Chapter 7
Effectiveness of Base Isolation Systems for Seismic Response Control of Masonry Dome

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Abstract Domes are constructed historically over the last many centuries. They are doubly curved structures, without angles and corners. The most important advantage of dome structures is that they enclose an enormous amount of column-free interior space, in addition to providing decent aesthetic sight. Historically, domes were built of masonry material. Masonry structures have very low ductility, and hence they are weak in resisting the lateral loads. Most of the domes are designed for gravity loads using simple geometrical rules, considering the dome as an arch of identical section. Due to the absence of reinforcement in the masonry domes, their thickness must be kept high to resist the tensile stresses. Because of their large thickness, masonry domes attract a large magnitude of seismic forces due to higher mass thus, making them vulnerable to earthquake excitations. Due to earthquake forces, the masonry domes are subjected to tensile forces at the bottom rings and as a result, cracks are developed in the bottom parts of domes. The conventionally designed and constructed masonry domes are vulnerable to severe damage or total collapse under strong seismic excitations. To preserve these ancient structures of historic importance from being damaged due to seismic excitations, base isolation can prove to be a very effective technique. In the present research, seismic response of the case study masonry dome of span 25 m, located in Maharashtra, India, is investigated analytically. The specific objectives of the study are (i) to analyse the seismic performance of the fixed base masonry dome structure under real earthquake ground motions, (ii) to analyse the seismic performance of the masonry dome installed with base isolation systems, viz. lead rubber bearings (LRB) and friction pendulum systems (FPS) and (iii) to compare the seismic performance of the fixed base masonry dome with that, installed with LRB and FPS. The response of the base-isolated dome is obtained using SAP2000 by performing nonlinear time history analysis and is compared with the corresponding response of the conventional dome without base isolators. The nonlinear time history analysis is performed considering real earthquake ground

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motions of PGA ranging between 0.1 g and 0.35 g. The effectiveness of base isolation technique in improving the response of the dome is explored. The major evaluation criteria considered are tensile stresses, base shear and displacement at the apex point of the dome. It is observed that the seismic response of the base-isolated dome diminishes significantly in comparison with the conventionally constructed dome, depicting the effectiveness of the base isolation strategy. Both, the elastomeric and sliding systems, are found to be very effective in decreasing the response quantities, substantially. The force–displacement loops for both the isolators show considerable energy dissipation. The original uniqueness and aesthetic value of the historical monumental dome are maintained unaltered, even after employing base isolators at the foundation level of the dome.

Keywords Masonry domes · Base isolation · Lead rubber bearing · Friction pendulum system · Nonlinear time history analysis · SAP2000

7.1 Introduction

Domes have been labelled as the 'kings' of all roofs, as they cover some of the most important structures. Domes enclose an enormous amount of space, thus providing a large column-free area. Despite their slimness, they are some of the strongest and stiffest structures in existence. Historically, domes were built of masonry, and they were designed only for gravity loads. The masonry dome is built without any supporting shuttering with small mud bricks laid in a mud mortar. Masonry domes are very weak in resisting lateral loads. The analysis of a shell is concerned with two stresses, (i) the stress that acts in the meridional direction and (ii) the stress that acts in the parallel direction. Meridional forces (like the meridians, or lines of longitude, on a globe) are compressive and increase towards the base, while hoop forces (like the lines of latitude on a globe) are compressive at the top and tensile at the base. The hoop compressive stresses are maximum at the top of the dome and go on reducing towards the bottom. At a roll-down angle of 51.8°, the hoop compressive forces become zero and then the hoop tensile forces start increasing. The hoop tensile forces are thus maximum at the bottom ring of the dome, and as a result, the dome is subjected to maximum tensile stresses at its bottom ring. Thus, over a period, severe tensile cracks are seen at its bottom portion.

Croci [1] studied the theory of the design of masonry dome and its failure modes. The seismic behaviour of masonry domes, viz. Vaults of Hagia Sophia in Istanbul and St. Francis in Assisi, are discussed. The study was performed on a dome of rise 54 m and diameter 43 m. Seismic elastic analysis showed that the behaviour of the dome was symmetrical in the two major directions. Matsagar and Jangid [2] analytically investigated the seismic responses of structures retrofitted using base isolation devices. The retrofitting of various important structures using seismic isolation technique, by incorporating the layers of isolators at suitable locations is studied.

