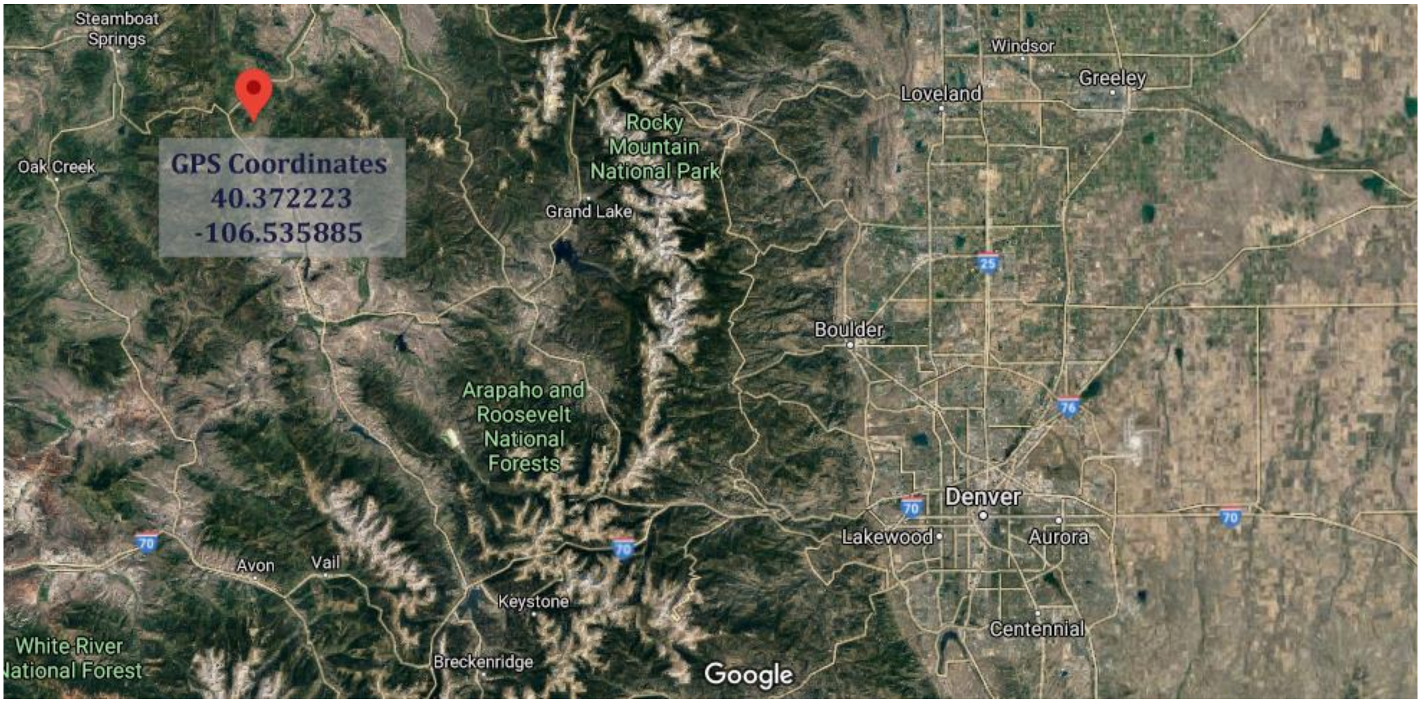


Figure 1, Location of dome structure

The geometry of dome is shown in Figure 2 and 3. As shown, the diameter of dome is 23 feet and its height is 11.5 feet.

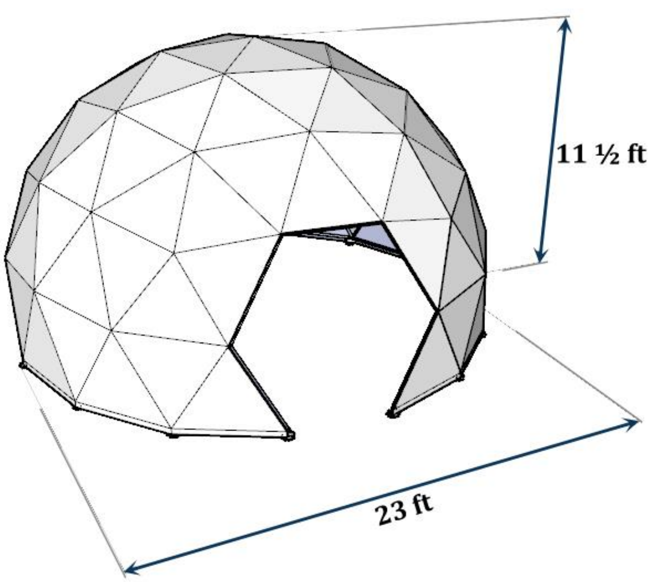


Figure 2, 3D View of geodesic dome

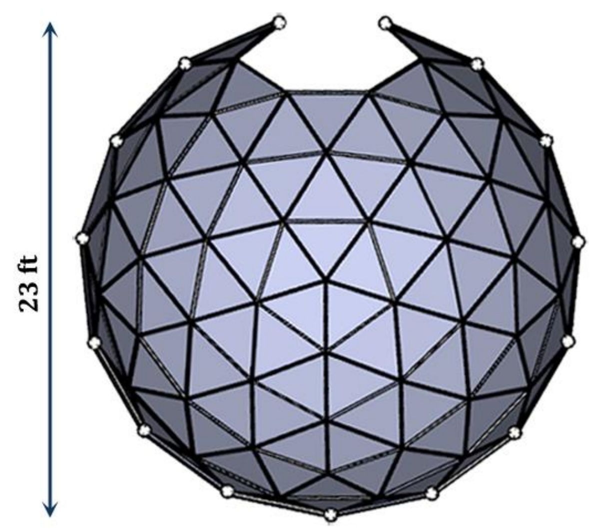


Figure 3, Bottom view of geodesic dome

1.2 Design Standards and Units

The required strength and stability of dome structure were checked according to the below provisions.

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

The units were used in this report were listed below.

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

1.3 Materials

Space frames are hot-rolled steel pipes and conform to Q235 material in Chinese standard GB/T 700. The specification of Q235 material is shown in below table.

Item	*Chemical composition*	*Yielding stress*	*Ultimate stress*
Q235	C (0.1%); Mn(0.36%); Si(0.11%); P(0.02%); S(0.021%)	32.6 Ksi	55.8 Ksi

The material of bolts is Hot-dip galvanized iron, grade B carbon steel with the ultimate stress of 120ksi. Roof cover is made by double PVC coated polyester and clear PVC. The yield strength of base plates is 32.6 ksi.

2. Modeling

2.1 Geometry

The structure was modeled in SAP2000 software. The space frames were modeled using pinned frames and the roof cover was modeled by zero-stiffness shell elements. The shell elements only used to distribute the area loads on the frames and are not expected to have the structural behavior.

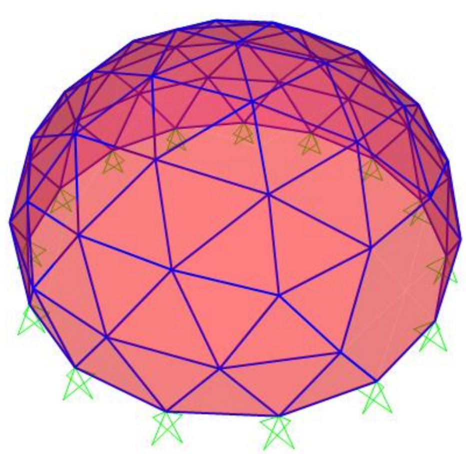


Figure 4, Modeled space frames and roof cover

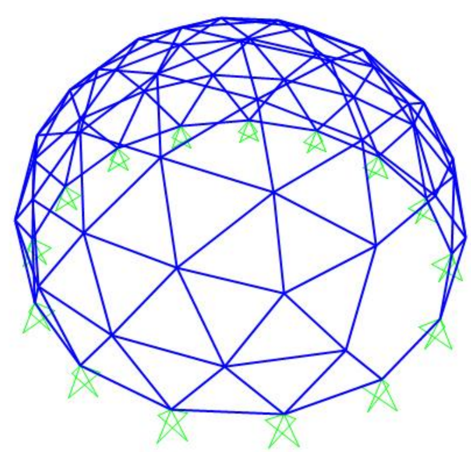


Figure 5, Modeled space frames

3. Loading

3.1 Dead Load

Dead loads include the weight of frames and roof cover. The weight of frame is considered automatically in software model. The total weight of roof cover is 185 lbf and is uniformly applied on the shells in gravity direction.

