SEISMIC RESPONSE OF 4-LEGGED SELF-SUPPORTING TELECOMMUNICATION TOWERS

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Abstract Telecommunication tower is an important component of the basic infrastructure of communication systems and thus preserving them in events of natural disasters - such as a severe earthquake - is of high priority. In past studies, researchers have mostly considered the effects of wind and earthquake-induced loads on 3-legged (triangular cross-section) self-supporting steel telecommunication towers. In this study, the seismic sensitivity of 4-legged telecommunication towers is investigated based on modal superposition analysis. For this purpose, ten of the existing 4-legged self-supporting telecommunication towers in Iran are studied under the effects of wind and earthquake loadings. To consider the wind effects on the prototypes, the provisions of the TIA/EIA code are employed and the wind is treated as a static load throughout the analysis. In addition, to consider the earthquake effects on the models, the standard design spectrum based on the Iranian seismic code of practice and the normalized spectra of Manjil, Tabas and Naghan earthquakes have been applied. Since Iran is considered to be located in a high seismic risk region, a base design acceleration of A = 0.35 g is used for normalization of the spectra. It was observed that in the case of towers with rectangular cross section, the effect of simultaneous earthquake loading in two orthogonal directions is important. At the end, a number of empirical relations are presented that can help designers to approximate the dynamic response of towers under seismic loadings.

Keywords Seismic Response, Earthquake and Wind Loadings, Self-Supporting 4-Legged Telecommunication Towers

چکیده دکل های مخابراتی بخش مهمی از زیر ساخت های سامانه ارتباطاتی هر کشور هستند. از این رو حفظ شرایط بهره برداری آنها پس از رخدادهای طبیعی، همچون زلزله بسیار حائز اهمیت است. تحقیقات انجام شده در گذشته بیشتر در رابطه با اثرات نیروهای حاصل از باد و زلزله بر دکل های سه پایه خود ایستا بوده است. در این تحقیق رفتار لرزه ای دکل های مخابراتی چهار پایه خود ایستا بررسی شده است. بدین منظور ده عدد از دکل های خود ایستا چهار پایه که در اقصی نقاط ایران بهره برداری می‌شوند، تحت اثر نیروهای استاتیکی باد و زلزله مورد مطالعه قرار گرفته اند. برای بارگذاری باد، ضوابط آئین نامه TIA/EIA مورد استفاده قرار گرفته و در سراسر تحلیل نیروهای باد بطور استاتیکی به دکل ها اعمال گردیده است. همچنین برای اعمال اثرات نیروی زلزله در تحلیل دکل ها طیف طرح آئین نامه ۲۸۰۰ ایران و طیف های نرمالیزه شده طبس، منجیل و ناغان بکار گرفته شده اند. برای نرمالیزه کردن طیف های فوق از شتاب مبنای طرح A = 0.35 g استفاده شده است. از نتایج بدست آمده دراین تحقیق می‌توان لزوم در نظر گرفتن اثرات بارگذاری جانبی بطور همزمان در دو جهت متعامد را بر شمرد. همچنین چند رابطه تجربی ارائه شده اند که مهندسین می‌توانند با بکارگیری آنها پاسخ دینامیکی دکل ها را تحت اثر نیروهای ناشی از زلزله، به طور تقریبی بدست آورند.

1. INTRODUCTION

In the contemporary era, the telecommunication industry plays a great role in human societies and thus much more attention is now being paid to telecommunication towers than it was in the past.

At times of occurrence of natural disasters, telecommunication towers have the crucial task of instant transmission of information from the affected areas to the rescue centers. In addition, performance of infrastructure such as dams, electric, gas, and fuel transmission stations, depends extensively on the information being transmitted via these telecommunication towers. Military and defense industries in addition to television, radio, and telecommunication industries are other areas of application for such towers and thus creates the necessity for further research on telecommunication towers.

There are three types of steel telecommunication towers mainly known to engineers as guyed towers, self-supporting towers, and monopoles. Guyed towers normally provide an economical and efficient solution for tall towers of 150 m and above, compared to self-supporting towers [1]. Self-supporting towers are categorized into two groups of 4-legged and 3-legged lattice towers. The monopoles are designed for use with cellular, microwave, broadcast, and other applications. Monopoles are most economical for heights under 55 m and are a viable solution for space limitation problems and rigid zoning codes. Industry separates monopoles from self-supporting towers with the latter being latticed.

Most research to date has been performed on 3-legged self-supporting towers and very limited attention has been paid to the seismic behavior of 4-legged self-supporting telecommunication towers. Since almost all of the self-supporting towers built in Iran are 4-legged, therefore, in this paper these types of towers are investigated based on the detailed dynamic analyses of ten existing towers erected in high seismic risk regions of Iran. Tower responses to seismic excitations are evaluated and then compared with those under the effect of statically applied wind forces. The design concepts of 3-legged towers are different from that of 4-legged towers and the results from this study could be useful for countries where the use of 4-legged towers is predominant.

Figure 1 illustrates a general view of a 4-legged 67-m self-supporting telecommunication tower. The transversal cross-section of such towers tapers down along the tower elevation. The member sections used are light equal-legged angles. All the members and the connection components such as bolts and nuts are galvanized. The vertical load acting on these towers is composed of the tower's own weight plus the weight of antennas and other related appurtenances attached to it. The horizontal forces exerted on the towers according to most of design codes are composed of forces induced by wind pressure acting on the projected areas of towers and antennas, in combination with ice in cold regions.

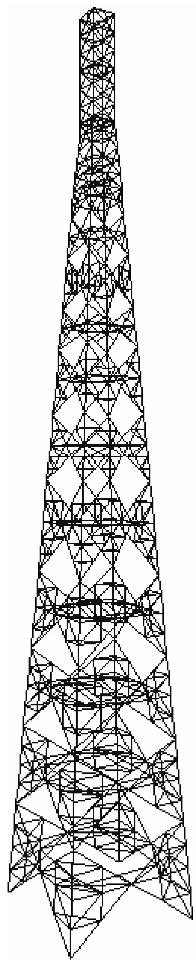


Figure 1. General view of the 67-m self-supporting 4-legged telecommunication tower.

2. BACKGROUND

Generally, the studies performed on telecommunication towers are divided into two categories of wind and earthquake loadings and the corresponding response of the towers to such loadings. In the early stages of telecommunication tower design, due to the lightness and height of such structures, much of the research efforts were focused on wind loading and its combination with ice. Nevertheless, in recent years, more attention is being paid to earthquake loading and its combination with ice due to high seismicity level of many regions where these towers are installed. In the latest editions of some of the accredited design codes, the topic of earthquake loading on such structures has been included.

Chiu and Taoka [2] were among the first to perform an experimental and theoretical study on the dynamic response of lattice self-supporting telecommunication towers under real and simulated wind forces. In their research, a 3-legged 46-m self-supporting telecommunication tower was investigated for its dynamic response under wind loading. The study showed that the tower response to wind-induced forces was dominated by the fundamental mode of vibration. In addition, the average damping for the fundamental mode was obtained to be 0.5 % of the critical viscous damping value, which is considered to be very low.

Venkateswarlu et al. [3] performed a numerical study on the response of lattice microware towers subjected to random wind loadings. The dynamic response could be estimated by the use of a stochastic approach. A spectral analysis method for evaluating the along-wind response and the corresponding gust response factor were introduced. The gust response factor is defined as the ratio of the expected maximum wind load effect in a specified time period to the corresponding mean value in the same time period. A 4-legged 101-m self-supporting tower was considered in their study. The gust response factor along the tower height was calculated with and without the contributions of second and higher modes of vibration. The results showed a maximum of 2 % change in the gust factor when employing higher modes of vibration.

