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Frequency and the Ultimate Lateral Load Capacity Calculation for Space Structures

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ABSTRACT

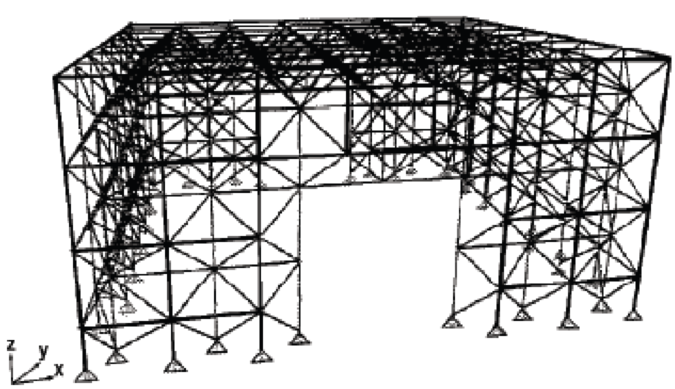
In order to implement space structures for residential, official and commercial buildings, special studies are required for these structures. Identifying dynamic variables and ultimate lateral load capacity for these types of structures are the steps of determining the requirements of space structures analysis and design which still paid no attention to their specific criteria in the codes. In this paper, a method for calculate the frequency and or time alternation of space structures is presented and also relations for calculation the ultimate lateral load capacity of the structures is presented. Results obtained from offered methods compared with the results of characteristic values non- linear and analyzes in both statical and dynamical states of above mentioned structures and it is shown that, results of provided theoretical methods were in a good agreement with those of analysis.

Key words: space structure, frequency, lateral capacity, statical, dynamical, nonlinear

1. INTRODUCTION

3D performance reticulated structure as a structural system composed of linear components' coupling is the most important trait of these structures. Although most of the studies were based on orthogonal loading on flat or with curvature surfaces of these structures, but this paper focus on effects of seismic forces which can be from three directions. In this study, space networks compose roofs and walls structures (Fig.1). Features of 3D behavior of structure and participation of set of components of structure provide intrinsic resistance and considerable hardness for this type of structures that increase the loading capability against asymmetricor out of plainloads and resistance against progressive rupture.

Fig.1 space structure of studied store



The non- linear behavior of materials is not separable from non- linear geometric behavior of structure with regards to interaction between them. In present study, in order to present behavioral model of single component space structures and to present components' axial shift-load diagram, the following was done. Under tension effect, the behavior of component considered perfect elasto-plastic and, capacity increasing, after yield threshold, i.e. strain hardness ignored. The effect of possible eccentric at behavioral diagram of component was considered, but the effects connections flexibility, joints and connections softness, and fastening junction screws were ignored.

Cylindrical profiles are common in space structures and they were used in this study. For determining post-buckling characteristics of compression tubes components, we benefitted time - shift behavioral diagram offered by FEMA [6] and also experimental behavioral diagram, reported by gholampour [3] that, led to proposing trilinear behavioral diagram in this paper (Fig.2)

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2.Optimizing components' position

In order to do this, three 7*7 m (S 7*7 structure),9*9 m (S 9*9) structure, and 11*11 m (S 11*11structure) space structures with height of 3.5 m and with different densities and opening dimensions of 2,4 and 6 m, respectively, were examined (Fig.1). First, each structure analyzed under 550 kg/m^2 dead load and 200 kg/m^2 live load and then, seismic load with regards to the mass emerged from vertical load applied on the structure and finally, according to UBC97 code the optimized design have been done. In addition, to optimizing components' position, nonlinear static analysis under lateral loads and slimming (slenderizing)restriction ratio in different part of space structure were applied. The results are shown in table (1) and figure 2. As can be seen in table (1),the analysis for each structure had been done from five views of: restriction at components slimming ratio at without restriction states, with restriction only around corners, with restriction at wall's components, all wall's components, and also total structure's components. It can be seen that, roof's horizontal components' weight gaining percent from slimming ratio restriction has not considerable effect on lateral loading capacity. Investigating the sequence of forming plastic joint in truss components of space structure shows that, components of web reach buckling status, at first. The ratio of weight increase to capacity increase shows that, the best results usually obtain when, web components restricted from slimming point.

