13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 215

SEISMIC BEHAVIOR OF 4-LEGGED SELF-SUPPORTING TELECOMMUNICATION TOWERS

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SUMMARY

The telecommunication masts are considered today as one of the basic infrastructures in the human societies. Due to their vital role, the preservation of these structures during natural disasters, such as a severe earthquake, is of utmost priority and hence their seismic performance should be properly evaluated. The researchers in their studies have considered the effects of wind and earthquake-induced loads mostly on the trussed steel masts of triangular cross sections. The main objective of this paper is to investigate the overall seismic response of 4-legged self-supporting telecommunication towers. For this purpose, ten of the existing 4-legged self-supporting telecommunication towers in Iran are studied under the effects of the design spectrum from the Iranian seismic code of practice and the normalized spectra of Manjil, Tabas, and Naghan earthquakes. As part of some of the results, it was observed that the first three flexural modes are sufficient for the dynamic analysis of such towers, even though in the case of taller towers, considering the first five modes would enhance the analysis precision.

INTRODUCTION

At the start of telecommunication tower design, due to lightness and height of such structures, much of the efforts of researchers were focused on the wind loading. Nevertheless, in recent years, more attention is being paid to earthquake loading due to adding the number of antennas mounted on the telecommunication towers and also due to the high seismicity level of the regions where the towers are installed. In the latest editions of world’s most accredited design codes, the topic of earthquake loading on such structures has been included.

There are two types of telecommunication towers mainly known to engineers as guyed towers and self-supporting towers. The self-supporting towers are categorized into two groups of 4-legged and 3-legged towers. Most researches to date have been performed on 3-legged self-supporting towers and very limited attention has been paid to the dynamic behavior of 4-legged self-supporting telecommunication towers. In

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this paper the effects of seismic forces acting on the 4-legged self-supporting towers have been studied based on the detailed dynamic analyses of ten existing towers. Figure 1 illustrates a typical 4-legged self-supporting telecommunication tower considered in this study.

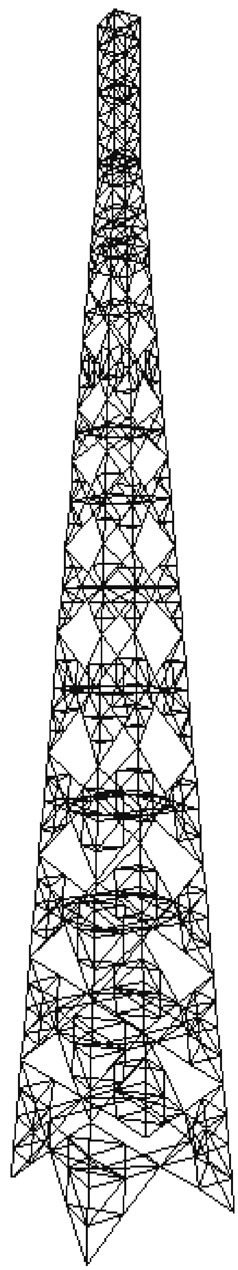

Fig. 1 Typical view of a 4-legged self-supporting telecommunication tower

BACKGROUND

Konno and Kimura [1] presented one of the first studies on the effects of earthquake loads on the lattice telecommunication towers. The objective of their studies was to obtain the mode shapes, the natural frequencies, and the damping properties of such structures. It was observed that in some of the members, the forces due to the earthquake were greater than those of the wind.

Mikus [2] investigated the seismic response of six 3-legged self-supporting telecommunication towers with heights ranging from 20 to 90 meters. The selected towers were numerically simulated as bare towers. Three accelerograms were considered as the real earthquake forces in the analysis. It was concluded that the lowest four modes of vibration would ascertain the sufficient precision. In addition, also, it was pointed out that the vertical component of earthquake-induced forces had no effects on the results.

Galvez [3,4] and Khedr [5,6,7] performed additional studies to introduce simplifying methods for the seismic analysis of telecommunication towers. Galvez investigated three different numerical models of 3-legged lattice steel towers with heights ranging from 90 to 121 meters that were subjected to 45 earthquake records. It was concluded that contribution of second and third transversal modes of vibration on the maximum acceleration at the top of the towers, depending on the tower type, varies from 15% to 50%. One of the main disadvantages of the Galvez method was the bilinear shape of the acceleration profile, which did not thoroughly include the towers with different geometries. Later, Khedr introduced a modified method for the horizontal acceleration profile, so that for every specified tower a separate acceleration profile be obtained. Moreover, in the latest edition of TIA/EIA code [8], provisions for seismic design of towers have been included.