Historical buildings are selected to investigate the effectiveness of the base isolation in seismic retrofitting, using isolation devices, such as elastomeric bearings and sliding systems. It is observed that the seismic response of the retrofitted structures reduces significantly in comparison with the conventional structures depicting the effectiveness of the retrofitting done through the base isolation technique.

Narayanan and Sirajuddin [3] described the properties of the masonry elements which are to be used in the software to perform the nonlinear analysis. Brick masonry exhibits distinct directional properties due to mortar joints, which act as planes of weakness, resulting in brick masonry structures showing complex and nonlinear mechanical behaviour. For the experimental study, the authors considered three varieties of brick and three mixed proportions of mortar. Compressive strength, water absorption, modulus of elasticity and Poisson's ratio of bricks; and compressive strength, modulus of elasticity, Poisson's ratio and density of different mortars were determined. The appropriate values of parameters for nonlinear FE analysis of masonry structures were recommended. Michiels et al. [4] performed a parametric study of the masonry roof shells. Singly curved (cylindrical shell) and doubly curved (spherical shell) were analysed. Real earthquake ground motions were applied, and the deformations and maximum principal stresses were computed. It was found that the key parameters which influenced the seismic response of the structure are the rise, span and thickness of the dome. Lupasteanu et al. [5] studied the behaviour of byzantine churches under seismic response and a shake table study. Preservation of historic structures was their main goal and hence several retrofitting techniques were not allowed, as they would change the architectural view of the heritage structure. The structural system of these churches consists of columns, walls, vaults, and sustained compressive gravity load stresses. Consequently, because the domes located at the top of these churches are prone to earthquake damage, the seismic protection techniques were extremely important. The authors compared the seismic response of the fixed base byzantine churches, strengthened byzantine churches and the base-isolated churches, in which it was found that the performance of the base-isolated byzantine churches under seismic loading was superior.

Masonry domes have represented the monumental structures thus increasing their grandeur. For the preservation of these massive, monumental structures against the strong seismic excitations, meticulous analysis of the masonry domes becomes a necessity. The specific objectives of the study are (i) to analyse the seismic performance of the fixed base masonry dome structure, (ii) to analyse the seismic performance of the masonry dome structure with lead rubber bearings, (iii) to analyse the seismic performance of the masonry dome structure with friction pendulum system and (iv) to compare the seismic performance of the fixed base masonry dome structure, LRB base-isolated structure and FPS base-isolated structure.

7.2 Base Isolation for Masonry Dome

Base isolation is one of the most powerful tools of passive structural vibration control technologies. The isolators decouple the superstructure from its substructure, thus protecting it from the damaging effects of an earthquake. The isolation can be achieved by using elastomeric and friction bearings. As masonry domes are unreinforced, their thickness is kept high to resist the tensile stresses. Due to their heavy mass, masonry domes attract large seismic forces, making them vulnerable to seismic excitations. Due to earthquake-induced forces, masonry domes are subjected to tensile stresses at the bottom rings, leading to the development of cracks at the bottom. The seismic performance of masonry domes can be significantly improved by implementing base isolators. The design basis report, architectural drawings and the structural drawings of the case study Sabhamandap dome located at Aurangabad, Maharashtra, India, are used for this research. The case study dome is isolated by the Lead Rubber Bearing (LRB) system and Friction Pendulum System (FPS).

7.3 Analysis Method

The masonry dome is analysed using nonlinear time history method. The method employed in SAP2000 is an extension of the Fast Nonlinear Analysis (FNA) developed by Wilson. The method is extremely efficient and is designed to be used for structural systems which are primarily linear elastic but have a limited number of pre-defined nonlinear elements. In the FNA method, all nonlinearity is confined to the link/support elements. Table 7.1 presents the particulars of the masonry dome considered for the study.

Figure 7.1 presents the elevation of the masonry dome and Fig. 7.2 presents the plan view.

