Seismic Analysis of Polygonal Face Dome, Triangular Face Dome and Zipper Braced Dome

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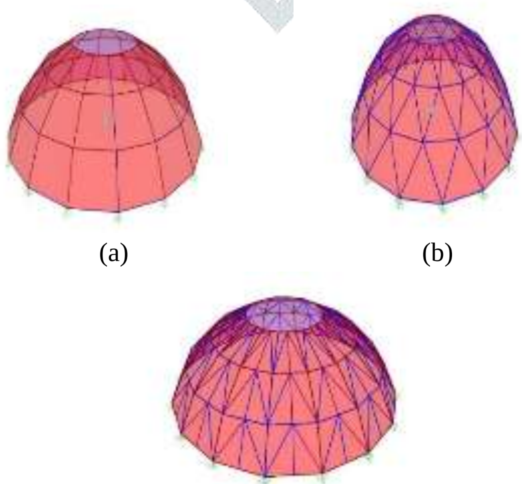
Abstract : This paper represents analysis and design work of three different domes. In this works the proposed dome is modeled, analysis and design to be done by using SAP2000 v19. The designed reinforced concrete domes have been made for normal braces i.e. polygonal faces, triangular faces, and zipper bracing system. The vertical member added in triangular faces i.e. strut is called as zipper frame. Analysis, design is done for these three types of domes and compared the results with each others. The used bracing systems i.e. zipper is works as column and beam in dome structure. The diameter and height (rise) of dome is kept common for all three cases i.e. 80m dia. and rise of dome is 27m. Various action of loads are considered on surface of dome such as Dead load, Live load, Static and dynamic earthquake loads As per IS 875 (Part-I, II, III). At last response spectrum analysis method has been used for analysis.

Keywords : Dome, Polygonal face dome, Triangular face dome, Zipper bracing dome, SAP2000.

Introduction

The shape that encloses the most volume with the smallest surface area is the sphere and a dome is part of a sphere. That means a dome is the shape that can enclose the largest space using the fewest materials. Domes are very efficient structure for covering large spans. The Sanchi Stupa is the oldest (3rd century BCE) stone dome structure of diameter 36.6m & height 16.46m was constructed by Great Ashoka at Madhya Pradesh, India. Gol Gumbaz, situated in Bijapur district of Karnataka, is the largest dome in India. Gol Gumbaz has a diameter of 124feet & is the second largest dome in the world, next only to St. Peter's Basilica in Rome. Gol Gumbaz dome was built by Muhammad Adil Shah in the year 1656.The domes on the circular-shaped base have a large usage – silos, tanks, warehouses for bulk materials, sportive facilities and exhibition halls. They are light, beautiful and can cover big spans, providing free space without intermediate columns. Domes can be constructed by variety of materials, from traditional masonary and concrete, to cast iron, timber and steel, lightweight materials such as architectural fabrics and cable. Domes can be highly efficient structures, similar to arches. They are self-supporting, stabilized by the force of gravity acting on their weight to hold them in compression.

In this study, the rcc dome is designed and it is modeled as triangular or polygonal faces that distribute the stresses within the structure itself. Dome is also modeled by using zipper frame which are used in building. It is designed as reinforced concrete structure. Three types of dome are considered as per design point of view i.e. dome with polygonal faces, triangular faces, and zipper frame for loads as gravity load, live load, earthquake loads (DL, LL, WL, static and dynamic earthquake load). Design of domes is based on to find out for each type which geometry is suitable for construction in normal and seismic areas.



(a) (b)

(c)

Fig 01: a) Polygonal face dome, b) Triangular face dome, c) Zipper braced dome.

