



SEISMIC BEHAVIOUR OF SPACE STRUCTURES

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SUMMARY

Investigations show that the seismic behaviour of space structures may fundamentally differ from the conventional structures. This paper presents the results of dynamic analyses of double-layer barrel vaults with different configurations and support conditions subjected to a severe earthquake, and highlights some basic influential elements in the seismic performance of the space structures. The significance of the presence of snow loads on dynamic characteristics and seismic performance of space structures is emphasized. Various hysteresis models were used, and the effects of strength reduction and energy dissipation in compression struts were investigated. The results of seismic analysis generally indicate that the induced seismic forces are enormously larger than the conventional seismic forces. The consequences of this finding on seismic design of space structures have been discussed, and remedial suggestions have been proposed.

INTRODUCTION

In a survey on the seismic behaviour of space structures during the 1995 Hyogoken-Nanbu earthquake in Japan, Saka and Tanigushi [1] did not reported any full or partial collapse amongst over one hundred double-layer and many gymnasiums with single-layer space structures in the meizoseismal region. In effect, many of these structures were used to shelter the people who lost their homes during the earthquake. This promising behaviour has been confirmed by other similar reports. The key to the superior behaviour of space structures is their relatively light self weight, high degree of redundancy, and appropriate 3-dimensional geometrical form.

During severe earthquakes, space structures behave differently from most ordinary structures in many ways:

- i. In space structures, the ratio of snow and wind loads to the dead load is markedly greater than in ordinary buildings. As a structure should be designed for these loads, in the absence of snow and wind, the space structures have a much more reserve capacity than the ordinary structures. This reserve capacity helps space structures to sustain relatively high seismic loads with little inelastic deformation, whereas ordinary structures need to undergo large inelastic deformation because their strengths are normally a small fraction of the elastic demand forces. This has a predominant influence on determining the factors that account for inelastic performance of structures in earthquakes such as the response modification factor R , and the ductility ratio μ .

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- ii. The probability of coincidence of snow and earthquake loads does not play a significant role in ordinary buildings, because the snow loads are usually a negligible fraction of the total weight. On the contrary, snow loads can be easily up to 2 to 3 times the self weight of a space structure. Therefore, it is essential to establish a strategy for combining snow and earthquake loads in the design stage.
- iii. Unless supported by an ordinary ductile substructure, many multiple-layer space structures cannot develop a safe failure mechanism with plastic hinges, and instead, they make inelastic excursions through buckling of axial members. Such failure mechanism is recognized as undesirable because it is accompanied by heavy strength deterioration, and may result in a progressive failure. Therefore, as opposed to ordinary structures where we rely upon their capability for inelastic deformation through safe failure mechanisms to discount seismic forces, space structures should be designed for much higher seismic forces due to their shortage of ductility.
- iv. In ordinary frame buildings, the horizontal and vertical modes are clearly separated and uncoupled, and the horizontal response of building is predominantly governed by the first mode known as the fundamental mode. Thus, the vertical component of earthquake does not usually affect the horizontal response significantly, and therefore, it is ignored in the design of seismic resistant systems such as bracings and shear walls. On the other hand, due to complicated nature of space structures, this may not prevail, and the effects of vertical component of earthquake and higher modes of vibration should be considered.

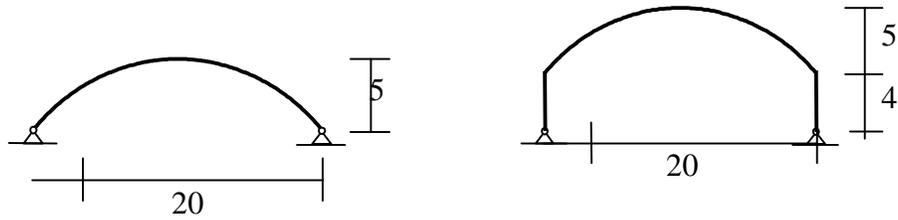
The main objective of the present study is to highlight some basic characteristics of seismic behaviour of space structures, in conjunction with the above issues.

During the last decade, numerous investigations on seismic behaviour of space structures have been reported [1-9]. Saka and Tanigushi [1] reported a number of cases where the delamination of concrete around anchor bolts and supports was found to be the dominant mode of failure during a severe earthquake. Pulling off and failure of anchor bolts at the bearing supports, inelastic buckling of members in the vicinity of supports, and fracture between ball joints and members were also reported [1]. It will be shown later that these types of failure result from underestimation of seismic loads in the design of joints and bearing supports. Similar observations have been reported by Kawaguchi [2]. Karamanos and Karamanos [3] discussed the effect of flexibility of supports and the progressive failure. Kunida [4] investigated the elastic response of shells to horizontal and vertical seismic excitations. Yamada [5] showed that the dynamic response of barrel vaults can be predicted by cylindrical shells with sufficient accuracy. Kato et al [6-8] conducted extensive research on seismic behaviour of space structures. They investigated the effect of vertical component of earthquake on single layer reticular domes [7]. They demonstrated that a dome designed for a gravity load of 180 kg/m² with a safety factor of 3, would initiate to yield under Kobe's vertical motion at a peak acceleration of 583 gal. It can be concluded that such structures may undergo yielding during many severe earthquakes.