3.2 Snow Load

The 50-year mean recurrence interval (MRI) ground snow loads P_g are specified in IBC Figure 1608.2. The ground snow loads are specified in greater detail in the *2016 Colorado Design Snow Loads.* Using Figure 1.1.b in this reference, the value of K is obtained equal to 18.

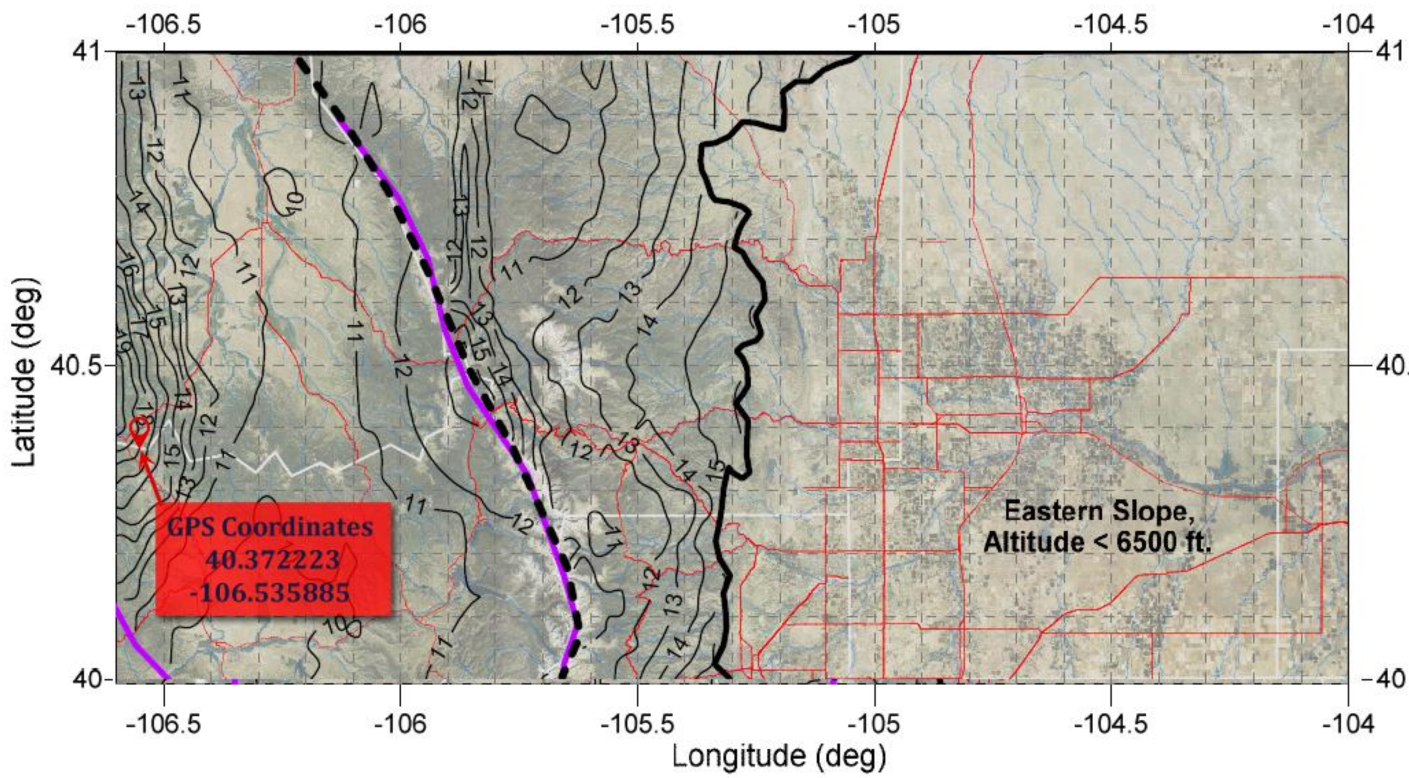


Figure 6, North Central Colorado Parameter Map for Determining Ground Snow Loads

The ground snow load is obtained from below equation which A is Altitude in thousands of ft and K is read from the map.

$$P_g = \max\left(\frac{k}{100}\cdot\ A^3\ .25\right) = \max(\frac{18}{100}\cdot\ 8.945^3\ .25) = 128.7\ psf$$

The provisions for the determination of snow loads on structures are available in Sections 7.3 to 7-6 ASCE7-16. Two types of snow loading shall be considered: Balanced and Unbalanced snow which were detailed in Figure 7.4-2 ASCE7-16.

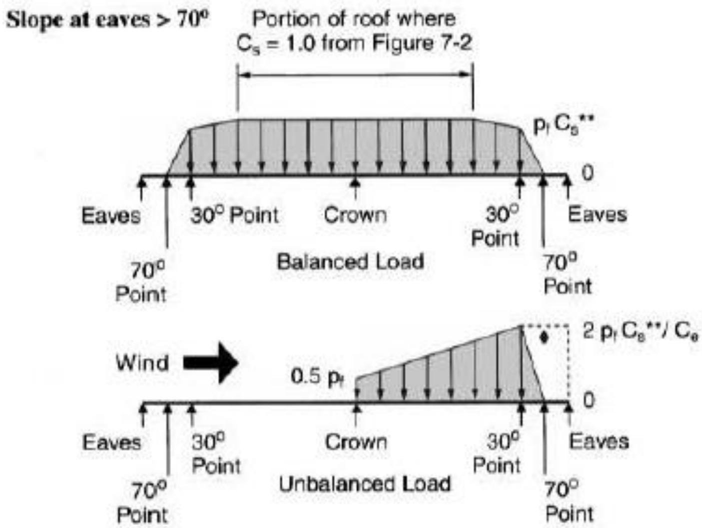


Figure 7, Balanced and Unbalanced loading on curved roofs

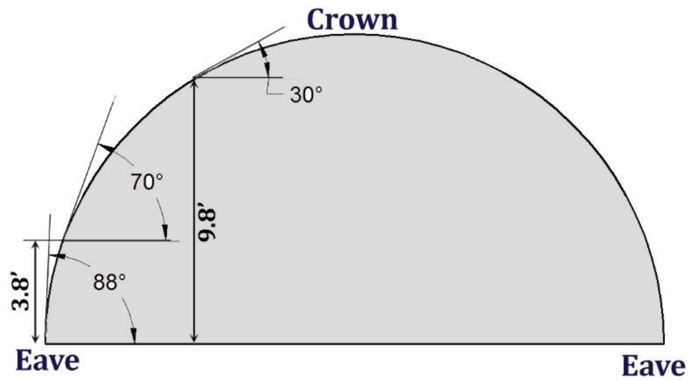


Figure 8, slope at eaves

For the flat roofs, balanced snow load is calculated from below equation.

$$P_f = 0.7\, C_e\, C_t\, I_s P_g = 0.7 \times 0.9 \times 1 \times 1 \times 128.7 = 81.1\, psf$$

The snow load parameters are summarized below.

Parameter	*Value*	*Description*	*Reference*
Risk Category	II	Risk Category of Buildings and Other Structures	Table 1.5-1 ASCE7-16
I_s	1	Snow importance factor	Table 1.5-2 ASCE7-16
Roughness	C	Surface Roughness Category	Section 26.7 ASCE7-16
Roof Exposure	Fully Exposed	Exposure of Roof	Table 7.3-1 ASCE7-16
C_e	0.9	Exposure Factor	Table 7.3-1 ASCE7-16
C_t	1	Thermal factor	Table 7.3-2 ASCE7-16
P_g	128.7	Ground snow load	Colorado Design Snow Loads

For the determination of C_e factor the category of C for the surface roughness and fully exposed roof were considered. So, the exposed factor equals to 0.9. C_t is the thermal factor which for the conditions of this dome, was considered equal to one. I_s is the importance factor of structure for the snow loads and equals to one for the risk category II. For the curved roof, the snow load is obtained as follows. The roof cover is considered as unobstructed slippery surface and therefore the C_s factor equals to 0.63 at 30° point. The summary of snow calculations is shown in Figure 9.