In two important papers written by Holmes [4,5] on the response of lattice towers under wind-induced forces, a number of relations for determining the gust response factor, both for the shearing force and the bending moment along the tower height, were suggested. The towers considered in his study had both linear taper and uniform solidity ratios. Therefore, a constant value for the drag coefficient was adopted. In addition, only the effect of fundamental flexural mode of vibration was considered. The advantage of Holmes' relations over the routine standard relations is the inclusion of additional parameters to account for the effects of various characteristics of wind and structure. Eventually the performed studies led to the introduction of a simple method for the distribution of the effective static load including resonant, background, and mean components of the wind load.

Konno and Kimura [6] presented one of the first studies on the effects of earthquake loads on lattice telecommunication towers atop buildings. The objective of their work was to obtain the mode shapes, the natural frequencies, and the damping properties of such structures. Simulation of a stick model of the tower using lumped masses and a viscous damping ratio of 1 % was used in their studies. It was observed that in some of the members, the forces due to earthquake were greater than those due to wind.

Mikus [7] 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 earthquake accelerograms were considered as input in the analysis. It was concluded that modal superposition with the lowest four modes of vibration would ascertain sufficient precision.

In order to obtain methods for simplifying the seismic analysis of telecommunication towers, Galvez [8,9] and Khedr [10-12] performed more studies. Galvez investigated three different models of 3-legged lattice steel towers with varying heights of 90, 103, and 121 meters under the effects of 45 earthquake records. It was concluded that the contribution of the 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 %.

In addition, the equivalent static method introduced by Galvez was based on the acceleration profile that was deduced from the modal superposition of the towers lowest three flexural modes of vibration. One of the main disadvantages of the Galvez method was the bilinear shape of the acceleration profile, which did not properly 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 is obtained. Moreover, much research has been performed to obtain the fundamental frequencies of 3-legged self-supporting towers [13]. In the latest edition of the TIA/EIA code [14], provisions for seismic design of towers have been included.

3. METHODOLOGY

This study relies entirely on numerical experiments, i.e. detailed three-dimensional full-scale simulations using the finite element method, because experimental results relating to the overall seismic behavior of self-supporting telecommunication towers are very rare, if not non-existent.

The tower models are subjected to wind and earthquake loadings, and are also detailed to include P-Delta effects to allow for more realistic dynamic response of the towers. The selection of the towers was made to cover a wide range of tower heights, from 18 to 67 m, in order to identify some common trends in behavior.

4. DESCRIPTION OF TOWERS

The ten towers considered in this study were chosen from amongst the existing self-supporting telecommunication towers, which have been designed, fabricated, and erected on the basis that wind forces were the dominant design forces. All the towers are 4-legged with square transversal cross-sections. Different types of bracings, such as x-bracing and inverted chevron bracing, have been utilized in each tower. The towers member cross-sections are made-up of single equal-legged angles. All the angle sections (profile number 10 and higher) are made from ST32 steel with a tensile yield strength of 3600 kg/cm^2, and the angle sections (smaller than number 10) are made from ST37 steel with a tensile yield strength of 2400 kg/cm^2. The modulus of elasticity and the unit weight of the steel materials used were 2.1×10^6 kg/cm^2 and 7.85×10^{-3} kg/cm^3, respectively. The connection type mainly used was bolts/nuts, and in the case of excessive usage of bolts, interface steel plates are used.

The total height and the corresponding total bare weight of each tower are listed in Table 1 and Figure 2. The weights of antennas mounted on the towers are excluded from this weight. The weights are composed of the weight of structure in conjunction with the weight of ancillary components such as ladder, platforms, feeders, lights for aircraft warning, and bolts/nuts as connections. Furthermore, other geometric characteristics such as the length of the 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.

Figure 3 shows the overall view of the tallest self-supporting telecommunication tower under study along with the grouping of the member cross-sections; in this figure letters H and X stand for horizontal and diagonal members, respectively. Tower heights versus tower weights have also been depicted in Figure 4.

5. NUMERICAL MODELING CONSIDERATIONS

5.1. Tower Modeling Assumptions Due to the presence of numerous members and the variety of cross-sections in such structures, the computer software used should be equipped with an appropriate graphical capability so that a fast and precise numerical configuration could be defined. Easy access to the input and output data is also desired. On these basis, SAP2000 [15], among the existing structural programs, was chosen for analysis.

To account for the mass of ancillary components in the analysis, their mass was proportionally distributed along the tower height by modifying the material properties. It is worthy to note that the weight of ancillary components is considerably high and its exclusion from the analysis can alter the results. In modeling the towers under study, all the members including the

TABLE 2. Characteristics of the Selected Self-Supporting Telecommunication Towers (See Figure 2 for Notations).

L_{total} (m)	W_{total} (kg)	D_L (m)	D_O (m)	L_{taper} (m)	$(L_{total}-L_{taper})/L_{total}$ %
18	4100	2	3.4	10	44.4
22	5700	2	3.9	14	36.4
25	6800	2	4.3	17	32.0
30	8700	2	5	22	26.7
35	10700	2	5.7	27	22.9
42	13400	2	6.7	34	19.0
48	16700	2	7.6	40	16.7
54	20300	2	8.4	46	14.8
60	24600	2	9.3	52	13.3
67	29500	2	10.3	59	11.9

horizontal, diagonal, and the redundant members have been considered. Regarding the connections, depending on the number of bolts/nuts used, the connections are classified into two types of pin-ended and fixed-ended, and thus, the members are identified as truss and beam elements, accordingly.

A lumped mass formulation is used for the tower members. Structural damping is modeled by using equivalent viscous damping with a value of 2 % of critical viscous damping. Earthquake loads are assumed to occur under still air conditions (IASS [16]) so aerodynamic damping is not included in the model. As for the wind loading, all the towers are subjected to the wind loads evaluated according to provisions of the TIA/EIA code [17] using a static approach. It should also be noted that tower foundations are assumed perfectly rigid. At first, a frequency analysis is performed in order to obtain the natural modes of vibration of the towers. Next, all the towers undergo a spectral analysis under the standard design spectrum of the Iranian seismic code of practice [18], and subsequently under the normalized spectra of Manjil, Tabas, and Naghan earthquakes. At the end, wind loading results are compared to those obtained from the earthquake loadings.

5.2. Earthquake Loading For the ground motion effects to be considered, the design spectrum from the Iranian seismic code of practice has been utilized [18]. The relations presented in this code are as follows:

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

$$B = 1 + S\ (T/T_0) \quad 0 \leq T \leq T_0 \tag{2}$$

$$B = S + 1 \quad T_0 \leq T \leq T_s \tag{3}$$

$$B = (S + 1)\ (T_s/T)^{2/3} \quad T > T_s \tag{4}$$

Where

Sa = Spectral acceleration

T = Fundamental period of structure in seconds

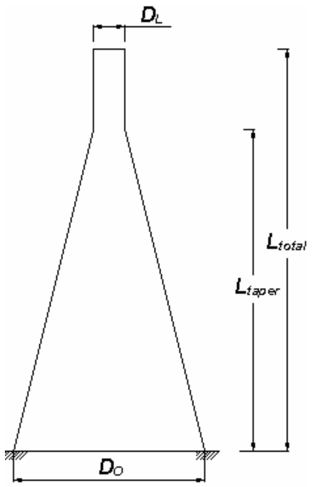


Figure 2. Notations of Table 1.

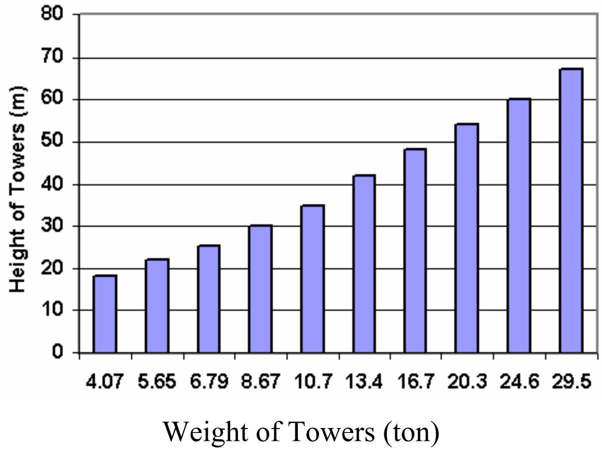


Figure 4. Tower weights versus tower heights.