Table 7.1 Particulars of the masonry dome

Parameter	Value
Rise (m)	5.93
Span/radius of dome (m)	24.78
Radius of curvature (m)	15.88
Roll-down angle	52°
Height of the wall/cylinder height (m)	6.61
Thickness of the wall (m)	0.47

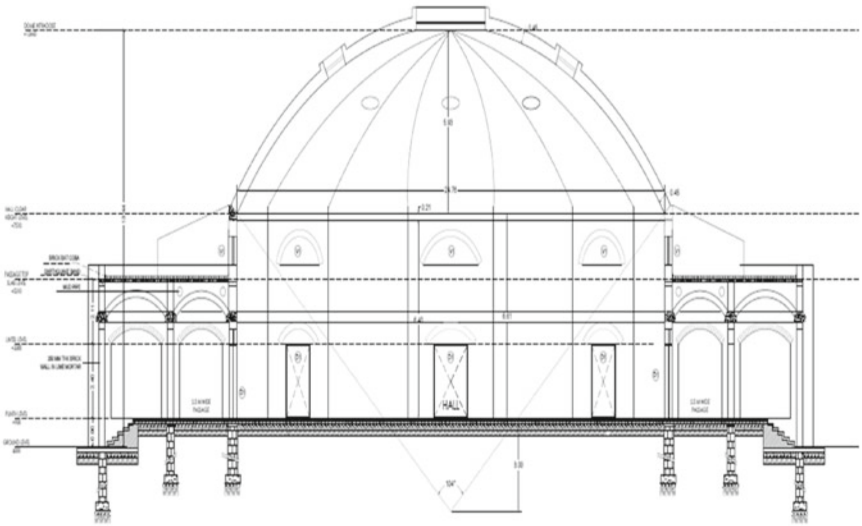


Fig. 7.1 Elevation of masonry dome. (Courtesy: Nandadeep Designers and Valuers Pvt Ltd, Aurangabad)

7.4 Simulation of Masonry Dome in SAP2000

The masonry dome structure is modelled using the shell element in SAP2000, as this element resists in-plane as well as out-of-plane bending moments. The radius of curvature of the dome, the roll-down angle (the angle with respect to vertical, up to which the dome extends) and the shell thickness are specified in SAP2000. The masonry material properties considered for simulation are modulus of elasticity = 2000 MPa, unit weight = 20 kN/m^3 and Poisson's ratio = 0.1. The masonry wall is simulated using a cylindrical shell element, which resists in-plane as well as out-of-plane bending moments. The geometry of the whole structure is created and gravity loads are applied. Further, nonlinear time history analysis is performed, considering three real earthquake ground motions, viz. El Centro (1940, PGA 0.33 g), Chamoli (1999, PGA 0.35 g) and Bhuj (PGA 0.1 g).

7.5 Masonry Dome Installed with Lead Rubber Bearings

Tables 7.2 and 7.3 respectively present the properties and dimensions of LRBs.

Parameters affecting the seismic performance of masonry dome, isolated by LRB are shear modulus of rubber and stiffness and damping of LRB, which are provided by the lead core. Table 7.4 presents the engineering properties of LRB used for nonlinear analysis.