$$P_s = C_s\, P_f$$

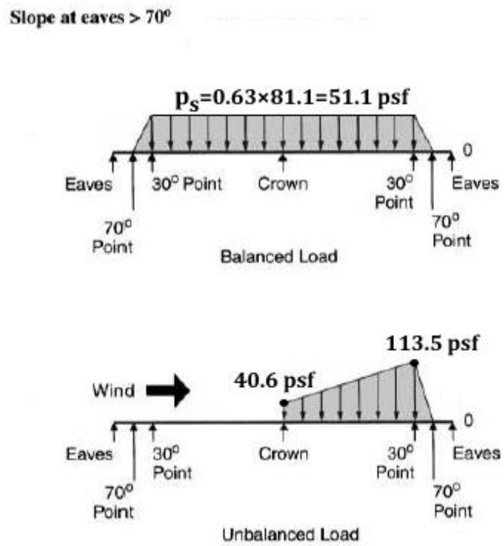


Figure 9, Applied Snow loads

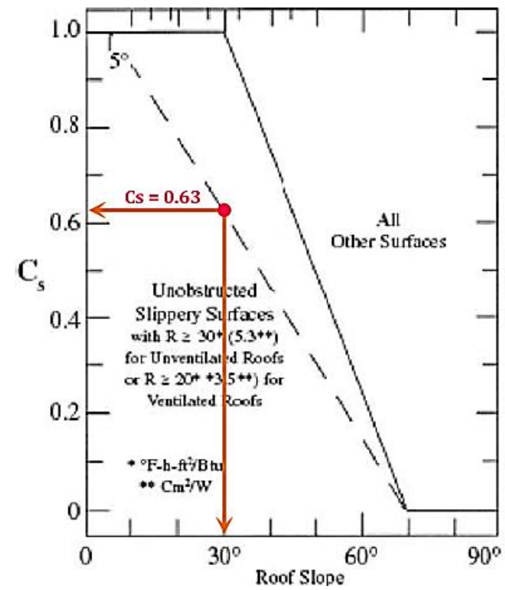


Figure 10, C_S , Roof Slope Factor for $C_t \leq 1$

The roof snow loads for both type of balanced and unbalanced were applied in the software. For the cases which the load changes linearly, the average value of ends has been applied. Below figures show the assigned snow loads in software.

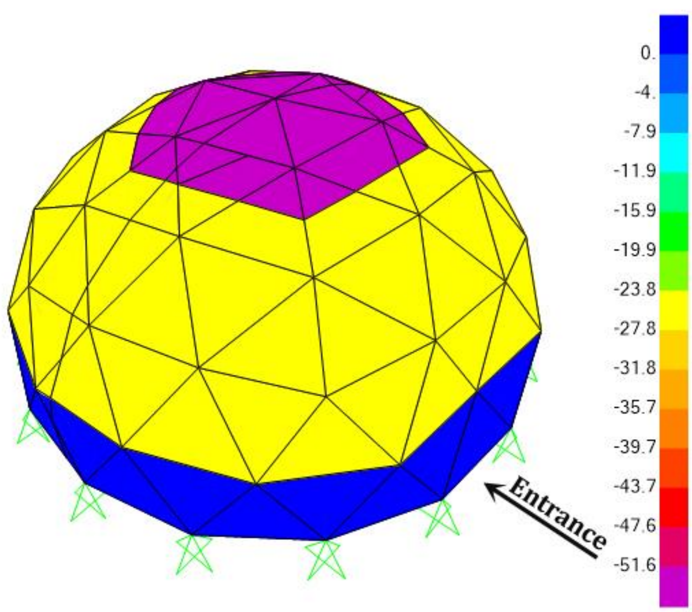


Figure 9, Applied balanced snow load (psf)

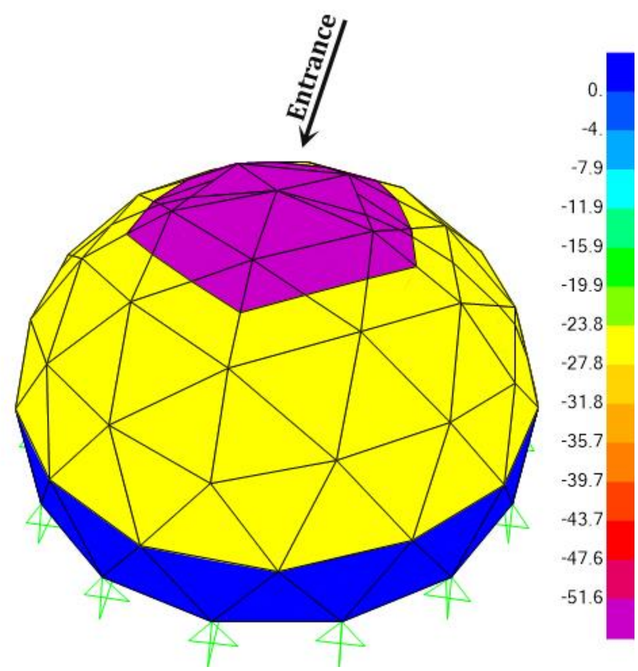


Figure 10, Applied balanced snow load (psf)

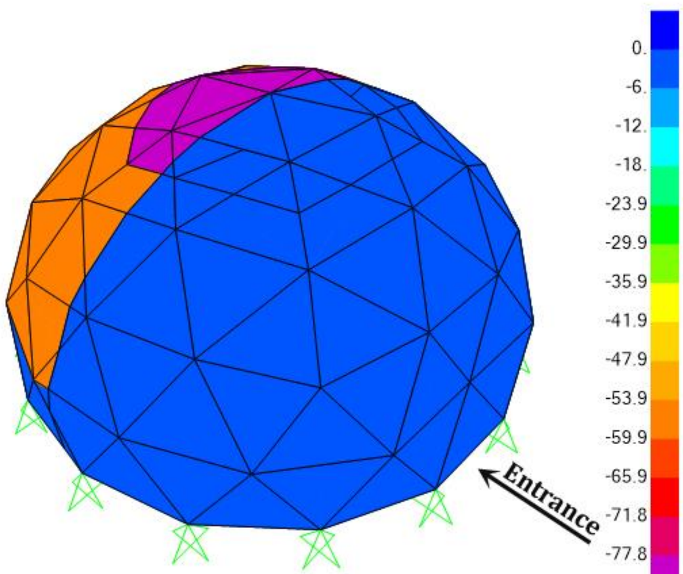


Figure 11, Applied unbalanced snow load (psf)

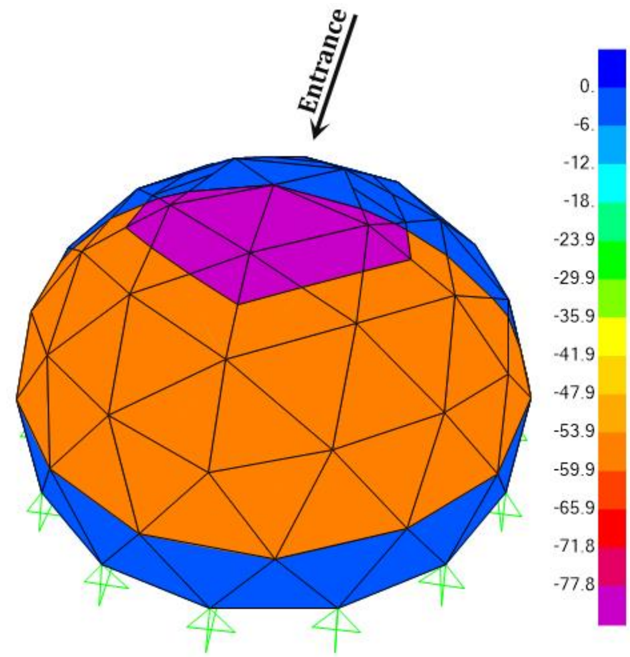


Figure 12, Applied unbalanced snow load (psf)

3.3 Wind Load

Provisions of ASCE7-10 section 27.3.2 were used for the wind loading. The wind speed is 106 mph for risk category II.

Figure 13, Wind speed map for the risk category II [ASCE7-16]

The wind load parameters are summarized below.

Parameter	*Value*	*Description*	*Reference*
Risk Category	II	Risk Category of Buildings and Other Structures	Table 1.5-1 ASCE7-16
I_w	1	Wind importance factor	Table 1.5-2 ASCE7-16
V	106 mph	Basic Wind Speed	ATC Council
K_d	1	Wind Directionality factor	Table 26.6-1 ASCE7-16
Exposure	C	Exposure Category	Section 26.7 ASCE7-16
K_{zt}	1	Topographic factor	Figure 26.8-1 ASCE7-16
K_e	0.723	Ground elevation factor	Section 26.9 ASCE7-16
G	0.85	Gust effects factor	Section 26.11 ASCE7-16
K_z	0.85	Velocity Pressure	Table 26.10-1 ASCE7-16
$G.C_{pi}$	±0.18	Internal wind pressure coefficient	Table 26.13-1 ASCE7-16

For the circular domes, the wind directionality factor is equal to one. The wind speed-up effect is included in the calculation by using the topographic factor. As shown below, the location of dome is below the half of hill height, so the topographic factor is assumed one [Section 26.8.1 ASCE7-16].