Many researches have been performed for obtaining the fundamental frequencies of self-supporting towers (e.g. Sackmann [9]). In most of these studies, especially in the field of seismic effects, the researches were focused on the 3-legged towers and very few studies have been conducted on the 4-legged masts. For this reason, in this paper, the dynamic sensitivity of 4-legged self-supporting telecommunication towers subjected to earthquake loadings are investigated.

DESCRIPTION OF TOWERS

Detailed three-dimensional full-scale numerical simulations using the finite element method have been carried out in this study. The tower models are very detailed to include geometric nonlinearities and to allow for potential dynamic response of the towers. The height limitation of 150-m is a common criterion to classify towers with respect to their heights. With this regard, the available data for the self-supporting towers shorter than 150-m were selected for the numerical simulations.

The towers member cross-sections are made-up of single equal-legged angles. All the angle sections (number 10 and higher) are made from the ST32 steel with a tensile yield strength of 3600 kg/cm2, and the angle sections (smaller than number 10) are made from the ST37 steel with a tensile yield strength of 2400 kg/cm2. The modulus of elasticity and the unit weight of the steel materials used were 2.1×10^6 kg/cm2 and 7850 kg/m3, respectively. The connection type mainly used was bolts and nuts, and in case of excessive usage of bolts, steel plates have been used as interface members. The total height and the corresponding total weight of each mast are listed in Table 1 and Fig. 2.

The detailed weights are composed of weight of structure in conjunction with weight of ancillary components such as ladders, platforms, feeders, lights for aircraft warning, and bolts/nuts as connections. Furthermore, other geometric characteristics such as the length of tapering portion of tower, the ratio of the length of prismatic portion to the total length of tower, and the span length between the legs at the extreme ends are also listed in Table 1. The vertical load acting on these towers is composed of tower own weight plus weight of antennas and other related accessories attached to it.

Table 1 Characteristics of the selected self-supporting towers (see Fig. 2 for notations)

W_{total} (kg)	D_L (m)	L_{total} (m)	D_O (m)	L_{taper} (m)	$(L_{total}-L_{taper})/L_{total}$ %
4072	2	18	3.4	10	44.4
5651	2	22	3.9	14	36.4
6791	2	25	4.3	17	32.0
8670	2	30	5	22	26.7
10721	2	35	5.7	27	22.9
13366	2	42	6.7	34	19.0
16700	2	48	7.6	40	16.7
20271	2	54	8.4	46	14.8
24627	2	60	9.3	52	13.3
29513	2	67	10.3	59	11.9

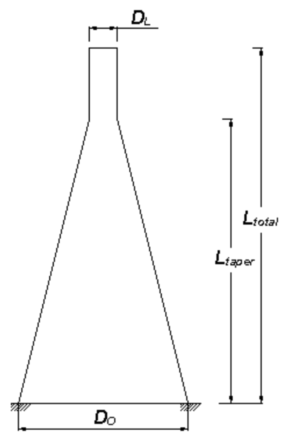

Fig. 2 Notations of Table 1

NUMERICAL MODELING CONSIDERATIONS

The computer program SAP2000 [10] is chosen for the analysis purposes. To account for the mass of ancillary components in the analysis, by modifying the material densities, these masses will be proportionally distributed along the tower height. It is worthy to note that the weight of ancillary components is considerably high and its exclusion from the analysis will have major effects in the results obtained. In modeling the towers under study, all the members including the legs, the horizontal and diagonal members, and the redundant members have been considered. Noting to the existing connections, in relation to their location and the number of bolts used, the connections are classified into two types of pin-ended and fixed-ended connections, and hence, the members are divided into two types of truss and beam elements. Foundations are assumed perfectly rigid. A lumped mass formulation is used for the mast members.

