A Comparative Study on the Seismic Behavior of Ribbed, Schwedler, and Diamatic Space Domes by Using Dynamic Analyses

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SUMMARY:
In this paper three steel single-layer common types of lattice domes have been studied under gravity and earthquake loads to obtain the optimal geometry of each type and introduce the more appropriate one to be used in seismic areas. The domes have the same span of 40m, and their height-to-span ratio varies from 1/2, 1/2.5 1/3, 1/4, 1/5, 1/6, 1/7 and 1/8. At first the domes have been designed based on their analysis statically under self-weight and snow loads. For earthquake loads equivalent static, spectral, and Time History Analysis (THA) methods have been employed. The THA cases have been conducted by using the accelerograms of a set of high-frequency earthquakes of both near- and far-source types. Numerical results show that the stress ratios in the considered domes subjected to near-field earthquakes are higher than far field earthquakes, and that Ribbed domes have better seismic behavior than Schwedler and Diamatic domes.

Keywords: Height/Span ratio, Effective mass in lateral or vertical directions, Optimal geometry of the dome

1. INTRODUCTION

Lattice domes are known as very efficient structures for covering large spans, and several types of them are presently in use worldwide. Some studies have been done so far on the response of these structures subjected to earthquake, but, due to their light weight, their seismic response has not been considered as determining in most parts of the world. However, in highly seismic areas, particularly in near filed, the seismic forces can be dominant in design of these structures. A few studies have been done in this regard among them some are mentioned here. Ogawa et al. (2003) conducted a study on earthquake response analysis of single layer lattice domes with response spectrum analysis, to examine the validity of response spectrum analysis that requires less time in computation. They used and compared the absolute summation, the SRSS the CQC methods for response spectrum analyses.

Lia et al. (2006) studied the structural optimization and dynamic analysis for double-layer spherical reticulated shell structures. Their study was concerned with the geometrical optimum design and the aseismic analysis of double-layer reticulated shell structures. They investigated the characteristic of free vibration of reticulated shell structures, with respect to geometric parameter. They also discussed variations of the eigenfrequency of shell structures, with respect to the height-to-span ratio, span, grid division frequency and thickness of shell.

Abdolpour et al. (2010) worked on estimation of statically equivalent seismic forces of single layer reticular domes. In that study, dynamic responses of single layer reticular domes subjected to horizontal earthquake motion by using scaled near field earthquake accelerations, different masses, densities and different members of domes were investigated. They proposed some relations for estimating base shear force and seismic forces in different levels of the dome. They claimed that by using those relations, seismic forces could be estimated accurately without any need to time-consuming dynamic analysis and complicated mathematical calculations.

In this paper three steel single-layer common types of lattice domes, including Ribbed, Shwedler, and Diamatic, have been studied under gravity and earthquake loads to find out the optimal geometric form of each type and introduce the more appropriate one among the three aforementioned types for use in seismic areas. The domes have the same span of 40m, and their height-to-span ratio of varies from 1/2, 1/2.5 1/3, 1/4, 1/5, 1/6, 1/7 and 1/8. The domes were firstly designed by analyzing statically under gravity (i.e. dead and snow) loads. For earthquake loads the equivalent static, spectral, and Time History Analysis (THA) methods have been employed. The THA cases have been conducted by using the accelerograms of a set of high-frequency earthquakes of both near-source and far-source types. The details of study are presented in the following sections of the paper.

2. GENERAL FEATURES OF THE DOMES AND THEIR PRELIMINARY DESIGN

Three types of domes, including Diamatic, Schwedler, and Ribbed were considered in this study, whose plan general views are shown in Figure 1 then elevation view of the Diamatic dome is shown in Figure 2.

