World Applied Sciences Journal 2 (6): 635-643, 2007
ISSN 1818-4952

Earthquake Amplification Factors for Self-supporting 4-legged Telecommunication Towers

[1]G. Ghodrati Amiri, [2]M.A. Barkhordari, [3]S.R. Massah and [3]M.R. Vafaei

[1]Center of Excellence for Fundamental Studies in Structural Engineering,
College of Civil Engineering, Iran University of Science & Technology, Tehran, Iran
[2]College of Civil Engineering, Iran University of Science & Technology, Tehran, Iran
[3]Department of Civil Engineering, Mazandaran University of Science & Technology, Babol, Iran

Abstract: Telecommunication towers are among the few steel structures that little attention has been paid to their seismic behavior in existing design codes. Due to lack of design provisions, the designers apply the existing building design codes to telecommunication towers. However, the seismic behavior and response of telecommunication towers is very different from that of building structures. For example, the base shear values obtained for telecommunication towers using the building codes is much higher than what it actually is. In this research, for obtaining the seismic amplification factors, ten of the existing self-supporting four-legged telecommunication towers in Iran with heights ranging from 18 to 67 meters are selected. Then, strong motion earthquakes are applied to these towers in both vertical and horizontal directions. By performing linear dynamic analysis, the base shear and vertical response of the towers are calculated. Dividing the obtained base shear or the vertical response by the product of the tower mass and maximum horizontal and/or vertical acceleration components will result in seismic amplification factors of both horizontal and vertical earthquake components. At the end, by drawing the amplification factors versus the fundamental flexural mode and the first axial mode of the towers, relations for estimating the base shear and the vertical response of the telecommunication towers are obtained.

Key words: Earthquake amplification factors · linear time-history analysis · self-supporting 4-legged telecommunication towers

INTRODUCTION

With advances in the telecommunication industry, the use of telecommunication towers is rapidly increasing. Yet, sufficient and consistent information regarding the seismic analysis of such structures, which their serviceability after the occurrence of natural disasters must be preserved, is not available.

Most studies performed up to now are related to either guyed masts or 3-legged self-supporting towers. The studies related to 4-legged self-supporting towers are rare if non-existence. More over, most of the design standards available on this subject introduce only few general guidelines instead of introducing some empirical relations useful for engineers.

Generally, telecommunication towers are divided into two categories of self-supporting and guyed towers. The self-supporting towers are divided into lattice towers and monopoles. The latticed self-supporting towers are either 3-legged or 4-legged. The connection types are mainly bolts and nuts. The sections usually used as primary and secondary members are hot-rolled angles.

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

Corresponding Author: Prof. G. Ghodrati Amiri, Center of Excellence for Fundamental Studies in Structural Engineering, College of Civil Engineering, Iran University of Science and Technology, P.O. Box 16765-163, Narmak, Tehran 16846, Iran

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 because of the high seismicity level of many regions where the towers are installed. In the latest editions of world's most advanced design codes, the topic of earthquake loading on such structures has been included.

Konno and Kimura [1] 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 [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 without considering the antennas and other accessories attached to them. 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.

Much research has been performed to obtain the fundamental frequencies of self-supporting towers (e.g. Sackmann [3]). In most of these studies, the research was focused on 3-legged towers and very few studies have been conducted on 4-legged towers. For this reason, in this paper, the dynamic sensitivity of ten existing 4-legged self-supporting telecommunication towers subjected to wind and earthquake loadings have been investigated.

In order to obtain methods for simplifying the seismic analysis of telecommunication towers, Khedr [4, 5] introduced a modified method for obtaining horizontal and vertical acceleration profiles. In his method, for every specified 3-legged tower a separate acceleration profile can be obtained. Moreover, in TIA/EIA code [6], CSA code [7], and most recently in ASCE [8], provisions for seismic design of towers have been included.

THE EARTHQUAKE ACCELEROGRAM RECORDS USED IN THIS RESEARCH

Ten earthquake records obtained from five different regions are selected to be employed as the tower base excitations. All the records include a maximum ground acceleration of greater than 0.05 g. All the earthquake accelerograms are assumed to be recorded on type I ground, according to the Iranian seismic code of practice [9]. Furthermore, the *A/V* ratio (peak ground acceleration to peak ground velocity) has been employed as a dynamic parameter for the selection of earthquake records. In this research the grouping of the records is based on the research done by Tso *et al*. [10].