Literature Review

Study of literature gives conclusion as the oval roof shape on a rectangular plane gives the designer more architectural freedom. Also, less effort required to ensure structure stability as well as the constraints of packing rooms together and flexibility of dimensioning. The important issues in the case of oval covers is the construction of excessive complication and expensive formwork systems by using conventional construction methods, and the design of the structure of such concrete shells, which is more complex than for the surface of revolution, it occurs due to second-order effects and edge forces, as well as the design of the buckling, which is not suitably covered by codes of practice [1]. The Ribbed domes have better seismic behaviour or performance than Schwedler and Diamatic domes. Schwedler domes are the most seismically vulnerable and ribbed domes are less vulnerable. Also behaviour of schwedler dome and ribbed dome are more similar in configuration. Buckling load on the member is higher for schwedler dome compared to the diamatic dome. Hybrid domes shows a lower deflection and higher load carrying capacity. The deflection of hybrid dome decreases with increase in the number of layers of diamatic dome. If axial force on members is considered as the deciding factor for selection of dome they can propose H1 domes. Maximum moment obtained for schwedler dome. As the number of schwedler layer in hybrid dome increases the moment value also increases [2] [13]. Various factors influences on the single layer lattice dome due to its dome geometry, slenderness of members, joint rigidity and load hypothesis. The combinations of geometric & structural parameters require avoiding the presence of critical point in the load displacement curve [3]. In Practical examination of performance of dome for hollow section, generally coefficient may vary as compared to full circular section. As well as errors are occurred upto15% as compared to analytical design of dome [4]. The potential regulation of the stress-strain state of the stone dome by means of changing the quantity and location of connecting members; stress strains in the stone dome are significantly decreased; supporting constructions are significantly unloaded due to the influence of horizontal forces, as the thrust is wholly absorbed by the reinforced concrete supporting ring. Therefore earthquake resistance of the stone domes increases significantly [5]. Increase in the dome thickness increases the frequency of vibration while the increase in dome height decreases the frequency of vibration of both spherical and paraboloidal domes. At large heights the frequencies of vibrations at all modes tend to converge to similar values for paraboloidal domes and different for spherical domes. For large dome thickness, the frequencies of vibrations of spherical domes diverge from each other while for paraboloidal domes they have the tendency to converge at certain values [6]. The failure of the concrete dome was the result of non-symmetric buckling at a local area affected by the bending effect, the material and geometric non-linear behaviour as well as imperfection. A critical location for failure of the dome was identified between the crown and the edge, where maximum dual compressive stresses occurred on the top surface accompanied by small dual tensile strains on the bottom surface. The maximum compressive stress was well below uniaxial compressive strength for the concrete. The failure of the concrete dome was the result of non-symmetric buckling at a local area affected by the bending effect, the material and geometric non-linear behaviour as well as imperfections. The buckling pressure from the FE analysis was higher than the experimental result when the analysis took account of the geometric nonlinear behaviour only, but used the initial modulus of elasticity [7]. For an earthquake with a return period of 75years the structures remains intact while some cracking occurs on some part of the structure. For short and normal duration of earthquake, the intact portion of column are expected to be able to prevent collapse of the structure but for an earthquake with long duration the damage may be so extensive that part of the structure may collapse at this earthquake level. And for return period of 2500yrs the damage will be so extensive that would cause collapse the structure at this earthquake level. [8]. Low level of stresses and displacement of the shells of revolution of covers when affected by the most severe earthquakes which proves that the cover constructions in the form of shells of revolution are highly effective where their seismic stability is concerned. Construction standards single out shell constructions as a separate group to make it easier to select true calculation factors, reduction factor K in particular. This is important due to the fact that shells of revolution are as a rule long-span structure [9]. Algeciras market dome and Semi-spherical dome are more efficient, because it posses minimum principal stress value and total deformation compared to Tori-spherical and Elliptical dome shapes. Generally these two shapes for dome can prefer for construction. Basically Crack failure is one of the major failures in concrete structures which developed by production of principal tensile stresses in body. From crack pattern of the domes, Algeciras market dome shape has more resistance to crack. Semi-spherical dome also gives a better result than that of Tori-spherical and elliptical dome shapes [10]. Donnell’s shell theory is suitable for the analysis of shallow domes from the perspectives of both accuracy and simplicity. Use of the simplified theoretical approaches is based on the age-adjusted effective modulus lead to very conservative results. The analysed model gives critical time to cause creep buckling at exponentially decays with the increase of the applied pressure [11]. Formation of Groove on spherical dome causes abrupt change on its wind pressure coefficient in the vicinity of the groove. For the single grooved dome, the effect of groove reaches its maximum fluctuation, when the groove axis has the angle θ =900 w.r.to wind direction. The groove effect on the wind surface pressure sustains its basic nature irrespective of the number of grooves, therefore applies to scallop dome [12]. Ribbed dome shows good performance against the vertical loads. Due to its structural symmetry and shape provide dome good performance against vertical loading. For providing lateral stiffness to the dome structures, providing diagonal elements to the dome structures seems a good practice. The provision of diagonal members to the dome structure can reduce the section for rib as well as Shell element shows significance effect in control of deflection due to horizontal loads. For improvement in performance of dome it is better to choose rise to span ratio in between 0.30 to 0.35 for ribbed dome [14].