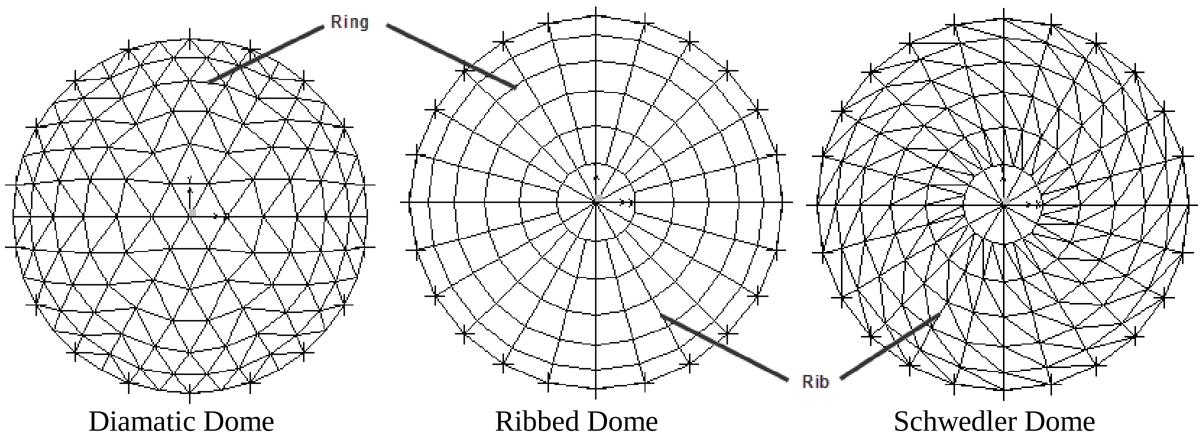

Figure 1. The plan view of the considered three types of domes

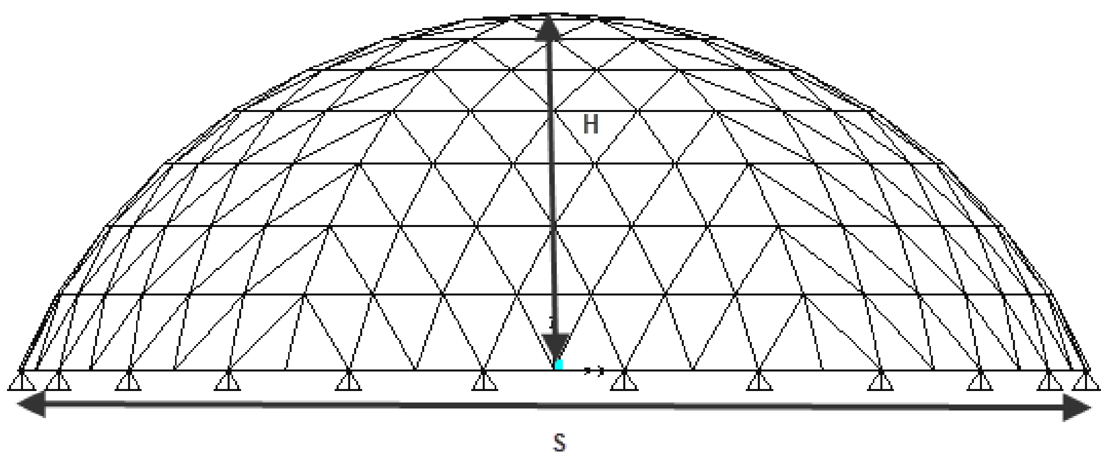

Figure 2. Elevation view of the Diamatic dome

All the considered domes are single layer with the moment resistant connections and the supports are hinge connection. In all cases the length of the dome elements varies in the range of 1.5 to 4.5 m, and they are of the tube section type. The snow load and the cover load have been assumed to be 150 and 15 kgf/m^2, respectively. The preliminary design of all does has been base on AISC-ASD 89, and the soil type of the site has been assumed to be of class B (Hajnasrollah 2012).

The first set of parameters which can be compared in the three considered types of domes is the number of whole structural elements required for each type, as well as the number of required ribs, and finally the number of required supports. These numbers are important for the amount of effort required for construction of domes, as well as the related costs. These numbers are shown in Table 1 for different H/S ratios of the three considered types of domes for comparison.