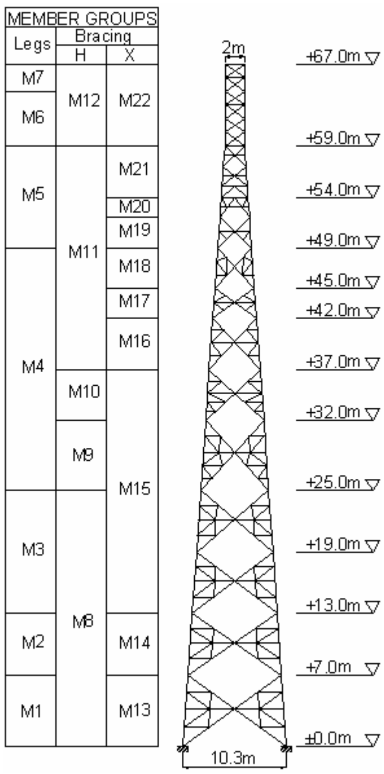


Figure 3. Member cross-section groupings along the 67-m tower.

A = 0.35 g (design base acceleration for regions with high seismicity level)

T_0 = 0.1 sec (for type 2 ground)

T_s = 0.5 sec (natural period for type 2 ground)

I = 1.2 (importance factor for structures with high importance)

R = 1.0 (reduction factor for the elastic behavior of structure)

S = 1.5 for very high seismicity level

Hence, the B curve in this research, after applying the above relations, will become as below (refer to Figure 5):

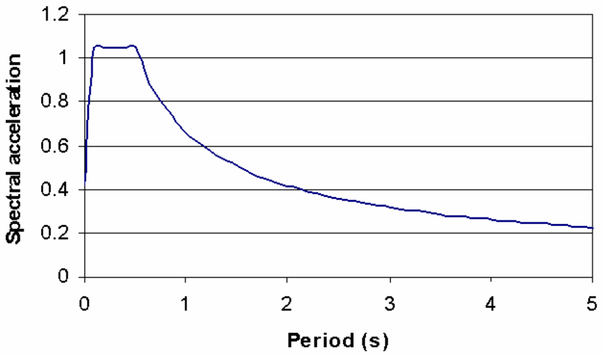
$$B = 1 + 15T \quad 0 \leq T \leq 0.1$$

$$B = 2.5 \quad 0.1 \leq T \leq 0.5$$

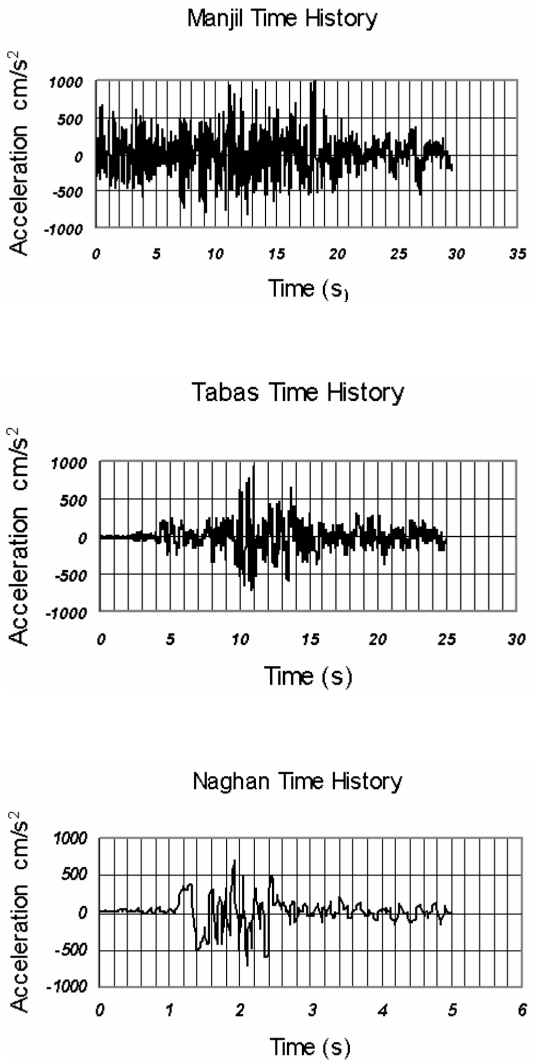
$$B = 2.5\,(0.5/T)^{2/3} \quad T > 0.5$$

The important point to note here is the low value chosen as the R factor for the telecommunication towers. Due to the stability and serviceability criteria required of such structures during and after the occurrence of an earthquake, i.e. elastic behavior of the structure, the selected value seems to be reasonable.

Furthermore, three classical horizontal earthquake accelerograms in Iran have been selected, representing different types of earthquake loading. The first is the 1988 Manjil earthquake with strong shaking and long duration; the second is the 1979 Tabas earthquake containing a wide range of frequencies and several episodes of strong ground motion; and the third is the 1978 Naghan



(a)



(b)

Figure 5. Earthquake loadings on towers, (a) Spectrum of the Iranian seismic code of practice [18], (b) Accelerograms of Manjil, Tabas, and Naghan earthquakes [20].

earthquake representing a single pulse loading. These earthquake records are selected to reflect realistic frequency contents as exhibited by real ground motions. 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 [18].

The standard design spectrum from the Iranian seismic code of practice and the three accelerograms of Manjil, Tabas, and Naghan earthquakes are illustrated in Figure 5. The ratios of peak ground acceleration to peak ground velocity (A/V) for the three mentioned earthquakes are 1.03, 0.97, and 0.83 respectively.

5.2.1. Consideration of earthquake force direction Since the towers under study are considered to be of regular geometry, i.e. no vertical/plan structural irregularities exist; the earthquake-induced forces may be applied separately in the two orthogonal directions. Nevertheless, as discussed afterwards, investigating the application of earthquake loads in the diagonal direction, or in the two orthogonal directions simultaneously (with loading proportions of 100 % and 30 %), is necessary in the case of 4-legged self-supporting telecommunication towers.

If we let, the axes normal to the tower horizontal members in its cross section be the principal axes denoted as X and Y, each of the tower surface planes can be considered as a structural truss and thus the tower legs acting as truss chords will behave axially when subjected to lateral loads. In other words, the resisting modes of tower faces act in flexure and hence the resisting modes of tower legs behave axially. Therefore, when the towers simultaneously drift laterally in both X and Y directions, the tower legs will jointly resist axially against the two lateral drifts. This effect is shown in Figure 6.

However, in reality, the earthquake motion is generally propagated vectorially in space and can be decomposed into two orthogonal horizontal directions and one vertical direction. For design purposes, the proportions of these motions are taken as 100 %, 30 %, and 40 %, respectively [18,19]. Therefore, in order to compute the internal forces in the leg members, the earthquake ground motions must be considered to act simultaneously in both X and Y directions, with the proportions of

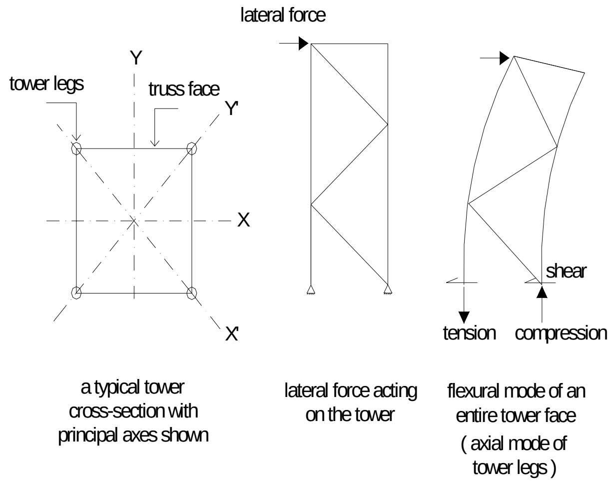


Figure 6. Tower behavior when subjected to lateral force.