Table 1: evaluation of the lateral loading increase with regard to components slimming ratio restrictions in different states

	EX					EY				
	Without restriction	Corner components	WEB components	Wall components	All components	Without restriction	Corner components	WEB components	Wall components	All components
S 7*7										
Ratio of weight increase (%)	0	23	62	39	76	0				
Ratio of lateral force capacity increase	0	61	204	132	204	0	57	197	138	197
Ratio of weight increase to capacity (%)	0	38	30	30	38		40	31	28	39
S 9*9										
Ratio of weight increase (%)	0	16	49	31	67					
Ratio of capacity increase	0	114	192	150	185	0	35	212	140	212
Ratio of weight/capacity increase (%)		14	26	21	36	0	46	23	22	32
S 11*11										
Weight increase (%)	0	10	37	24	53					
Capacity increase (%)	0	58	122	69	122	0	32	135	85	135
Ratio of weight to capacity increase (%)	0	17	30	35	43	0	31	27	28	39

Fig. 2: Load-shifting diagram of components for slimming ratios of 50, 70, and 90-suggested diagram based on experiment, FEMA, and theoretical diagram

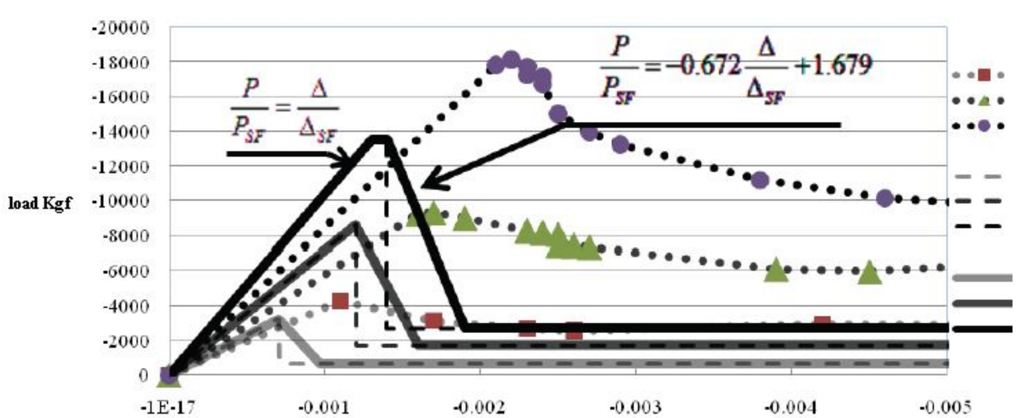
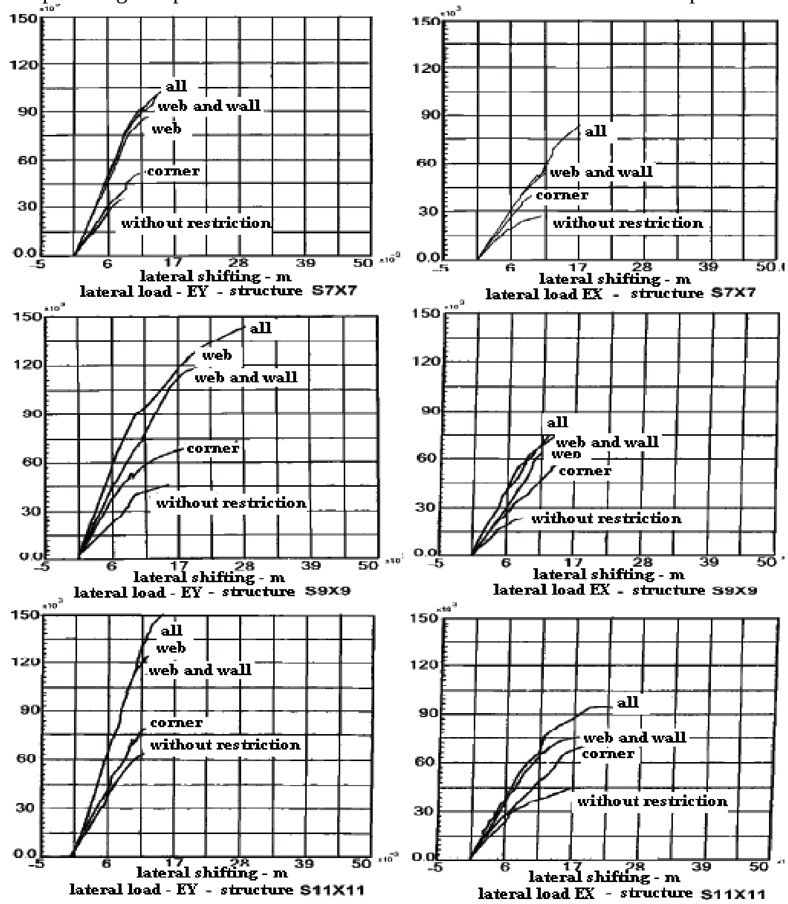