EARTHQUAKE LOADING

To analyze the towers, at first, a modal analysis is performed in order to obtain the natural modes of vibration. Next, all the towers undergo a spectral analysis under the standard design spectrum of the Iranian seismic code of practice [11], and subsequently under the normalized spectra of Manjil, Tabas, and Naghan earthquakes. The relations presented in the said code and its related parameters are as follows:

$$Sa\,(T) = ABI/R \tag{1}$$

$$B = 2.5\,(T_0/T)^{2/3} \leq 2.5 \tag{2}$$

Where

Sa = spectral acceleration
A = 0.35 g (design base acceleration for regions with high seismicity level)
T_0 = 0.5 natural period of ground type (in this case for ground type 2)
T = fundamental period of structure in seconds
I = 1.2 importance factor (for structures with high importance)
R = 1.0 reduction factor (considering the elastic behavior of the structure) [12]

To analyze the towers, first, a modal analysis is performed in order to obtain the natural modes of vibration, then, all the towers undergo a spectral analysis under the standard design spectrum of the Iranian seismic code of practice, and subsequently under the normalized spectra of Manjil, Tabas, and Naghan earthquakes. The earthquake records were scaled by A=0.35 g, the design base acceleration of the Iranian seismic code of practice for the highest seismicity level. The standard design spectrum from the Iranian seismic code of practice and the accelerograms of Manjil and Tabas earthquakes are illustrated in Fig. 3.

RESULTS OF DYNAMIC ANALYSIS

Frequency analysis

For a simple and accurate recognition of vibration modes of a mast in a modal analysis, the member masses must be lumped at the joints intersecting the mast leg members with the main horizontal and diagonal members, in order to avoid the creation of local modes. Of course, before performing the analysis, it must be noted that the mass of redundant members should also be taken into account when assigning the lumped masses.

The frequency analysis has only been performed on the bare masts, ignoring the mass of antennas attached to them. Among the towers under study, the tallest mast with 67-m height was the most flexible having a fundamental period of 0.58 seconds. The shortest mast with 18-m height exhibited the highest stiffness with the fundamental period of 0.168 seconds. After examining the figures and the tables corresponding to the towers under study, it can be concluded that approximately 90% of the total effective mass participation is attributed to the first three flexural modes, which is indicative of its sufficiency for the performance of a dynamic analysis. Of course, considering the lowest five flexural modes, especially in the case of taller towers, will greatly enhance the analysis accuracy. Notably, the results obtained here are in line with those obtained by Galvez [3,4], Khedr [5,6,7], and Sackmann [8] on the frequency analysis of self-supporting telecommunication towers.

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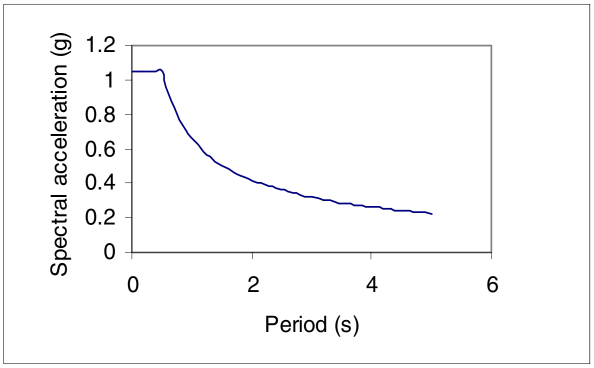
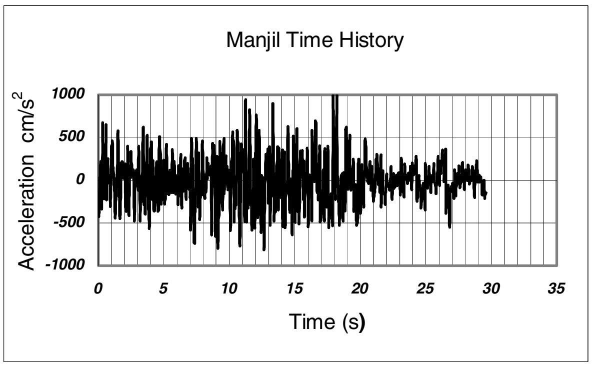
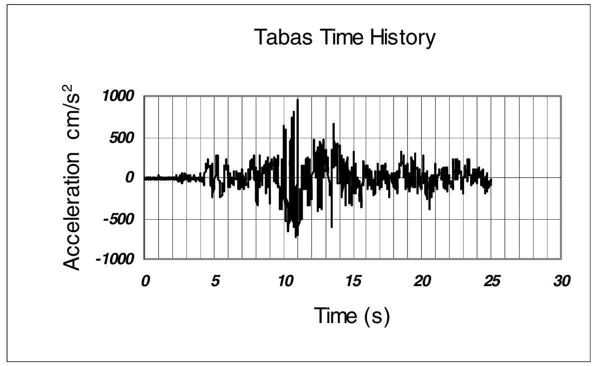


