UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
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STRUCTURAL BEHAVIOUR OF STEEL LATTICE TELECOMMUNICATION TOWERS UNDER THE SEISMIC ACTION WITH OTHER ACTIONS

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IN
CIVIL ENGINEERING**

ŞADİYE DEFNE PARTAL

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Structural Behaviour of Steel Lattice Telecommunication Towers Under the Seism[illegible]ion with Other Actions

M.Sc. Thesis
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Supervisor
Assist. Prof. Dr. Esra METE GÜNEYİSİ

Şadiye Defne PARTAL
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Prof. Dr. Ramazan KOÇ
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Mustafa GÜNAL
Head of Department

This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Assist. Prof. Dr. Esra METE GÜNEYİSİ
Supervisor

Examining Committee Members	Signature
Assoc. Prof. Dr. Mehmet GESOĞLU	
Assist. Prof. Dr. Esra METE GÜNEYİSİ	
Assist. Prof. Dr. Faruk GEYİK	

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all materials and results that are not original in this work.

Şadiye Defne PARTAL

ABSTRACT

STRUCTURAL BEHAVIOUR OF STEEL LATTICE TELECOMMUNICATION TOWERS UNDER THE SEISMIC ACTION WITH OTHER ACTIONS

PARTAL, Şadiye Defne
M.Sc. in Civil Engineering
Supervisor: Assist. Prof. Dr. Esra METE GÜNEYİSİ
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The service without interrupted is a crucial design criterion for utilities exposed to natural hazards. As one of the utilities, telecommunication lines play a considerable role in the operation of a reliable telecommunication system. Recent earthquakes reveal that telecommunication towers are fundamental components since the impacts of shaking to numerous telecommunication tower structures led to delays in the systems of national communication all through the most significant rescue and recovery period, specifically the hours immediately following the earthquake. Thus, in order to minimize the disruption to telecommunication systems, the reliability and the safety of these towers against natural forces should be assessed. In this study, actually built 4-legged steel telecommunication towers having two different heights of 40 m and 80 m were selected in order to assess its structural response under the effect of earthquake. All computer simulations were conducted using a finite element modeling software of SAP 2000. The structural analysis considered as acting vertical loads as well as wind effects over the steel towers. Moreover, four different natural ground motion records, namely Chi-Chi 1999, Kocaeli 1999, Landers 1992, and Hector Mine 1999 were utilized to identify the dynamic behavior of the tower under seismic excitations. As a result, the structural response of the telecommunication tower and its behavior under seismic and wind forces were discussed comparatively.

Keywords: Earthquake, Finite element modeling, Seismic response, Steel tower, Structural analysis

ÖZET

ÇELİK TELEKOMİNÜKASYON KULELERİNİN DEPREM YÜKLERİ VE DİĞER ETKİLER ALTINDA YAPISAL DAVRANIŞLARININ ARAŞTIRILMASI

PARTAL, Şadiye Defne
Yüksek Lisans Tezi, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Yrd. Doç. Dr. Esra METE GÜNEYİSİ
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Doğal afetlerde kesintisiz servis sağlanması kamu hizmetleri için temel bir kriterdir. Kamu hizmetlerinden olan telekomünikasyon ağları, güvenli iletişim sistemlerinin işleyişinin sağlanmasında önemli rol oynamaktadır. Geçmiş yıllardaki depremlerde birçok çelik telekomünikasyon kulesinde deprem sarsıntısının, özellikle depremden hemen sonra hayati önem taşıyan kurtarma ve iyileştirme sürecinde iletişimin sağlanmasında gecikmelere sebep olduğu görülmüş ve telekomünikasyon kulelerinin iletişim ağının önemli unsurları olduğu ortaya çıkmıştır. Dolayısıyla, iletişim sistemindeki deprem etkisiyle oluşabilecek aksaklıkların en aza indirilebilmesi için, iletişim kulelerinin deprem etkilerine karşı da güvenilirliliğinin değerlendirilmesi gereklidir. Bu çalışmada, 40 m ve 80 m yüksekliği olan mevcut iki çelik telekomünikasyon kulesinin, rüzgar yükü ve sismik etkiler altındaki yapısal davranışı incelenmiştir. Kulenin yapısal olarak modellenmesi ve analizleri sonlu eleman modelini kullanan SAP 2000 programı ile yapılmıştır. Kulenin analizlerinde hem düşey yükler hem de kule üzerindeki rüzgar yükleri dikkate alınmıştır. Diğer taraftan, kulenin sismik hareketler altındaki davranışlarını belirlemek için dört farklı doğal deprem ivme kaydından yararlanılmıştır. Chi-Chi 1999, Kocaeli 1999, Landers 1992 ve Hector Mine 1999 deprem kayıtları kullanılmıştır. Sonuç olarak, bu çalışmada örnek olarak seçilen kulelerin sismik yükler ve rüzgar yükleri altındaki davranışları yapısal tepkisi de göz önüne alınarak karşılaştırmalı olarak incelenmiştir.

Anahtar kelimeler: Deprem, Sonlu elemanlar yöntemiyle modelleme, Deprem performansı, Çelik Kule, Yapısal Analiz

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LIST OF SYMBOLS/ABBREVIATIONS

A_n	Pseudo-acceleration spectrum ordinate $A(T_n, \zeta_n)$
$A_n(t)$	Pseudo-acceleration of n-th mode SDF system
C	Damping matrix
CQC	Complete quadratic combination
ρ_{in}	The result of the peak responses in the i-th and the n-th modes and the coefficient of the correlation
K	Stiffness
M	Diagonal mass
MDOF	Multi-degree-of-freedom
PGA	Peak ground acceleration
PGV	Peak ground velocity
r	Earthquake accelereation record
$r(t)$	Any response quantity
$r_n(t)$	n-th mode constribution
r_o	The peak value of the total responses
r_{no}	Peak modal response
RC	Reinforced concrete
RSA	Response spectrum analysis
SRSS	Square root of sum squares
THA	Time history analysis
u	Displacements of the structures
$\dot{u}$	Velocities of the structures
$\ddot{u}$	Acceleration of the structure

CHAPTER 1

INTRODUCTION

Necessity of uninterrupted communication is so effective for human life. In the current century, this field has become significantly important and has been named communication era. Telecommunication towers have essential role in this industry. They support radio, television, and telephone antennas to transmit telecommunication signals over long distances. In the case of emergency, these towers play an important role for transmitting news from damaged area to the rescue centers (medical services, fire fighting, and police stations). Therefore, damage to them can significantly increase loses due to natural disasters. Also, infrastructures such as dams, electricity power stations, gas and fuel stations, etc. for their operation need these towers for transmitting their data and these towers are very important for such facilities. Therefore, the protection of these towers during natural disasters is of major importance and accordingly the performance of such structures under these loadings should be properly evaluated (Amiri and Boostan, 2002).

Among several types of the towers, the steel lattice towers are widely used in the telecommunication industry. They are commonly categorized into two groups of the 3-legged and the 4-legged lattice towers. Most researches to date have been performed on the 3-legged self-supporting towers and very limited attention has been paid to the dynamic behavior of the 4-legged self-supporting telecommunication towers (Amiri et al., 2004; Amiri et al., 2007). The demand to make a design of a lattice tower for resonant dynamic response because of the wind force arises if the fundamental frequency of the structure is low adequate to be stimulated by the turbulence in the natural wind (Harikrishna et al., 1999). Moreover, such towers are frequently designed considering the influences of the wind as only source of lateral forces with no attention specified to the earthquake (Khedr and McClure, 2000).

seismically induced member loads might go beyond loads derived from the service and wind force computations. Amiri and Booston (2002) studied the dynamic response of antenna-supporting structures. In this regard, self-supporting steel telecommunication towers with different heights were evaluated considering the wind and earthquake loads. A comparison is made between the results of wind and earthquake loading. These comparisons resulted in the necessity of considering earthquake loads in tower analysis and design. In the study of Amiri et al. (2007), the seismic response of 4-legged self-supporting telecommunication towers was examined. It is found 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, they proposed a number of empirical relations that can help designers to approximate the dynamic response of towers under seismic loadings. In the literature, it is also suggested that for tower structures placed in high risk earthquake regions, the influence of earthquake on towers might not be negligible in comparison to the effects of wind and should be taken into account as a design verification at least in a simplified manner (Khedr and McClure, 2000).

1.1. Objectives of the Thesis

The aim of this thesis is to further examine the structural behavior of steel lattice telecommunication towers under the seismic action with other actions. For this purposes, actually built 4-legged steel telecommunication towers having two different heights were utilized so as to assess its structural response under the effect of seismic loading. In the structural analysis, vertical loads as well as wind effects over the steel towers were considered. All computer simulations were performed using a finite element modeling software of SAP 2000. Moreover, different earthquake ground motion records, namely Chi-Chi 1999, Kocaeli 1999, Landers 1992, and Hector Mine 1999 were employed to identify the behavior of the steel telecommunication towers under seismic excitations. The responses of the towers under seismic forces were determined by using both response spectrum analysis and time history analysis. The results of analyses carried out on the 40 m and 80 m towers are presented in terms of their mode shapes, axial forces occurred in the leg, horizontal, and diagonal members in accordance with different load combinations, and then critically discussed.