Table 1. Number of structural elements in the three considered types of domes with respect to the H/S ratio

H/S	Dome type								
	Diamatic			Ribbed			Schwedler		
	No. of elements	No. of rings	No. of supports	No. of elements	No. of rings	No. of supports	No. of elements	No. of rings	No. of supports
1/2	600	8	24	496	8	32	720	8	32
1/2.5	600	8	24	496	8	32	720	8	32
1/3	600	8	24	496	8	32	720	8	32
1/4	342	6	18	368	6	32	528	6	32
1/5	342	6	18	368	6	32	528	6	32
1/6	342	6	18	368	6	32	528	6	32
1/7	342	6	18	368	6	32	528	6	32
1/8	342	6	18	368	6	32	528	6	32

Table 1 shows that for relatively higher domes number of rings is 8, while for relatively shorter ones this number is 6. Furthermore, it can be seen in Table 1 that the number of supports for shorter domes are a little less than that of higher ones in case of Diamatic domes, while this number is the same for all case of Ribbed and Schwedler domes, which is also more than that number in Diamatic domes. This is because of the fact that Ribbed and Schwedler domes have same number of ribs in their design, however, Diamatic domes have different interconnection pattern and, therefore, different number of supports. The second important parameter for comparison is the total weights of the domes and its variation with respect to the H/S ratio. Table 2 shows the total weights of the three considered types of domes with respect to the H/S ratio.

Table 2. Total weights of the three considered types of domes with respect to the H/S ratio

H/S	Dome type		
	Diamatic (ton)	Ribbed (ton)	Schwedler (ton)
1/2	22.1	28	42.8
1/2.5	15.8	11.1	17.4
1/3	14.6	10.1	15.8
1/4	14	9.8	15.5
1/5	**13.4**	9.5	15
1/6	14.1	9.2	14.5
1/7	14.1	**9.1**	**14.2**
1/8	19.2	10.3	15.9

It can be seen in Table 2 that in all cases the maximum weight corresponds to largest H/S value, and with decrease in the H/S ratio the dome weight of any type decreases first and reaches a minimum value (shown in the Table with **bold** figures), and then start increasing again. This means that for each type of domes there is an optimal H/S ratio. Comparing the three types, it is seen that the minimum weight in achieved in case of Ribbed dome with H/S ratio of 1/7, and this type is generally more economical, and the other two types are almost the same, as long as the weight is concerned. The increase of weight for very low values of H/S ratio can be due to the domination of bending action in the dome rather than the compression is such cases.

3. DYNAMIC CHARACTERISTICS OF THE DESIGNED DOMES

As the dynamic characteristics of the three considered types of the designed domes their dominant mode (mode with the highest modal mass ratio) in either horizontal or vertical direction and its period, and their corresponding modal mass ratio have been considered. These values related to horizontal direction of dome motion are given in Table 3, with respect to H/S ratio.

Table 3. Dominant modes and related periods and modal mass ratios for horizontal direction of domes motion

H/S	Dome type								
	Diamatic			Ribbed			Schwedler		
	No. of the Dominant Mode	Period (sec)	Modal Mass Ratio	No. of the Dominant Mode	Period (sec)	Modal Mass Ratio	No. of the Dominant Mode	Period (sec)	Modal Mass Ratio
1/2	1	0.166	0.726	1	0.258	0.649	2	0.165	0.598
1/2.5	1	0.150	0.389	3	0.233	0.453	1	0.164	0.596
1/3	1	0.141	0.505	1	0.225	0.278	2	0.158	0.457
1/4	111	0.052	0.238	1	0.234	0.335	1	0.172	0.158
1/5	16	0.053	0.4	146	0.055	0.193	148	0.048	0.342
1/6	110	0.051	0.457	146	0.056	0.296	147	0.048	0.271
1/7	110	0.050	0.548	146	0.056	0.289	147	0.048	0.315
1/8	110	0.045	0.634	144	0.055	0.338	146	0.046	0.321

It is seen in Table 3 that in all case, for domes with relatively large value of H/S ratio the dominant mode is one of the first modes (modes with the lower frequencies), while for lower values of H/S ratio one of the higher modes, sometimes modes with very high frequencies, are the dominant mode. It is also seen in Table 3 that the natural periods of domes vary in range of 0.045 to 0.258 sec, with the largest value related to the highest Ribbed dome, and the smaller value related to either Diamatic or Schwedler dome with smallest height. On this basis, it can be said that domes of the considered types and sizes are generally high frequency structures, and therefore, are more sensitive to high frequency earthquakes, including most of near-source ones. However, the level of their sensitivity is different, even in each type of dome, due to the different values of modal mass ratio of the dominant mode with for different values of H/S ratio. The dominant mode (mode with the highest modal mass ratio) in vertical direction of the domes motion and its period, and corresponding modal mass ratio for all considered domes have been shown in Table 4 with respect to H/S ratio.