100 % and 30 %, respectively.

If the diagonal axes of the square tower cross-section is allowed to be the principal axes, (i.e. X' and Y'), the joint axial resistance of the legs will no longer exist in time of simultaneous drift in both the X' and Y' directions. The reason for this is the fact that in every overall tower bending action (e.g. in X' direction), the axes perpendicular to that (i.e. Y') will act as neutral axes. Therefore, in order to evaluate the internal forces in the tower legs, the earthquake ground motion acting along the diagonal axes can be considered independently.

In addition, this idea has conceptually been discussed in the Iranian seismic code of practice for braced steel frame buildings [18]. According to the provisions outlined in this code, all the columns at the intersection of two perpendicular braced frames must be designed for the maximum axial forces resulting from the seismic forces acting in a direction with an appropriate angle that would lead to such maximum axial forces. This could be attained by the simultaneous application of earthquake loads in both orthogonal directions of the structure, i.e. X and Y, with proportions of 100 % and 30 %, respectively.

Applying the seismic forces obtained from the standard design spectrum of the Iranian seismic code of practice in three different conditions, i.e. perpendicular to the plane surface (X or Y direction), along the diagonal (X' or Y' direction), and perpendicular to principal directions with proportions of 100 % and 30 %, the axial forces in all the leg members, horizontal and diagonal members will be evaluated. The results obtained for the 67-m tower are shown in Figure 7. It must

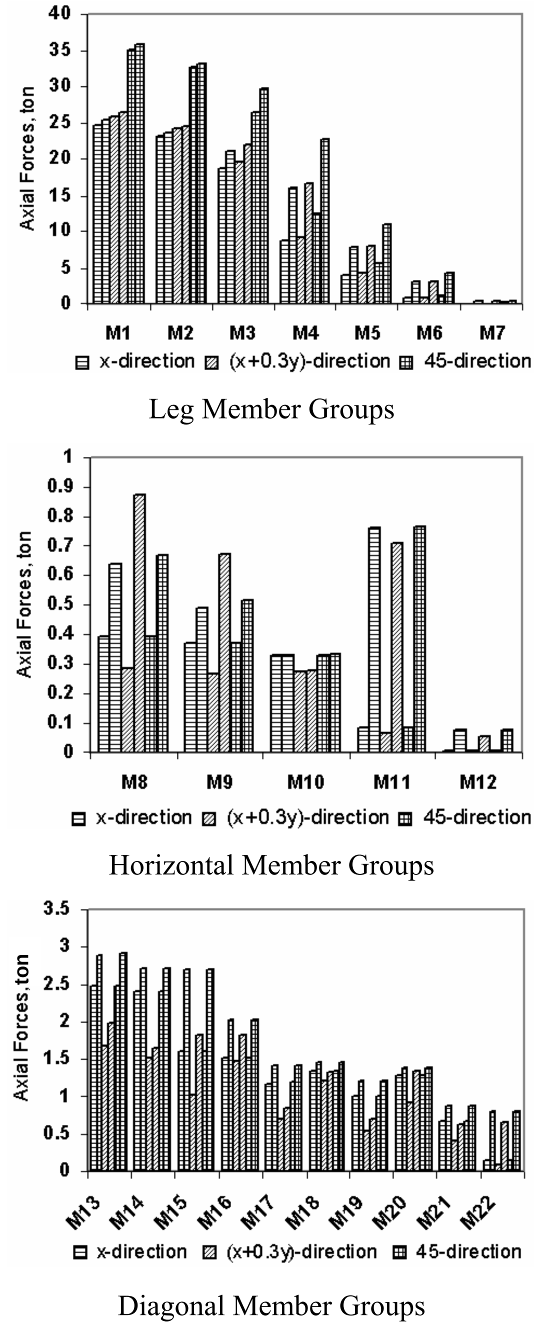


Figure 7. Axial member forces for the 67-m tower subjected to the spectrum of the Iranian code under three different loading conditions.

be noted that the axial forces were evaluated by considering a bare tower without taking into account the mass of antennas, while under the application of the standard design spectrum from the Iranian seismic code of practice.

The results indicate that for all the towers the axial forces in the leg members under the effect of seismic load in the diagonal direction attain the highest value. Nevertheless, in the case of axial forces in the horizontal and diagonal members, the results obtained from the three different conditions differ marginally with respect to one another. Furthermore, the value of axial forces in these members is small which proves that the slenderness criterion is the dominant factor in the design of bracing members in telecommunication towers.

Regarding the obtained results, the application of earthquake forces along the diagonal direction for the design of leg members in 4-legged self-supporting telecommunication towers can be considered as predominant. Noting that since most studies by researchers in the past were focused on the 3-legged towers, this subject had not been brought up until now.

5.3. Wind Force Specifications For applying the wind forces to the towers under study, the TIA/EIA code has been utilized. To calculate the wind-induced lateral forces, the base wind velocity of 160 km/hr with a return period of 50 years and duration of 3 seconds has been considered. According to the above code, in case of wind loads acting in conjunction with ice, an ice thickness equivalent to one inch is used. Moreover, since the transversal cross-sections of the towers under study are square, wind loads are applied along the diagonal axes.

6. RESULTS OF DYNAMIC ANALYSIS

6.1. Frequency Analysis For a precise distinction of vibration modes in a frequency analysis, i.e. to prevent the formation of local vibration modes, the mass of members must be lumped at the intersecting joints of the tower legs with the main horizontal and diagonal members. 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 bare towers. Amongst the towers under study, the tallest tower with 67-m height was the most flexible having a fundamental period of 0.58 seconds. The shortest tower with 18-m height exhibited the highest stiffness with a fundamental period of 0.17 seconds.

The lowest three flexural and torsional mode shapes of the 67-m tower are shown in Figure 8. The periods for the first twenty modes in conjunction with the corresponding participating modal mass ratios for the 67-m tower are listed in Table 2. After examining the figures and the tables corresponding to the towers under study, one may recapitulate as:

1. In all the towers, the flexural frequencies in every principal direction are very well separated.
2. The first torsional mode, for the towers shorter than 30 meters, is in between the first two flexural modes, whereas in the case of 30-m high and taller towers, the first torsional mode takes place after the second flexural mode.
3. The participating modal mass for the first flexural mode, in all the towers, is less than 50 percent.
4. In all the towers, 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, the analysis accuracy and precision will be greatly enhanced.
5. The first axial mode for the towers with heights of 18 and 22-m happens in the sixth mode, whereas for the towers with heights ranging from 25 to 67-m it occurs in the ninth mode.
6. In all the towers, the second torsional mode takes place close to the third flexural mode.

Notably, the results obtained here are in great agreement with the results of the studies performed by Galvez [8,9], Khedr [10-12], and Sackmann [13] on the frequency analysis of self-supporting 3-legged telecommunication towers.