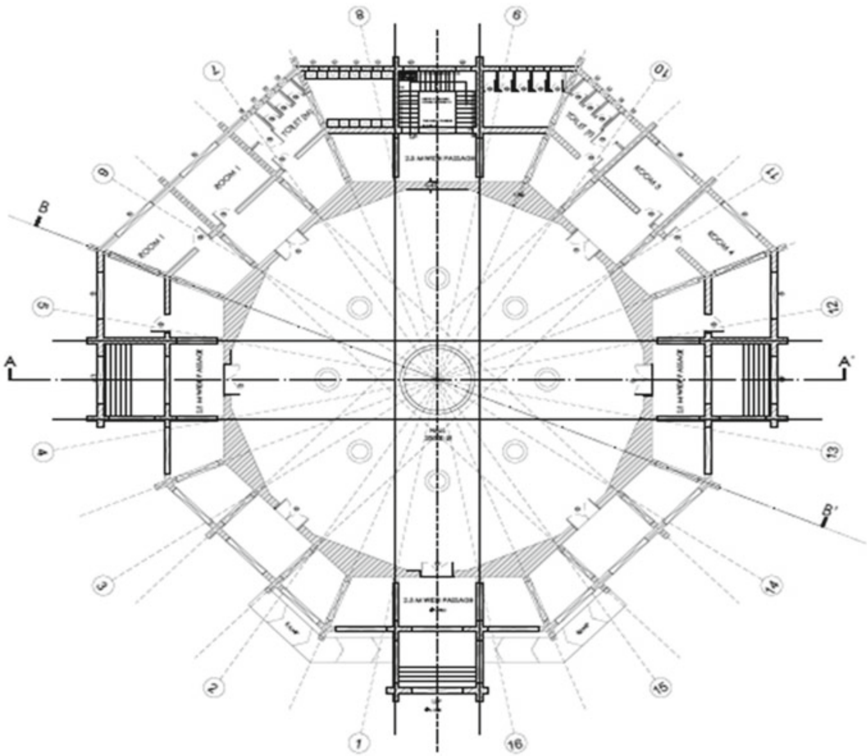


Fig. 7.2 Plan of masonry dome. (Courtesy: Nandadeep Designers and Valuers Pvt Ltd, Aurangabad)

Table 7.2 Properties of lead rubber bearings

Parameter	Value	Parameter	Value
Shear modulus (G) (MPa)	0.4	Bulk modulus (K) (MPa)	1500
Ultimate elongation (ε)	6.5	Elasticity modulus (E) (MPa)	1.35
Material constant (k)	0.87	Lead yield strength (σ_y) (MPa)	8

Table 7.3 Dimensions of lead rubber bearings

Property	Value	Property	Value
Plan dimensions (mm^2)	550 × 550	Side cover (mm)	10
Rubber layer thickness (mm)	12	Steel shim thickness (mm)	3
Number of rubber layers	20	Load plate thickness (mm)	25.5
Lead core diameter (mm)	120	Total height (mm)	348

Table 7.4 Engineering properties of LRB for nonlinear analysis

Parameter	Value
Initial horizontal stiffness (kN/m)	4663.19
Yield force (kN)	99.10
Post-yield stiffness ratio	0.08704

7.6 Masonry Dome Installed with FPS

Parameters affecting seismic analysis of masonry dome structure, isolated by FPS are the coefficient of friction (μ), radius of sphere (R), and the restoring force provided by the system. The post-sliding stiffness is determined by the geometry and supported weight as (W/R). The total force resisted by a spherical slider bearing is directly proportional to the supported weight. The engineering properties of FPS used for nonlinear analysis are presented in Table 7.5.

Figure 7.3 shows the elevation of the masonry dome modelled in SAP2000.

Table 7.5 Engineering properties of FPS for nonlinear analysis

Parameter	Value
Radius of pendulum (m)	1.924
Coefficient of friction (fast)	0.05
Coefficient of friction (slow)	0.11
Rate parameter	1.5
Maximum vertical load acting on each FPS (kN)	962
Effective horizontal stiffness (kN/m)	1000

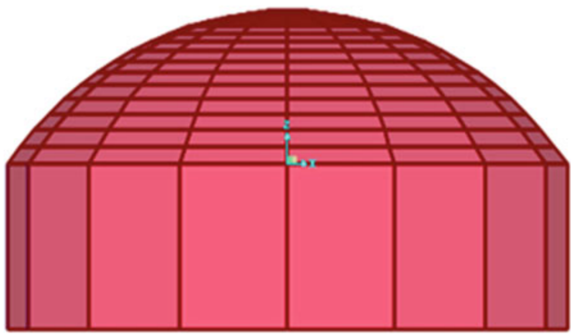


Fig. 7.3 Elevation of the dome structure (SAP2000)

7.7 Nonlinear Time History Analysis

In the nonlinear time history analysis in SAP2000, the isolators are modelled as link elements, and all nonlinearity is restricted to the link (support) elements. The results of the nonlinear time history analysis, viz. base shear, bending moments, tensile stresses, isolator forces and displacements are obtained for three cases, viz. (i) the fixed base dome, (ii) dome installed with LRB and (iii) dome installed with FPS. The analysis is performed considering three earthquake ground motions; however, the time history results are presented for El Centro (1940) earthquake.

(i) **Seismic Performance of Fixed Base Masonry Dome**

From modal analysis, the fundamental time period of the fixed base masonry dome is found to be 0.1838s, which reduces to 0.1097 s the in the 12th mode. Figure 7.4 and Table 7.6 present the response of fixed-base masonry dome.