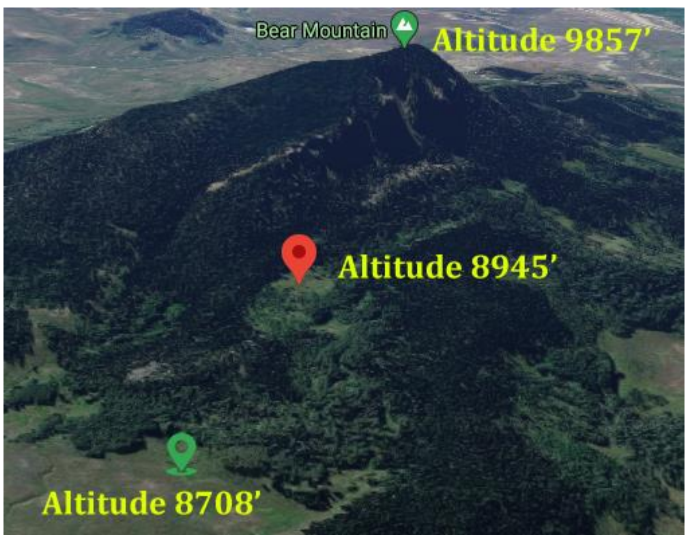


Figure 14, Location of dome at hillside of Bear Mountain

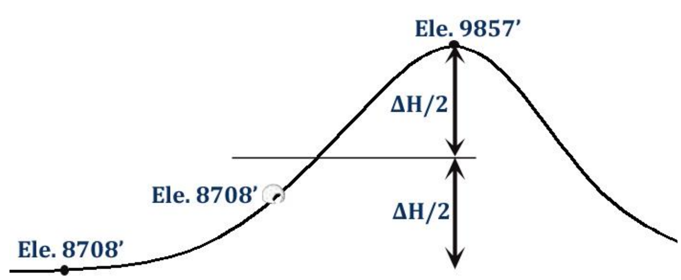


Figure 15, Elevation of dome relative to hill

Velocity pressure is obtained from equation 26.10-1 ASCE7-16.

$$q_z = 0.00256\, K_z\, K_{zt} K_d K_e V^2 = 0.00256 \times 0.85 \times 1 \times 1 \times 0.7234 \times 106^2 = 17.69\, psf$$

The internal velocity pressure is assumed equal to external velocity pressure. The external pressure coefficients are obtained from Table 28.3-2 ASCE7-16 and for the nodes A, B and C are equal to +0.8, -1.2 and zero respectively. External wind pressure for the nodes A, B and C are as follows:

$$p_{ext} = qGC_p = 17.69 \times 0.85 C_p$$

Node	*A*	*B*	*C*
p_{ext} (psf)	12.03	-18.04	0

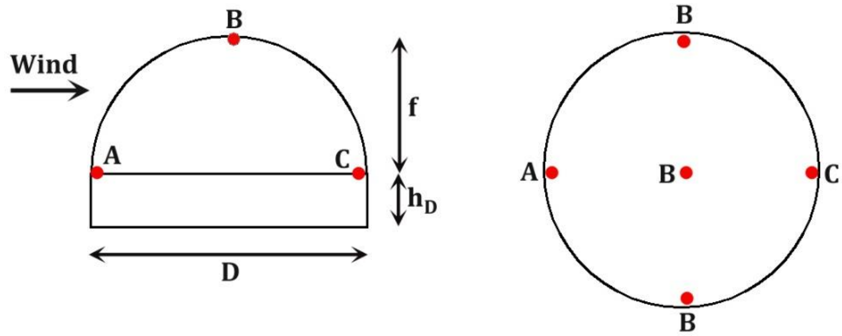


Figure 16, External Pressure Coefficient for Domes with Circular Base

Two wind load cases were considered. For the first Case (A), C_p values between A and B and between B and C are determined by linear interpolation along arcs on the dome parallel to the wind direction. In Case B, C_p is the constant value of A for θ ≤ 25 degrees and is determined by linear interpolation from 25 degrees to B and from B to C. The external load pressure for both cases are shown in below figures.

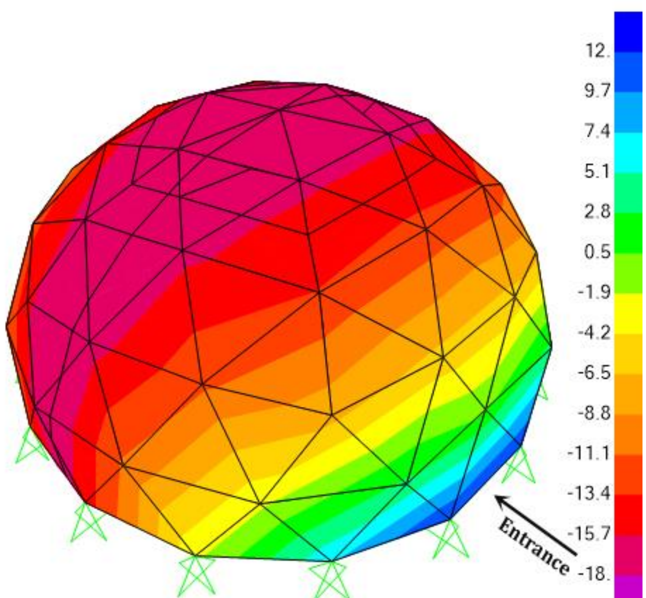


Figure 17, Applied external wind pressure (Case A) (psf)

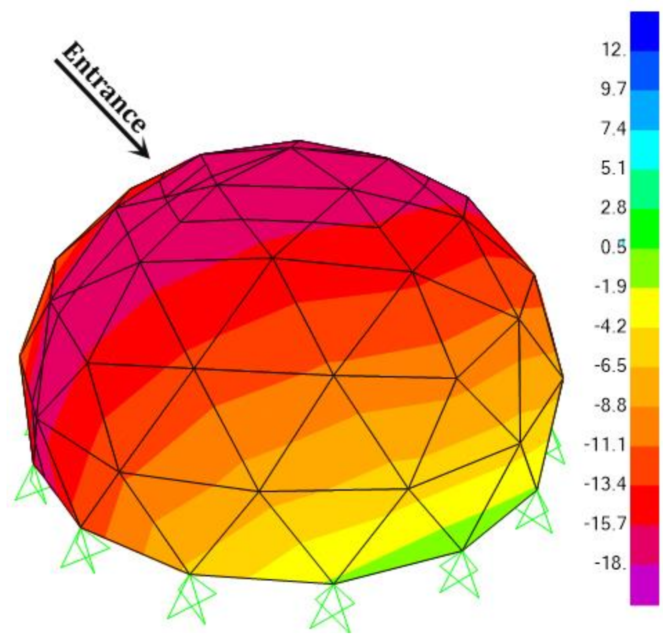


Figure 18, Applied external wind pressure (Case A) (psf)

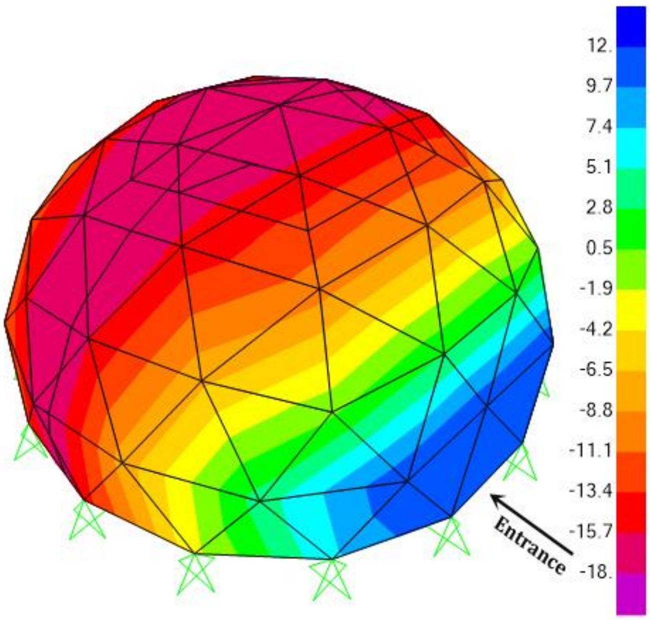


Figure 19, Applied external wind pressure (Case B) (psf)