Table 1: Properties of the selected earthquake records as the base excitation of towers in this study

Record No.	Station	DUR. (sec)	Year	m_b	M_s	FD	ED	Soil Class
1347-4	SIRCH	18.6	89	5.6	5.7	32	38	1
1425	SIRCH	7.66	92	4.6	3.9	15	8	1
1492-16	ZARRAT	43.5	94	5.8	5.8	18	26	1
1043	GHAEN	19.54	76	5.8	6.4	10	10	1
1084-48	TABAS	15.24	78	4.9	4.8	10	64	1
1082-1	DEYHUK	58.4	78	6.7	7.3	17	37	1
1492-6	ZARRAT	33.24	94	5.5	5.3	16	15	1
1519-4	ZARRAT	20.44	94	5.0	4.5	16	29	1
1084-21	TABAS	12.39	78	4.9	4.5	10	21	1
1084-46	TABAS	16.16	78	5.0	4.7	22	17	1

Table 2: Peak ground acceleration, peak ground velocity and related A/V ratio of the selected records

Record No.	PGA	$PGV\left(\frac{m}{s}\right)$	$\frac{PGA}{PGV}\left(\frac{s}{v}\right)$
1347-4	0.068g	0.0312	2.18g
1425	0.051g	0.0186	2.74g
1492-16	0.156g	0.115	1.35g
1043	0.13g	0.108	1.2g
1084-48	0.143g	0.0495	2.88g
1082-1	0.324g	0.195	1.66g
149-6	0.224g	0.083	2.69g
1519-4	0.067g	0.0174	3.85g
1084-21	0.125g	0.0436	2.86g
1084-46	0.099g	0.0472	2.09g

The selected records along with their characteristics are shown in Table 1. As it is shown in the table, both near field and far field records are included in the selected set of records. The maximum longitudinal acceleration and velocity of the records along with their *A/V* ratios are given in Table 2. The vertical components of the records are employed as the vertical excitation of the towers.

TOWERS USED IN THIS RESEARCH

Ten of the existing self-supporting 4-legged latticed steel towers, which have been erected at different locations in Iran, were selected for this study. The geometrical characteristics of these towers are illustrated in Table 3. In addition, in Table 4, one can observe the

Table 3: Geometry of the selected towers

L_{total} (m)	Top width(m)	Base width(m)	L_{taper} (m)
18	2	3.4	10
22	2	3.9	14
25	2	4.3	17
30	2	5.0	22
35	2	5.7	27
42	2	6.7	34
48	2	7.6	40
54	2	8.4	46
60	2	9.3	52
67	2	10.3	59

Table 4: Weight characteristics of the selected towers

Tower Height (m)	Actual Weight (kgf)	Nominal Weight (kgf)	Actual weight / Nominal weight	Modified Material Density(kgf/m^3)
18	4072	2547	1.599	12551
22	5651	3513	1.609	12627
25	6791	4195	619.1	12707
30	8670	5590	551.1	12174
35	10721	7223	484.1	11652
42	13366	9448	415.1	11105
48	16700	11873	407.1	11041
54	20271	14497	398.1	10976
60	24627	17640	396.1	10959
67	29513	21660	3631.	10696

nominal and actual weights of the selected towers. In this study, the actual weights of towers have been considered which includes non-structural accessories such as ladders, feeders, etc. The three-dimensional numerical modeling of towers is based on the assumption that elements are mass less. To include the weight of accessories in the overall tower weight, the element material densities are modified. The tower legs are assumed to be fixed at the foundation. The bracing and leg members are modeled as truss and beam elements, respectively.

ANALYSIS METHOD

Linear dynamic approach has been employed for tower analysis. For this purpose, time history acceleration records are translated to SAP2000 [11] format. Using the first twenty modes of vibration, the modal mass participation of all towers in flexural modes and axial modes reaches to more than 90 percent and 85 percent, respectively. The damping ratio used in all modes is equal to 3 percent as suggested by IASS [12], i.e. for towers with regular bolt/nut connections.