(ii) **Seismic Performance of Masonry Dome isolated with Lead Rubber Bearings**

From modal analysis, the fundamental time period of the LRB-isolated masonry dome is found to be 1.8702 s, which reduces to 0.1063 s in the 12th mode.

From Fig. 7.5c, it is observed that up to a yield force of 99.10 kN, the initial stiffness of the isolator is high, i.e. 4663.19 kN/m. Hence, the deflection is very less up to the yield force. However, as the lead yields, it undergoes plastic deformation and thus deflection goes on increasing (see Table 7.7).

(iii) **Seismic Performance of Masonry Dome isolated with Friction pendulum System**

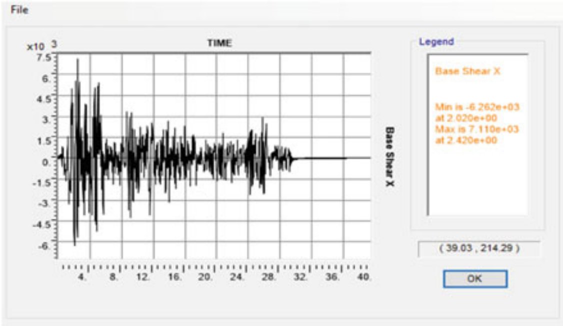
From modal analysis, the fundamental time period of the FPS-isolated masonry dome is found to be 1.7427 s, which reduces to 0.0701 s in the 12th mode. The time period is same in both directions, as the dome structure is symmetric in two directions. For the first two modes itself, the mass participation is 100%. As a result, the dynamic response is concentrated in these two modes itself (see Fig. 7.6 and Table 7.8).

7.8 Comparison of Fixed Base, LRB-Isolated and FPS-Isolated Masonry Dome

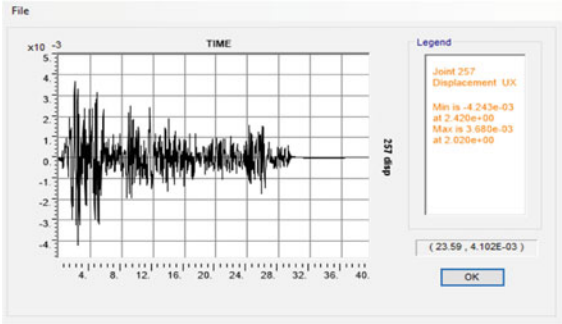
(a) Time period

From Fig. 7.7, the highest natural fundamental time period is found to be that of the LRB-isolated structure followed by FPS. However, it is observed that the first two modes have same time period as the structure is symmetric in nature. After first three modes, it can be observed that Fixed base, LRB, FPS have almost the same time periods for the next subsequent modes.

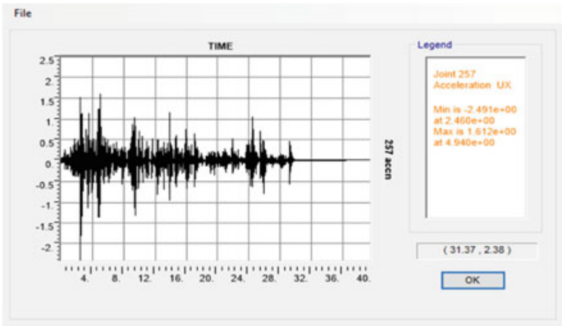
Fig. 7.4 Response of fixed base dome: El Centro earthquake (1940)



(a) Base shear response of dome

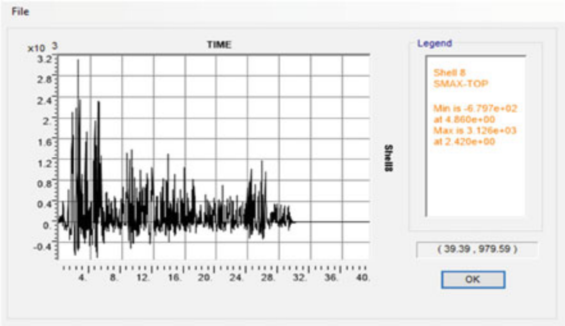


(b) Displacement at top joint of the dome



(c) Acceleration at top joint of the dome

Fig. 7.4 (continued)