Table 4. Dominant modes and related periods and modal mass ratios for vertical direction of domes motion

H/S	Domes type								
	Diamatic			Ribbed			Schwedler		
	No. of the Dominant Mode	Period (sec)	Modal Mass Ratio	No. of the Dominant Mode	Period (sec)	Modal Mass Ratio	No. of the Dominant Mode	Period (sec)	Modal Mass Ratio
1/2	203	0.030	0.207	196	0.056	0.244	185	0.051	0.126
1/2.5	194	0.055	0.315	97	0.110	0.152	27	0.106	0.128
1/3	193	0.062	0.476	117	0.113	0.089	134	0.079	0.146
1/4	109	0.076	0.67	134	0.078	0.718	114	0.084	0.509
1/5	6	0.090	0.363	128	0.093	0.808	109	0.094	0.671
1/6	109	0.102	0.708	122	0.107	0.852	106	0.107	0.765
1/7	103	0.117	0.756	114	0.123	0.875	101	0.121	0.799
1/8	73	0.118	0.385	101	0.132	0.874	85	0.131	0.781

One can see in table 4 that there is a general trend of decrease in the dominant mode number in vertical direction of domes motion as the H/S ratio decreases. However, contrary to the case of horizontal motion, the value of dominant period increases almost in all three types of the considered domes as the value of H/S ratio decrease. These periods are in the range of 0.030 to 0.132 sec, which is relatively shorter than those related to the horizontal direction of domes motion.

4. SEISMIC ANALYSIS OF THE DESIGNED DOMES

For seismic analysis, including Time History Analysis (THA), of the designed domes in order to evaluate their seismic behavior, six set of three-component earthquake accelerograms have been chosen according to the soil type of class B, of which three ones correspond to near-source earthquake and the other three ones correspond to far-source, as shown in Table 5.

Table 5. Three-component earthquake accelerograms chosen according to the soil type of class B

Earthquake	Near-source			Far-source		
	Northridge	Morgan Hill	Palm Spring	San Fernando	Loma Prieta	Whittier Narrow
PGA UP	0.562	0.485	0.634	0.185	0.191	0.081
PGA 00	0.313	0.266	0.350	0.350	0.357	0.350
PGA 90	0.350	0.350	0.329	0.289	0.325	0.289

As shown in Table 5, in all of the selected earthquakes the Peak Ground Acceleration (PGA) value of one of the horizontal components is exactly equal or very close to 0.35g, used and the design value. It is also seen that among the selected earthquakes the vertical PGA values of Northridge, Palm Spring and Morgan Hill earthquakes are larger than their corresponding horizontal PGA values, regarding the fact that they are near-source records. As samples of acceleration time history and response spectra of the selected earthquakes, the acceleration record of Northridge earthquake are shown in Figure 3, and the pseudo acceleration spectra related to vertical and the dominant horizontal component of all selected earthquakes are shown in Figures 4 to 9.