Fig .3: Optimizing components location for slimness ratio restriction in studied space structures

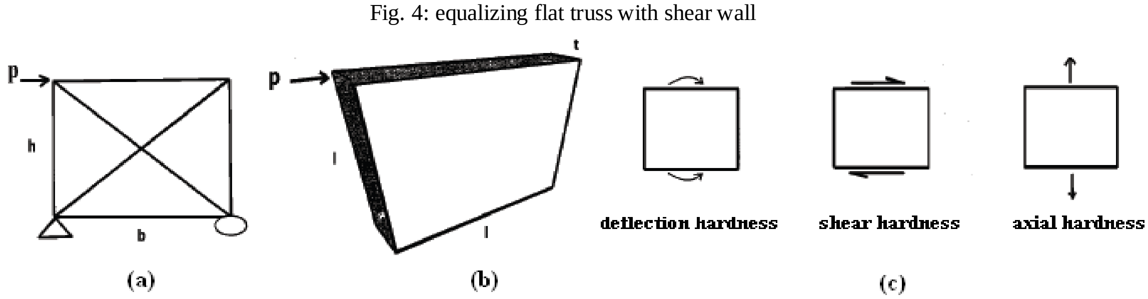


3. Calculating natural frequency, shifting, and ultimate lateral load capacity

Identifying natural frequency of structure vibration is one of the most important factors of dynamical analysis. Here, we suggest a method for calculation natural period for space structure that is, based on axial hardness of space structure components. Also, we suggest a relational to estimate ultimate lateral load capacity and its accordant ultimate lateral shifting, and then results compared with the results of nonlinear statically and dynamical analyses.

With the use of simplified2D flat truss model (Fig.4a) and equalizing it with shear wall (Fig.4b), deflection, shear, and axial hardness of two mentioned structures will be equalized as following (Fig.4c).

Fig. 4: equalizing flat truss with shear wall



Assuming stress behavior we have:

A-Deflection hardness: to calculate deflection hardness of truss using axial hardness of structure columns(Fig.3a) around central axis of structures we have: Assuming equal elastic coefficient for truss and wall materials

$$EI = 2EA(\frac{b}{2})^2 = \frac{Etb^3}{12} \quad [1]$$

Where, E is elasticity coefficient, I is wall inertia, A is cross sectional area off truss components, b and h are wall dimensions, and t is equivalent wall thickness.

B - Shear hardness: to calculate shear hardness of truss using horizontal diagonal components of structure (Fig .3a) we have:

$$\frac{GA}{L} = \frac{2EA_D Sin^2\theta}{L} = \frac{Gbt}{h} = \frac{Ebt}{2(1+\upsilon)h} \qquad [2]$$

L is the length of diagonal truss component, G is shear coefficient of wall material, and ν is the Poisson proportion.
C- Axial hardness: axial hardness of truss can be calculated from vertical axial components and diagonal member component.

$$\frac{EA}{L} = \frac{2EA_C}{h} + \frac{2EA_D Sin^2\theta}{L} = \frac{Ebt}{h} \qquad [3]$$

Where, A_D and A_C are diagonal and vertical (column) components' cross section, respectively.
According to results the optimizing performance of diagonal components in capacity of lateral loading, and generalizing results of a flat truss to pyramidal square network and equalizing shear hard nessof truss with shear hardness of wall we have:

$$\frac{h}{Gtb} = \frac{l^* l}{nEAC_x^2} = \frac{2(1+\upsilon)h}{Etb} \quad , \quad t = \frac{2(1+\upsilon)h^* nAC_x^2}{l^* b} \qquad [4]$$

Where, C_X is guiding cosine of longitudinal axis of web component. The wall in the studied space structure made from continuous pyramidal networks. It is worth mentioning that, the number of pyramids in both horizontal and vertical densities must be considered (number of pyramids in vertical and horizontal direction).The lateral shifting by shear, Δ_1, can be calculated by following equation.