(d) Shell stresses at the top of the dome

Table 7.6 Results of fixed base dome under different earthquakes

Parameter	El Centro	Chamoli	Bhuj
Maximum shell stresses (MPa)	3.12	0.33	1.152
Base shear (kN)	7110	6754	3311
Top displacement (mm)	4.24	3.97	1.964
Top acceleration (m/s^2)	2.491	0.84	1.08

(b) Maximum tensile stresses

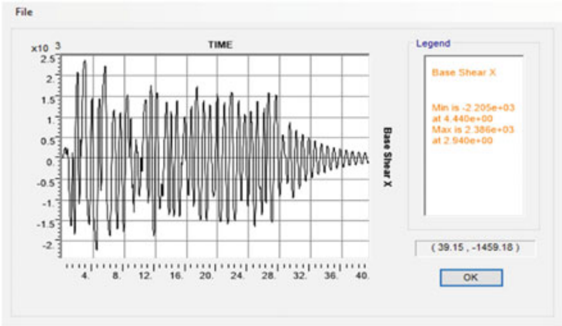
The maximum tensile stresses are found at the bottom ring of the masonry dome structure. The stresses are maximum for fixed base structure when subjected to El Centro earthquake input with 3.12 MPa the maximum value. It can be observed that though PGA of Chamoli earthquake is more than that of El Centro, tensile stresses are lesser. For Bhuj earthquake, FPS gives the best results, while LRB gives the best results for El Centro and Chamoli (see Fig. 7.8).

(c) Base shear

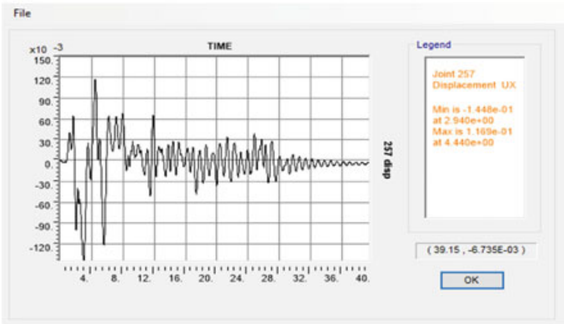
The base shear is maximum for fixed base structure when subjected to El Centro earthquake input with 7110 kN the maximum value. It can be observed from Fig. 7.9 that though PGA of Chamoli earthquake is more than that of El Centro, base shear is lesser. For Bhuj and El Centro earthquake, FPS gives the best results, while LRB gives a maximum reduction in base shear for Chamoli earthquake ground motion.

(d) Displacement at the top of the dome

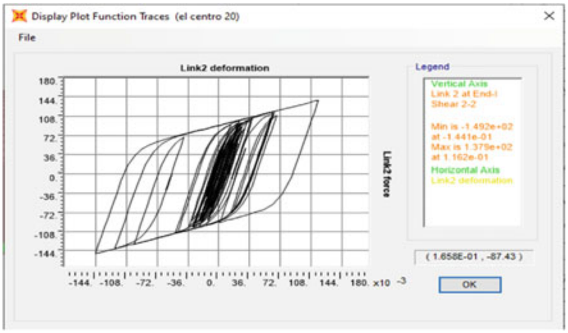
The displacement at the top of the dome is maximum for fixed base structure when subjected to El Centro earthquake input with 4.24 mm as the maximum value. It can be observed that though PGA of Chamoli earthquake is more than that of El Centro, displacement at the top is lesser. For Bhuj earthquake, FPS gives the best results, while LRB gives the best results for Chamoli and El Centro (see Fig. 7.10).



(a) Base shear response of LRB-isolated dome

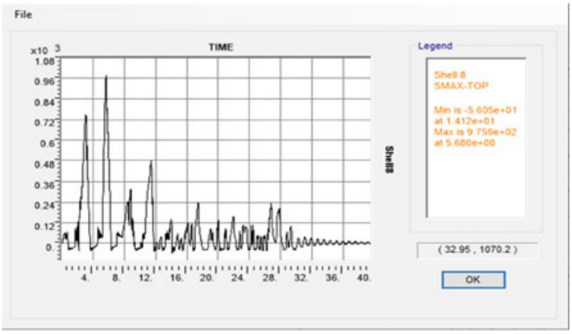


(b) Displacement at top joint of the LRB-isolated dome



(c) Hysteresis loop of the LRB isolator

Fig. 7.5 Response of LRB-isolated dome: El Centro earthquake (1940)