Determining the natural frequencies of a

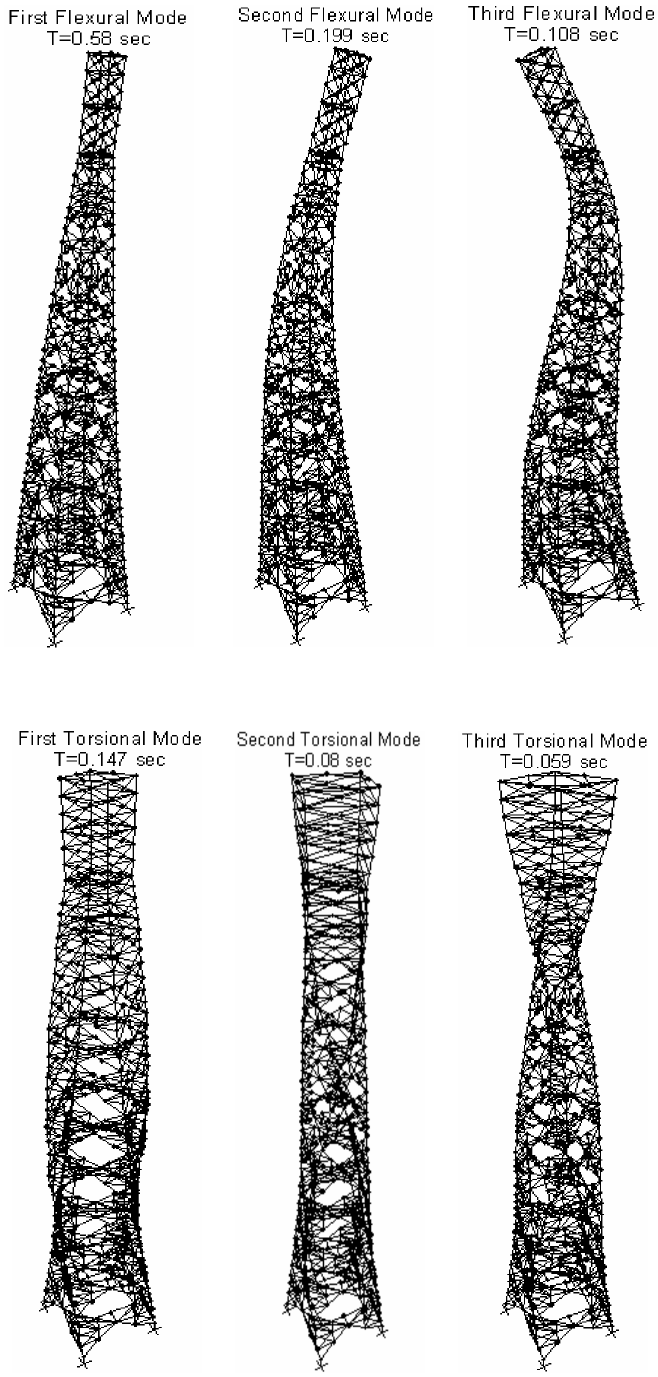


Figure 8. Flexural and torsional mode shapes for the 67-m tower.

structure followed by its corresponding mode shapes is of great importance in the assessment of dynamic amplification factors. In Figures 9-11, the periods of the lowest three flexural modes are drawn versus the different tower heights. In these figures, the proposed relations for determination of the periods of the lowest three flexural modes with respect to the overall tower heights are expressed as:

TABLE 2. Results of the Frequency Analysis on the 67-m Tower.

MODAL MASS PARTICIPATION RATIOS							
Mode	Period	Individual Mode (%)			Cumulative Sum (%)		
		UX	UY	UZ	UX	UY	UZ
1	0.579762	40.4327	0.3034	0	40.4327	0.3034	0
2	0.579762	0.3034	40.4326	0	40.7361	40.736	0
3	0.199251	20.3978	3.1791	0	61.1339	43.9151	0
4	0.199251	3.179	20.3974	0	64.313	64.3125	0
5	0.147289	0	0	0	64.313	64.3125	0
6	0.107851	12.3028	1.5844	0	76.6157	65.8969	0
7	0.107851	1.5847	12.3028	0	78.2005	78.1996	0
8	0.099287	0	0	0	78.2005	78.1996	0
9	0.086543	0	0	70.8667	78.2005	78.1996	70.8667
10	0.080001	0	0	0	78.2005	78.1996	70.8667
11	0.07011	0	9.1329	0	78.2005	87.3325	70.8667
12	0.070109	9.1351	0	0	87.3356	87.3325	70.8667
13	0.064666	0	0	0.2897	87.3356	87.3325	71.1564
14	0.062679	0.1018	0.0121	0	87.4374	87.3446	71.1564
15	0.062679	0.0122	0.1019	0	87.4496	87.4465	71.1564
16	0.061289	0	0	0	87.4496	87.4465	71.1564
17	0.059364	0	0	0	87.4496	87.4465	71.1564
18	0.058381	0	0	0	87.4496	87.4465	71.1564
19	0.051016	0.9033	6.2412	0	88.3529	93.6877	71.1564
20	0.051016	6.2375	0.903	0	94.5904	94.5907	71.1564

$$T_1 = 0.0102\ H^{0.958} \tag{5}$$

$$T_2 = 0.0027\ H^{1.027} \tag{6}$$

$$T_3 = 0.0010\ H^{1.13} \tag{7}$$

Where

T = Natural period in seconds

H = Overall tower height in meters

Furthermore, it should be noted that the proposed relations have been presented in accordance with the tower geometries under study. Naturally, the application of these relations to other telecommunication towers with different geometries will not lead to sufficiently accurate results, and therefore are only applicable to the 4-legged self-supporting telecommunication towers of similar geometrical configurations.

6.2. Spectral Dynamic Analysis In this section, all the towers under study went through the spectral analysis using the design spectrum

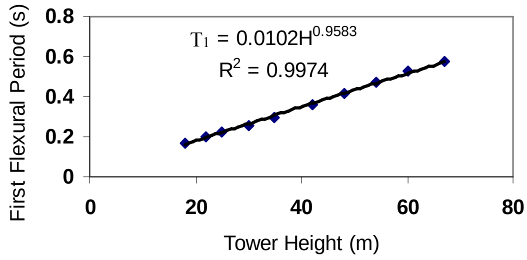


Figure 9. Period of the fundamental flexural modes versus tower height.

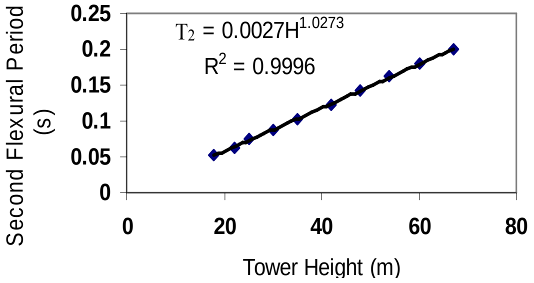


Figure 10. Period of the second flexural modes versus tower height.

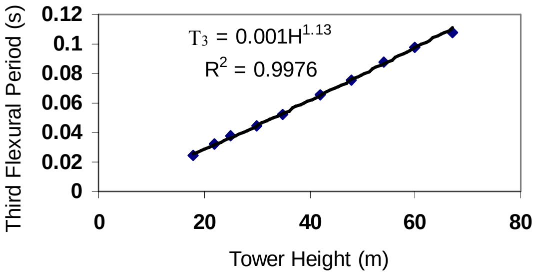


Figure 11. Period of the third flexural modes versus tower height.

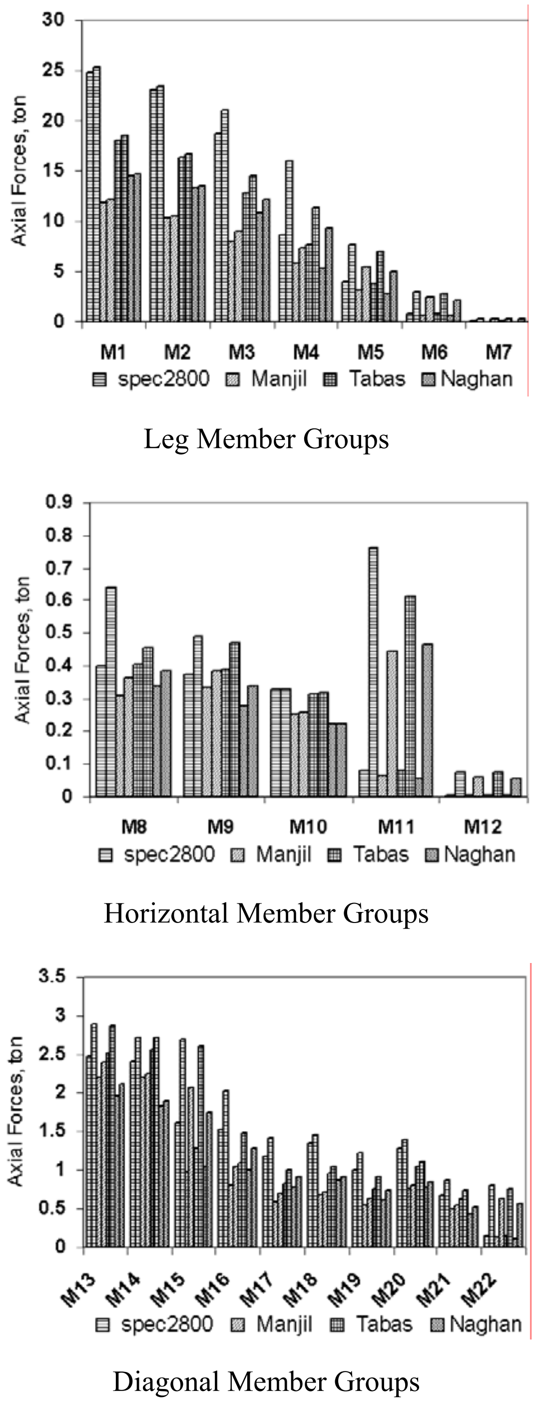


Figure 12. Axial member forces for the 67-m tower subjected to the four spectra.