1.2. Outline of the Thesis

Chapter 1-Introduction: Aim and objectives of the thesis are introduced.

Chapter 2-Literature review and background: A literature survey based on this thesis is given. For this, firstly, studies on telecommunication towers as well as their classification in the literature are described. Secondly, the utilization of different towers in the structural system is given. Afterward, the properties and use of steel lattice systems in relation with the structural applications and available studies on this issue in the literature are summarized.

Chapter 3-Analytical study: This chapter provides a description of analytical models of the case study towers. Additionally, the methodology used in the analysis and design of the structures is summarized and details of every step are given in this chapter. Moreover, the properties of the ground motion records used in the analysis are described in this chapter.

Chapter 4-Discussion of the results: Results obtained from the analysis of the tower structures with different load combinations are presented. Discussion on the results of the analysis is given in this chapter.

Chapter 5-Conclusion: Conclusions based on the results of this study are summarized.

CHAPTER 2

LITERATURE REVIEW

2.1. General

The service without interrupted is a fundamental design criterion for utilities and chosen structures exposed to man made or natural hazards. The 1995 Kobe (Japan) earthquake and the 1999 Chi-Chi (Taiwan) seismic activity emphasized the criticality of a quick restoration of failed telecommunication systems to both the recovery attempts and the normalization of business and civic life (Schiff, 1998; ASCE-TCLEE, 1999). During those seismic activities, the impacts of shaking to numerous telecommunication towers brought about the delays in public communication systems throughout the most important rescue and recovery period, namely the hours immediately following the earthquakes. Consequently, the significance of telecommunication towers, due to their rescue and recovery functions, needs not simply post-incident survival but minimal overall failure, without interrupting the service (Kalkan and Laefer, 2004).

Actually, the design provisions regarding the self-supporting tower for rehabilitating the available towers subjected to seismic events are not yet addressed in current industry regulations (TIA/EIA 222-F, 1996), even though simplified methods for the earthquake design of new poles are given in UBC (1997) and IBC (2000). Supplementary provisions are required to concentrate on the increasing prevalence and geometric complication of the towers in earthquake areas.

There is even more pressure to get greater capacity stages from existing tower structures by including new antennas and reflectors with added carries. Attempts to raise the forces of existing structures have further complicated efforts at earthquake retrofitting due to the additional and undesirable loads. More accurate knowledge of tower structure behavior under seismic effects is vital for suitable code development.

More analyses are also needed to appraise the potential of different rehabilitating solutions (Kalkan and Laefer, 2004).

2.2. Telecommunication Towers

The towers used for the telecommunication system are among the few steel structures which little attention is paid to their earthquake behavior in current design codes. Because of the lack of design codes, the structural engineers utilize the available building design provisions for the telecommunication towers. But, the earthquake behavior and performance of the telecommunication towers is considerably different in comparison to the building structures. For instance, the values of base shear achieved for such towers utilizing the building provisions are much greater than its actual values. (Amiri et al., 2007).

Communication industry has a unique situation in the history of human life. In the current century, this field has become significantly important and has been named communication era. Telecommunication towers have essential role in this industry. They support radio, television, and telephone antennas to transmit telecommunication signals over long distances. In the emergency situation, these towers play an important role for transmitting news from damaged area to the rescue centers (medical services, fire fighting and police stations). Therefore, damage to them can significantly increase loses due to natural disasters. Also, infrastructures such as dams, electricity power stations, gas and fuel stations, etc. for their operation need these masts for transmitting their data and these towers are very important for such facilities. Therefore, the protection of these masts during severe events is of major importance and accordingly the performance of such structures under these loadings should be properly evaluated (Amiri and Boostan, 2002).

2.3. Types of 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 (Amiri, 2002). 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. The monopoles are very economical (particularly for the heights less than 55 m) and provide a feasible solution for space limitation problems and rigid zoning provisions. Moreover, industry distinguishes the monopoles from the self-supporting towers with the latter being latticed (Amiri et al., 2007).

2.3.1. Guyed Towers

Guyed towers are typically tall structures. The functionality of these towers is explained as carrying elevated antennas for radio and television broadcasting, telecommunication, and two-way radio systems. Therefore, immediate serviceability or even continuous function of first-aid-station infrastructure is of significantly high priority in the case of a disaster. Figure 2.1 illustrates a 111-m mast owned by Hydro-Québec in St-Hyacinthe, Québec, Canada, as an example (Faridafshin et al., 2008).

Figure 2.1 Example of a 111.2 m guyed tower located in St. Hyacinthe, Québec, Canada (Faridafshin et al., 2008)

A schematic representation of the fundamental components of a guyed communication tower is given in Figure 2.2. As seen from the figure, this comprises: a) a series of guys and anchors, b) the central mast, c) the system of mast foundation, and d) environmental effects. The first three of these constituents are physical or functional parts which might be directly evaluated for the duration of inspections. Environmental force is a total system constituent believed to be composed of wind, ice, and seismic loadings. In Figure 2.3, the constituents of guy anchor system: a) guy anchor showing gusset plate and anchor rod extending into ground surface, b) feature of gusset plate, c) characteristic of tensioning system, and (d) property of cable-to-anchor connection are illustrated (Likos and Salim, 2005).

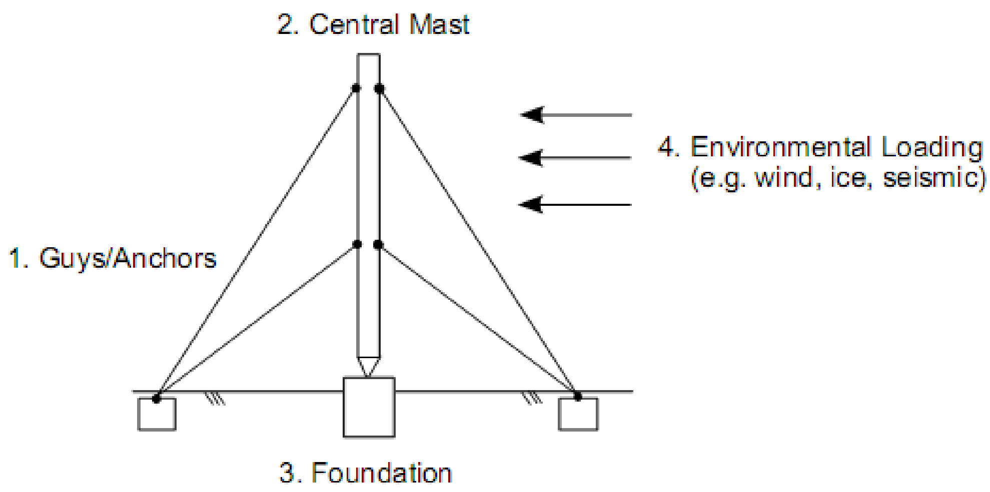


Figure 2.2 Four main parts of a guyed tower (Likos and Salim, 2005)

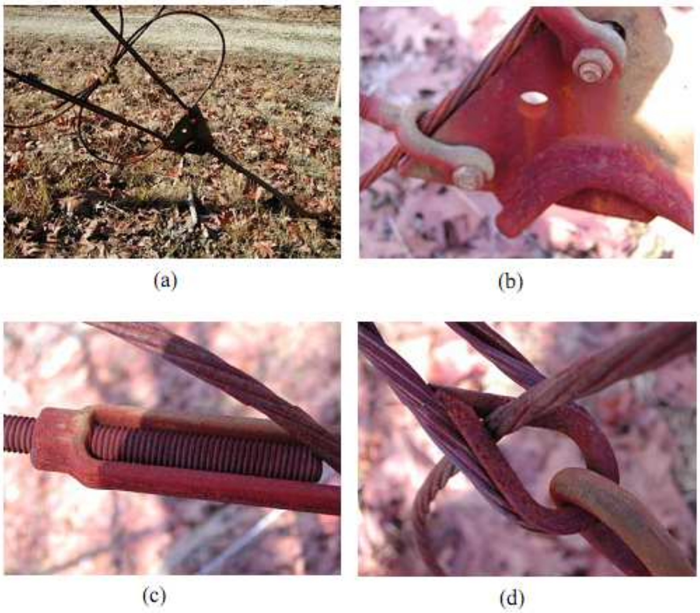


Figure 2.3 Components of guy anchor system (Likos and Salim, 2005)

Even under the conditions of working, a guyed telecommunication tower is considered as an illustration of a structure having a non-linear behavior. That non-linear behavior is because of the variation in the stiffness of the guy cable with the variation in the guy tension causing a non-linear load-displacement relation for the structure, and also by reason of the large deformations taking place in the structure even under normal design forces (Wahba et al., 1998).

The suggestions for guyed masts reported by International Association for Shell and Spatial Structures (IASS, 1981) provide two satisfactory models for the structural analysis of the towers. They are referred as a) an elastic beam-column to characterize the shaft, with non-linear elastic supports at the guy attachment points indicating guys, and b) a finite element demonstration of the guyed mast utilizing elastic beam-column members for the shaft and cable components for the guys. For more than 20 years, the first type of model is employed widely; that is very easy as answers could be achieved rapidly and the forecast of the structural response is satisfactory. On the other hand, the majority of the codes have a tendency to use limit states design. In this approach, the structural systems are analyzed considering the maximum factored forces (such as excessive loading situations or close to collapse). Moreover, when the telecommunication system is being more sophisticated, the demand for raised accuracy is being fundamental (Wahba et al., 1994).