(a)





(b)

Fig. 3 Earthquake forces subjected to towers: a) Spectrum of the Iranian seismic code of practice b) Accelerograms of Manjil and Tabas earthquakes

In addition, to determine the periods of the lowest two torsional modes, the following relations are described in terms of the periods of second and third flexural modes, respectively, as shown in Fig. 4.

$$T_{t1} = 0.5781T_{f2} + 0.0302 \tag{3}$$

$$T_{t2} = 0.6097T_{f3} + 0.0163 \tag{4}$$

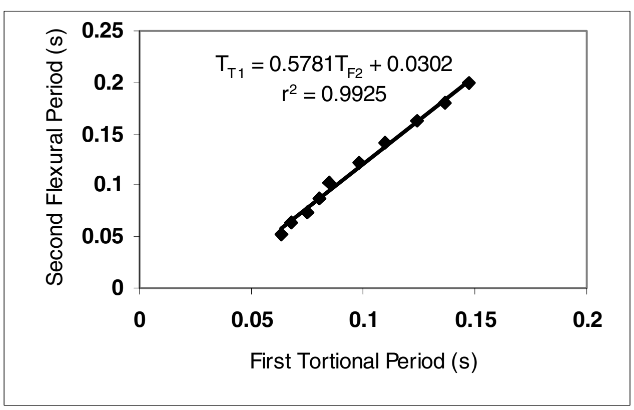


Fig. 4 Period of the first torsional modes versus period of the second flexural modes

Where

T_{t1} = the period of the 1st torsional mode in s
T_{t2} = the period of the 2nd torsional mode in s
T_{f2} = the period of the 2nd flexural mode in s
T_{f3} = the period of the 3rd flexural mode in s

Spectral dynamic analysis
In this section, all the towers under study went through the spectral analysis using the design spectrum from the Iranian seismic code of practice [11]. Then, the results were compared to that obtained from the spectral analysis under the effect of spectra extracted from the scaled accelerograms of Manjil, Tabas, and Naghan earthquakes.

The results show that in most of the towers, the axial forces in the leg members, when under effect of the design spectrum from the Iranian seismic code, have the highest values. While in the case of short masts, eg. 25-m and shorter, the normalized spectra of Manjil or Tabas are most predominant. The reason to this is the resistance of the leg members against the over-turning that is caused by the lateral forces with their magnitudes depending on the frequency content of the earth movement.

Base shear
The maximum base shear values for all the masts subjected to the four defined spectra are shown in Fig. 5.

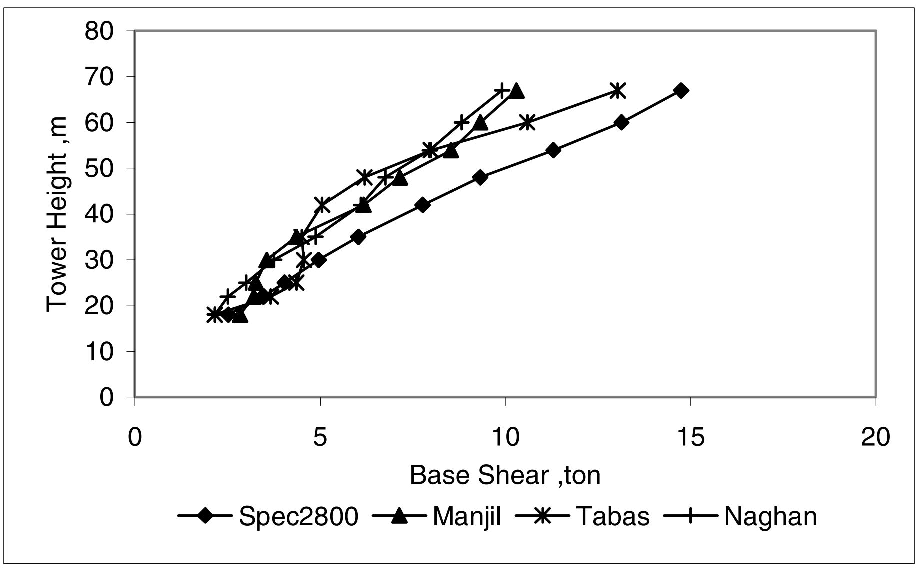


Fig. 5 Maximum base shear versus tower heights
* spec2800: The spectrum from the Iranian seismic code of practice

As it can be observed, the base shear values in most of the towers, under the effect of design spectrum from the Iranian seismic code, have the highest values, whereas in short masts the normalized spectra of Manjil or Tabas acclerograms are most predominant.