In the present study, double-layer barrel vaults with two different configurations were subjected to seismic loads at presence of snow load, and their response was studied in elastic and inelastic ranges. In some models, columns were provided to act as a flexible substructure. In inelastic analyses, various hysteresis models were used for axial members to represent different types of post buckling behaviour.

MODELS

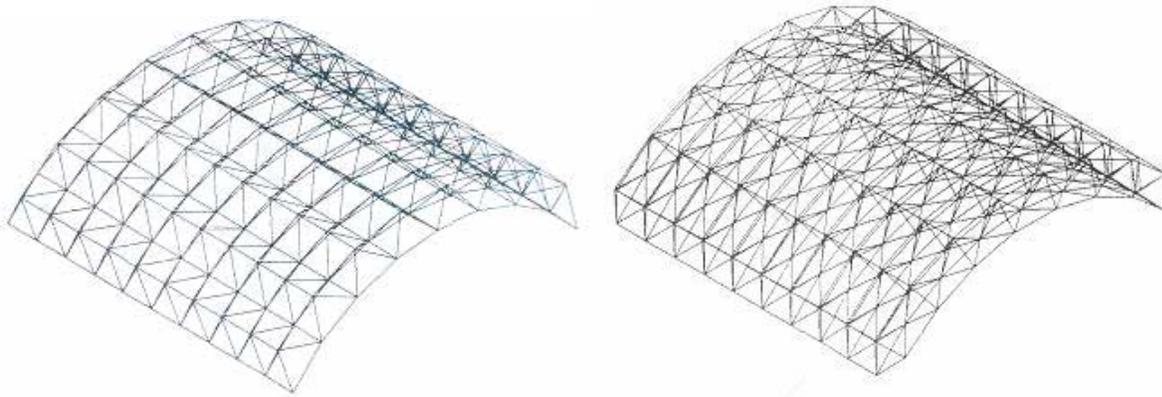
4 different barrel vault models were chosen. These models are designated as M1, M2, M3, and M4. As shown in Fig. 1, in M1 and M2 there is no column available, and they are seating directly onto the ground. Conversely, in M3 and M4, the arch has been seating on columns. The models consist of two-way double layer grids. As shown in Fig. 2, in M2 and M3, a rectangular grid is used for both upper and lower layers, and in M1 and M4, the lower layer is replaced with a diagonal grid.



a. models M1 and M2

b. models M3 and M4

Figure 1. geometry and support conditions



a. models M1 and M4

b. models M2 and M3

Figure 2. configuration

Model Idealization

To minimize the amount of computation efforts, it was decided to analyze one module instead of the whole structure. These modules were also designated as M1, M2, M3, and M4. As shown in Fig. 3, the width of each module is 2 m. It was assumed that no displacement can occur in transverse direction. The total mass was distributed evenly between the nodes.

To study the seismic performance of the models, they were needed to be designed first. The dead and the snow loads were assumed as equal to 50 kg/m², and 150 kg/m², respectively. Therefore, the total gravity load, equals to 8 ton for each module (2x20x200 kg). This gravity load was applied as vertical point loads to the nodes, and the models were proportioned to sustain these forces. Therefore, the models were primarily designed for loading combination D+S, where D and S represent dead weight and snow, respectively. The members were designed from hollow steel tubes. The slenderness ratio of the compression members was limited to 200. Subsequently, the models were subjected to the above loading, and the design was modified accordingly. This went on until no further modification was needed.