from the Iranian seismic code of practice. 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.

Performing the spectral analysis on the towers, the axial forces in all the members are computed. Some of the results obtained from the dynamic analysis of the 67-m tower are shown in Figure 12. In this figure, the extremum values of axial forces in each group of members are presented.

The results show that in most of the towers, the axial forces in the leg members, when under the effect of the design spectrum from the Iranian seismic code, have the highest values. While in the case of short towers, e.g. 25-m and shorter, the normalized spectra of Manjil or Tabas are mostly predominant. Nevertheless, in the case of axial forces in the horizontal and diagonal members, the results obtained from the four spectra did not show a significant difference. In addition, the axial forces in these members are very small which is in line with the fact that slenderness is the dominant factor in design of tower bracing members. These results indicate that the leg members have more sensitivity to frequency content of earth movement than the bracing members do. The reason for this is the resistance of the leg members against the overturning moments that are caused by the lateral forces with their magnitudes depending on the frequency content of the earth movement.

6.2.1. Base shear Since the base shear is one of the important response indicators for earthquake resistant design purposes, the maximum base shear of the towers is investigated more closely.

The maximum base shear values for all the towers subjected to the four defined spectra are shown in Figure 13. 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 towers the normalized spectra of Manjil or Tabas earthquakes are predominant. For a more appropriate comparison of the base shear forces in the towers, Figure 14 illustrates these values as the percentage of the overall weight of bare towers. In the case of application of the design spectrum from the Iranian seismic code of practice, the base shear value given as a percentage of the total tower weight varies in the range of 50 % to 62 %. While, for Manjil, Tabas, and Naghan spectra, these values are in the ranges of 35 % to 70 %, 37 % to 67 %, and 34 % to 53 %, respectively.

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 design spectrum from the Iranian seismic code of practice), has been depicted versus the period of the fundamental flexural mode of vibration, as

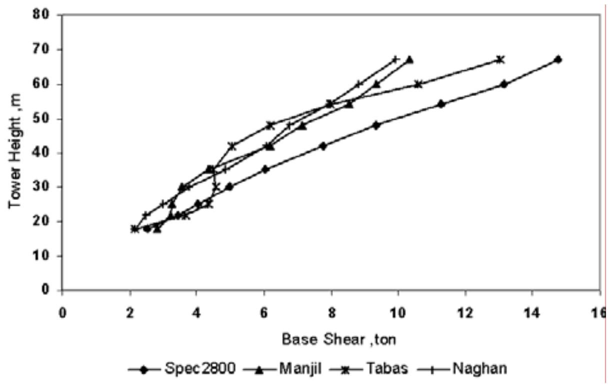


Figure 13. Maximum base shear versus tower height.

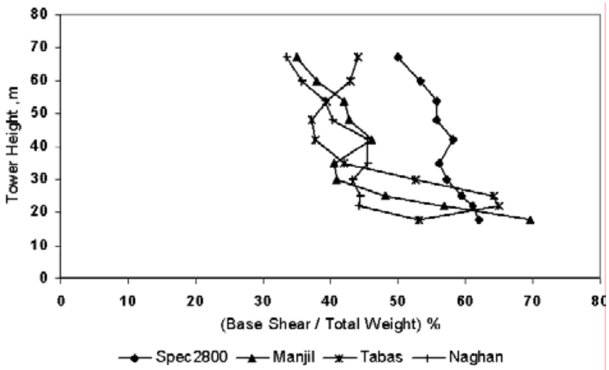


Figure 14. Ratios of the base shear to total weight versus tower height.

illustrated in Figure 15. From this figure, a linear relation for determining the base shear is introduced as follows:

$$V_h = M\,A_h\,(1.86 - 0.66\,T_f) \qquad (8)$$

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 tower.

The above relation is in agreement with the relations that were proposed by Khedr and McClure [12], which were obtained from the application of 45 earthquake records to 3-legged lattice towers.

As it can be observed, in shorter towers that

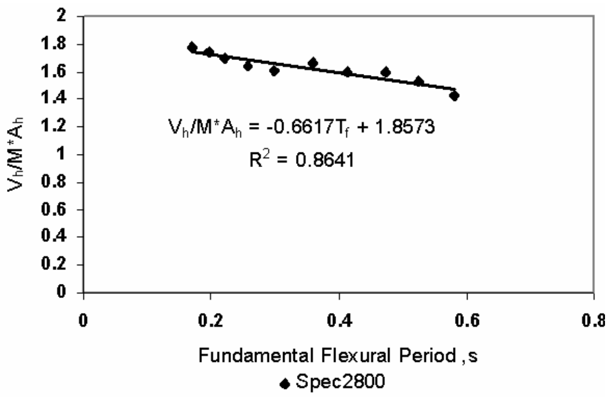


Figure 15. The ratio Vh/M.Ah versus the period of fundamental flexural mode for all the towers when subjected to design spectrum from the Iranian seismic code of practice.

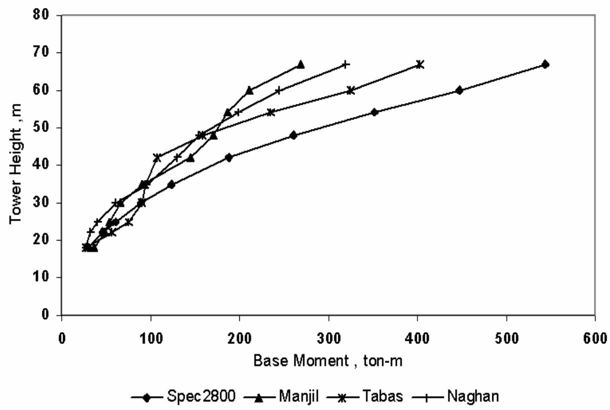


Figure 16. Maximum bending moment versus tower height.

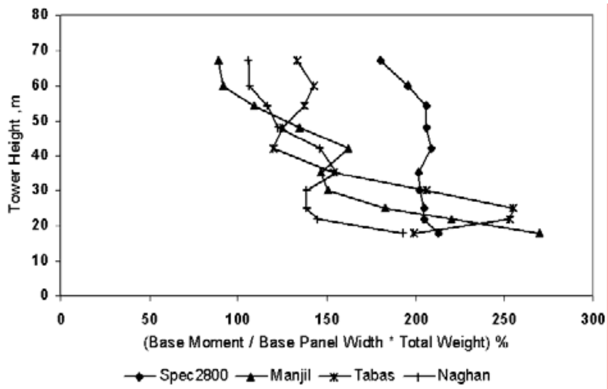


Figure 17. Ratios of the maximum bending moment to the product of the base panel width and total weight versus tower height.

have a shorter period for the first flexural mode, the ratio of the base shear to the product of mass and maximum acceleration of earth movement will increase, and with the increasing height and period of the first flexural mode, this ratio would decrease.