$$\Delta_1 = \frac{2ml}{2nEAC_x^2} \qquad [5]$$

Assuming linear distribution on wall section, deflection hardness can be presented as follows.

$$K = 2EA\left(\frac{2}{6}b\right)^2 . \frac{m}{2} = 0.12mb^2 EA \quad , \quad \Delta_2 = \frac{l^{3*}}{3K} \quad , \quad \Delta = \Delta_1 + \Delta_2 \qquad [6]$$

Where, n is the number of web components in a horizontal row and mis the number of columns in wall width made by vertical members of sample pyramid. The following equation can give frequency or natural period of structure.

$$w^2 = \frac{k}{m} \quad , \quad T = \frac{2\pi}{w} \qquad [7]$$

Moreover, there are studies investigating two and three story space structures that are, in fact, extended model of above mentioned one storeyin terms of height.

4. Calculating ultimate load capacity for studied one, two, and three storey structures

Given the results of optimizing stage and given that, in the increasing load method, half of web components will reach buckling, to calculate total capacity of buckled members:

The coefficient of 0.5 was selected for compression members and, the total capacity they obtained from adding buckled members with non-buckled members. In this state n is the number of all members placed in a row in order to increase upon it.

If all web components and other components would select with unequal lateral capacity, the total load capacity will decreased because buckling phenomenon, in addition to diagonal components, occurs in wall components, particularly, in column's corner components and causes the instability of structure.

Table (2) shows the results of increasing load analysis in without opening direction. In all studied structures, tube profiles of structure components, except web components were selected of P1.5 type(tube with 1.5 inch diameter).The difference between proposed load capacity and the capacity resulted from increasing loading method at Table (2), is due to the buckling of vertical components before buckling of web components. If all components of two storey structure web will be of p.5 type, the maximum load capacity will be equal to 43 tons and in fact, truss shear will not be the sum of ultimate shear capacity of lower storey and, total buckling must be controlled separately for each storey. Figure (4), shows the position of buckling components in three storey structures at final boundary state.

Table.2:load capacity of components and load capacity of lateral load capacity of structure

Number of store	Tube profile of web component	Component's buckling capacity(kg)	Lateral load capacity by proposed method(ton)	Lateral load capacity by increasing load(ton)	Error rate of increasing load results	Proposed period (s)	Numerical method period(S)	Error rate
One	p.5	2011	44	43	-0.02	0.0175	0.174	0.00
Two	p.5	2011	44	43	-0.02	0.272	0.278	0.02
Two	P1	6137	145	128	0.11	0.187	0.185	0.01
Two	P1.5	11514	273	190	0.30	0.157	0.154	0.02
Three	p.5	2011	43	43	0.00	0.172	0.197	0.07
Three	P1	6137	145	128	0.11	0.257	0.273	0.06
Three	P1.5	11514	273	190	0.30	0.213	0.173	0.19

In order to analyze historical data, the Altadena modified acceleration mappings(station 24 402, 1987) with maximum horizontal acceleration of 439 cm/s/s and Newhall (station 24279, 1987) with maximum horizontal acceleration of 578 cm/s/s have been used. Time interval between recorded data was 0.02 seconds. Innon- linear integral, the New mark method with α and β parameters equal to0.5 and 0.25, respectively have been used.

In table 3, t-max is the occurrence time of maximum from beginning of recording acceleration mapping, Td is the effective time of acceleration mapping, and Tpis the dominant period of acceleration mapping. The comparison of non- linear statically and dynamical analyses show that, the ultimate capacities resulted from two methods are different. It is due to the nature of dynamic behavior to because, the acceleration mapping actions are shuttling and buckling components during the force direction change, could not tolerate the load, and the positions of bearing components are variable, depended to load intensity effect which itself affects from dynamical variables. While in static non- linear loading, loading direction and compression components direction are always constant. Therefore, in dynamic historical data analysis it is required to present proposed ultimate lateral loading capacity as follows.

Figures (5) and (6) show the results of dynamic non- linear analysis in opening direction for a one storey structure. In both cases the acceleration mappings calculated in single component and three component modes, the value of base shear in opening direction shows about 10 % difference with proposed equation. The difference of ultimate base shear value resulted from three acceleration mappings analyses is smaller than 3% for each single component and three component state. Figure (7) shows the base shear value with regards to post- buckling slope in component's behavior based on the results of experiment [1] indicating that, in the studied structure this value in in a good agreement with value of proposed equation.