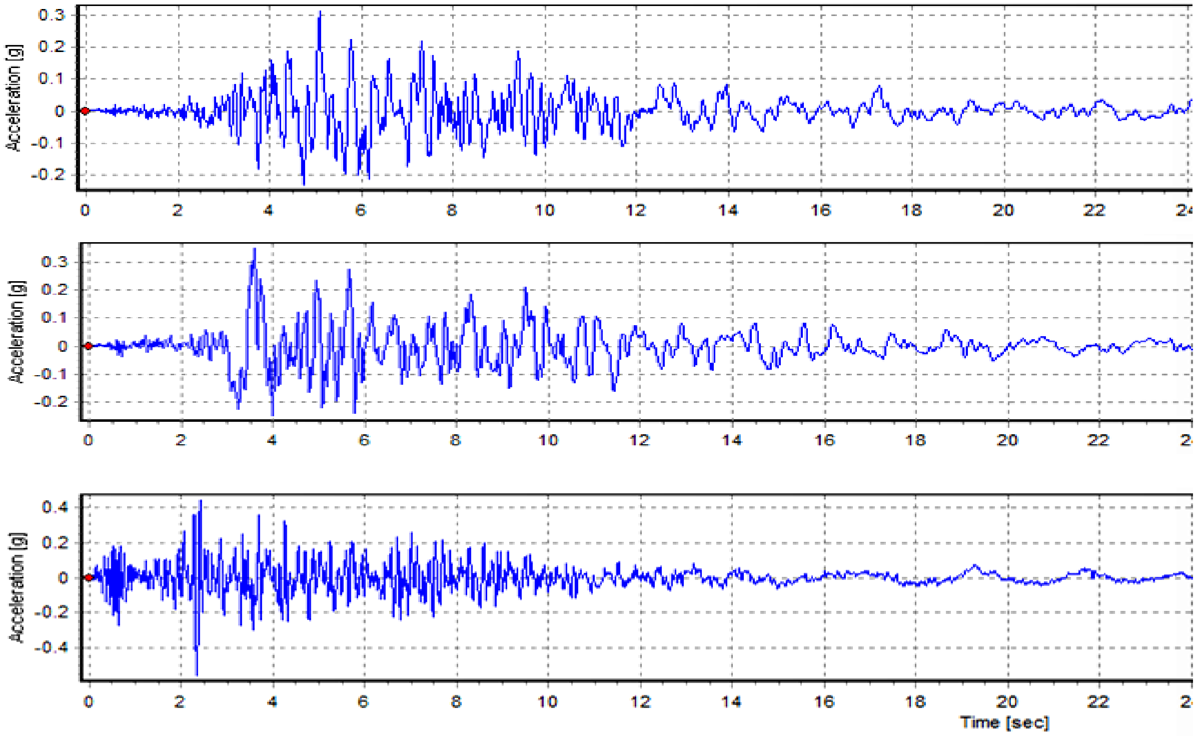

Figure 3. The acceleration time histories of vertical and horizontal components of Northridge earthquake

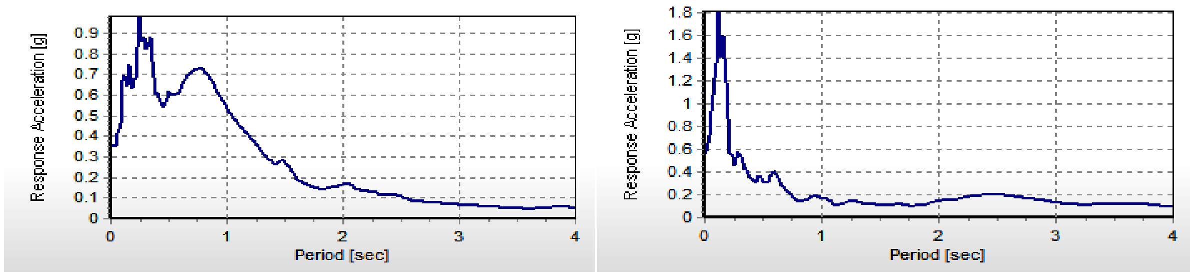

Figure 4. The pseudo acceleration spectra of the horizontal and vertical components of Northridge earthquake

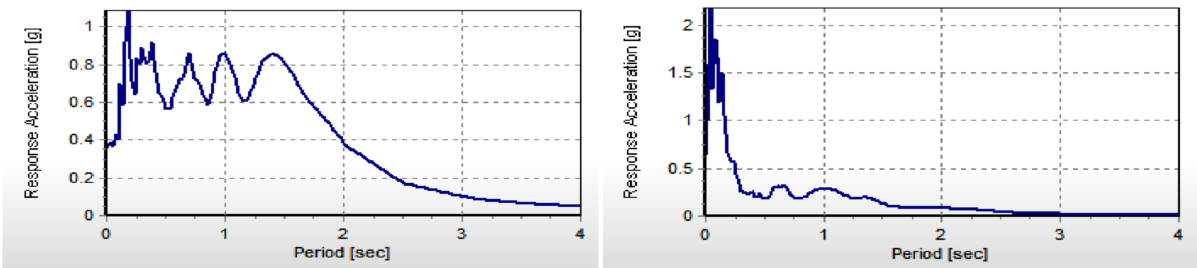

Figure 5. The pseudo acceleration spectra of the horizontal and vertical components of Palm Spring earthquake