Estimating the tower base shear using Iranian Seismic Code (ISC): Fundamental differences exist between the structural behavior of telecommunication towers and building structures. The greatest response of buildings occurs in their first vibration mode, while, in telecommunication towers at least the first four modes of vibration have to be considered for achieving reliable results [4]. Additionally, the seismic design level for common building structures is to prevent the structure from collapse, but in telecommunication towers, the seismic design level must be immediate serviceability after occurrence of a strong earthquake. In order to observe the difference between the results obtained by using the equivalent static method and the results obtained by the dynamic analysis, tower base shears are evaluated using both methods. At first stage, the base shears, using the equivalent static method from the Iranian seismic code are evaluated.

$$V = CW \tag{1}$$

Where;

$$C = ABI/R \tag{2}$$

$A = 0.35$	design base acceleration for high seismic risk zone
$I = 1.2$	importance factor
$R = 1$	structural behavior factor
$B = 2.5(T_0/T)$	structural seismic response factor
$T_0 = 0.4$	ground type factor for type I

At second stage, the base shears, using the response spectra of Tabas, Manjil, and Naghan earthquakes are computed. The results are illustrated in Figure 1.

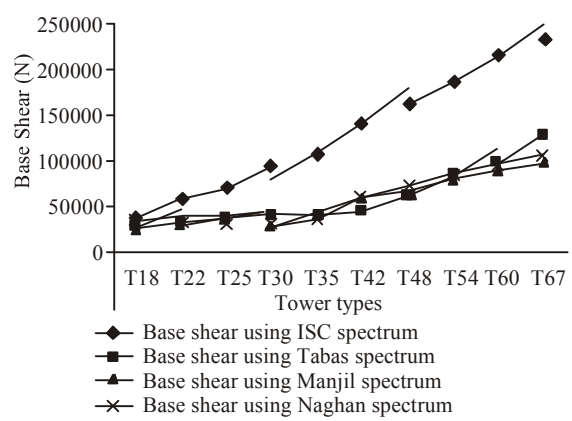

Fig. 1: Comparison of tower base shears obtained using response spectra of Manjil, Tabas and Naghan earthquakes with base shears obtained using equivalent static method

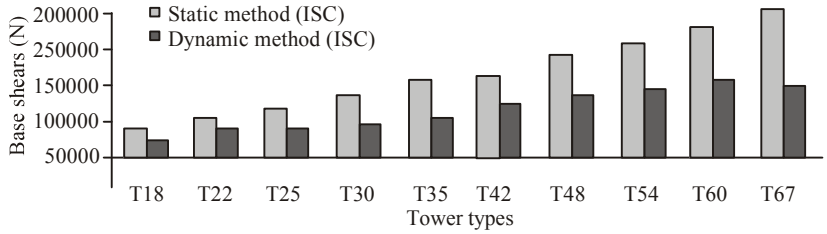

Fig. 2: Comparison of tower base shears obtained using dynamic time-history approach with base shears obtained using static equivalent method

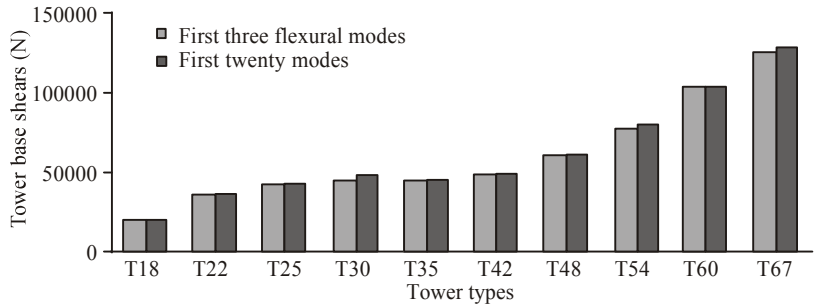

Fig. 3: Comparison of tower base shears using Tabas response spectrum; considering the first three flexural modes and the first twenty modes