(d) Shell stress at top of the of LRB-isolated dome

Fig. 7.5 (continued)

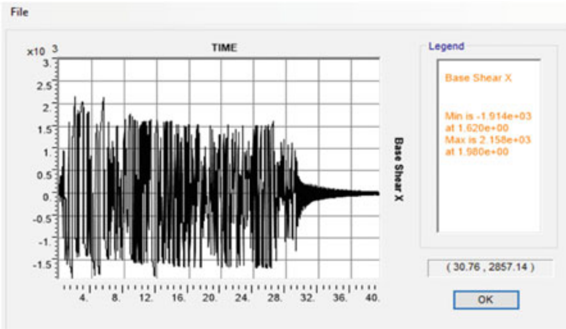
Table 7.7 Results of LRB-isolated dome under different earthquakes

Parameters	El Centro	Chamoli	Bhuj
Maximum shell stresses (MPa)	0.975	0.6	0.243
Base shear (kN)	2386	2850	1845
Top displacement (mm)	0.3	0.79	0.47

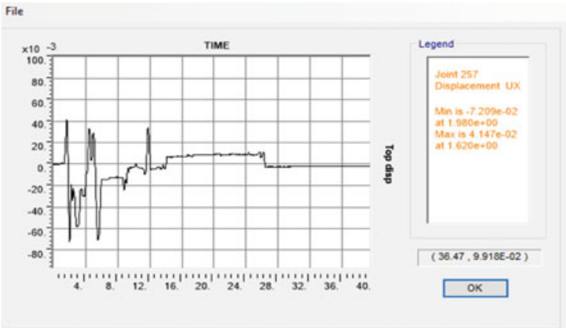
7.9 Conclusions

Response of fixed base and base-isolated dome is studied for three real earthquake ground motions. LRBs and FPSs are employed at the base of the dome. From the nonlinear time history analysis performed in SAP2000, the following conclusions are drawn.

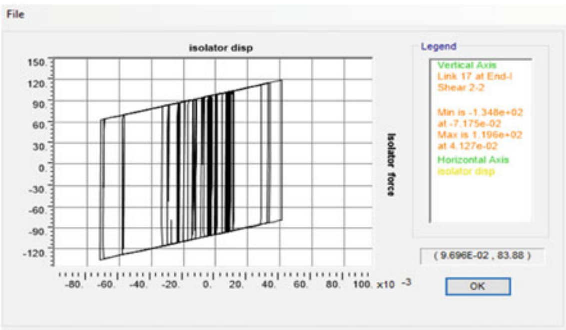
1. LRB and FPS isolators significantly reduce the tensile stresses, base shear and displacement of the top of the dome, compared to the fixed base dome, thereby reducing the structural damages during strong ground shaking.
2. Base isolation increases the time period of the structure resulting in reduced earthquake-induced forces on the structure.
3. When subjected to low seismic forces, the FPS is comparatively more efficient in the reduction of the base shear than the LRB. Further, it also reduces the tensile stresses and displacements of the apex of the dome than LRB does and thus resulting in comparatively low structural damage.
4. When subjected to high PGA earthquake ground motions, the LRB system is more efficient in the reduction of tensile stresses, displacements and the base shear of the masonry dome structure.



(a) Base shear response of the FPS-isolated dome

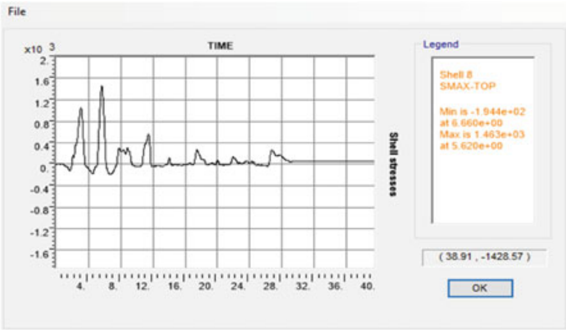


(b) Displacement at top joint of the FPS-isolated dome



(c) Hysteresis loop of the FPS isolator

Fig. 7.6 Response of FPS-isolated dome: El Centro earthquake (1940)