In the study of Murty et al. (1998), the guyed masts were analyzed under the effect of dynamic loadings and their responses were investigated. In addition, the experimental and analytical results were evaluated comparatively. In order to determine the natural frequencies of guyed masts, two models were considered, in the first one, the mast was modeled as truss elements. In the second one, the beam-column elements were taken into account. In both of the cases, the cable elements were used in the model of guys. The natural frequencies obtained from these models provided almost similar results. To validate the finite element modeling, two triangular steel latticed scale-modeled guyed masts were constructed and tested on a shake table. The geometry of the model tower is shown in Figure 2.4. The selected towers had different properties. For example, they had dissimilar height, number of guy levels, wind and ice load condition. A total of six available masts indicating representative guyed masts in Canada were utilized for assessing the effect of icing, initial guy tension and torsion resistors on the natural frequencies of such structures. The height of the existing towers varied from 46 m to 122 m. According to the results of the analyses performed on the guyed masts, the effects of ice accretion, guy initial tensions, and torsion resistors on the dynamic features were argued. Consequently, it was pointed out that both experimental results and those obtained from the finite element analysis were close to each other. In addition, the height of the tower was found to be very effective parameter on its response, especially at the lowest natural frequency.

Hensley and Plaut (2007) studied the seismic response of guyed masts. In their research, the three-dimensional analysis was performed. The performance of a 120 m-tall guyed mast under seismic excitation was determined using the finite-element program of ABAQUS. Two seismic records were used, namely Northridge and amplified El Centro earthquakes. The mast was pinned at its base, and guy cables were attached at four equally spaced points along its height. Figure 2.5 demonstrates the views of guyed mast and direction of strong horizontal component. Nonlinear function derived from tests was utilized to model the dynamic tension in the guys.

Three-dimensional beam elements were employed for the mast. The following factors such as displacements, bending moments, and base shears were evaluated for the mast, along with guy tensions. The influences of guy stiffness and pre-tension, mast weight, and directionality of the earthquake acceleration were evaluated. Results indicated that it was essential to consider all three components of the earthquakes when analyzing the performance of a guyed mast. Moreover, It was imperative to comprise three-dimensional movements of the mast that might be reasonably complex during the response.

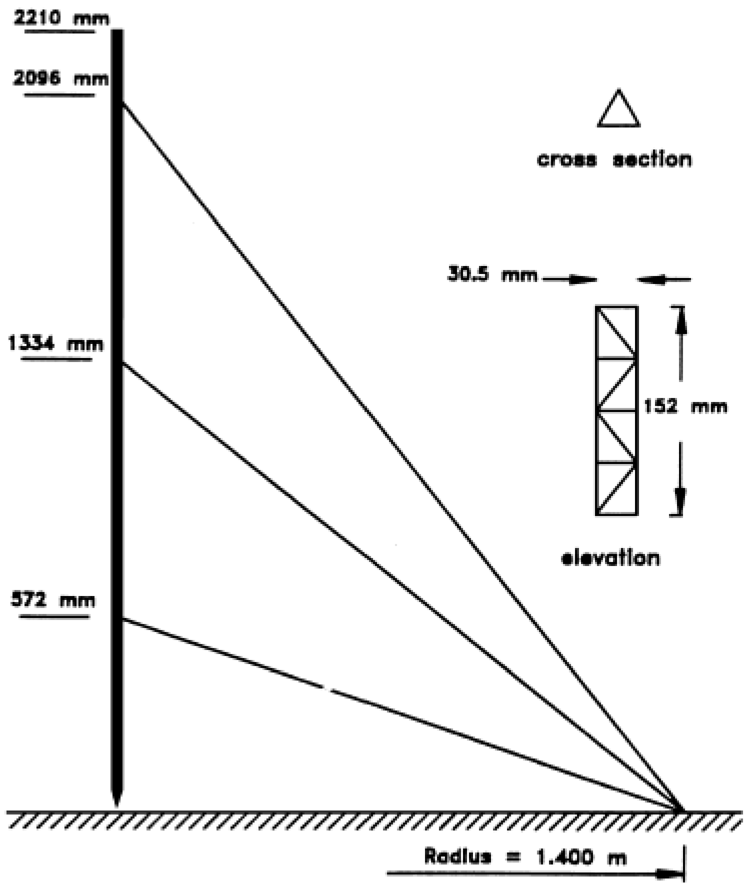


Figure 2.4 View of the experimental model guyed mast (Murty et al., 1998)

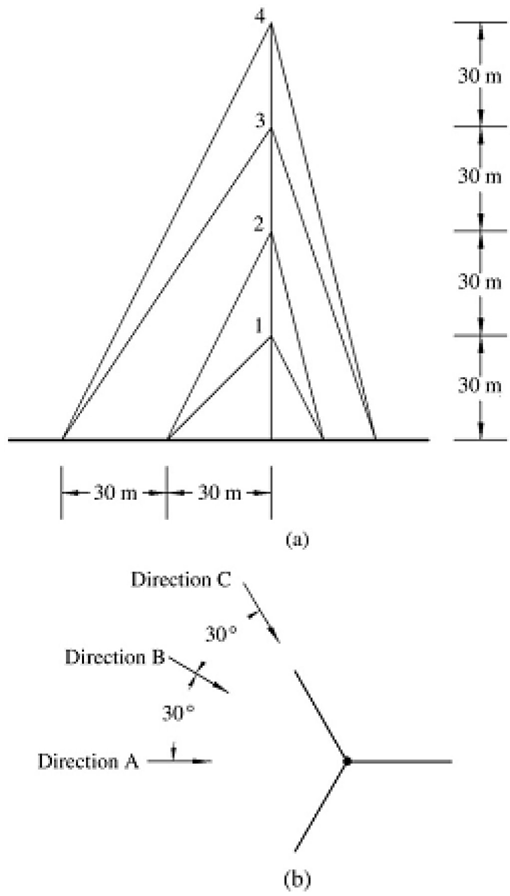


Figure 2.5 Details of guyed mast: (a) profile view of guyed mast and (b) plan view of direction of strong horizontal component (Hensley and Plaut, 2007)

Faridafshin et al. (2008) examined the performance of tall guyed telecommunication masts to seismic wave propagation. The main goal of this study was to demonstrate the importance of considering realistic three-dimensional ground motion with asynchronous input when evaluating the seismic response of these tall multi-support structures. The case study masts having different heights of 213, 313 and 607 m and various guy cable arrangements were modeled and investigated in detail with three classical historical earthquake records using a commercial finite element program (ADINA). The geometric characteristics of the three masts are summarized in Table 2.1 and a schematic of the 607 m mast is illustrated in Figure 2.6. Both synchronous and asynchronous ground shaking were considered. The effect of asynchronism in multiple support excitations was determined by using various shear wave velocity of the surface traveling wave associated with dissimilar levels of soil rigidity. The three towers were shown sensitivity to asynchronous shaking of their supports. It was also found that softer soil conditions resulted in more severe response. However, the mast with a height of 607 m seemed to be sensitive even for comparatively stiff soils.

Table 2.1 Geometry of the three masts studied (Faridafshin et al., 2008)

Height (m)	No. of stay levels	No. of anchor Groups	Panel width (m)	Panel Height (m)
607	9	3	3.00	2.25
313	5	2	2.14	1.52
213	7	2	1.52	1.52

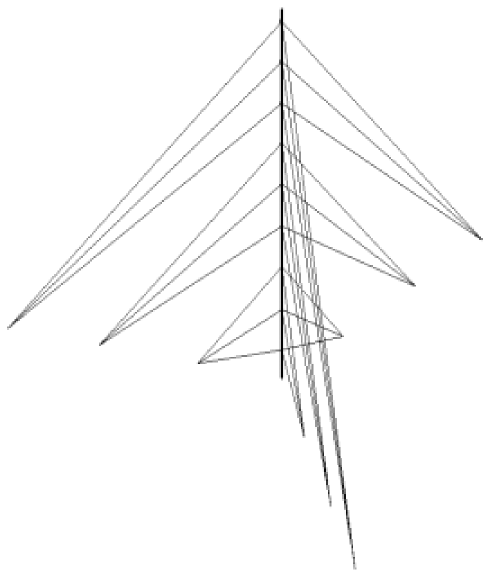


Figure 2.6 Geometry of the 607 m mast ((Faridafshin et al., 2008)

2.3.2. Monopole Towers

Monopole towers are typically utilized to sustain the required cellular apparatus, coaxial wires and antennas. Community resistance to building new towers is familiar, so reuse and rehabilitation of available towers is critical and often the only option to carry the supplementary services. Figure 2.7 shows the typical monopole supporting wireless services. The monopoles are characteristically constructed with high-strength steel and varied from 7 and 75 m in height. They encompass different cross-section such as a round or polygonal shape having shaft thicknesses changing from 15 to 120 mm. The towers are either directly implanted into the concrete foundation or connected to the concrete foundation with the anchor rods (Lanier et al., 2009).