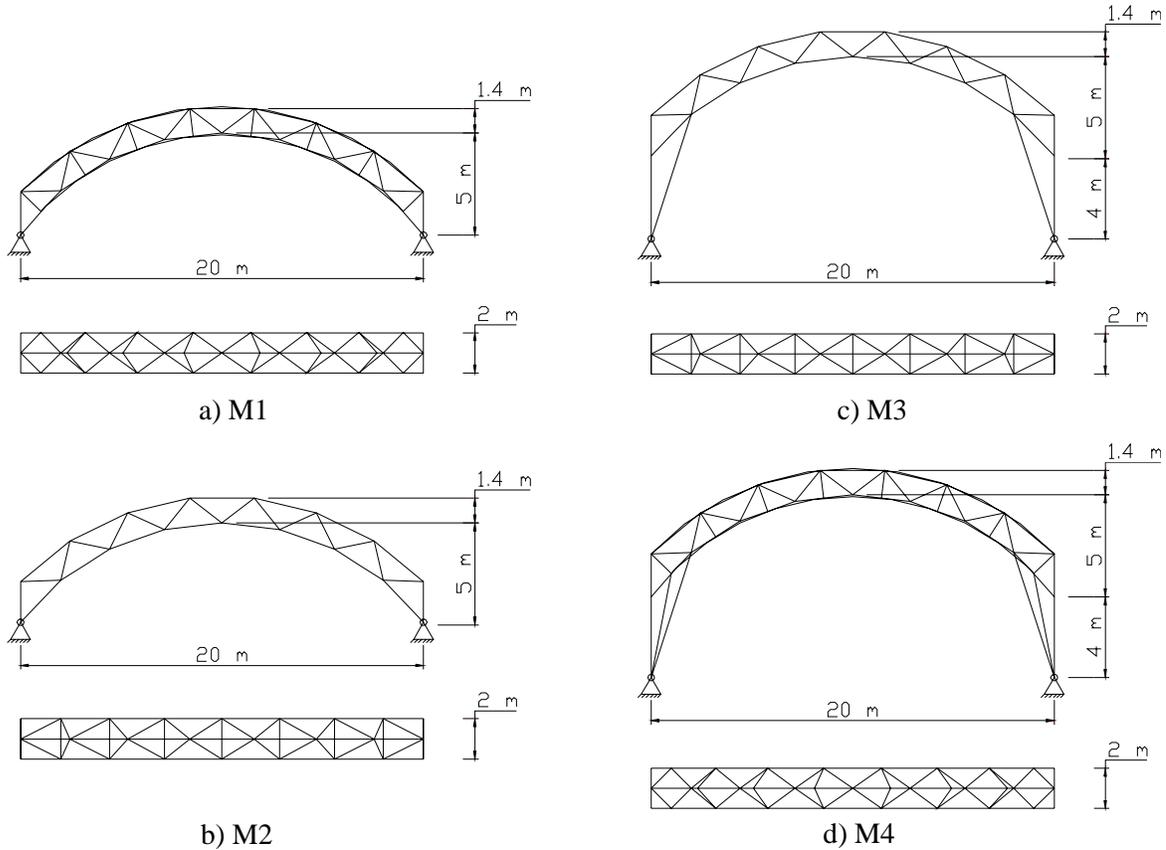


Figure 3. Module Models

LINEAR SEISMIC ANALYSIS

Seismic Loading

The models were subjected to two types of loading: i. Nominal seismic loads, ii. Naghan earthquake 1977. The nominal seismic loads were calculated according to the Iranian seismic code with a response modification factor R of 6. The amplification factor B was assumed as equal to 2 for all modes. This resulted in an identical shear coefficient of $C=0.1167$ for all modes.

The second type of seismic loading consists of a ground motion acceleration applied to the supports. The acceleration response of Naghan earthquake is shown in Fig. 4. The Naghan earthquake was applied both in horizontal and vertical direction. However, in vertical direction a scaling factor of $2/3$ was used. Due to low level of damping in space structures as compared with ordinary buildings, the damping ratio was assumed as equal to 0.02.

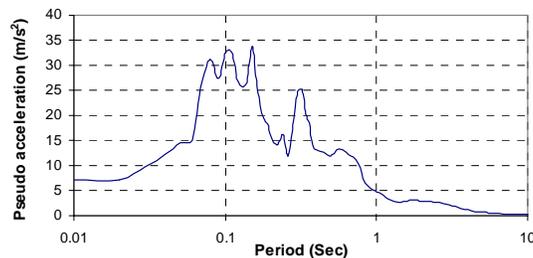


Figure 4. Acceleration response spectrum of Naghan Earthquake 1977 for 2% damping.

Modes and Frequencies

One significant factor in evaluating the effect of earthquake on space structures is the amount of snow mass that is present during an earthquake. In ordinary buildings, the presence of snow does not affect the seismic response, because the ratio of snow mass to the total mass is very small. However, this ratio can easily exceed 1 in space structures locating in mountainous or cold regions. The presence of snow can produce various effects: i. Inertial seismic forces are increased due to an increase in mass, ii. The overall strength decreases as the snow load is applied, iii. Frequency decreases with an increase in snow mass. In the present study, various amounts of snow loads were applied to the models, and these effects were investigated. Fig. 5 shows the variation of the fundamental period with respect to the amount of snow. The results indicate that the presence of snow load increases the fundamental periods of all four models. For example, the fundamental period of M1 increases from 0.133 to 0.211 Sec as the snow mass increases from 30 to 150 kg/m². Similarly, the fundamental period of M3 increases from 0.306 to 0.483 at this range. It can also be concluded that configuration has a negligible effect on the fundamental period, as in this figure M1 and M3 are, respectively, very close to M2 and M4. On the other hand, the results indicate that the geometry plays a significant role, as the presence of columns in M4 has resulted in an average increase of 145% in period with respect to M1.