5. Conclusion

Studies show that, the results obtained from equations offered for calculating frequency and ultimate lateral load capacityfor space structure, have a desirable precision at comparison with the results of numerical analytical methods whether on increasing load basis or dynamic non- linear basis. This result is achieved in similar works (see [3, 8, 9 and 10]). Also Comparison of base shear of without post-buckling slope effect one store structure at single component Altadena, Newhall and Sylmar acceleration mapping at opening direction and Comparison of base shear of without post-buckling slope effect one storey structure at three component Altadena, Newhall and Sylmar acceleration mapping at opening direction show that this findings are related to previous works.

Fig. 5: Comparison of base shear of without post-buckling slope effect one store structure at single component Altadena, Newhall and Sylmar acceleration mapping at opening direction

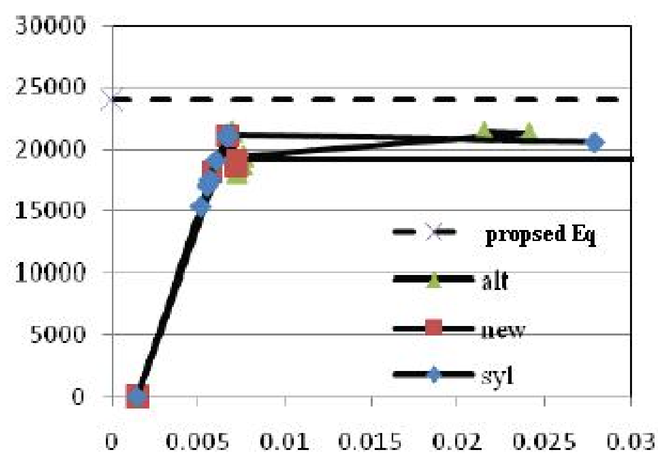


Fig. 6: Comparison of base shear of without post-buckling slope effect one storey structure at three component Altadena, Newhall and Sylmar acceleration mapping at opening direction

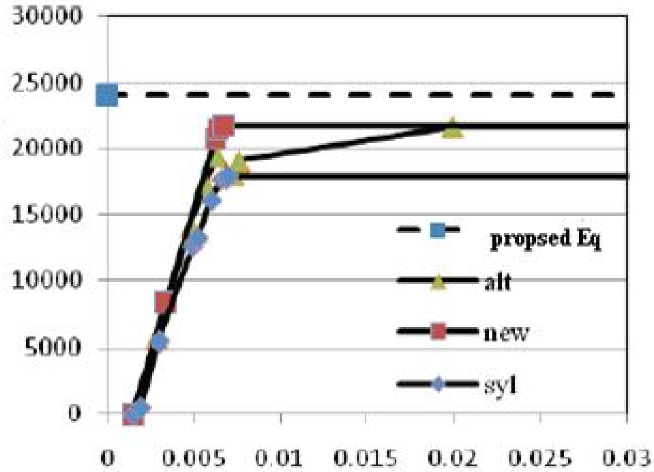
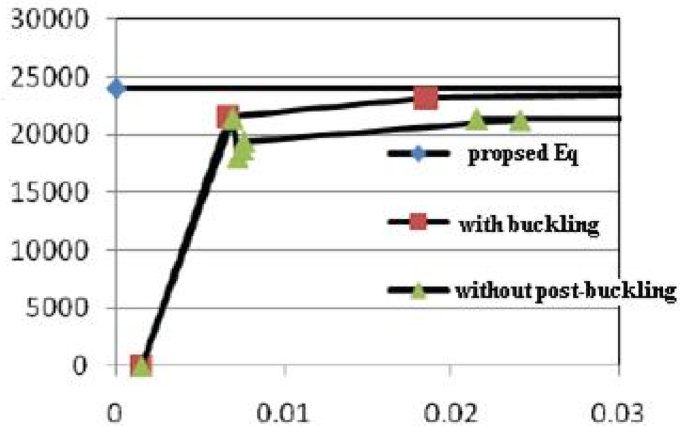


Fig.7: Base shear value with regards to post- buckling slope in component's behavior based on the results of experiment



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