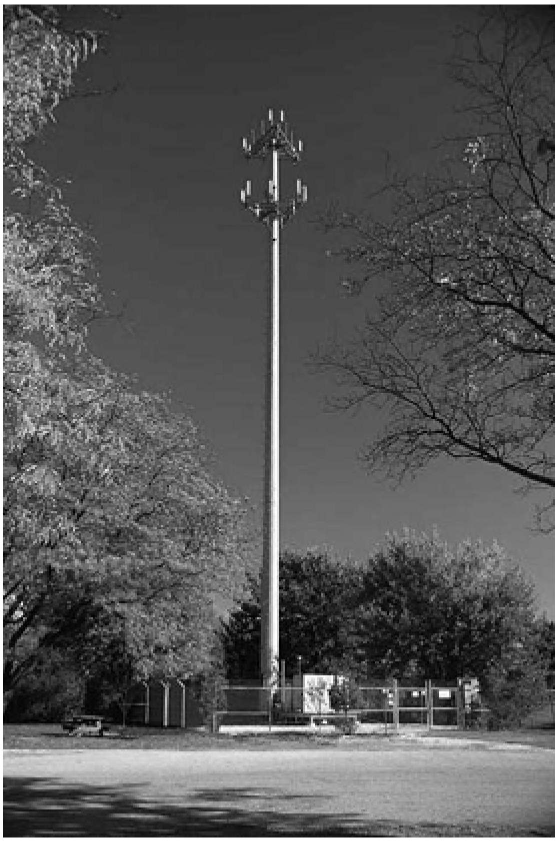


Figure 2.7 Typical monopole supporting wireless services (Lanier et al., 2009)

Considering a variety of combinations of dead, wind, seismic, and ice loads, the monopole towers are designed and constructed. It is stated that the most of the applied design load is due to wind and seismic lateral forces. The wind loads typically include 80–90% of the structure design rule for this kind of towers while the dead loads usually make up smaller than 5% of the tower load. Bending resistance to the wind loads is generally the significant structural loading for the monopole towers because of their height and loading situation. The monopoles are designed in accordance with the guidelines of ANSI/TIA-222-F or TIA-222-G. These regulations use wind, ice and seismic criterion as stated in ASCE7-88 and ASCE7-02, respectively with revision F using an allowable stress steel design while revision G imposes a force and resistance factor steel design (Lanier et al., 2009).

Pagnini and Solari (2001) carried out a study on measuring the damping of the steel poles and tubular towers. In their study, a total of four towers were examined by supplying the logarithmic damping decrement as a function of motion amplitude for each of them. A slender cantilever tubular shaft having a linear elastic behavior was used in modeling the column (Figure 2.8). Its vertical axis matched up with z-axis. The foundation of the towers is fixed and placed on a viscoelastic soil. The findings obtained from this investigation were related to the formulations of appropriate damping models and analyzing the importance of this factor in the wind-excited response of the towers. It was pointed out that the dissipation of energy noticeably augmented through the motion amplitude in the studied structures.

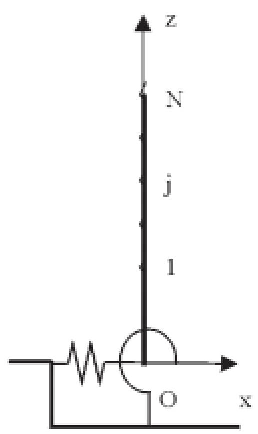


Figure 2.8 Structural model (Pagnini and Solari, 2001)

Kalkan and Laefer (2004) investigated the retrofitting of steel and reinforced concrete (RC) telecommunication poles by means of a finite element model analysis. For this purpose, two reinforced concrete poles with 38 and 50 m in height and two steel poles with 33 and 53 m in height were assessed. Mast flexibility, changeable damping on dynamic response and significance of period on base shear amplification were the variable of the study. The efficiency of strengthening against the base excitation was determined for reinforced concrete and steel poles by the function of modal and response spectrum analyses. A group of strong ground motion records during the 1994 Northridge earthquake was used. Figure 2.9 and Table 2.2 presents the view and geometry features of RC and steel poles, respectively. The commercial finite element modeling software of SAP2000 was employed in the analysis of the towers. Four node quadrilateral shell elements were used in simulating the steel sleeve sections and existing poles. The elements consisted of the separate membrane and plate bending behaviors. It was founded that the steel jacketing was very effective in enhancing load carrying capacity of the poles as a result of enabling stress redistribution.

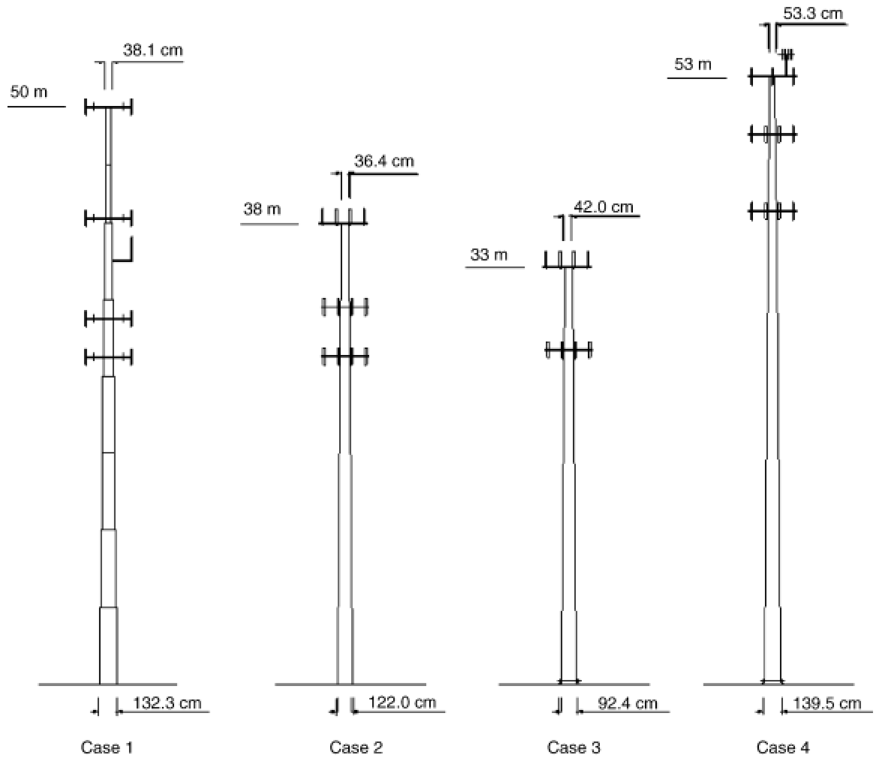


Figure 2.9 Views of RC and steel monopoles (Kalkan and Laefer, 2004)

Table 2.2 Properties of RC and steel monopoles (Kalkan and Laefer, 2004)

Type of poles	RC		Steel	
	Case-1	Case-2	Case-3	Case-4
Pole configuration	Tapered & Stepped	Tapered	Tapered	Tapered
Number of platforms	4	3	2	3
Weight of the poles (kN)	275	190	38	160
Section thickness (bottom) (cm)	10.2	10.2	0.8	1.3
Section thickness (top) (cm)	10.2	10.2	0.8	1.3

Hu (2011) reviewed and discussed the crucial matters on the design of cover plate of beams and stringers, and addressed the cover plate reinforcement design for the monopole. Based on the analytical examination, the design suggestions were forwarded and discussed in detail. Figure 2.10 shows the most common reinforcing schemes for monopole reinforcement. As seen from the figure, different methods such as rebar, cover plate, WT, and channel were utilized. It was noted that the channel could flip inside out and the locations on the surface could be adjusted. The arrangement of reinforcing members was supported to provide the symmetry. The total number of reinforcing member could be three, four, six, eight, and nine or even more, depending mainly up on the side number of the monopole. The response of the monopoles was examined considering the structural and loading characteristics and the improvement due to the cover plate. It was reported that the design rule for the cover plate reinforcement of the monopole was different because of the distinctive structural features and loading characteristics. In comparison to beam and stringer, the monopole possessed distinctive structural and loading performances. The greatest loading and overstress frequently happened at the bottom end section of the monopole while the highest loading and overstress took placed at the middle span for the beams and stringers. It was found that the cover plate design for the monopole required various unusual details. It was recommended that the grade of the cover plate steel might be no less than the monopole shaft as well as the thickness of cover plate could not be higher than two times the thickness of the shaft of the monopole. It was also noted that narrower cover plate could be favored though wider cover plate permitted.