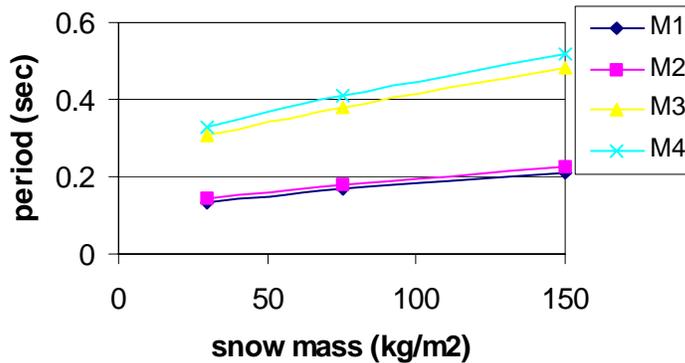


Figure 5. variation of period with snow mass

The frequencies and participating masses of M1 and M3 are displayed in Table 1. To compare the results with ordinary frame buildings, the frequencies and participation masses of a typical 20 storey building are also given. The results indicate that the fundamental frequency of M3 is much less than M1. This is due to the presence of columns in M3 that acted as a rather flexible substructure. It can also be seen that the participation of the fundamental mode exceeds 80% in both M3 and the 20 storey structure. On the other hand, in M1, the fundamental mode has participated in dynamic response by only 59.7%. Hence, it can be concluded that the number of modes that contribute in dynamic response is higher in space structures with rigid supports as compared with ordinary buildings. As shown in Table 1, the first mode of M1 is a horizontal mode with a frequency of 7.5 Hz, and the second and third modes are vertical modes with frequencies of 13.9 and 21.7 Hz, respectively. Therefore, due to presence of both horizontal and vertical ground motion in real earthquakes, there should occur a marked coupling between horizontal and vertical modes, whereas these modes can be easily uncoupled in ordinary buildings.

Table 1. Frequencies and participating masses

Mode No	20 storey building		M1				M3			
			frequency		Participating mass%		frequency		Participating mass%	
	Freq.	P. mass %	Horiz . mode	Verti. mode	Horiz .	Vert.	Horiz . mode	Verti. mode	Horiz .	Vert.
1	1	80.7	7.5		59.7		3.2		93.5	
2	3.02	10.1		13.9		27.2		9.8		29
3	5.15	3.4		21.7		51.6	19.6		4.2	
4	7.78	1.75	23.8		16.8			27		31.2
5	9.49	1.06		32.3		4.3		30		1.9
6	11.72	0.72	35.7		17.7		38.5		.7	
7	13.99	0.52	40		6.4		41.7		0	
8	16.31	0.39		41.7		1.2	41.7		0	
9	18.69	0.30	41.7		.2		41.7		.7	
10	21.11	0.24		66.7		13.7		47.6		.6

Seismic Forces

As discussed before, all models were subjected to nominal seismic forces, as well as ground motion. Due to the significance of coincidence of earthquake and snow for space structures, the models were loaded with various amounts of snow from 20% to 100% (i.e. 30 to 150 Kg/m²). The base shear force was calculated in each case. It was found that M1 and M2 have very similar base shears. The same applies to M3 and M4. Therefore it can be concluded that the configuration of these space structure models has not affected the seismic loading. The base shears are illustrated in Fig. 6 for M1 and M3. These results correspond to nominal seismic loads and the Naghan earthquake. The results indicate that the nominal seismic loads are far less than what a structure experiences during a real earthquake. For example, as the nominal seismic load for M1 with 30 Kg/m² snow load was obtained as 0.24 ton, the structure would experience some 4.95 ton in Naghan earthquake in the elastic region. However, the structure may not sustain such a large shear force, and therefore, it responds inelastically. In such case, a reduction in base shear is normally observed. This will be discussed later. The elastic base shear in M3 is even larger, as it reaches 9.08 ton under a snow load of 150 kg/m². The total weight of the model including the snow is 8 ton (2x20x200 kg). Therefore, it can be deduced that the elastic seismic forces can exceed the weight of structure.

It is interesting to notice that the presence of snow load increases the seismic loads significantly. For example, as the snow load increases from 30 to 150 kg/m², the base shear in M1 increase from 0.24 to 0.60 ton under nominal seismic loading, and from 4.95 to 7.76 under Naghan earthquake, respectively. Hence, it can be concluded that as opposed to ordinary buildings, the probability of coincidence of snow and earthquake plays a significant role in seismic response of most space structures, and therefore, it should be taken into account by seismic codes in developing regulations for seismic loading of space structures.

Fig. 6 also indicates that the vertical base reaction due to vertical excitations can be relatively large. For example, the vertical base shear of M1 was equal to 7.76 ton at a snow load of 150 kg/m². Thus, the vertical components of earthquakes can produce vertical seismic forces up to the weight of a structure, and even more. As the strategy for allowing a structure to respond inelastically can be different for horizontal and vertical loads, the generation of such large values of vertical forces in real earthquakes should be dealt with special attention.