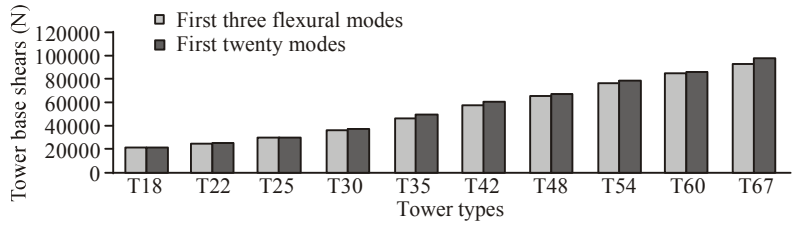

Fig. 4: Comparison of tower base shears using Naghan spectrum; considering the first three flexural modes and the first twenty modes

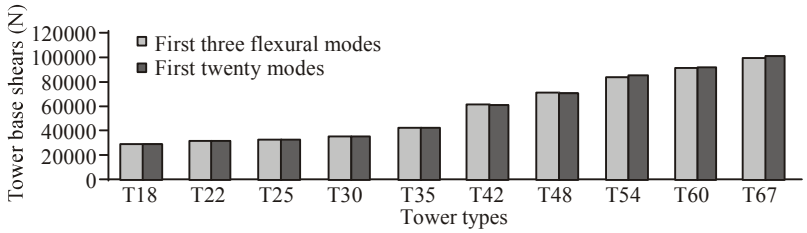

Fig. 5: Comparison of tower base shears using Manjil spectrum; considering the first three flexural modes and the first twenty modes

It is observed that the results obtained from the first method are much higher than that in the second method. Figure 2, also, illustrates the results of the first static method compared with that of the design spectrum from the Iranian seismic code.

Investigating the number of necessary vibration modes required for dynamic analysis of self-supporting 4-legged towers: To investigate the necessary number of vibration modes for dynamic analysis of 4-legged self-supporting towers, three different approaches have been employed for evaluating tower base shears. First, using the response spectrum method, the three spectra of Tabas, Manjil, and Naghan earthquakes are applied for obtaining the base shears, by considering only the first fundamental vibration mode. Second, using the same spectrum, the first three flexural vibration modes are considered for evaluating the tower base shears. Third, the first twenty vibration modes are considered for calculating the tower base shears. Figures 3 to 5 compare the base shears

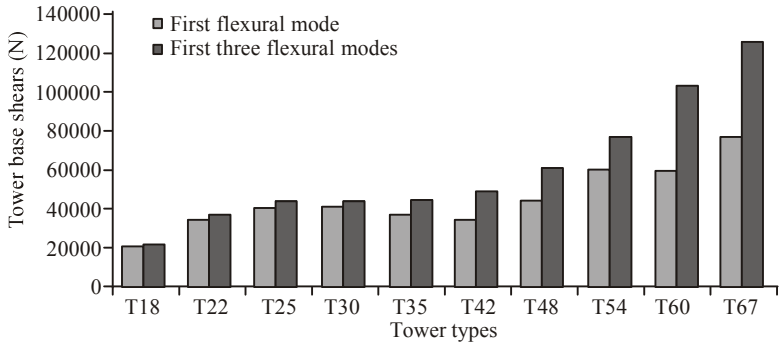

Fig. 6: Comparison of tower base shears using Tabas response spectrum; considering the first flexural mode and the first three flexural modes

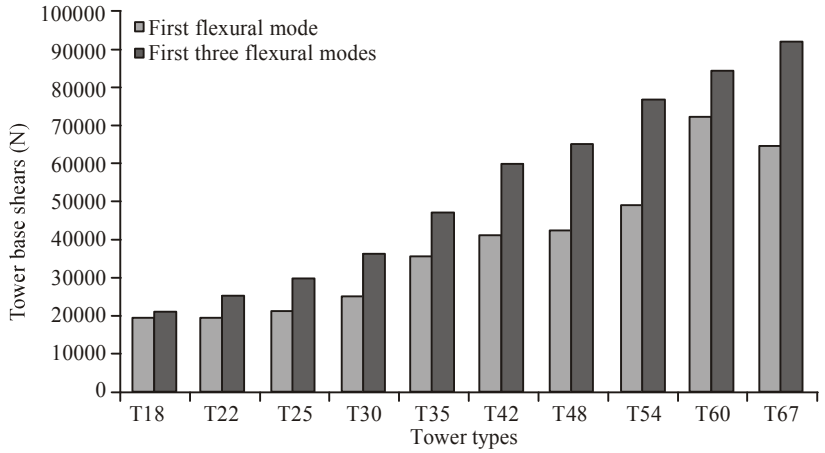

Fig. 7: Comparison of tower base shears using Naghan response spectrum; considering the first flexural mode and the first three flexural modes