In order to present some relations for determining the base shear, the ratio of the base shear to the product of mass and maximum acceleration of earth movement (when subjected to the normalized spectra of Manjil, Tabas, and Naghan earthquakes), has been depicted versus the period of the fundamental flexural mode of vibration, as illustrated in Fig. 6. From this figure, a linear relation for determining the base shear is introduced as follows:

$$V_h = M\,A_h\,(1.7598 - 1.3177\,T_f) \quad (5)$$

Where V_h is horizontal base shear, M is tower mass, A_h is maximum horizontal ground acceleration, and T_f is fundamental flexural period of the mast. The above relation is in accord with the relations that were proposed by Khedr and McClure (1999) [7], which were obtained from the application of 45 earthquake records to their 3-legged prototypes.

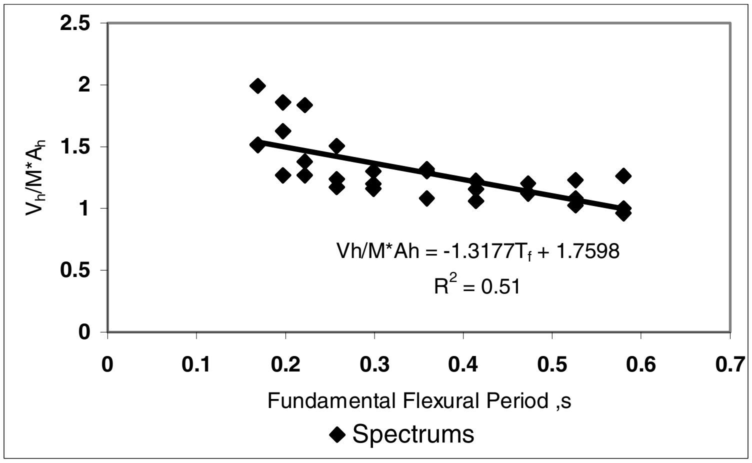


Fig. 6 The ratio Vh/M.Ah versus the period of fundamental flexural mode for all the towers when subjected to normalized spectra of Manjil, Tabas, and Naghan earthquakes

Maximum lateral displacements

As shown in Fig. 7, a relation for determining the maximum lateral displacement of towers with respect to their overall height, when subjected to the normalized spectra of Manjil, Tabas, and Naghan earthquakes, is presented as follows:

$$d_{max} = 0.0016H - 0.0202 \quad (6)$$

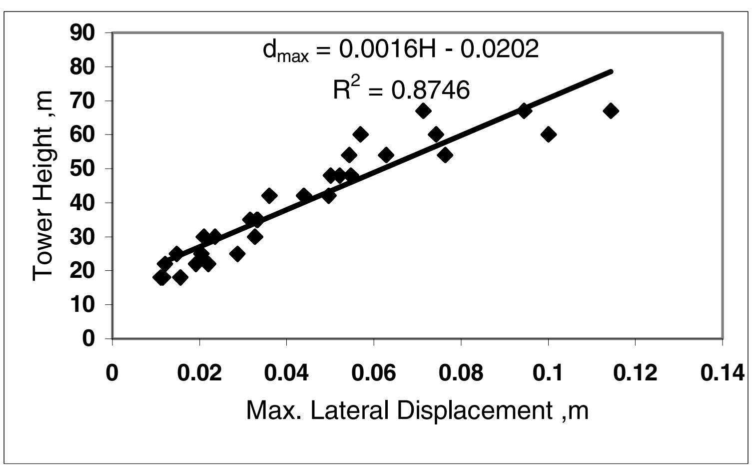


Fig. 7 Maximum lateral displacements versus tower heights when subjected to normalized spectra of Manjil, Tabas, and Naghan earthquakes