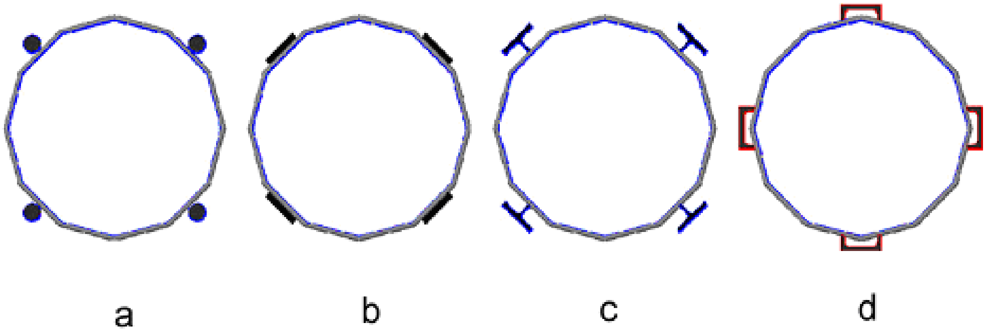


Figure 2.10 Details of the strengthening schemes for the monopole with a) rebar, b) plate, c) WT, and d) channel (Hu, 2011)

2.3.3. Self-Supporting Towers

Lattice steel towers are utilized for different purposes, specifically, telecommunications, radio and television broadcasting, observation, power transmission and lighting supports, etc. (Harikrishna et al. 1999). The major components are legs, braces, and attached antennas. The bracing has various patterns as cross bracing, portal bracing, cranked K type bracing (TIA/EIA-222-G.5, 2006). Figure 2.11 illustrates the views of a self supporting tower with K-type bracing while Figure 2.12 shows representative bolted connections between secondary horizontal or bracing elements with the main elements.

Self-supporting towers are categorized into two groups of 4-legged and 3-legged lattice 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 (Amiri et al., 2004; Amiri et al., 2007). The demand for designing a lattice tower against resonant dynamic response caused by the wind force increases as the fundamental frequency (natural vibration frequency) of the tower is low sufficient for exciting by the turbulence in the natural wind (Harikrishna et al., 1999). Moreover, such structures are frequently designed with the influence of the wind since the sole source of lateral loads is thought about such effect, and therefore there is no adequate attention given to earthquake (Khedr and McClure, 2000). However, in the recent work, the importance of seismic load has been emphasized. For instance, Lefort (1998) studied the effects of wind and earthquake loads on the self-supporting antenna towers and it was pointed out that for the towers, the force of the members under the effects of seismic loads might go beyond that of the members derived from service and wind force computations.

Figure 2.11 Typical view of a self supporting steel lattice telecommunication tower (Shi, 2007)

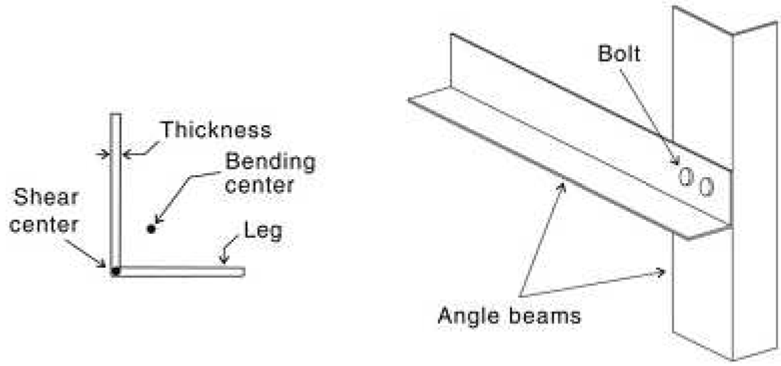


Figure 2.12 View of the angle beam and typical eccentric connection of angle beams utilized in lattice steel structures (Lee and McClure, 2007)

Chiu and Taoka (1973) performed 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.

One of the first researches on the influences of seismic forces on the lattice telecommunication towers was carried out by Konno and Kimura (1973). The purpose of their works was to accomplish the shape of mode, the natural frequency, and the damping characteristics of such towers. Analysis of the results showed that in a number of the elements of the structures, the loads because of the seismic activity were higher than those of the wind.

Mikus (1994) looked into the seismic behavior of 3-legged self-supporting telecommunication towers. A total of six towers were investigated, and their heights were in the range of 20 to 90 meters. The chosen structures were numerically simulated as bare towers. In the analysis, three accelerograms were considered as the real earthquake forces. It was found that the smallest four modes of vibration could determine the adequate accuracy. In addition, also, it was pointed out that the vertical component of earthquake-induced forces had no effects on the results.

In the studies of Galvez (1995) and Galvez and McClure (1995), they investigated three different numerical models of 3-legged lattice steel towers. They had various heights from 90 to 121 meters and 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 method was the bilinear shape of the acceleration profile, which did not thoroughly include the towers with different geometries.

Glanville and Kwok (1997) investigated the deflections of free-standing lattice towers induced by the wind. Full scale deflection measurements were conducted on a 67 m 3-legged steel frame tower (as Prospect Tower) and a 233 m 3-legged steel truss tower (as TCN9 tower) under the wind force. In the frequency domain analysis, STRAND6 finite element software was utilized. Various ways were used for the measurements and evaluated with along-wind theoretical predicts. The latter were acquired utilizing a simplified frequency domain forecast which includes several practical examinations and comments. In addition to this, in their study, the dynamic cross wind properties of lattice structures was monitored and conferred. The natural frequency was derived from a power spectral analysis of the wind caused tower acceleration. Structural damping of the tower by the side of the east and west axis was predicted approximately as 0.7% of critical damping. Structural damping towards the north and south axis was calculated approximately as 1% of critical damping. Measurements on the TCN9 tower yielded the resonant component of displacement. It was observed that the cross wind background displacement was found to be just about half the along wind background displacement of the Prospect Tower. Both of the resonant displacement components were roughly equivalent for the Prospect and TCN9 towers.

In the work of Khedr and McClure (1999), the seismic amplification parameters for the base shear and the total vertical reaction of self-supporting 3-legged lattice telecommunication towers were proposed derived from the modal superposition analysis. This analysis was carried out on 10 existing structures that were exposed to a group of strong ground motions. The horizontal and vertical directions were taken into consideration individually. Analysis of the result was forwarded particularly for two telecommunication towers studied. They were 66 and 121 m in height. Figure 2.13 demonstrates the geometric views of the towers studied (TC3 and TC10). The modal superposition technique for the examination was realized by using the commercially available software of SAP 90. The towers were modeled as three dimensional lattice structures. Statistical regression analysis was adopted on the data and then the base shear and vertical reaction amplification parameters were achieved from the results of this analysis. Those amplification parameters were represented based mainly on the highest flexural period or highest axial period of vibration of the towers. It was also reported that the findings from the study could be utilized by practitioners with the purpose of predicting the probable stage of dynamic loads occurred in lattice telecommunication structures because of a seismic activity.

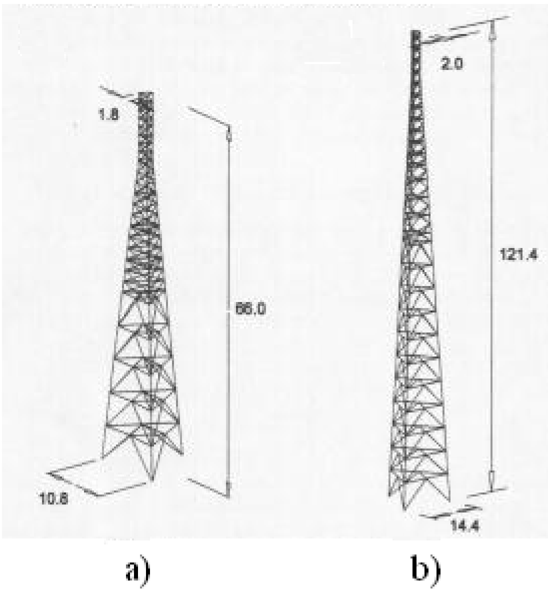


Figure 2.13 Sketch of the 3-legged towers utilized in the case studies: a) TC3 and b) TC10 (Khedr and McClure, 1999)

Khedr and McClure (2000) also proposed a simplified static procedure for forecasting the member loads in self-supported 3-legged lattice telecommunication towers under the effect of the earthquake. The horizontal and vertical components of seismic excitations were considered. In the proposed static procedure, the model superposition and response spectrum methods were utilized. It was supposed that the smallest three flexure modes of vibration were adequate for predicting the response of the structure to the horizontal excitation. However, only the smallest axial mode was enough to capture the response to the vertical excitation. By means of the prediction of the smallest three flexural modes or the smallest axial mode as well as the spectral acceleration associated with the natural periods, an acceleration profile over the tower's elevation was described in their study. Subsequently, the structure was examined statically considering the influence of these forces to appraise the member loads. The layout of example telecommunication towers is presented in Figure 2.14 while the characteristics of the towers are given in Table 2.3. That method was developed based on more detailed dynamic evaluation of 3-legged self-supporting telecommunication towers. A total of ten towers having 30-120 m in heights were analyzed. The results derived from two structures with heights of 66 and 83 m were expressed to make obvious the accuracy and practicality of the suggested procedure.