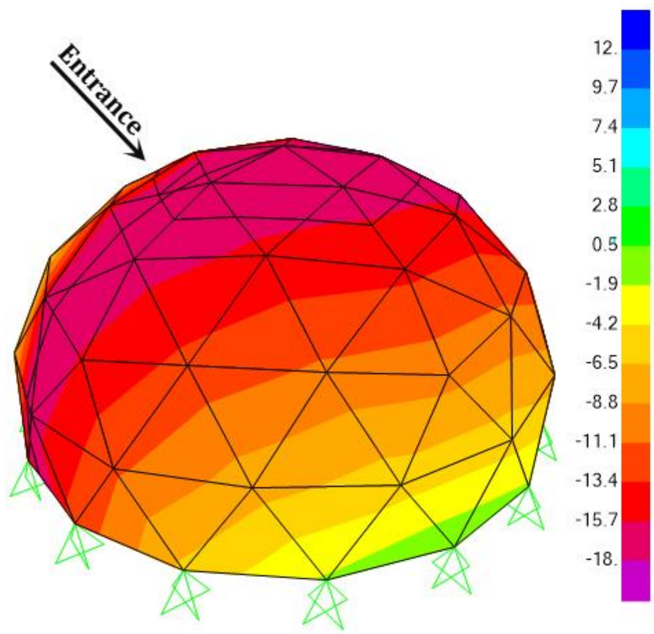


Figure 20, Applied external wind pressure (Case B) (psf)

Design wind pressure is calculated according to equation 27.3-1 ASCE7-16.

$$p = qGC_p - q_i(GC_{pi}) = 17.69 \times 0.85 \times C_p - 17.69 \times (\pm 0.18)$$

For applying the wind loads in software, 4 different wind loads were defined to consider the sign of internal pressure and both cases of external pressure (A and B).

3.4 Load Combinations

The basic load combinations for LRFD design method were listed in the below tables.

Load Combination	*Description*
LRFD01	1.4DL
LRFD02	1.2DL+ 0.5S_{Bal}
LRFD03	1.2DL+ 0.5S_{Unbal}
LRFD04	1.2DL + 1.6S_{Bal} + 0.5W_{A1}
LRFD05	1.2DL + 1.6S_{Bal} + 0.5W_{A2}
LRFD06	1.2DL + 1.6S_{Bal} + 0.5W_{B1}
LRFD07	1.2DL + 1.6S_{Bal} + 0.5W_{B2}
LRFD08	1.2DL + 1.6S_{Unbal} + 0.5W_{A1}
LRFD09	1.2DL + 1.6S_{Unbal} + 0.5W_{A2}
LRFD10	1.2DL + 1.6S_{Unbal} + 0.5W_{B1}
LRFD11	1.2DL + 1.6S_{Unbal} + 0.5W_{B2}

Load Combination	*Description*
LRFD12	1.2DL + W_{A1} + 0.5S_{Bal}
LRFD13	1.2DL + W_{A2} + 0.5S_{Bal}
LRFD14	1.2DL + W_{B1} + 0.5S_{Bal}
LRFD15	1.2DL + W_{B2} + 0.5S_{Bal}
LRFD16	1.2DL + W_{A1} + 0.5S_{Unbal}
LRFD17	1.2DL + W_{A2} + 0.5S_{Unbal}
LRFD18	1.2DL + W_{B1} + 0.5S_{Unbal}
LRFD19	1.2DL + W_{B2} + 0.5S_{Unbal}
LRFD20	0.9DL + W_{A1}
LRFD21	0.9DL + W_{A2}
LRFD22	0.9DL + W_{B1}
LRFD23	0.9DL + W_{B2}

An envelope load combination has been defined to report the envelope values of all 23 load combinations.

4. Analysis Results

4.1 Joint Reactions

Joint reaction forces under LRFD04, LRFD20 and ENVLRFD load combinations are shown in the below tables. Joint labels are shown in the figure. For the vertical reaction forces (Rz), the negative sign represents the uplift action and positive sign is bearing force.

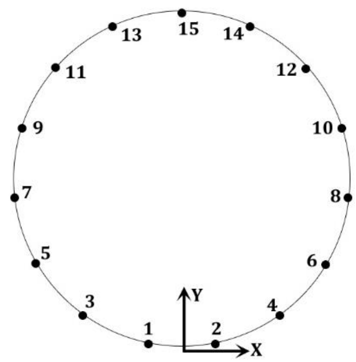


Figure 21, Base joint labels

Joint reaction forces under ENVLRFD load combination

Joint	Step Type	Rx (Lb)	Ry (Lb)	Rz (Lb)
1	Max	232.43	441.89	1995.2
	Min	-1273.8	-183.79	-362.73
2	Max	1302.28	352.61	1996.68
	Min	-240.24	-170.96	-358.09
3	Max	416.91	-0.57	2208.86
	Min	11.58	-315.88	-359.5
4	Max	-11.54	1.08	2207.16
	Min	-442.39	-264.7	-358.99
5	Max	221.43	19.12	1984.99
	Min	6.67	-349.02	-382.7
6	Max	-6.23	23.42	1984.36
	Min	-262.67	-322.62	-382.48
7	Max	195.28	276.68	2008.48
	Min	7.52	-310.52	-464.77
8	Max	-7.66	296.47	2007.73
	Min	-219.53	-282.41	-464.77
9	Max	251.93	62.13	2557.17
	Min	5.46	-145.66	-413.53
10	Max	-6.11	92.36	2560.06
	Min	-241.41	-128.44	-413.36
11	Max	159.46	21.16	3553.69
	Min	3.64	-219.03	-368.95
12	Max	-4.62	28.97	3553.61
	Min	-186.18	-196.79	-368.58
13	Max	53.93	17.74	3875.93
	Min	0.63	-260	-315.94
14	Max	0.13	21.35	3875.76
	Min	-89.92	-251.3	-315.75
15	Max	1.26	17.65	3664.51
	Min	-16.96	-233.9	-280.69

Joint reaction forces under LRFD04

Joint	Rx (Lb)	Ry (Lb)	Rz (Lb)
1	-1273.8	430.06	1995.2
2	1302.28	338.24	1996.68
3	357.4	-175.43	2156.74
4	-377.48	-121.9	2156.19
5	158.78	-49.95	1955.47
6	-164.18	-26.66	1955.21
7	171.6	-110.03	1995.47
8	-185.81	-84.21	1995.13
9	130.38	-87.86	1896.25
10	-141.45	-68.03	1896.02
11	99.3	-105.1	2105
12	-113.32	-89.8	2104.95
13	44.11	-122.64	2143.92
14	-61.16	-115.08	2143.89
15	-8.1	-112.99	1999.91

Joint reaction forces under LRFD20

Joint	Rx (Lb)	Ry (Lb)	Rz (Lb)
1	135.06	-149.07	-219.39
2	-143.38	-138.45	-216.35
3	86.51	-167.22	-137.37
4	-106.88	-151.63	-138.55
5	91.72	-203.06	-194.21
6	-119.57	-187.64	-194.57
7	81.76	-156.59	-281.42
8	-103.79	-142.82	-281.97
9	69.07	-11.2	-244.44
10	-69.69	-0.6	-244.75
11	55.66	21.16	-193.02
12	-52.13	28.97	-193.02
13	26.43	17.74	-141.57
14	-23.66	21.35	-141.58
15	1.26	17.65	-116.75

4.2 Frames forces

For some load combination, the internal forces of frames are shown in the below figures.