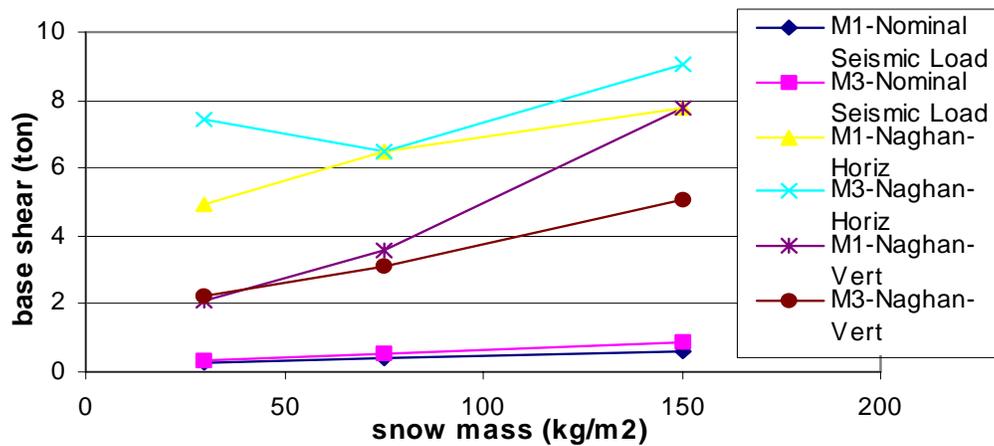


Figure 6. Seismic base shear force s of M1 and M3

NONLINEAR SEISMIC ANALYSIS

Hysteresis models

In the present study, 8 hysteresis models were employed and incorporated in a dynamic computer programme to analyze M1, M2, M3, and M4 in nonlinear range. As shown in Fig. 7, these hysteresis models are designated as P, Q, U, V, W, X, Y, and Z. X describes an unlimited linear elastic behaviour, and has been used as a reference point. V corresponds to an ideal elasto-plastic behaviour, without any strength deterioration and stiffness degradation. This model represents the characteristics of either very stocky compression struts where no buckling occurs, or those with a yielding fuse. Y is similar to V in tension, and it behaves differently in compression as it unloads along the line of zero stiffness until it reaches the last yielding threshold. This model idealizes the hysteresis behaviour of elastic slender struts around their zero-stiffness plateau. In this model, no energy dissipation takes place in compression cycles. Q is similar to Y, except that it asymptotes by a hyperbolic curve to a reduced strength equal to 80% of the buckling load. In Z, an 80% reduction takes place immediately after buckling, and the unloading path is similar to ideal elasto-plastic systems. It should be noticed that the member can regain its full compressive strength only after it has undergone some tensile yielding. In W, the strength decreases after buckling along a hyperbolic line, and asymptotes to 20% of the buckling load. Once unloaded, it runs towards the last threshold of tensile yielding, and it regains its full compressive strength after undergoing tensile yielding. P and U are similar to W, except that their asymptotic compressive strengths are equal to, respectively, 50% and 80% of the buckling load.