SYSTEM MODELING

Dome Details: To study the behaviour of Rcc building under high Seismic forces as here taken

1) Architectural details:-

Diameter of Dome : 80 m
Total Rise of the Dome : 27 m

2) Codes used for analysis of the structure:-

R.C.C. design : IS 456: 2000
Earthquake load : IS1893: 2016
Wind load : IS875: part 3
Code for Dead load : IS875: Part 1
Code for Live load : IS875: Part 2

3) The basic parameters assumed for the Analysis and design:-

Roof Slab Thick :125 mm thick
Plinth beam size : 600x600 mm
Supporting beams : 230x450 mm
Columns sizes : 300x650 mm
Zipper bracing : 300x300 mm

4) Notations used:-

DL : Dead Load
LL : Live Load
WX : Wind Load In X
EX : Static Earthquake In X
EY : Static Earthquake In Y
DEX : Dynamic Earthquake In X
DEY : Dynamic Earthquake In Y

5) Earthquake parameters considered:-

Zone : II (Aurangabad)
Soil type : Hard soil
Importance factor : 1
Time period : Based on IS 1893

Table01: Time Period of the structure under Static and Dynamic load consideration

Modal Participating Mass Ratios			**Polygo-nal Dome**	**Triang-ular Dome**	**Zipper Dome**
Output Case	**Step Type**	**Step No**	**Period**	**Period**	**Period**
Text	Text		Sec	Sec	Sec
Modal	Mode	1	0.2098	0.5650	0.3119
Modal	Mode	2	0.2096	0.5650	0.3119
Modal	Mode	3	0.1733	0.3899	0.2340

Table 02: Base Reaction under Service Load (DL+LL) Consideration (Polygonal face dome)

TABLE: Joint Reactions(All cases are combination)						
Joint	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	**KN**	**KN**	**KN**	**KN-mm**	**KN-mm**	**KN-mm**
1	254.087	4.47E-13	561.59	-9.63E-10	61064.2	-4.39E-10
2	255.348	121.856	570.06	-29282.7	99237.4	5537.44
5	189.709	190.285	548.46	-	74850.4	-13.99

				74958.2		
7	103.159	248.206	548.87	-97432.6	40243.1	-69.51
9	0.028	268.6	548.36	-105694	307.56	167.35

As we analysis the dome for the only DL+LL consideration for the normal dome we observed the above reactions whereas F1 shows the direction x similarly F2 show the direction y and F3 indicate the direction z and if we consider the direction means F3 the maximum reaction is in the range of 500 onwards .where as if we consider the moments in the different case the suitable value is shown in above table.

Table 03: Base Reaction under Service Load (DL+LL) Consideration (For Triangular face dome)

Table: Joint Reactions(All case types are combination)						
Joint	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	**KN**	**KN**	**KN**	**KN-mm**	**KN-mm**	**KN-mm**
1	-683.873	-0.00109	1603.918	27.68	102437.8	16.96
2	-631.815	-261.707	1603.916	-39177	94649.05	15.79
5	-483.572	-483.57	1603.918	-72450.3	72418.23	-14.07
7	-261.706	-631.817	1603.919	-94629.8	39225.63	16.12
9	0.000423	-683.873	1603.917	-102437	27.59	16.66

The direction means F3 the maximum reaction is in the range of 1600 onwards and this is because the mass is increasing in the form of bracing the reaction is also getting on higher side. And if we consider the moment the moments are going to be increasing in the case of triangular face dome when it will compared with the polygonal face dome.