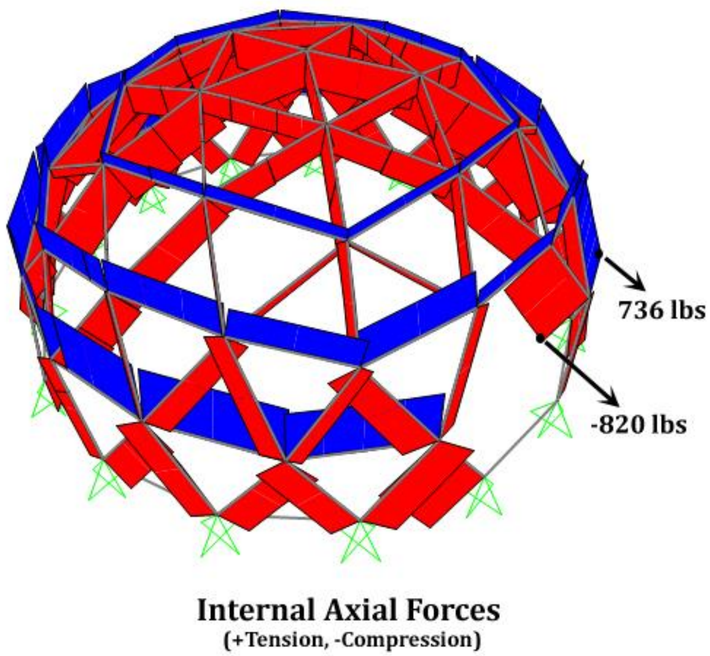


Figure 22, Axial forces of frames under LRFD02

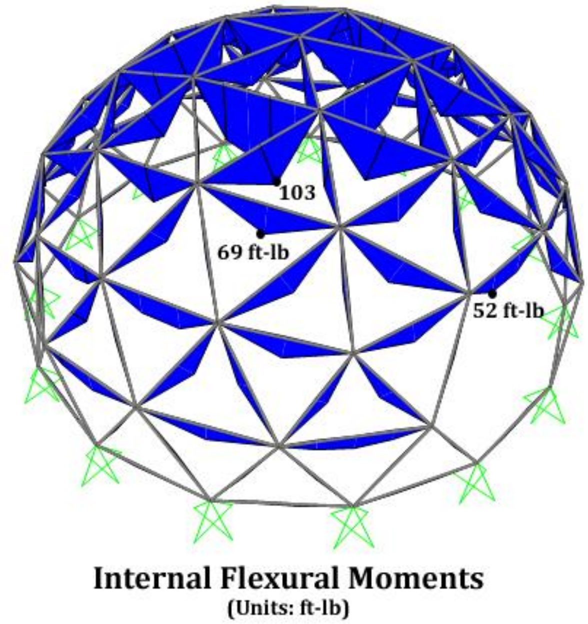


Figure 23, Flexural moment of frames under LRFD02

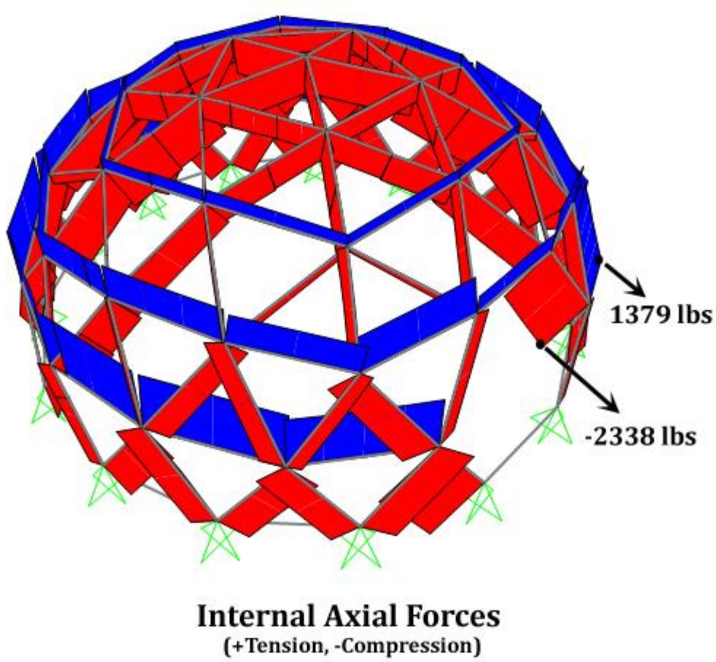


Figure 24, Axial forces of frames under LRFD02

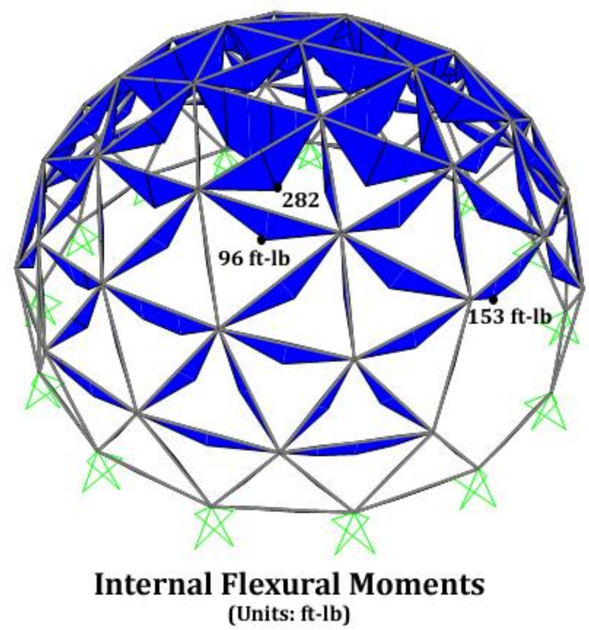


Figure 25, Flexural moment of frames under LRFD02

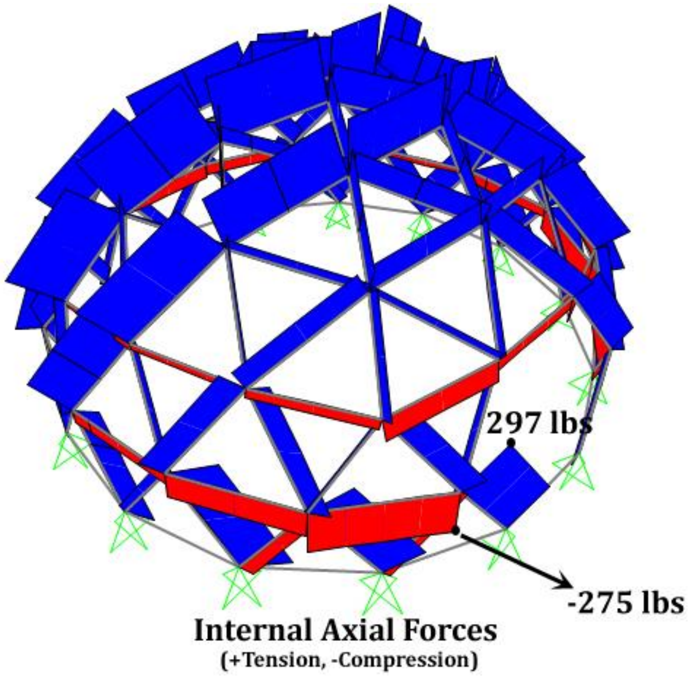


Figure 26, Axial forces of frames under LRFD20

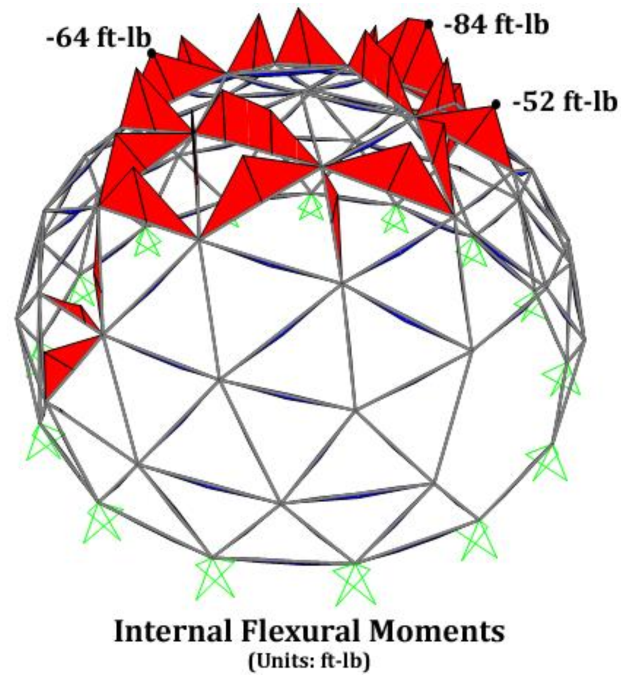


Figure 27, Flexural moment of frames under LRFD20

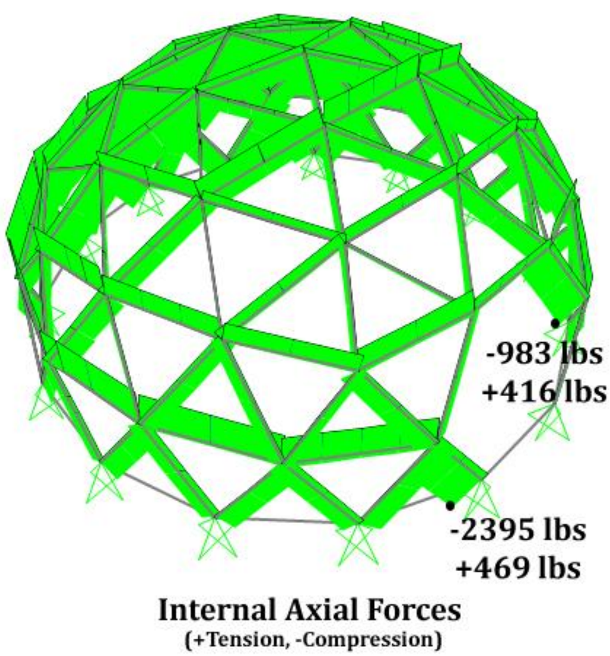


Figure 28, Axial forces of frames under ENVLRFD

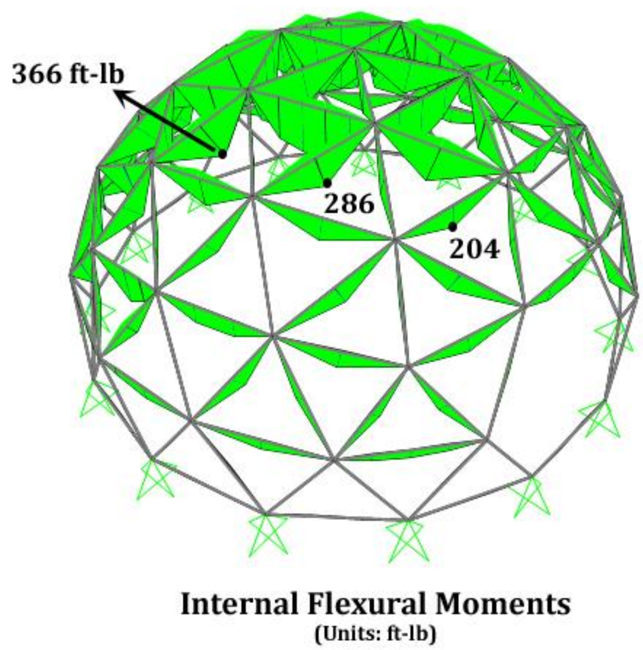


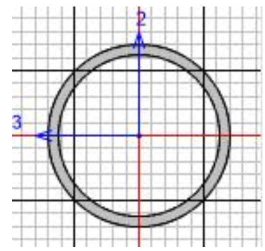
Figure 29, Flexural moment of frames under ENVLRFD

5. Design/Check

5.1 Frame Design

The section of all frames is Pipe48x2.5mm with the below properties.

Item	*Symbol*	*Unit*	*Value*
Outside Diameter	D	in	1.89
Wall Thickness	t	in	0.0984
Section Area	A	in^2	0.554
Moment of Inertia	Ix, Iy	in^4	0.223
Section Modulus	Sx, Sy	in^3	0.236
Plastic Modulus	Wx, Wy	in^3	0.3162
Radius of Gyration	rx, ry	in	0.6344



AISC360-16 standard has been used for frame design/check. The label of frames and PMM ratios of them are shown in the below figures.

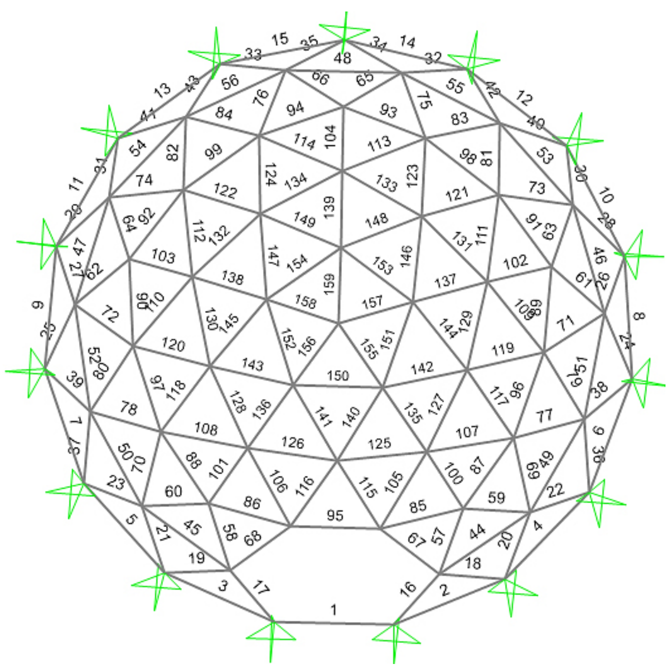


Figure 30, Frames labels

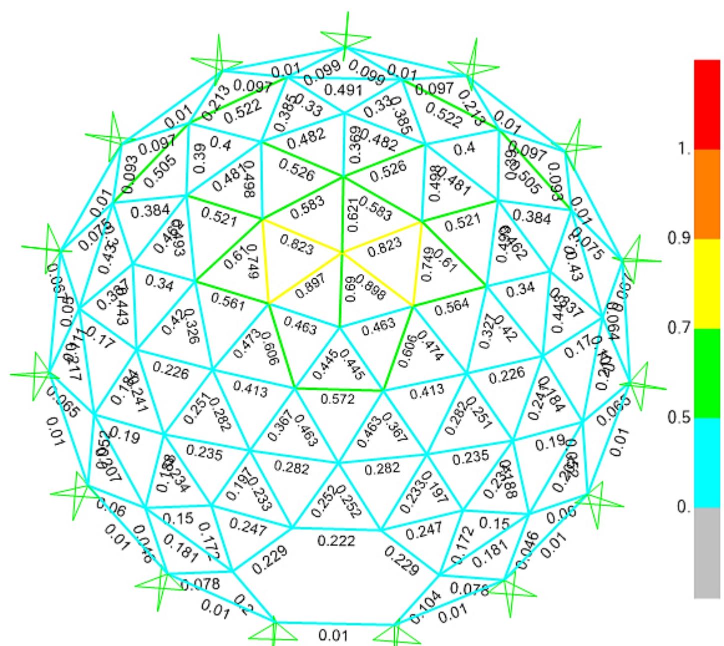


Figure 31, PMM ratios of frames

The details of design for the frame 153 are as follows.

```
AISC 360-16 STEEL SECTION CHECK    (Summary for Combo and Station)
Units  :  Kip, in, F

Frame :  153      X Mid:  369.629   Combo:  LRFD08-D-S-W    Design Type:  Beam
Length:  59.495   Y Mid:  169.689   Shape:  Pipe48x2.5mm    Frame Type:  OCBF
Loc   :  29.747   Z Mid:  135.16    Class:  Compact         Princpl Rot: 0. degrees

Provision: LRFD   Analysis: Direct Analysis
D/C Limit=1.      2nd Order: General 2nd Order       Reduction: Tau-b Fixed
AlphaPr/Py=0.113  AlphaPr/Pe=0.113  Tau_b=1.         EA factor=0.8     EI factor=0.8

PhiB=0.9          PhiC=0.9          PhiTY=0.9        PhiTF=0.75
PhiS=0.9          PhiS-RI=1.        PhiST=0.9

A=0.554           I33=0.223         r33=0.634        S33=0.236         Av3=0.278
J=0.446           I22=0.223         r22=0.634        S22=0.236         Av2=0.278
E=29000.          Fy=32.633         Ry=1.            z33=0.316
RLLF=1.           Fu=55.84                           z22=0.316

STRESS CHECK FORCES & MOMENTS (Combo LRFD08-D-S-W)