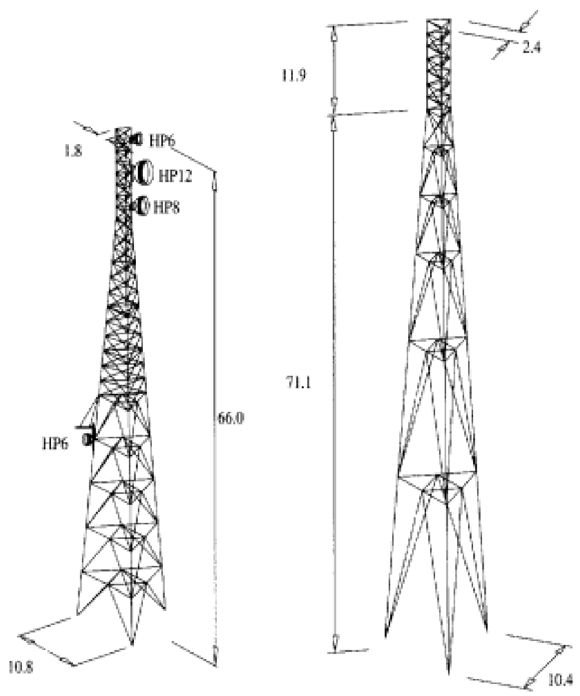


Figure 2.14 Layout of 3-legged steel lattice telecommunication towers (Khedr and McClure, 2000)

Table 2.3 Features of the telecommunication towers under the investigation (Khedr and McClure, 2000)

Towers	I	II
Height (m)	66	83
Base width (m)	10.8	10.4
Top width (m)	1.8	2.4
Total mass (kg)	27000	27000
Flexural period 1 (s)	0.54	0.69
Flexural period 2 (s)	0.21	0.37
Flexural period 3 (s)	0.11	0.23
Fundamental axial period (s)	0.071	0.073

McClure et al. (2004) studied the performance of telecommunication towers placed on the rooftops of the building. In their study, time history analyses were used to search the relationship between the building accelerations and the maximum earthquake base shear together with the base overturning moment of towers placed on the rooftops of the building. The models included two medium-rise buildings combined with two self-supporting 3 legged lattice steel towers subjected to 45 horizontal accelerograms with varied frequency content. Figure 2.15 shows the combined model for University building with tower GSM30 and GSM40 while Figure 2.16 shows the lowest frequencies and corresponding mode shapes of the combined model. The towers were assumed mounted at the position of the elevator shaft on the roof of the buildings. This position minimized torsional effects on the building as a whole and on the top portion, in particular. As for the interface roof-tower in the model, a stiff, triangular plate was pinned to the three main tower legs, and the centroidal joint was merged into the core joint. The principal directions of the towers are coincident with the main axes (X-Y) of the buildings. The tower base shear results were compared with the predictions based on a simplified formula proposed in building codes for secondary structures.

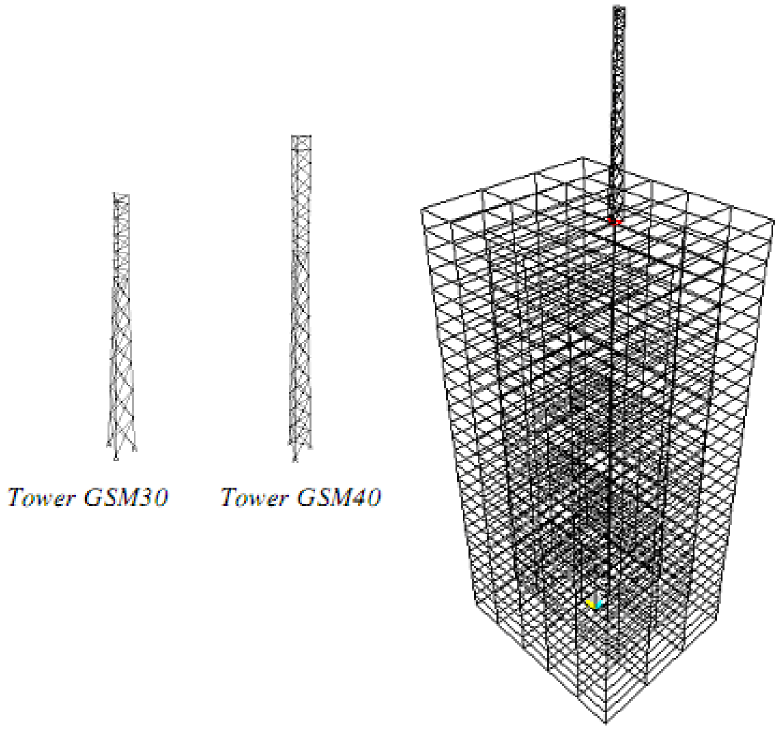


Figure 2.15 Building with 3-legged lattice towers GSM30 and GSM40 (McClure et al., 2004)

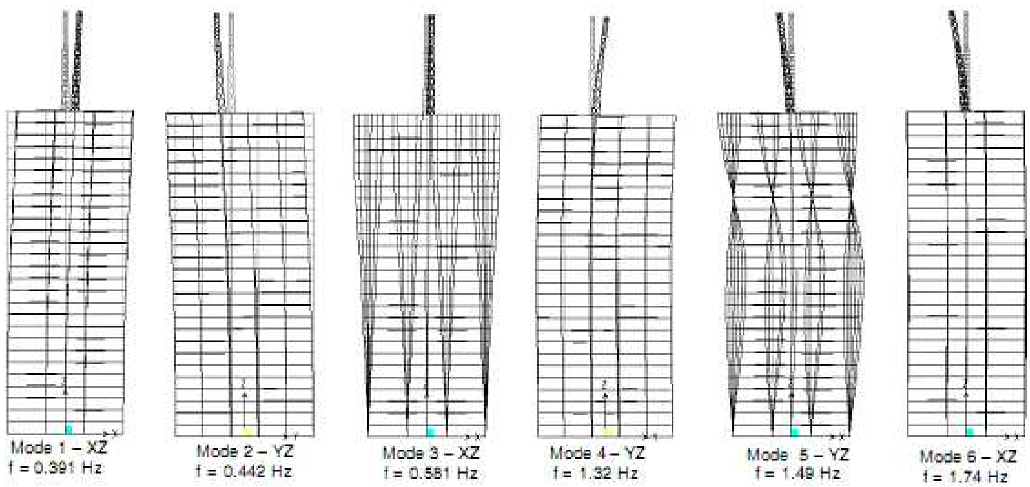


Figure 2.16 Mode shapes of the combined model (McClure et al., 2004)

In the work of Venkateswarlu et al. (1994), the response of steel lattice microware towers subjected to random wind loadings was investigated numerically. The dynamic response was predicted by the use of a stochastic approach. A spectral analysis method for determining the response to wind and the corresponding gust response parameters were established. The gust response factor was described by dividing the expected maximum wind force influence in a specific time period to the corresponding average value at similar time period. A 4-legged 101-m self-supporting tower was taken into consideration in their study. The gust response parameter along the elevation of the tower was evaluated considering the contributions of second and greater modes of vibration. The analysis of results indicated that there was a maximum of 2 % change in the gust factor when employing higher modes of vibration.

Gusella and Materazzi (2000) carried out a study on the wind response analysis of telecommunication tower structures. Presuming turbulence as a homogeneous Gaussian field, the dynamic performance of the tower structures was determined by utilizing the power spectral density of the wind load in the case of the standard Gaussian analysis and by utilizing both the power spectral density and bi-spectrum in the case of non-Gaussian analysis. The statistical evaluation on the basis of the response was employed to get the distribution of the peak value. A comprehensive study was performed assuming the change in the relative damping of all modes ranging from 0.01 to 0.11 with a step of 0.02. The analysis was conducted on a 4-legged steel lattice broadcasting antenna tower. For comparison, the dynamic analysis in time domain was also made to appraise the reliability of the spectral techniques. The tower studied (see Figure 2.17) had a height of 90 m. The lower part of the structure displayed a pyramid shape and its cross section was a square. The length of the section linearly varied from 15 m at the bottom to 4 m at the height of 65 m. Moreover, its upper part was prismatic. In the study, the first 10 mode shapes and frequencies were obtained from the finite element dynamic analysis. Then, a condensed cantilever model based mainly on five nodes was adopted to execute the numerical analyses. The properties of the towers were described by using the first three natural modes. Based on the results of the frequency domain analysis, the non-Gaussian approach adapted as well as the performance of the towers was discussed comparatively.