Table 04: Base Reaction under Service Load (DL+LL) Consideration (For Zipper brace dome)

TABLE: Joint Reactions(All cases are combination)						
Joint	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	**KN**	**KN**	**KN**	**KN-mm**	**KN-mm**	**KN-mm**
1	-714.656	-0.315	1867.451	10.25	116877.3	3.32
2	-660.944	-273.985	1868.932	-44535.3	108007.6	97.64
5	-506.253	-505.647	1868.901	-82748.1	82282.49	-168.92
7	-273.55	-659.868	1866.946	-108185	44701.37	-54.7
9	1.432	-713.777	1866.056	-117091	1196.81	613.42

The direction means F3 the maximum reaction is in the range of 1600 onwards and this is because the mass is increasing in the form of bracing the reaction is also getting on higher side. In case of moments the moments are higher than the above both triangular and normal frame as shown in above tables.

Table 05: Displacement Consideration: For DL & Wind (For Polygonal Face Dome)

TABLE: Joint Displacements					TABLE: Joint Displacements			
Joint	**Case**	**U1**	**U2**	**U3**	**Case**	**U1**	**U2**	**U3**
Text	Text	mm	mm	Mm	Text	Mm	mm	mm
1	DL	0	0	0	WX	0	0	0
2	DL	0	0	0	WX	0	0	0
3	DL	0.16511	0.063274	0.00422	WX	-0.2	-0.13718	-0.09083
4	DL	0.215378	-1.4E-16	0.031793	WX	0.02	-0.09152	0.02776
5	DL	0	0	0	WX	0	0	0
6	DL	0.122536	0.119103	0.004451	WX	-0.1	-0.09546	-0.04959
7	DL	0	0	0	WX	0	0	0
8	DL	0.065134	0.154422	0.003505	WX	0.01	0.047652	0.02013
9	DL	0	0	0	WX	0	0	0
10	DL	-0.00042	0.170203	0.004479	WX	0.01	0.046622	0.02009
11	DL	0	0	0	WX	0	0	0
12	DL	-0.06711	0.160799	0.006021	WX	0.06	-0.12798	-0.0496
13	DL	0	0	0	WX	0	0	0
14	DL	-0.12372	0.123518	0.006172	WX	0.14	-0.20421	-0.09044
15	DL	0	0	0	WX	0	0	0
16	DL	-0.16032	0.066569	0.005739	WX	-0.1	-0.0766	0.02595
17	DL	0	0	0	WX	0	0	0
18	DL	-0.1725	1.74E-17	0.005305	WX	-0.3	-0.06033	0.16286
19	DL	0	0	0	WX	0	0	0
20	DL	-0.16032	-0.06657	0.005739	WX	-0.3	-0.1231	0.15832
21	DL	0	0	0	WX	0	0	0
22	DL	-0.12372	-0.12352	0.006172	WX	-0.1	-0.07028	0.08648
23	DL	0	0	0	WX	0	0	0
24	DL	-0.06711	-0.1608	0.006021	WX	-0	0.046847	0.02136
25	DL	0	0	0	WX	0	0	0
26	DL	-0.00042	-0.1702	0.004479	WX	0.02	0.041759	0.02144
27	DL	0	0	0	WX	0	0	0
28	DL	0.065134	-0.15442	0.003505	WX	0.09	-0.11351	0.08664
29	DL	0	0	0	WX	0	0	0
30	DL	0.122536	-0.1191	0.004451	WX	0.24	-0.23335	0.15832
31	DL	0	0	0	WX	0	0	0
32	DEAD	0.16511	-0.06327	0.00422	WX	0.29	-0.18452	0.16246
33	DEAD	-0.16041	-0.07388	-0.42113	WX	-0.5	-0.32063	-0.42032
34	DEAD	-0.15704	-4.7E-16	-0.40398	WX	-0.1	-0.24096	-0.06067
35	DEAD	-0.13308	-0.14272	-0.43067	WX	-0.2	-0.14797	-0.15779