    Location            Pu         Mu33        Mu22         Vu2         Vu3          Tu
    29.747          -2.036        7.456          0.   5.474E-04  -3.722E-05          0.

PMM DEMAND/CAPACITY RATIO   (H1-1b)
    D/C Ratio:    0.898 = 0.095 + 0.803 + 0.
                       = (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1b)
    Factor                   L           K1          K2          B1          B2          Cm
    Major Bending           1.           1.          1.          1.          1.          1.
    Minor Bending           1.           1.          1.          1.          1.          1.

                          Lltb         Kltb          Cb
    LTB                     1.           1.       1.171

                            Pu      phi*Pnc     phi*Pnt
                         Force     Capacity    Capacity
    Axial               -2.036       10.693       16.27

                            Mu       phi*Mn      phi*Mn      phi*Mn
                        Moment     Capacity      No LTB        Cb=1
    Major Moment         7.456        9.288       9.288       9.288
    Minor Moment            0.        9.288

                            Tu           Tn      phi*Tn
                        Moment     Capacity    Capacity
    Torsion                 0.        9.716       8.745
SHEAR CHECK
                                                     Vu      phi*Vn      Stress      Status
                                                  Force    Capacity       Ratio       Check
    Major Shear    5.474E-04        4.881          0.          OK
    Minor Shear    3.722E-05        4.881   7.625E-06          OK
```

Figure 32, SAP2000 output report for design of frame 153 (Units: kip, in)

5.2 Frames Connection

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

Figure 33, frames and their connections

Under ENVLRFD load combination, the maximum shear force acts on the critical bolt is equal to 9.94 kips. According to the Section J3.6 AISC360-16, the shear strength of bolts is calculated as follows.

$$\varphi V_n = \varphi A_b F_{nv} \ ; \ F_{nv} = 0.45 F_u = 54 \ ksi$$

For M18 (0.7”) bolt, the shear capacity is calculated as follows.

$$\varphi V_n = 0.75 \times \frac{1}{4} \times \pi (0.7)^2 \times 54 = 15.6 \ kips > 9.94 \ kips \quad \textbf{OK}$$

5.3 Connection to Ground

5.3.1 Base Plate

Fifteen base plates are used to carry the supports loads to the ground. The below figure is shown its dimensions.