Distribution of lateral earthquake forces with tower elevation

Here, the shear force distribution along the tower height for the towers under study is investigated. The shear force is resulted from the spectral analysis using the design spectrum from the Iranian seismic code of practice. The results indicate that the distribution of the lateral forces along the tower height has approximately a triangular shape. Figure 8 shows the obtained results for the 67-m tower. The TIA/EIA code [8] has introduced the following relations for the distribution of the lateral seismic forces along the tower elevation:

$$F_x = \frac{V w_x h_x}{\sum w_i h_i} \qquad (7)$$

Where

w_i, w_x = the portion of weight at the i^{th} or x^{th} level
h_i, h_x = the relative height with respect to the ground level at the i^{th} or x^{th} level
V = total design shear force

To check the accuracy of the above formula, an equivalent simple cantilever model of the towers was created. Then each tower was assumed to be divided into several portions. The weight of each portion was evaluated and then assigned to the center of each tower portion as projected on the cantilever. Considering the elevation of each portion as the distance from the ground level to the center of each portion, the lateral earthquake force at each member was evaluated with respect to the base shear force resulted under the design spectrum of the Iranian seismic code of practice.

By using the above relation, the shear force distribution along the height of each tower is evaluated. The results, as shown in Fig. 8, indicate that the shear force distribution along the tower height is in good accord with that obtained from the relation introduced by the TIA/EIA code. This relation can be used to properly approximate the distribution of lateral seismic forces along the height of self-supporting telecommunication towers.

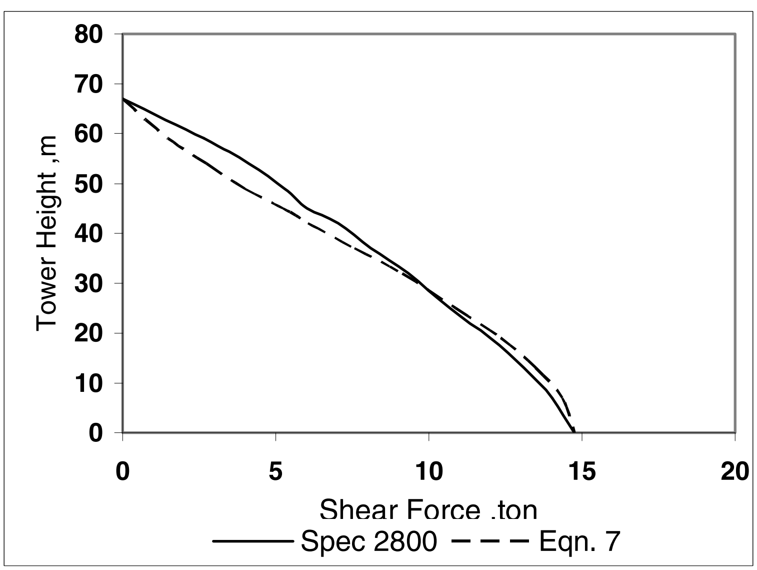


Fig. 8 Distribution of the base shear force along the tower height for the 67-m tower

CONCLUSIONS

The results obtained from this study can be summarized as follows:

1. By investigating the tower mode shapes, it can be concluded that the lowest three flexural modes of vibration are sufficient for the dynamic analysis of self-supporting telecommunication towers. Although, considering the lowest five modes, especially in the case of taller towers, would enhance the analysis precision.

2. It was observed that the axial forces in the towers leg members, when under the design spectrum from the Iranian seismic code of practice, attained the highest values. Although, in the case of short towers, the normalized spectra of Manjil and Tabas were predominant, but the axial forces in the horizontal and diagonal members show only a marginal difference when compared to those of Iranian seismic code. This is an indication of a greater sensitivity of the leg members to the frequency content of earth movement.

3. Investigating the shear force distribution along the tower height when subjected to the design spectrum from the Iranian seismic code of practice, the accuracy of the relation (eqn. 7) presented in the TIA/EIA code relating the distribution of lateral seismic forces is verified.

4. To determine the tower base shear due to the normalized spectra of Manjil, Tabas, and Naghan earthquakes in terms of the maximum acceleration of the ground movement, the overall tower mass, and the period of the fundamental flexural mode, the relation (5) is proposed. Furthermore, to obtain the maximum lateral displacement of the towers in terms of their overall height when subjected to the normalized spectra of Manjil, Tabas, and Naghan earthquakes, the relation (6) is presented.

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