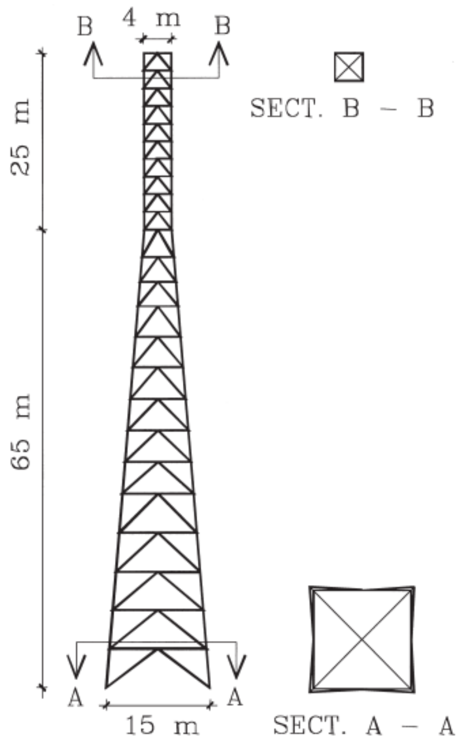


Figure 2.17 Schematic view of tower (Gusella and Materazzi, 2000)

In the research of Amiri et al. (2004), the dynamic behavior of 4-legged self-supporting telecommunication towers was studied. For this purpose, the existing 4-legged self-supporting telecommunication towers in Iran were studied under the effects of the earthquakes. Detailed three-dimensional full-scale numerical simulations using the finite element method were carried out. The tower models were very detailed to include geometric nonlinearities and to allow for potential dynamic response of the towers. The towers member cross-sections were made-up of single equal-legged angles. The connection type mainly used was bolts and nuts, and in case of excessive usage of bolts, steel plates were utilized as interface members. Foundations were assumed perfectly rigid. A lumped mass formulation was used for the mast members. It was reported that the height limitation of 150-m was a common criterion to classify towers with respect to their heights. With this regard, the available data for the self-supporting towers less than 150-m were taken for the numerical simulations. As part of some of the results, it was observed that the first three flexural modes were satisfactory for the dynamic analysis of such structures, even though in the case of taller towers, considering the first five modes would enhance the analysis precision.

Amiri et al. (2007) also investigated the seismic amplification parameters for self-supporting 4-legged telecommunication towers. For this purpose, ten existing towers in Iran were studied and their heights were varied between 18 and 67 meters. The properties of the structures are given in Table 2.4. The three-dimensional numerical modeling of towers was conducted by using the commercial software of SAP2000. Accelerations with strong motion characteristics were utilized considering vertical and horizontal directions. The base shear and vertical performance of the tower structures were achieved from linear dynamic analysis. The ratio of the obtained base shear or the vertical response to the product of the tower mass and maximum horizontal and/or vertical acceleration components would yield earthquake amplification factors in terms of both horizontal and vertical seismic components. Subsequently, by sketching the amplification parameters against the fundamental flexural mode and the first axial mode of the tower structures, relationships for predicting the base shear and the vertical performance of such structures were acquired. Analysis of the results in terms of natural periods concerning each first three flexural modes is illustrated in Figure 2.18 while that of the correlation of earthquake amplification factors and natural fundamental periods of towers is shown in Figure 2.19.

Table 2.4 Geometry of the selected towers (Amiri et al., 2007)

L_{total} (m)	Width at top (m)	Width at base (m)	L_{upper} (m)
18.0	2.0	3.40	10.0
22.0	2.0	3.90	14.0
25.0	2.0	4.30	17.0
30.0	2.0	5.00	22.0
35.0	2.0	5.70	27.0
42.0	2.0	6.70	34.0
48.0	2.0	7.60	40.0
54.0	2.0	8.40	46.0
60.0	2.0	9.30	52.0
67.0	2.0	10.30	59.0

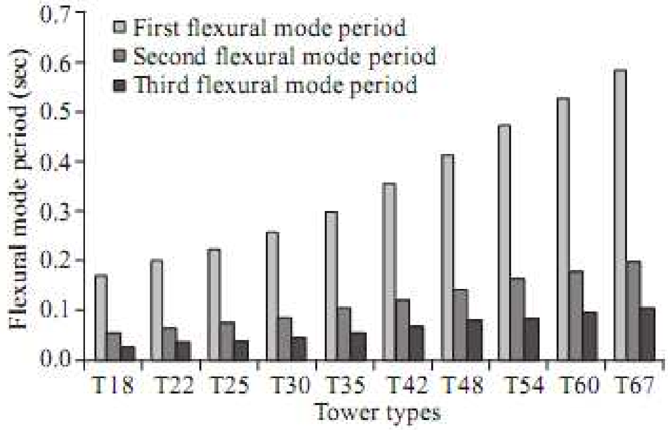


Figure 2.18 Natural periods of towers (Amiri et al., 2007)

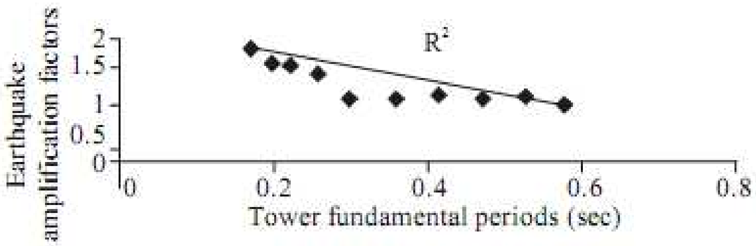


Figure 2.19 Earthquake amplification factors vs. natural periods (Amiri et al., 2007)

CHAPTER 3

ANALYTICAL STUDY

3.1. Description of Steel Lattice Towers

Free-standing steel lattice towers are three-legged or four-legged space trussed structures with usual heights between 30 m and 160 m. In this study, two actual free standing four-legged steel lattice towers with square transversal cross-sections were selected as case study towers. The selected towers have heights of 40 m and 80 m. The sample towers and panels over the elevation of the towers are shown in Figures 3.1-3.4.

The structural members used in the towers are single equal leg angles, and the steels are S235 JR with a tensile yield strength of 235 MPa and S355 JR with a tensile yield strength of 355 MPa. The modulus of elasticity and the unit weight of the steel materials used were 210 GPa and 7850 kg/m^3, respectively. For the members, S235 JR and S355 JR, which are rolling steel to using the area of the structure and construction, are utilized.

The sample steel lattice towers of 40 m and 80 m possess a square cross section divided into two segments: the lower part is pyramidal while the upper part is trapezoidal, as shown in Figures 3.1 and 3.3, respectively. It is noted that the 40m tower has 728 frames and 284 connection points while the 80 m tower possesses 1116 frames and 566 connection points.

At the end of the analysis, with the purpose of better present the results, the tower of 40 m was divided into 7 panels whereas that of 80 m was divided into 11 panels. General view of the towers and all diagonal, horizontal, and leg members particularly grouped according to these panels are shown in Figures 3.2 and 3.4 for the 40 m and 80 m tower, respectively.

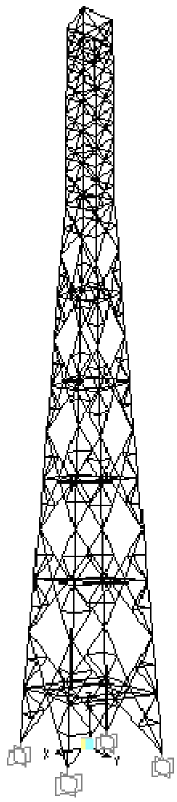


Figure 3.1 General view of the steel lattice telecommunication tower (40 m)

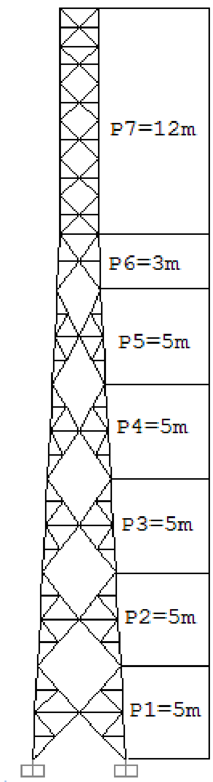


Figure 3.2 Panel view of the steel lattice telecommunication tower (40 m)

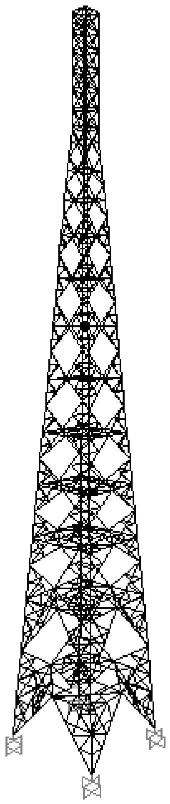


Figure 3.3 General view of the steel lattice telecommunication tower (80 m)

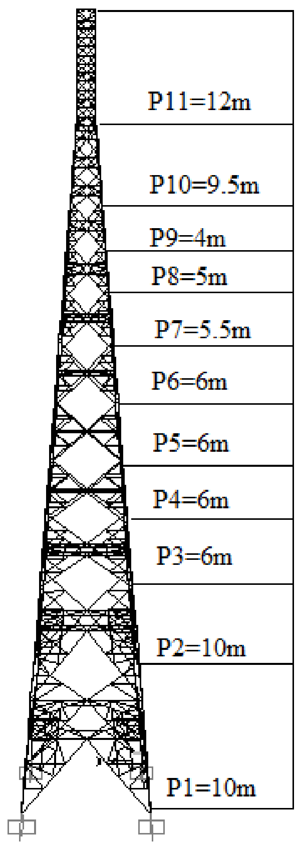


Figure 3.4 Panel view of the steel lattice telecommunication tower (80 m)

3.2. Modeling of Structures

The finite element model of the selected lattice towers were carried out by means of SAP 2000 Nonlinear version 14 (SAP 2000, 2009) that is a general purpose structural analysis program. The study taken into consideration as acting vertical forces such as self weight of the tower, stairs, internal platforms, vertical carriers, cables, etc. 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 (Amiri et al., 2007).