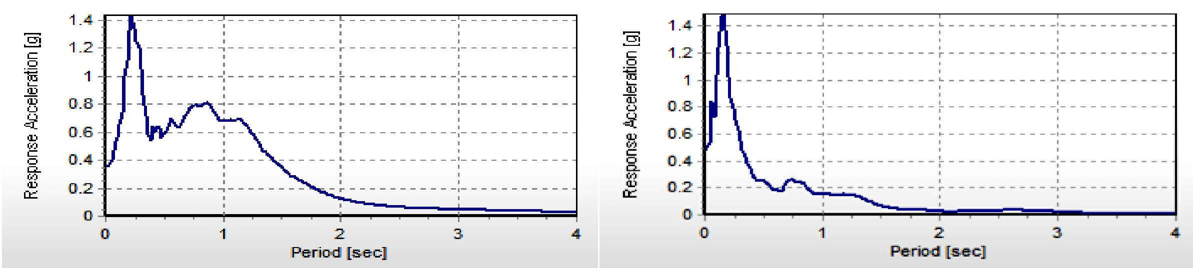

Figure 6. The pseudo acceleration spectra of the horizontal and vertical components of Morgan Hill earthquake

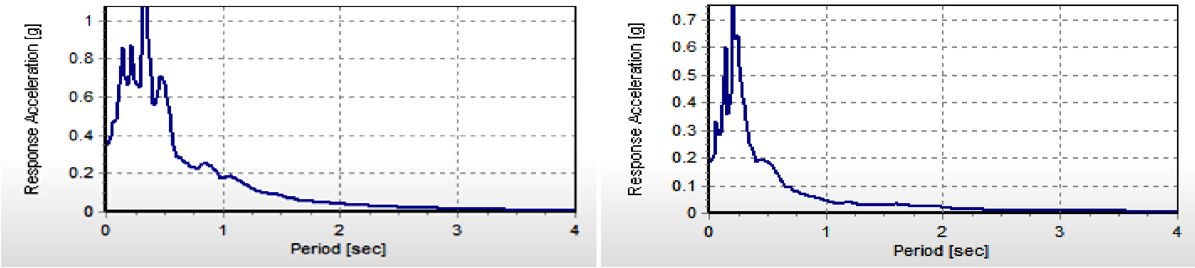

Figure 7. The pseudo acceleration spectra of the horizontal and vertical components of San Fernando earthquake

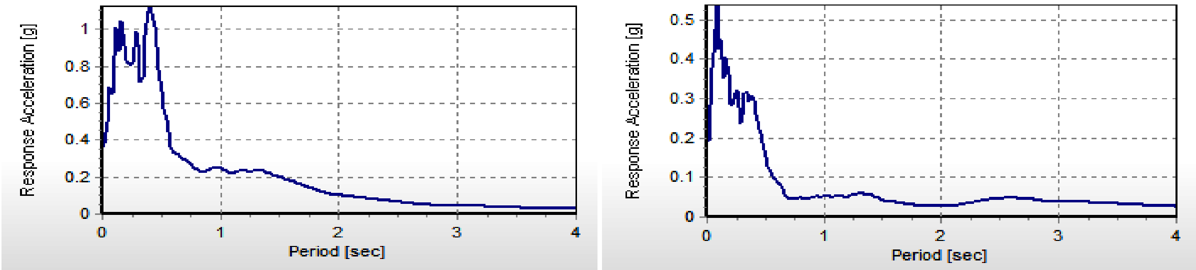

Figure 8. The pseudo acceleration spectra of the horizontal and vertical components of Loma Prieta earthquake

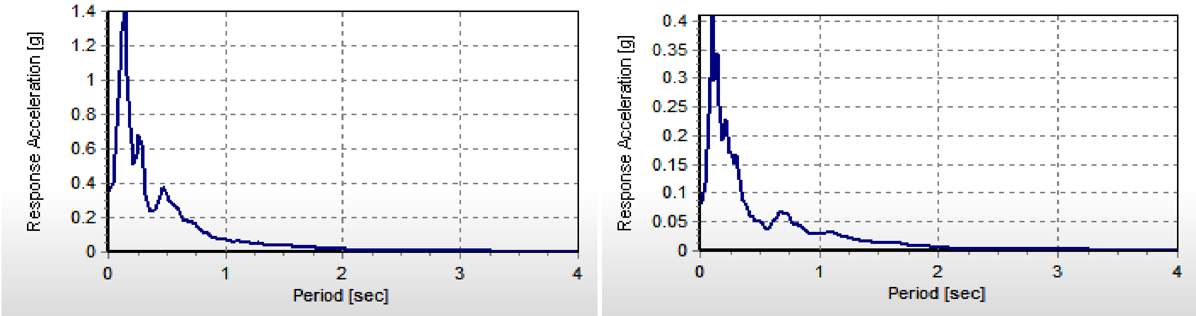

Figure 9. The pseudo acceleration spectra of the horizontal and vertical components of Whittier Narrow Earthquake