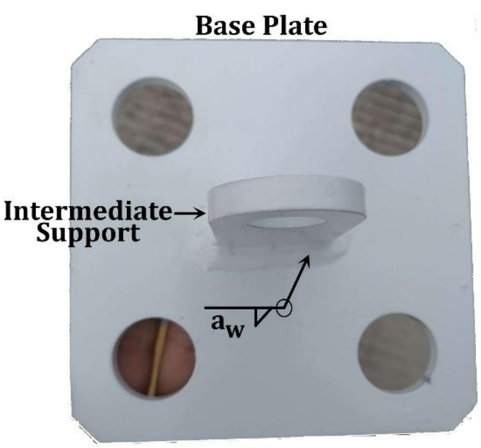


Figure 34, Base plates and intermediate plate

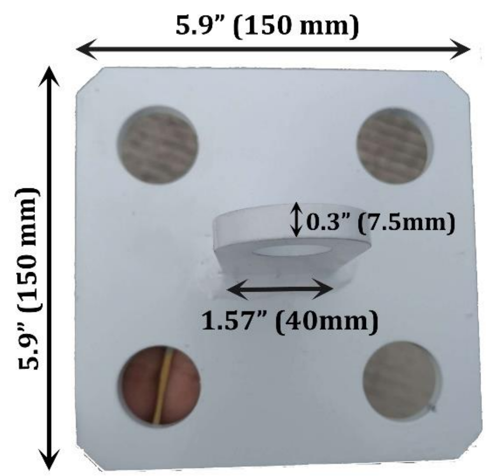


Figure 35, Base plates dimensions

Under ENVLRFD load combination, the maximum compression and uplift forces are shown in below table.

Load Combination	*Max. Compression Force*	*Max. Uplift Force*
ENVLRFD	Rz = 3875.76 lb	Rz = 464.77 lb

The bearing stress below the plate, the flexural moment at the critical point and the required thickness of plate are calculated as follows:

$$\sigma_{Bearing} = \frac{3875.76\ lbs}{5.9\ in \times 5.9\ in} = 0.11\ ksi$$

$$M_u = 0.11 \times 5.9 \times \frac{2.8^2}{2} = 2.54\ kip - in$$

$$\varphi M_n = \varphi Z F_y \geq M_u \quad \rightarrow t \geq \sqrt{\frac{4M_u}{\varphi b F_y}} = \sqrt{\frac{4 \times 2.54}{0.9 \times 5.9 \times 32.6}} = 0.24"$$

The obtained bearing stress would pass the requirements if is the maximum settlement be in the acceptable range. The subgrade modulus of soil is assumed 75 lb/in^3 and the maximum settlement under the heaviest snow load in the critical support is calculated.

Load Combination	*Max. Compression Force*
DL + $Snow_{max}$	Rz = 2506 lb

$$\delta_{max} = \frac{2506\ lb}{75\ lb/in^3\ \times 5.9^2} = 0.96"$$

Due to the temporary application of dome, the obtained settlement for the heaviest snow load would be acceptable. The required thickness of intermediate support is calculated so that it can carry the flexural moment due to F_1 and F_2 reaction forces. F_1 and F_2 are the maximum reaction forces which were projected to the normal and tangential directions of dome base.

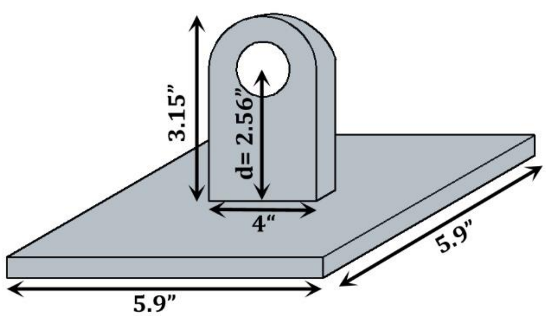


Figure 36, Dimensions of base plate and intermediate support

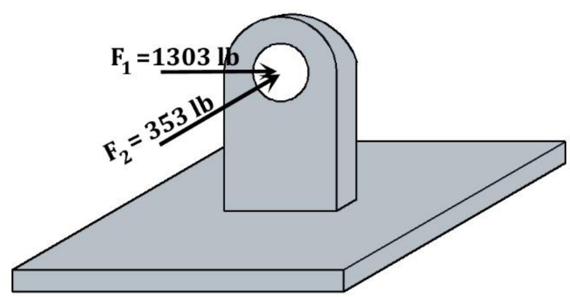


Figure 37, Max loads act on support

$$M_{u1} = F_1 \times d = 3.34\ kip - in \qquad \rightarrow t \geq \frac{4M_u}{\varphi b^2 F_y} = \frac{4 \times 3.34}{0.9 \times 4^2 \times 32.6} = 0.05" < 0.3" \quad \textbf{OK}$$

$$M_{u2} = F_2 \times d = 0.91\ kip - in \qquad \rightarrow t \geq \sqrt{\frac{4M_{u2}}{\varphi b F_y}} = \sqrt{\frac{4 \times 0.91}{0.9 \times 4 \times 32.6}} = 0.18" < 0.3" \quad \textbf{OK}$$

The existing thickness of the intermediate support can carry the maximum uplift force as well. The weld connection of intermediate support to the base plate was checked. The required fillet weld dimension is less than minimum specified by standard. According to Table J2.4 AISC360-16, the minimum fillet weld size should be equal to 3/16” (5mm)

5.3.2 Anchors

Duckbill earth anchors are used to anchor the base plates to the ground. Four Duckbill anchors shall be used to anchor each base plate to the ground. The required uplift force of each anchor shall be at least 155 lbs. Below table shows the working load capacity of Duckbill anchors in normal soils. (To know more about Duckbill anchors view Link1, and Link2).

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

6. Miscellaneous

6.1 Stability and Overturning

Due to anchorage of base to the ground, the overturning of dome is not possible.

6.2 Deformations

According to the AISC Design Guide 3, two load combinations are suggested for vertical deflections of framing members.

Load Combination	*Description*
Servicibility1	DL + LL
Servicibility2	DL + 0.5Snow

The allowable deflection of roof members is L/150 according IBC Table 1604.4. Maximum deformation value is equal to 0.17 inch which is less than L/150. The length of frames is equal to 60 inches.

$$\delta_{max} = 0.17"$$

$$\delta_{max} \leq L/150 = 60/150 = 0.4" \quad \textbf{OK} \quad ; \quad \delta_{max} \leq L_{span}/360 = 276/360 = 0.77" \quad \textbf{OK}$$

Also, vertical deflection at dome’s crown is 0.03” which is too less than the allowable value.

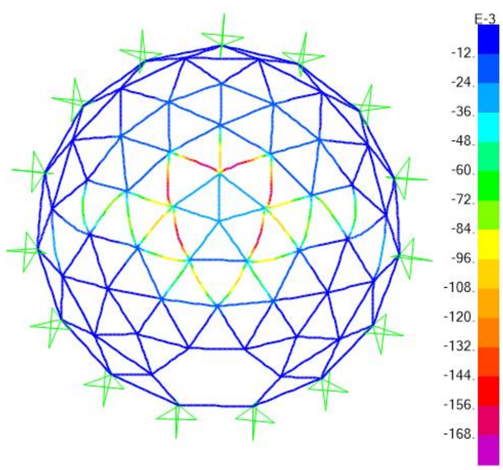


Figure 38, Deformation under Servicibility2