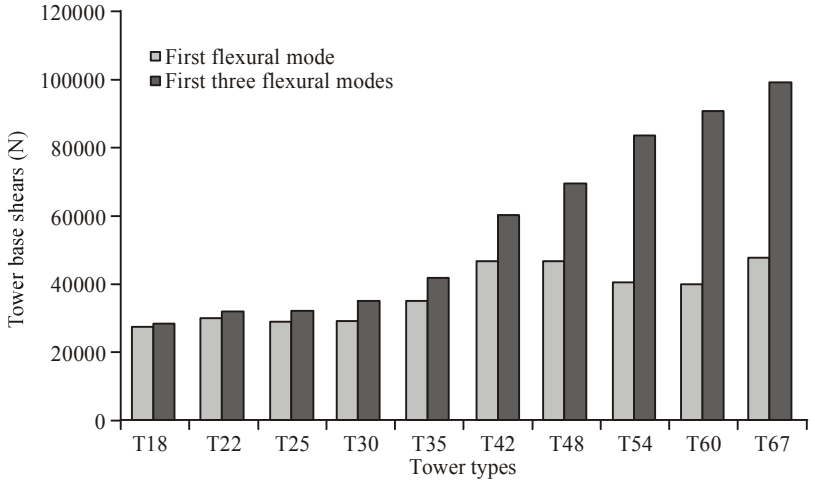

Fig. 8: Comparison of tower base shears using Manjil response spectrum; considering the first flexural mode and the first three flexural modes

obtained by employing the second and third approaches. As one can observe, small differences exists in base shear results obtained by the two latter approaches. In Figures 6 to 8, the tower base shears obtained by considering only the fundamental flexural mode has been compared with the base shears obtained by considering

the first three flexural vibration modes. As it is shown, considering only the first flexural mode in the dynamic analysis of towers, results in smaller base shear values as compared with the actual base shear values.

Investigating the towers natural vibration modes and periods: Since SAP2000 software has no limitations regarding the number of vibration modes considered in analysis, therefore, the first twenty modes of vibration have been considered in the dynamic analysis in this study. By investigating mode shapes, natural periods and percent mass participations, the following results are concluded:

- By increasing tower height, natural period of the first flexural mode increases from 0.17 seconds for the 18-meter tower to 0.58 seconds for the 67-meter tower. It is observed in Fig. 9 that the second and third flexural modes also increase as the tower height increases. In addition, the difference between the natural periods of the first and second flexural modes increases as the tower height increases. This conclusion also stands for the natural periods of the second and third flexural modes.
- The first torsional mode for towers shorter than 30 meters occurs in the third mode of vibration, while for taller towers it occurs in the fifth mode.
- The first axial mode for towers with heights of 18 and 22 meters occurs in the sixth mode while for other towers it occurs in the ninth mode.
- Since the effective modal mass participation in some of the flexural modes has quantities in both orthogonal directions, thus applying seismic forces in directions inclined to each tower face, other than the perpendicular directions, seems necessary.
- It is shown in Fig. 10 that as the tower height increases, the natural periods in first, second, and third torsional modes decreases, in contrary to the increase in flexural modes. In addition, the difference in natural periods for the first two torsional modes in shorter towers is more than that in taller towers.
- As shown in Fig. 11, the natural period for the first axial mode increases with the increase in tower height.
- The first three flexural modes, for the 18 and 22-meter towers, occurs in first nine vibration modes, for towers with heights of 25 and 30 meters, it occurs in first eight vibration modes, and for the remaining towers, it occurs in first seven vibration modes. Therefore, as the tower height increases, the number of necessary modes to consider the first three flexural modes decreases. Thus, it can be concluded that the number of vibration modes required for a satisfactory dynamic analysis of the selected towers would be the first nine modes of vibration.
- It is observed that the percent mass participation of the first three flexural modes reaches 90 percent, and, hence it can be concluded that the first three flexural vibration modes are sufficient for the dynamic analysis of towers. Figure 12 shows that the first