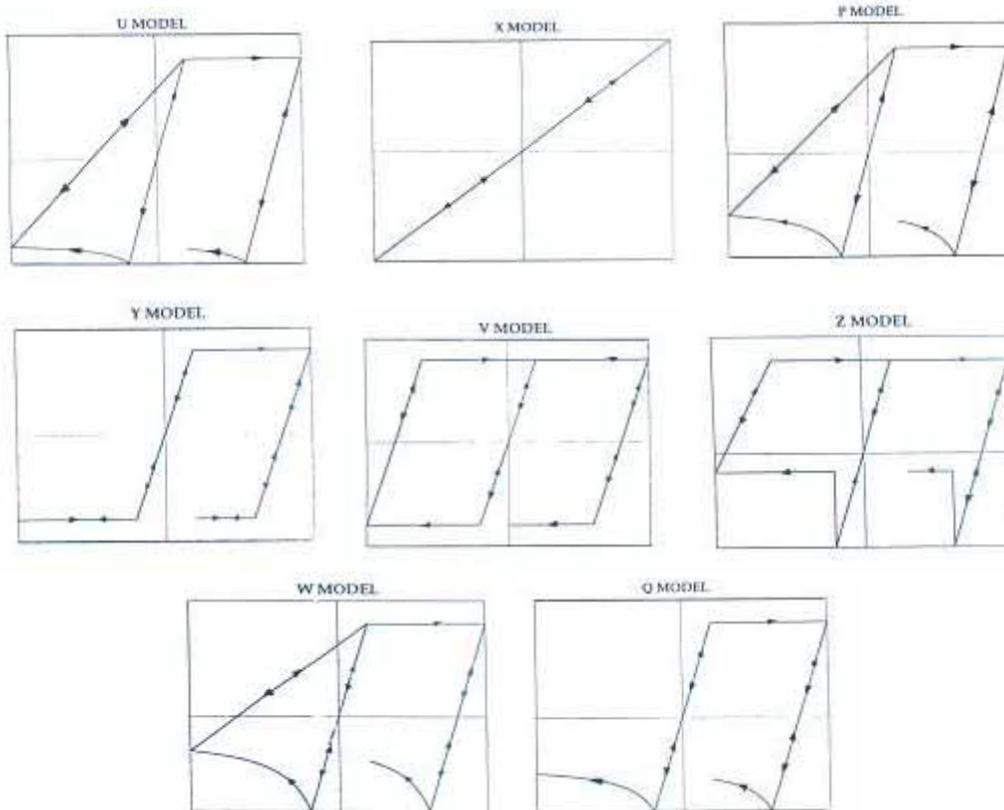


Figure 7. Hysteresis models

Analyses

The computer program SOHRAB was written for nonlinear dynamic analysis of space structures. The program applies no limit to the number of nodes, and it can take any type of hysteresis model. The accuracy of this program was checked with DRAIN-2DX. The results of several analyses were found to match quite satisfactorily. This program was then used to conduct nonlinear analyses. Models M1 to M4 were subjected to the horizontal acceleration of Naghan earthquake with a scale factor R varying in the range of 0.1 to 2.5. The maximum horizontal and vertical displacements of M1 and M4 are shown in Figs. 8 and 9. These figures display the variation of displacement versus the scale factor R (of the ground acceleration) for various hysteresis models.

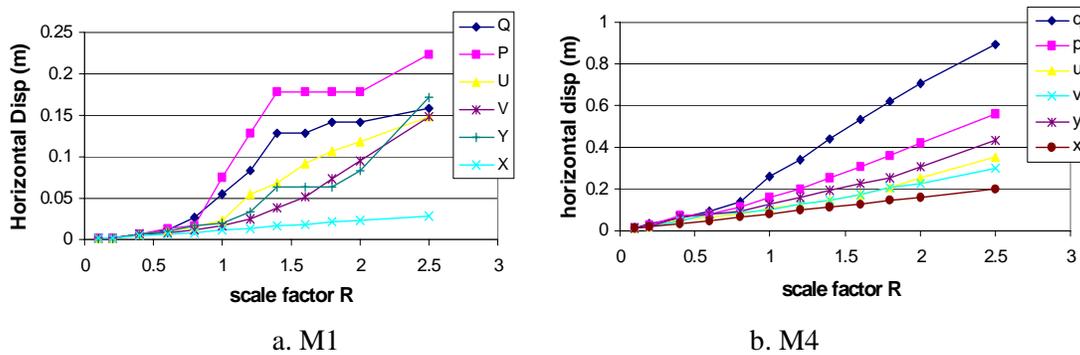
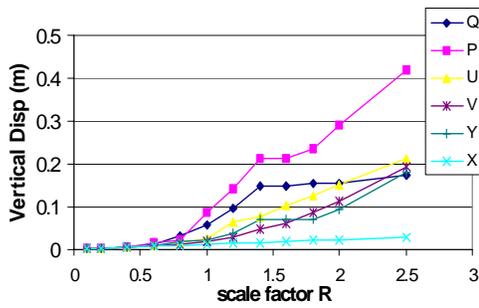
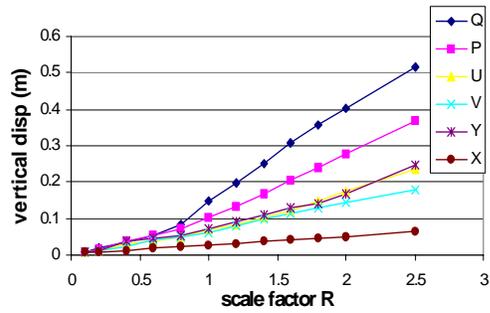


Figure 8. Horizontal peak displacement of M1 and M4 subjected to the Naghan earthquake with various scale factors.



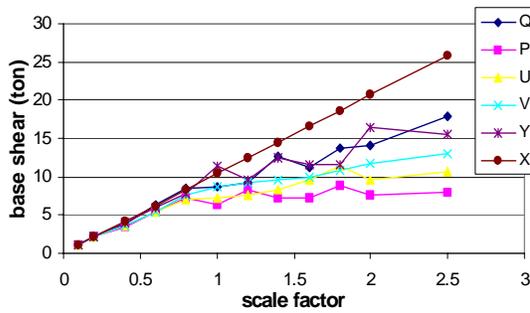
a. M1



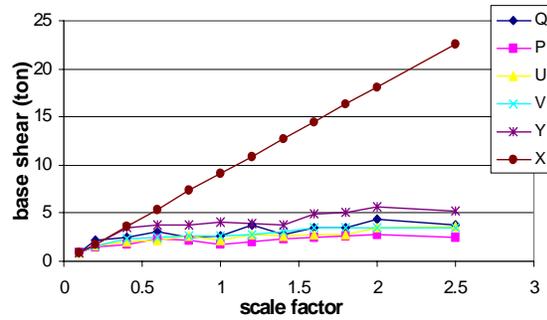
b. M4

Figure 9. Vertical peak displacement of M1 and M4 subjected to the Naghan earthquake with various scale factors

The maximum horizontal base shear has been given in Figs. 10 and 11 for various hysteresis models. Fig. 10 shows the variation of the horizontal base shear with the acceleration scale factor R, and Fig. 11 demonstrates maximum base shear versus displacement. The hysteresis models have been designated as P, Q, U, V, X, Y, and Z. To make a better judgment on the results, it is reminded that the total mass of each model was equal to 8 ton, including the snow load.