6.2.2. Maximum moment The values of the maximum bending moments for all the towers when subjected to each of the four defined spectra are shown in Figure 16. In order to make a more appropriate comparison of the maximum bending moments, they have been described in percent of the ratio of the maximum bending moment to the product of the base panel width and total tower weight, as shown in Figure 17.

6.2.3. Maximum lateral displacements The maximum lateral displacements in all the towers when subjected to each of the four spectra are shown in Figure 18. For a better comparison of the maximum lateral displacements, they have been depicted as a percentage of the tower overall height, as shown in Figure 19.

In Figure 20, a relation for determining the maximum lateral displacement of towers with respect to their height, when subjected to the design spectrum presented in the Iranian seismic code of practice, is expressed as follows:

$$d_{max} = 3E\text{-}5\ H^{2.0435} \qquad (9)$$

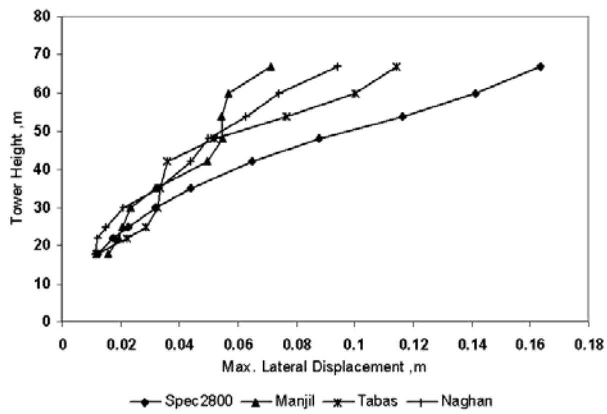


Figure 18. Maximum lateral displacement versus tower height.

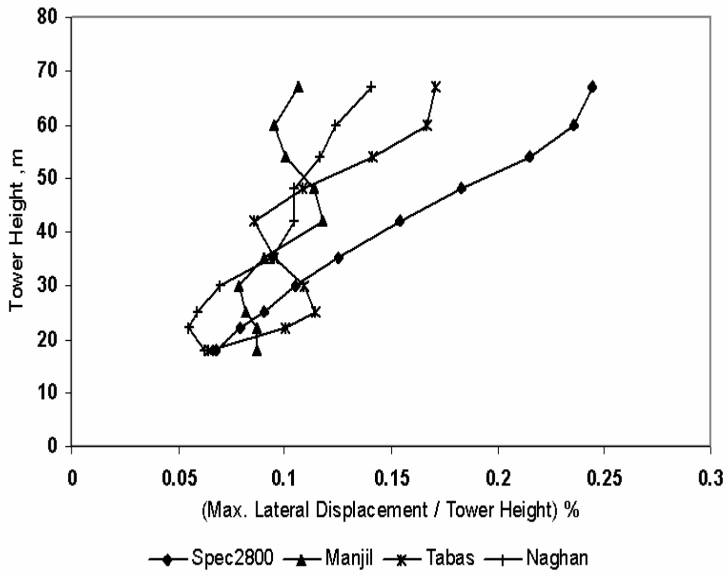


Figure 19. Ratios of the maximum lateral displacement to tower height versus tower height.

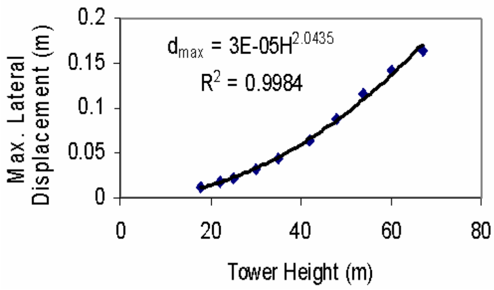


Figure 20. Maximum lateral displacement versus tower height when subjected to design spectrum from the Iranian seismic code of practice.

6.2.4 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 obtained from 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 21 shows the results for the 67-m tower. The TIA/EIA code [17] has introduced the following relations for the distribution of the lateral seismic forces along the tower elevation:

$$F_x = \frac{V w_x h_x}{\sum w_i h_i} \tag{10}$$

Where

w_i, w_x = Portion of weight at the i^{th} or x^{th} level

h_i, h_x = 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

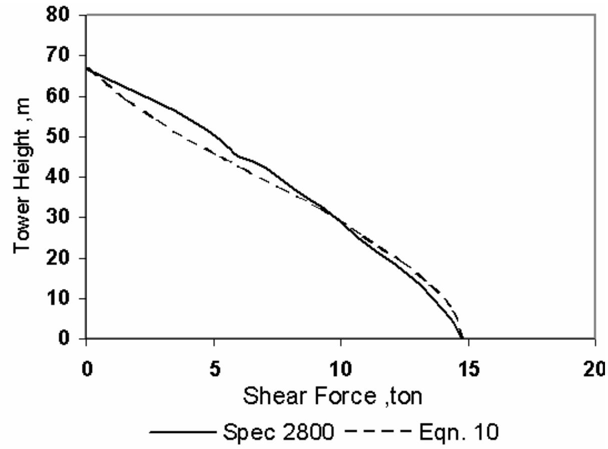


Figure 21. Distribution of the base shear force along the tower height for the 67-m tower.

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 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 is evaluated with respect to the base shear force 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 Figure 21 indicate that the shear force distribution along the tower height is in agreement with that obtained from the relation (Equation 10) 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.

6.2.5. Effects of concentrated mass of antennas
All the results presented in the previous sections are for bare towers. The mass distribution and the tower mode shapes will vary, depending on the antennas' mass, position, and height at which they are mounted on the towers. This matter is more significant for the towers with heavy antennas being attached to their top portion. Of course, when the application of new digital communication systems prevails in the future, the antennas used would have a much smaller diameter and weight than the presently used antennas, which greatly reduces the inertial effects of antennas on the tower response.

In any case, to account for the mass of heavy antennas installed at the top portion of the towers under study, 5 % and 10 % of the total mass of each tower is considered to be lumped at the top portion of the tower. Then, all the towers underwent a spectral analysis in accordance to the design spectrum from the Iranian seismic code of practice. The axial forces in the tower leg members were compared to each respective existing axial forces of bare tower. The results for the 67-m tower are shown in Figure 22. In this figure, the extremum values of the axial forces for each group of members are presented.

By examining this figure, it can be stated that the effect of inclusion of the mass of antennas on the leg members in the proximity of antenna attachments are much greater than those of bare towers. Moreover, the amount of axial forces in the leg members close to the ground level, especially in the case of the taller towers, have little difference with that obtained due to seismic forces acting on bare towers. This subject was also studied for the case of horizontal and vertical bracings. The results indicated that similar to the leg members, the axial forces in the bracing members near the antenna attachments (tower upper parts) are much larger than those in bare towers. These forces tend to equalize with the

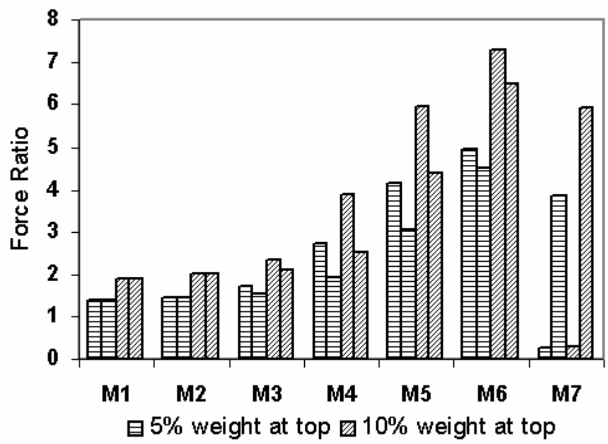


Figure 22. Ratio of the axial forces in leg members for the 67-m tower when the antenna mass is included.

values attained due to the application of seismic forces to the bare towers, as getting closer to ground level.