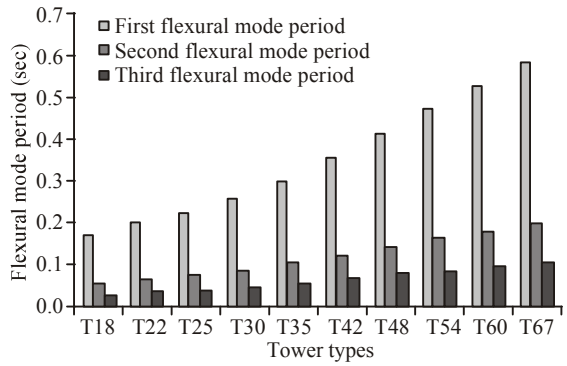

Fig. 9: Tower natural periods with respect to each of the first three flexural modes

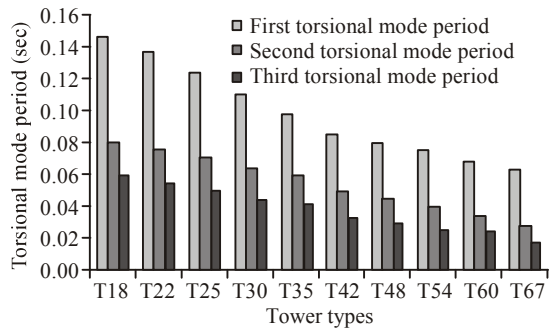

Fig. 10: Tower natural periods with respect to each of the first three torsional modes

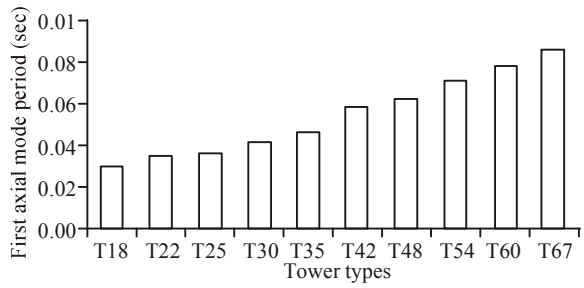

Fig. 11: Tower natural periods with respect to the first axial mode

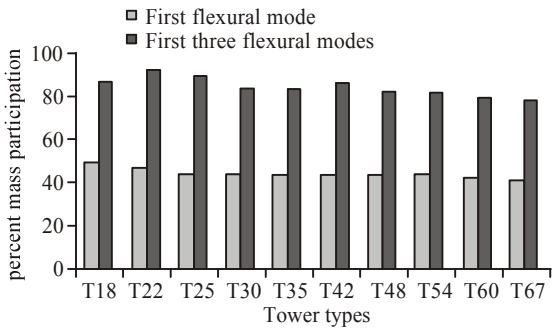

Fig. 12: Towers percent mass participation considering the first flexural mode and the first three flexural modes

flexural mode contributes only to 60 percent of the total mass being participated in the dynamic analysis. Additionally, while considering the first three flexural modes in the analysis, as the tower height increases, the percent modal mass participation of towers will decrease.

SEISMIC EXCITATION OF TOWERS FOR EVALUATING THE EARTHQUAKE AMPLIFICATION FACTORS

Horizontal earthquake force component: At the beginning, all the towers undergo base excitations using the ten selected records for obtaining maximum base shears by applying the time-history method of analysis. Next, for determining the relation between tower base shear and peak ground acceleration (A), peak ground velocity (V), and A/V ratio, Figures 13 to 15 are depicted for the 67-meter tower. In each case, by applying linear regression analysis, the related correlation factors are calculated. A strong correlation exists between the tower base shear and peak ground acceleration, as well as the peak ground velocity; though the relation between tower base shear and A/V ratio seems to be weak. Based on the results obtained by regression analysis, the horizontal earthquake amplification factors are described by the ratio of maximum base shear to the product of tower mass and horizontal peak ground acceleration. The amplification factors are evaluated for the selected set of records and then depicted against the fundamental flexural period of each tower as shown in Fig. 16. Using the linear regression analysis, the following relation is obtained for estimating tower base shear with respect to its fundamental flexural period:

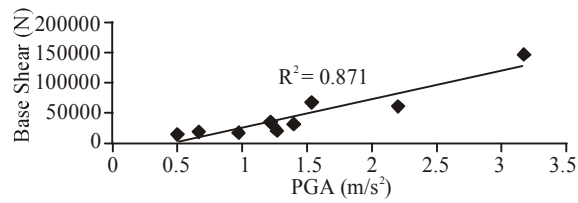

Fig. 13: Correlation factor of the base shear and the peak ground accelerations for the 67-meter tower

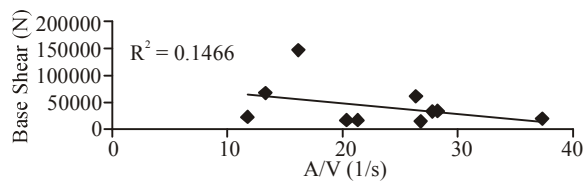

Fig. 14: Correlation factor of the base shear and the A/V ratio for the 67-meter tower

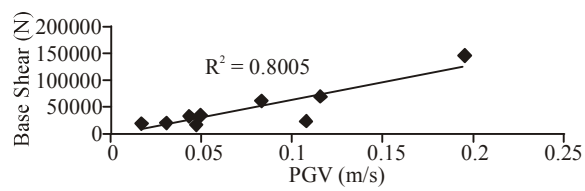

Fig. 15: Correlation factor of the base shear and the peak ground velocity for the 67-meter tower

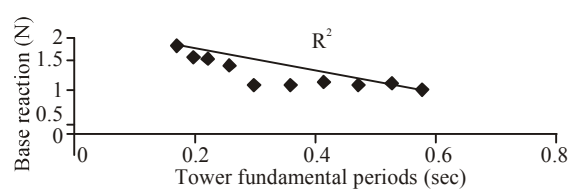

Fig. 16: Correlation of towers earthquake amplification factors and natural fundamental periods

$$V_h = Ma_h(-1.85T_f+1.9) \quad (3)$$

Where;

V_h = maximum tower base shear (N)
M = tower mass (kg)
A_h = peak horizontal ground acceleration (m/sec^2)
T_f = fundamental flexural period of towers (sec)

Vertical earthquake force component: Vertical components of the set of earthquake records are applied to towers as the vertical base excitation. For this purpose, after performing the linear dynamic time-history analysis, the vertical tower reactions are evaluated. Tower base reactions and peak vertical ground accelerations are depicted in Fig. 17 for the 67-meter tower. A strong

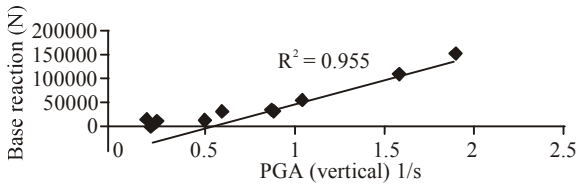

Fig. 17: Correlation of the maximum vertical base reaction and vertical peak acceleration for the 67-meter tower

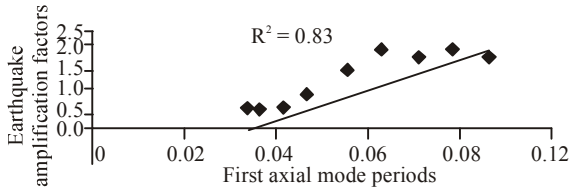

Fig. 18: Correlation between earthquake amplification factors (for vertical components) and first axial modes

correlation exists between tower base reaction and peak vertical ground acceleration (R^2=0.87). Based on the results obtained from the regression analysis, the earthquake amplification factor for the vertical tower reaction is described as the ratio of maximum vertical reaction (P) to the product of tower mass and peak vertical ground acceleration (MA_V). The tower amplification factors are calculated for the selected set of records and then depicted against the first axial period of towers as shown in Fig. 18. Applying the linear regression analysis, the following relation is obtained for estimating tower vertical base reactions:

$$P = MA_V(28.96T_a - 0.35) \quad (4)$$

Where;
P= tower vertical reaction (N)
M= tower mass (kg)
A_V= peak vertical ground acceleration (m/sec^2)
T_a= first axial period of tower (sec)