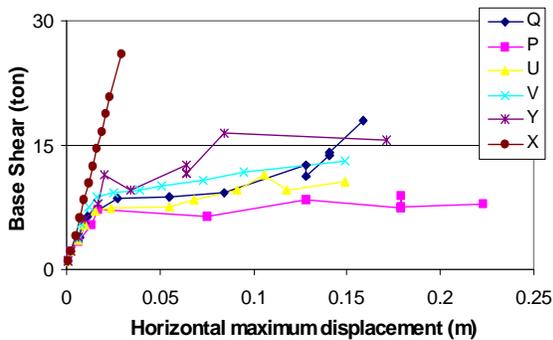


a. M1

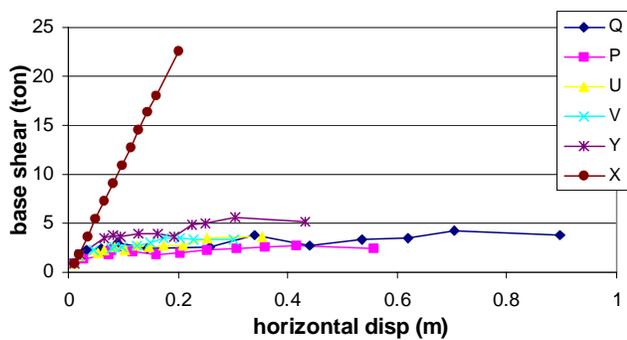


b. M4

Figure 10. maximum horizontal base shear



a. M1



b. M4

Figure 11. maximum horizontal base shear

Effect of Hysteresis Behaviour on Seismic Response

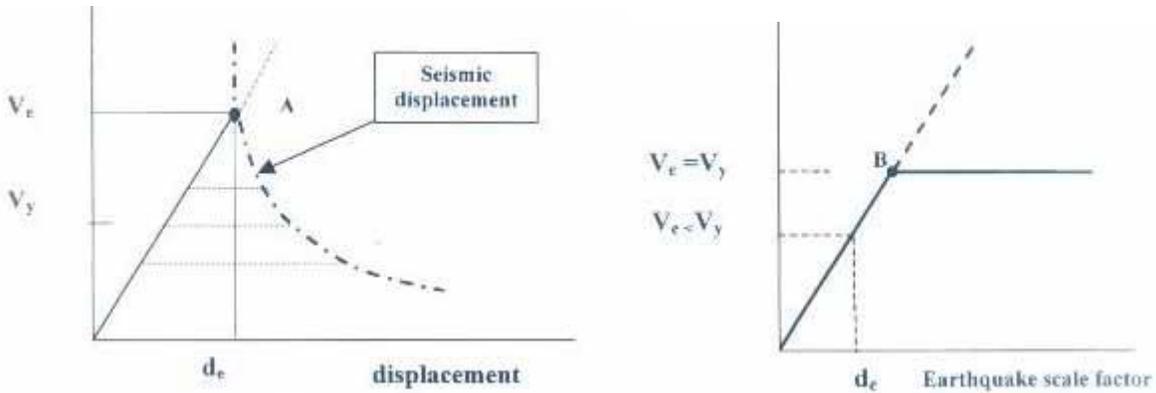
The effect of hysteresis behaviour on seismic response has been presented in Figs. 8-11. As shown in Fig. 7, X represents an ideal elastic behaviour, and all the others correspond to inelastic behaviour of axial members. Figs. 8 and 9 indicate that both horizontal and vertical displacements of the elastic model X are in general less than the inelastic models. The seismic responses of W and Z are not shown in these figures as their displacement response became enormously large, and the system was regarded as unstable. This was due to the marked reduction of postbuckling strength (up to 80%) in these models. Similar trend can be observed as for the other models. For example, P and Q underwent larger displacements than the others because of their relatively larger strength reduction (50% in P and Q as compared with 20% in U, and 0% in V and Y). On the other hand, V and Y with no strength reduction underwent the least deformation among all inelastic models. Therefore, it can be concluded that postbuckling strength has a predominant effect on seismic response, and in general, displacements increase as postbuckling strength decreases.

It can be seen in Fig. 7 that the hysteresis models Q and P are similar, except that Q does not dissipate energy during excursions in postbuckling region. On the other hand, P dissipates energy during excursions in postbuckling region. A comparison of the displacement response of these models in Figs. 8 and 9 indicates that the two are relatively close.