The normalized spectra of Manjil, Tabas, and Naghan earthquakes were also applied to the towers when the mass of antennas were included. In these cases, as for the design spectrum from the Iranian seismic code of practice, the axial forces in members near the antenna attachment points were by far greater than those obtained when the earthquake loads were acting on bare towers.

One can notice the decrease in the value of tower base shear when compared to the seismic forces acting on the bare towers. The reason for this is that with increasing the effective mass of towers, especially in their upper parts, the period of the first flexural mode increases. According to the shape of the defined spectra, the spectral acceleration reduces as the period increases.

It can be said that with the increase in the mass of antennas mounted on the towers, the base shear values due to the seismic force decreases, while the axial force in the members near the proximity of antenna mountings increases. This is consistent with the results of the research done by Khedr [10].

6.3. Static Wind Analysis In this section, a concise comparison among the results obtained from the wind and seismic forces acting on the towers under study is presented. Numerous parameters are involved in both types of loadings.

In the case of wind loading, the parameters such as loading combinations, angle of attack of the wind force, wind velocity or wind pressure, ice radial thickness, and number and weight of mounted antennas affect the tower behavior.

Taking into account the above points, comparisons between results obtained from the wind and the earthquake loads for the bare towers were presented. For this purpose, according to the TIA/EIA code [17], a wind speed of 160 km/h was selected. In addition, for the spectrum analysis, the design spectrum from the Iranian seismic code of practice was considered. The results obtained for the bending moments, and maximum lateral displacement of the towers are plotted in Figures 23-24.

As it can be observed, higher values have been attained when the tower was under wind load compared to seismic loads. Since the results due to the seismic loads are close to those resulting from the wind loads, further investigation of the seismic loads is essential.

Consequently, as most design codes introduce the wind load as the dominant force in the design of self-supporting telecommunication towers; the designer, using the empirical formulas presented in this paper, can quantitatively estimate the seismic response of these towers. Comparing the results with those obtained from the wind effects, the designer can decide on the necessity of seismic load application or otherwise. Ostensibly, increasing the number of antennas and consequently the tower weight, in addition to the high seismicity level of the regions where the towers are installed, leads to the conclusion that the seismic loads would have a much larger effect on such structures than it was previously thought.

7. CONCLUSION

It should be noted that the results obtained in this study are only applicable to bare self-supporting 4-legged latticed telecommunication towers with heights ranging from 23 to 67 meters. Additionally, the seismicity level considered is constant with PGA = 0.35 g which corresponds to high seismic risk regions in Iran. The results obtained in 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. In addition, a modal mass participation of less than 50 % has been achieved for the first flexural mode in all the towers.
2. The proposed relations (Equations 5-7) corresponding to the towers under study, and any other tower with similar geometries, have been introduced in order to calculate the lowest three flexural modes with respect to the overall height of the towers.

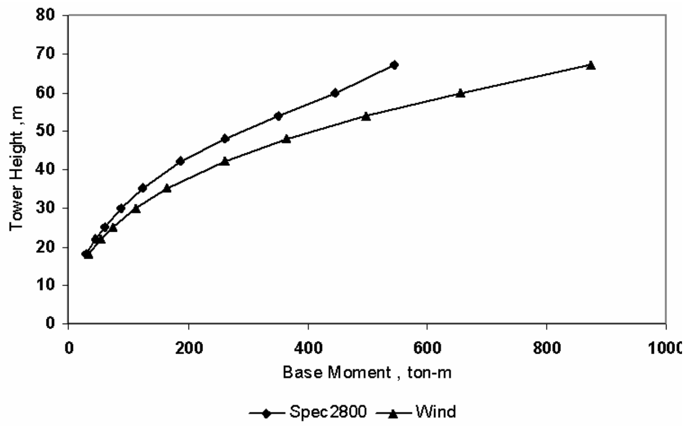


Figure 23. Maximum base moment versus tower height under the Iranian seismic code of practice and the wind loading.

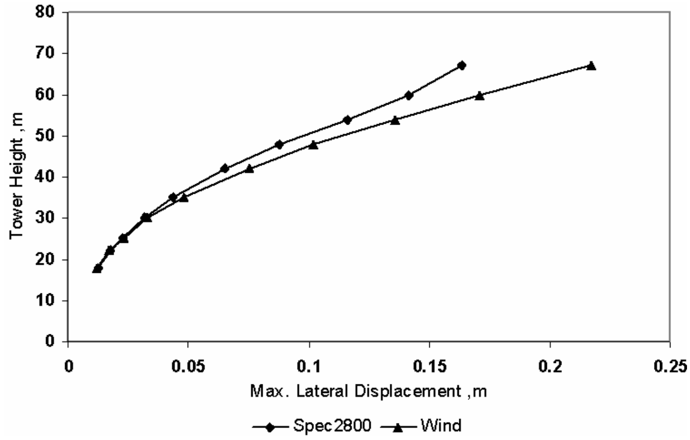


Figure 24. Maximum lateral displacement versus tower height under the Iranian seismic code of practice and the wind loading.

3. To perform a spectral analysis, the towers were subjected to four design spectra; the spectrum from the Iranian seismic code of practice, and the normalized spectra of Manjil, Tabas, and Naghan earthquakes. It was observed that the axial forces in the tower 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 earthquakes were predominant, but the axial forces in the horizontal and diagonal members show only a marginal difference when compared to those of the Iranian seismic code. This is an indication of a greater sensitivity of the leg members to the frequency content of earth movement. In addition, it is observed that the highest values are attributed to the base shear, the maximum bending moment, and the maximum lateral displacement of the towers, when subjected to the design spectrum from the Iranian seismic code of practice. Even though in short towers, the normalized spectra of Manjil or Tabas earthquakes are predominant.

4. Having the towers subjected to the design spectrum of the Iranian seismic code of practice, from different directions, it is observed that the highest axial forces in the leg members are attained when the seismic excitations are induced along the diagonal principle axes. However, the amount of axial forces in the horizontal and diagonal members, under the above seismic loading, does not show a considerable variation with respect to one another.
5. To determine the tower base shear due to the design spectrum from the Iranian seismic code of practice in terms of the maximum acceleration of the ground movement, the overall tower mass, and the period of the fundamental flexural mode, the relation (8) is proposed. Furthermore, to obtain the maximum lateral displacement of the towers in terms of their overall height when subjected to the design spectrum from the Iranian seismic code of practice, the relation (9) is presented.
6. Considering 5 % and 10 % of the total mass of each tower as a lumped mass placed at the top portion of the tower, the mass effects of the heavy antennas mounted on the upper parts of the towers have been investigated. The results showed that the axial forces in all the tower members that were near the antenna attachment points to the tower, are considerably higher than those in the bare towers. Of course, it has to be noted that by utilization of new digital communication systems, much smaller and lighter antennas would be used which will greatly reduce the mass effects of such antennas on the towers.
7. It may be concluded that the wind loading is prevailing with respect to the seismic loading, but, since the results from these two loadings are very close to each other, more investigation of the seismic forces is necessary.
8. The designers can estimate the effects of seismic forces by the usage of the empirical formulas presented in this paper. Comparing the obtained approximate values to that obtained from the wind forces, the designer can decide on the need for implementing the effect of the seismic forces or otherwise.

8. NOMENCLATURE

A	Design base acceleration
A_h	Maximum horizontal ground acceleration
B	Dynamic magnification factor
d_{max}	Maximum lateral displacement
F_x	Lateral seismic forces along the tower elevation
H	Tower height
h_i, h_x	Tower height from ground level at the i^{th} or x^{th} level
I	Importance factor
M	Tower mass
PGA	Peak ground acceleration
PGV	Peak ground velocity
R	Reduction factor
Sa	Spectral acceleration
T	Fundamental period of structure
T_0	Natural period of ground type
T_f	Fundamental flexural period
T_s	Parameter related to ground type
V	Total design shear force
V_h	Horizontal base shear
w_i, w_x	Portion of tower weight at the i^{th} or x^{th} level
S	Parameter related to seismicity level

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