Summary of results: By a thorough investigation of the results obtained in this study the followings can be concluded:

- The tower base shears obtained by using the equivalent static method are much higher than the values obtained by dynamic analysis. Therefore, the existing relations in the building codes are not appropriate for use in latticed tower design.
- The number of mode shapes needed for consideration in a dynamic analysis, for a more precise and accurate results, are at least the first three flexural modes. Considering only the first vibration mode will result in much less base shear values than in reality.
- Weight of non-structural accessories mounted on towers should be included in the model analysis, since the actual total weight of each tower, including the weight of all accessories, is 30 to 60 percent more than the tower nominal weight.
- The results of frequency analysis of towers show that by increasing tower height, the natural vibration period of the fundamental flexural mode and the first axial mode will increase, whereas the period of the first torsional mode decreases. In addition, as the tower height increases, the first axial and torsional modes occur in higher modes.
- Since, in some of the flexural modes, the effective mass signifies in both principle directions, applying the seismic forces in directions other than the principal directions seems necessary.
- Strong correlation exists between the maximum tower base shear and peak horizontal ground acceleration and peak horizontal ground velocity. However, the relation between tower base shear and A/V ratio is week.
- According to the relation determined for estimating the tower base shear, (Eq. 3), as the tower fundamental period increases (in other words as the tower height increases), the tower seismic amplification factor decreases. Meaning that if considering two towers with identical mass and on same ground conditions, ex., placed on bedrock, the tower with higher period for first flexural mode of vibration will have smaller base shear.
- Strong correlation exists between the maximum tower vertical base reaction and peak vertical ground acceleration. This correlation is weak when considering peak horizontal ground acceleration.
- According to the relation obtained for estimating the maximum tower vertical base reaction, (Eq. 4), as the period of the first axial mode increases, the tower seismic amplification factor increases.

REFERENCES

1. Konno, T., and E. Kimura, 1973. Earthquake effects on steel tower structures atop buildings. Proceedings of the 5th World Conference on Earthquake Engineering, Rome, Italy, Vol. 1: 184-193.

2. Mikus, J. 1994. Seismic analysis of self-supporting telecommunication towers, M. Eng. Project Report G94-10. Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada.
3. Sackmann, V., 1996. Prediction of natural frequencies and mode shapes of self-supporting lattice telecommunication towers, M. Eng. Project, 1996. Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada.
4. Khedr, M.A., and G. McClure, 2000. A simplified method for seismic analysis of lattice telecommunication towers. Canadian Journal of Civil Engineering, 27: 533-542.
5. Khedr, M.A., and G. McClure, 1999. Earthquake amplification factors for self-supporting telecommunication towers, Canadian Journal of Civil Engineering, 26(2): 208-215.
6. TIA/EIA-222-F. Structural Standards for Steel Antenna Towers and Antenna Supporting Structures, 1996. American National Standard Institute.
7. CSA., 1994. Antennas, towers, and antenna-supporting structures. Standard CSA S37-94, Canadian Standards Association, Etobicoke, Ontario, Canada.
8. ASCE. Madugula, M.K.S. Dynamic Response of Lattice Towers and Guyed Masts, Task Committee on the Dynamic Response of Lattice Towers, American Society of Civil Engineers, 2001.
9. Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800, 2nd edition, Building & Housing Research Center, Tehran, Iran, 1999.
10. Tso, W.K., T.J. Zhu, and A.C. Heidebrecht, 1992. Engineering implication of ground motion A/V ratio. Soil Dynamics and Earthquake Engineering, 11(3): 133-144.
11. Wilson, E.L., and A. Habibullah, 1999. SAP2000 User's Manual. Computers & Structures, Inc., Berkeley, California.
12. IASS. Recommendations for guyed masts. Madrid, Spain: International Association for Shell and Spatial Structures; Working Group No. 4, 1981.