Seismic Forces

The seismic forces in elastic region are dependant on the following factors: i. intensity and frequency content of earthquake, and ii. Dynamic characteristics of structure such as frequencies, mode shapes, and damping. In inelastic region, some other factors also become influential such as the hysteresis behaviour of members and connections, the amount and distribution pattern of strength within a structure, response modification factor, and ductility. Thus, in inelastic region, the shortage of strength with respect to elastic demand is compensated by the ability to sustain inelastic deformation, usually referred to as ductility. Fig. 12 (a) illustrates this trend schematically. So far as the strength exceeds the elastic demand V_e , the seismic force equals V_e , and the displacement equals a constant value of d_e . In this region, the seismic force does not depend on the strength and ductility of structure. However, as the strength falls below V_e , the structure undergoes inelastic deformation, and the seismic force apparently becomes equal to strength. This is also depicted in Fig. 12 (b). For V_e 's less than V_y , the seismic force is dependent on the elastic characteristics of structure such as frequency, mode shape, and damping, as well as the intensity and frequency content of earthquake. As V_e reaches V_y , the seismic force remains constantly equal to V_y , although V_e continues to increase. Therefore, a significant conclusion can be drawn for single degree of freedom systems: In inelastic region, the seismic forces do not neither depend on the type and magnitude of earthquake, nor on the elastic characteristics of structure such as frequency and damping; they are merely dependent on the strength of structure. However, for multiple-degree-of freedom systems, the seismic forces depend on both elastic and inelastic characteristics of structure [8], and therefore a more complicated situation prevails.

For most ordinary buildings, the inelastic seismic forces vary in a range of 15% to 30% of the weight of structure, depending on the seismic design loads suggested by code, the type of structure, and overstrength. On contrary, the space structures show a fundamentally different behaviour as they can develop much larger forces in inelastic region. The horizontal base shear forces are drawn in Figs. 10 and 11. Fig. 10 (b) indicates that as the scale factor increases, the seismic base shear in M4 also increases until it reaches an ultimate value. This behaviour resembles the behaviour of ordinary structures as shown in Fig. 12 (b). The base shear force for ideal elasto-plastic system V was obtained as 3.5 ton. Therefore, the seismic force in M4 is equal to 44% of the weight. This is almost 1.5 to 3 times the seismic force for ordinary buildings. It should be noticed that we can reduce the base shear by changing the flexibility and strength of substructures (columns in M4). This, however, leads to an increase in displacements.



12 (a)

12 (b)

Figure. 12. Effect of inelastic behaviour on seismic response

Fig. 10 (a) indicates that unlike the ordinary buildings, in space structures with rigid supports the base shear continues to increase up to large values close to elastic demand. For example, the base shear in elasto-plastic model V rises up to 13 ton that is equal to 163% of the weight of structure. This is about 8 times the average seismic forces in ordinary buildings. Thus, the following conclusions can be drawn.

1. The inelastic seismic forces that are induced in space structures during severe earthquakes can be much more than ordinary buildings. These forces can be even greater than the weight of structure.
2. These seismic forces can be reduced by decreasing the strength of substructure, although such measure leads to an increase in displacements.
3. The specifications and recommendations at present seismic codes are mainly based upon the seismic behaviour of ordinary buildings. Since space structures do behave differently in earthquakes, we need to develop specific set of rules for their seismic design. However, it is thought that we will generally end up with a lower response modification factor R , and a higher seismic base shear for space structures as compared with ordinary buildings.
4. It should be noticed that such large seismic forces in space structures can lead to the failure of supports, as well as the failure of members in the vicinity of supports. This phenomenon has been repeatedly reported by investigators. The reason for such undesirable failure is that the supports and joints are designed for nominal seismic forces which are far below the real seismic forces in space structures. Until appropriate specifications are developed for seismic design of space structures, it is recommended that when using present codes, the response modification factor R should be assumed as equal to 1 for determining the seismic forces in supports and joints.

CONCLUSIONS

On the basis of linear and nonlinear analyses of space structure models subjected to earthquake ground motion, the following conclusions can be drawn.

1. Space structures exhibit an outstanding performance in severe earthquakes. Light weight, appropriate geometry, redundancy, and large reserve strength are the key elements in such superior behaviour.
2. As opposed to ordinary buildings, in space structures of non-plane geometry (like domes and barrel vaults) higher modes and vertical modes contribute in dynamic response effectively.
3. The barrel vault models underwent a marked vertical displacement as they were subjected to horizontal excitation, whereas in ordinary buildings, there occurs no significant vertical displacement under horizontal excitation.

4. Post-buckling behaviour proved to have a predominant effect on seismic response. In general, displacement response increases with an increase in strength reduction after buckling.
5. The real seismic forces that are induced during a severe earthquake in inelastic region can be enormously larger than those in ordinary buildings, and they may even exceed the weight of structure. These large forces can lead to the failure of supports and joints that, according to many existing codes, are designed for nominal seismic forces. Suggestion has been made to avoid this type of failure. It was also shown that these forces can be reduced by introduction of a rather flexible substructure with relatively lower strength. Provision of a safe failure mechanism in substructure can protect the main structure from developing undesirable brittle modes of failure.

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