It is seen in Figures 4 to 9 that the dominant period of all the considered records is less than 0.30 sec and mostly less than 0.25 sec. It can also be seen that the dominant period of the vertical component in each case is less than that of the horizontal component. As the first set of results obtained from seismic analysis of the designed domes, the maximum stress ratios in the members of each dome based on its spectral analysis by using all of the pseudo acceleration spectra of the considered earthquakes, shown in Figure 4 to 9, are given in Table 6, and are shown in Figure 10 for better comparison. In these calculations a damping value of 2% has been assumed.

Table 6. Max stress ratios obtained by spectral analysis

H/S	Dome type		
	Diamatic	**Ribbed**	**Schwedler**
1/2	0.848	0.744	0.757
1/2.5	0.937	0.929	0.954
1/3	0.846	0.940	0.978
1/4	0.838	0.768	0.879
1/5	0.946	0.789	0.797
1/6	0.838	0.883	0.943
1/7	0.926	0.978	0.995
1/8	0.890	0.907	0.981

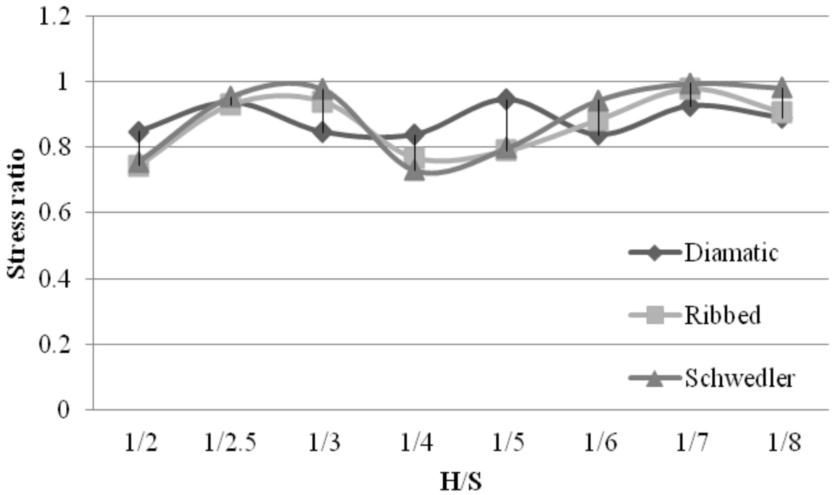

Figure 10. Maximum stress ratios versus H/S values

It is seen in Table 6 and Figure 10 that in all cases the stress ratio values are between 0.75 and 1.0, which means that all of the designed domes satisfy the criteria of spectral design as well. As the second set of the seismic analyses results, the maximum stress ratios obtained based on linear THA by using the three-component accelerograms of the six selected far- and near-fault earthquakes are given in Tables 7 to 12, where the maximum for each dome type are shown in **bold** figures to be better seen, and cases in which the stress level in a member has exceeded the buckling level are shown as Not Calculated (N/C). In all cases of THA a damping value of 2% has been assumed for all types of domes.

Table 7. Max stress ratios in Northridge

H/S	Domes type		
	Diamatic	**Ribbed**	**Schwedler**
1/2	1.317	1.007	0.929
1/2.5	**1.718**	1.689	1.695
1/3	1.063	1.139	1.207
1/4	1.076	1.043	1.017
1/5	1.301	1.445	1.638
1/6	1.184	**2.687**	**4.197**
1/7	1.171	2.62	1.676
1/8	N/C	2.347	1.908

Table 8. Max stress ratios in Palm Spring

H/S	Domes type		
	Diamatic	**Ribbed**	**Schwedler**
1/2	1.185	1.036	0.95
1/2.5	1.407	1.738	1.956
1/3	1.037	1.144	1.225
1/4	1.118	1.137	1.179
1/5	**1.629**	1.383	1.645
1/6	1.079	1.848	2.509
1/7	1.206	**2.304**	1.88
1/8	N/C	0.975	**2.516**