(d) Shell stress at top of the of FPS-isolated dome

Fig. 7.6 (continued)

Table 7.8 Results of FPS-isolated dome under different earthquakes

Parameter	El Centro	Chamoli	Bhuj
Maximum shell stresses (MPa)	1.463	0.971	0.2
Base shear (kN)	2156	3091	1642
Top displacement (mm)	0.34	0.9	0.09

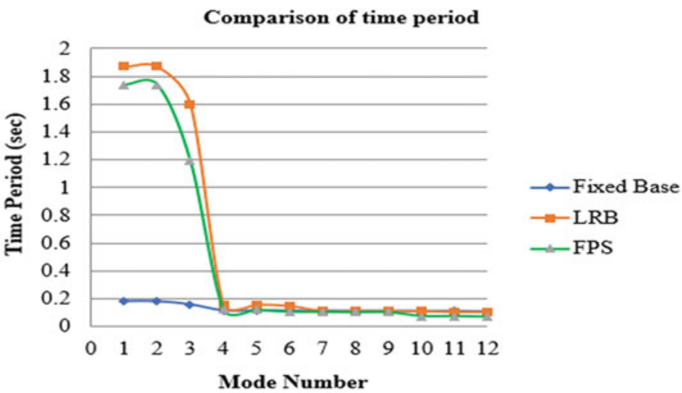


Fig. 7.7 Comparison of fundamental time period

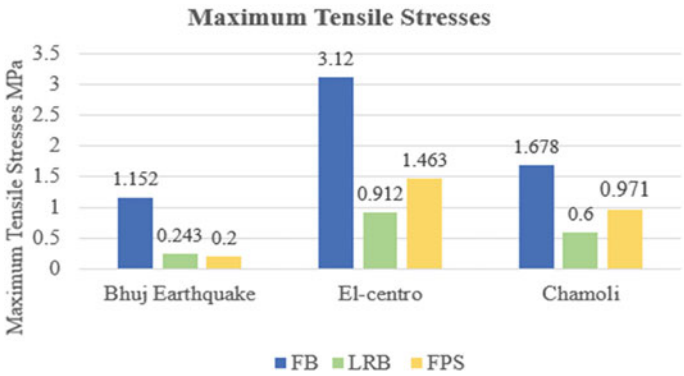


Fig. 7.8 Comparison of maximum tensile stress

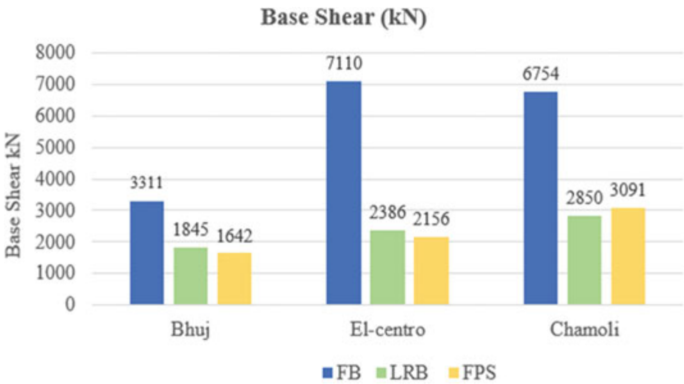


Fig. 7.9 Comparison of base shear

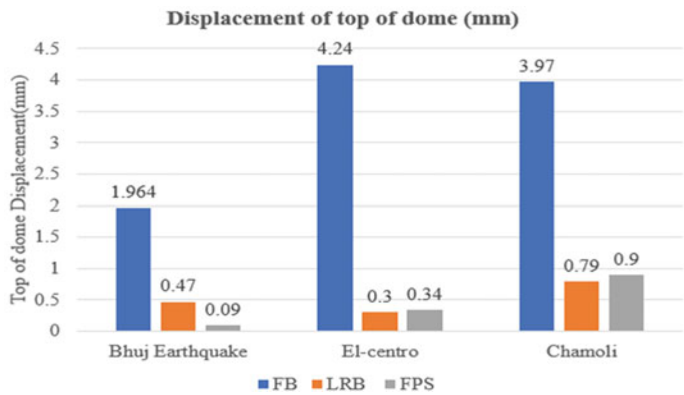


Fig. 7.10 Comparison displacement at the top of the dome

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