Table 06: Displacement Consideration: For DL & Wind (For Triangular Dome)

TABLE: Joint Displacements for triangular face dome								
Joint	**Case**	**U1**	**U2**	**U3**	**Case**	**U1**	**U2**	**U3**
Text	Text	mm	mm	mm	Text	mm	mm	mm

1	DL	0	0	0	WX	0	0	0
2	DL	0	0	0	wx	0	0	0
5	DL	0	0	0	wx	0	0	0
7	DL	0	0	0	wx	0	0	0
9	DL	0	0	0	wx	0	0	0
11	DL	0	0	0	wx	0	0	0
13	DL	0	0	0	wx	0	0	0
15	DL	0	0	0	wx	0	0	0
17	DL	0	0	0	wx	0	0	0
19	DL	0	0	0	wx	0	0	0
21	DL	0	0	0	wx	0	0	0
23	DL	0	0	0	wx	0	0	0
25	DL	0	0	0	wx	0	0	0
27	DL	0	0	0	wx	0	0	0
29	DL	0	0	0	wx	0	0	0
31	DL	0	0	0	wx	0	0	0
33	DL	-0.23815	-0.0986	-1.024	wx	0	0	0
34	DL	-0.25777	-6E-07	-1.024	wx	0	0	0
35	DL	-0.18227	-0.1823	-1.024	wx	0	0	0

Table 07: Displacement Consideration: For DL & Wind (For Zipper Brace Dome)

TABLE: Joint Displacements For Zipper Brace Frame				
Joint	**Case**	**U1**	**U2**	**U3**
Text	Text	mm	mm	mm
1	DL, WX	0	0	0
2	DL, WX	0	0	0
3	DL, WX	0	0	0
4	DL, WX	0	0	0
5	DL, WX	0	0	0
6	DL, WX	0	0	0
7	DL, WX	0	0	0
8	DL, WX	0	0	0
9	DL, WX	0	0	0
10	DL, WX	0	0	0
11	DL, WX	0	0	0
12	DL, WX	0	0	0
13	DL, WX	0	0	0
14	DL, WX	0	0	0
15	DL, WX	0	0	0
16	DL, WX	0	0	0
17	DL, WX	0	0	0
18	DL, WX	0	0	0

19	DL, WX	0	0	0
20	DL, WX	0	0	0
21	DL, WX	0	0	0
22	DL, WX	0	0	0
23	DL, WX	0	0	0
24	DL, WX	0	0	0
25	DL, WX	0	0	0
26	DL, WX	0	0	0
27	DL, WX	0	0	0
28	DL, WX	0	0	0
29	DL, WX	0	0	0
30	DL, WX	0	0	0
31	DL, WX	0	0	0
32	DL, WX	0	0	0
33	DL, WX	0	0	0
34	DL, WX	0	0	0
35	DL, WX	0	0	0

As shown in above table 07, values of Joint displacement for (Dead load and Wind load) are zero for Zipper Brace Frame.

CONCLUSION

1. As per the model results we conclude that the modal value for the zipper brace dome is quite good when it will compared with both normal and triangular face dome.
2. As the stiffness will increase the displacement value is going to be reduced in case of the zipper braced dome.
3. Zipper braced dome provide the better stability in the case of Earthquake and wind consideration.
4. When we compared the all three dome the Polygonal face dome is having lesser base reaction that the remaining two domes but the polygonal brace dome is having higher displacement values in case of earthquake and wind consideration.
5. In case of triangular face dome the displacement values are quite good but the zipper brace dome is having the displacement less than that of triangular dome.
6. Over all if we check the analysis results we found that the zipper frame is having more stability against wind and earthquake than the other two domes (Polygonal face & triangular face dome).

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