Table 9. Max stress ratios in Morgan Hill

H/S	Domes type		
	Diamatic	**Ribbed**	**Schwedler**
1/2	**2.472**	1.028	1.071
1/2.5	1.47	1.474	1.848
1/3	1.063	1.122	1.162
1/4	1.079	1.116	0.894
1/5	1.194	1.383	1.11
1/6	1.029	1.467	1.719
1/7	1.241	**2.687**	**2.241**
1/8	N/C	1.74	2.117

Table 10. Max stress ratios in San Fernando

H/S	Domes type		
	Diamatic	**Ribbed**	**Schwedler**
1/2	1.096	0.975	0.929
1/2.5	**1.34**	1.115	1.318
1/3	1.119	1.054	1.125
1/4	1.001	0.948	0.797
1/5	1.129	0.979	0.912
1/6	0.964	1.058	1.099
1/7	1.053	**1.29**	1.173
1/8	N/C	1.171	**1.327**

Table 11. Max stress ratios in Loma Prieta

H/S	Domes type		
	Diamatic	**Ribbed**	**Schwedler**
1/2	1.442	0.99	1.97
1/2.5	**3.295**	1.368	**2.559**
1/3	1.075	1.139	1.204
1/4	1.038	1.095	0.849
1/5	1.195	1.093	0.951
1/6	1.047	1.261	1.113
1/7	1.122	**1.579**	1.226
1/8	N/C	1.079	1.187

Table 12. Max stress ratios in Whittier Narrows

H/S	Domes type		
	Diamatic	**Ribbed**	**Schwedler**
1/2	**1.238**	0.972	0.966
1/2.5	N/C	1.347	**1.318**
1/3	1.155	1.057	1.188
1/4	1.014	1.047	0.902
1/5	1.284	1.089	0.967
1/6	1.096	1.125	1.15
1/7	1.058	**1.351**	1.16
1/8	N/C	1.047	1.148

It can be seen in Tables 7 to 12 that stress ratios in several cases has gone above 1.0, and has even exceeded 2.0, 3.0 or 4.0 in some cases, which means that in those earthquakes some elements of the dome structure will experience inelastic deformation and the dome will get damage. Table 13 shows the dominant periods as well as their corresponding Spectral Acceleration (SA) values for all three components of the six earthquakes.

Table 13. Peak SA values and their corresponding period values for the six earthquake components

Earthquake	Near-source						Far-source					
	Northridge		Morgan Hill		Palm Spring		San Fernando		Loma Prieta		Whittier Narrow	
	Acceleration (g)	Period (sec)	Acceleration (g)	Period (sec)	Acceleration (g)	Period (sec)	Acceleration (g)	Period (sec)	Acceleration (g)	Period (sec)	Acceleration (g)	Period (sec)
PGA UP	1.80	0.12	1.50	0.12	1.35	0.05	0.8	0.25	0.58	0.10	0.41	0.10
PGA 00	0.92	0.50	1.15	0.20	1.07	0.20	1.10	0.35	1.30	0.40	1.40	0.20
PGA 90	0.97	0.50	1.45	0.20	0.83	1.75	0.70	0.24	1.15	0.20	0.83	0.30

It can be seen in Table 13 that vertical component of Northridge earthquake has the highest SA value among the used earthquakes, while Morgan Hill earthquake has the highest SA in horizontal components as well as overall. The peak frequencies of the used records are not similar, and that is why in each case of H/S ratio and each dome type some specific earthquake has resulted in the maximum response in the structure.

4. CONCLUSIONS

Based on the numerical results of this study it can be concluded that:

- Behaviors of Schwedler and Ribbed domes, which are more similar in configuration, are also more similar.
- In all three types of studied domes near-source earthquakes have been more effective.
- Among the three types of designed domes Schwedler domes are the most seismically vulnerable, and Ribbed domes are the less seismically vulnerable domes.
- The largest stress ratio are observed in Schwedler domes, while the largest number of member in which the amount of axial force exceeds the buckling load of the member is observed Diamatic domes.
- As expected higher domes with higher H/S values (say more than 1/4 or 1/5) are more vulnerable to horizontal earthquakes are more effective, and domes with lower H/S values are more susceptible to the vertical earthquake.

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