In addition to these vertical loads, the wind forces over the telecommunication tower were the first type of horizontal load thought about in the structural analysis of the tower. For determination of the wind forces and wind forces with ice, ANSI/TIA 222-G was utilized (ANSI/TIA 222-G, 2005). In the calculations of wind forces, the base wind velocity of 110 km/h was considered. Additionally, for the calculation of wind loads acting in conjunction with ice, an ice thickness equivalent to 1 mm was used.

3.3. Seismic Analysis

Static procedures are appropriate for short, regular buildings. In these buildings generally the greater mode effects are not considerable. However, for tall structures, those with torsional irregularities, or non-orthogonal systems, a dynamic procedure is needed.

In the linear dynamic method, the structure is modeled as a multi-degree-of-freedom (MDOF) system considering a linear elastic stiffness matrix and an equal viscous damping matrix. There are two types of linear dynamic analysis: a) response spectrum analysis and b) analysis in time domain. Therefore, in this study, the responses of the towers under seismic forces were determined by using the response spectrum analysis as well as time history analysis methods.

3.3.1. Response Spectrum Analysis

Response spectrum analysis (RSA) is a technique for dynamic analysis of structures exposed to seismic activity; however, it decreases the dynamic analysis to a series of static analysis.

The response spectrum procedure was firstly initiated in 1932 in the doctoral thesis of Maurice Anthony Biot at Caltech. This study presents a method to find earthquake response of structures utilizing waves or vibrational mode shapes. The numerical rules of oscillations in n-degree-of-freedom systems were obtained principally from the theories proposed by Rayleigh. Biot reported that a structure possess an assured number of so called normal modes of vibration which correspond a certain frequency. In the study of Biot, fourier amplitude spectrum was utilized to discover the highest amplitude of a system motion which calculates the sum of amplitudes for each separate mode of vibration (Trifunac and Todorovska, 2008).

However, prior to the digital processing age, the calculation of structural response took too much time and the outcomes were unreliable (Trifunac, 2003). By the late 1960s and early 1970s, the response spectra were performed as a result of the digitization of analog accelerograph records and the digital calculation of ground motion. Moreover, the modern era of the response spectrum procedure was started after the incidence of the San Fernando, California, seismic activity (in 1971). The California earthquake was monitored by various accelerographs, and by means of the combination of those data with all previous strong ground motion accelerations, the first detailed empirical scaling analyses of response spectral amplitudes were performed (Lee 2002, Trifunac and Todorovska, 2008).

In the response spectrum analysis, the peak modal response (r_{no}) of the n-th mode constribution $r_n(t)$ to response $r(t)$ can be obtained from the earthquake design spectrum. The peak value of $A_n(t)$ is available from the pseudo acceleration spectrum as its ordinate $A(T_n, \zeta_n)$ denoted as A_n. Thus, r_{no} the peak modal response which relates to the peak value of the $A_n(t)$ is calculated as:

$$r_{no} = r_n^{st} A_n \tag{3.1}$$

All response quantities $r_n(t)$ associated with the n-th mode reach their peak values at the same time instant as $A_n(t)$ reaches its peak value (Chopra, 2000).

With the intention of computing the peak value of the total response (r_o), the peak modal responses r_{no} were combined. There are some approaches in coming together the peak modal responses determined from the seismic response spectrum, since there is no information when these peak modal values take place. One of the approaches is the modal combination rule of absolute sum (ABSSUM) which provides an upper bound for the peak value of the total response. In this approach, it is assumed that the modal peak happens all together, and their algebraic sign is ignored (Chopra, 2000).

$$r_o \leq \sum_{n=1}^{N} \left| r_{no} \right| \tag{3.2}$$

Another approach is square root of sum of squares (SRSS) way developed by E. Rosenbluet's PhD thesis (in 1951). In this rule, the peak response is squared in each mode, the squared modal peaks are added and the square root of the sum gives an forecasting of the peak total response (Chopra, 2000):

$$r_o \cong \left(\sum_{n=1}^{N} r_{no}^2 \right)^{1/2} \tag{3.3}$$

Another modal combination rule that is relevant for a wider class of structures is the complete quadratic combination (CQC). In accordance with CQC:

$$r_o \cong \left(\sum_{n=1}^{N} . \sum_{n=1}^{N} \rho_{in} r_{io} r_{no} \right)^{1/2} \cong \left(\sum_{n=1}^{N} r_{no}^2 + \sum_{i=1}^{N} . \sum_{n=1}^{N} \rho_{in} r_{io} r_{no} \right)^{1/2} \tag{3.4}$$

The equation (3.4) in its right hand side includes the result of the peak responses in the i-th and the n-th modes and the coefficient of the correlation ρ_{in} for these modes, which alters between 0 and 1, and $\rho_{in} = 1$ for $i=n$ (Chopra, 2000).

As it is mentioned above, response spectrum analysis (RSA) reduces dynamic analysis to a series of static analysis. For each mode considered, static analysis of the structure under the influence of loads provides the modal static response, which is multiplied with the spectral ordinate to obtain the peak modal response. However, the RSA is still a type of dynamic analysis, since it utilizes the properties of natural frequencies, modes, damping ratios of the structural system and also the dynamic properties of the ground motions through its response or design spectrum (Chopra, 2000).

In this study, for response spectrum analysis, the design spectrum given in Turkish Seismic Code 2007 (TEC, 2007) for Z4 local site class was considered. The design spectrum was scaled by effective acceleration coefficient of 0.4 given for the highest seismicity level. As in the study of Amiri et al. (2007), due to the stability and serviceability criteria required of such structures during and after the occurrence of an earthquake, the elastic behavior of the structure was considered and the seismic load reduction factor was taken as 1.

3.3.2. Time History Analysis

Time history analysis is used for determining the dynamic characteristics of a structural system to arbitrary loading. The equations of dynamic equilibrium to be solved are yielded by (Chopra, 2000):

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = r(t) \tag{3.5}$$

where K is the stiffness matrix, C is the damping matrix, M is the diagonal mass matrix, u, $\dot{u}$, and $\ddot{u}$ are the displacements, velocities, and accelerations of the structure, and r is the earthquake acceleration record.

In the dynamic analysis in time domain, a total of four natural ground motion records (PEER, 2010) with a single criterion the compatibility of the elastic spectra of these ground motions with the code spectrum utilized in the response spectrum analysis of the tower were chosen. The relationship between the code spectrum and the elastic spectra of the chosen earthquake accelerations of Chi-Chi 1999, Kocaeli 1999, Landers 1992, and Hector Mine 1999 is given in Figure 3.5. The properties of the sample earthquakes used in this investigation are represented in Table 3.1 and the variations in the acceleration with time for each ground motion are given in Figures 3.6-3.9.

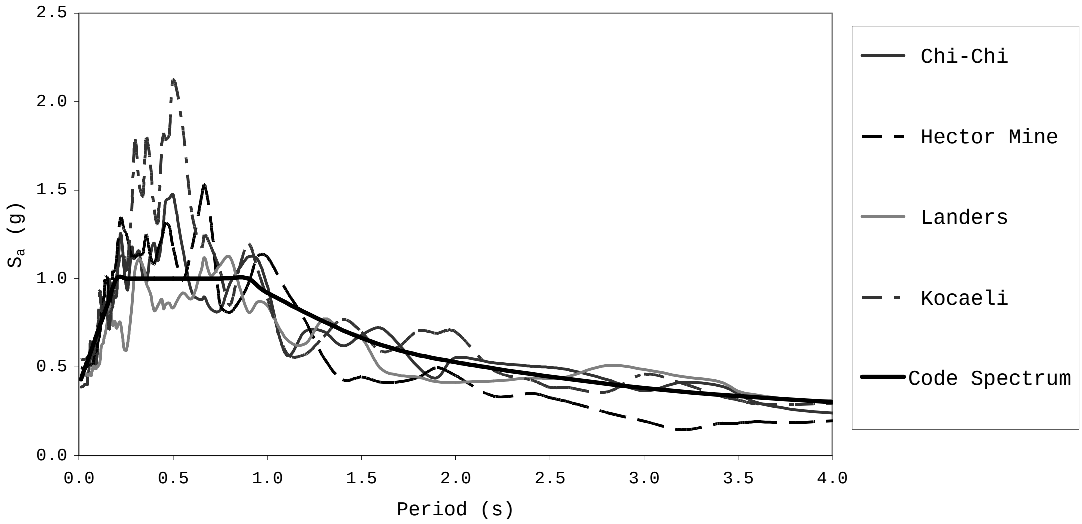


Figure 3.5 Comparison of the normalized elastic spectra of the natural ground motions with the code spectrum

Table 3.1 The properties of the natural ground motions used in the study

Earthquake Location	Recording Station	Date	Magnitude	Scale factor	PGA (g)	PGV (cm/s)	Soil (Vs30)(m/s)
Chi-Chi, Taiwan	TCU056	1999	7.62	3.05	0.39	132.81	440.2
Kocaeli, Turkey	Airport Station	1999	7.51	6.02	0.54	148.63	424.8
Landers, USA	Downey – Co Maint Bldg	1992	7.28	9.22	0.45	112.10	271.9
Hector Mine,USA	Whittier-Scott&Whittier	1999	7.13	12.53	0.49	66.99	338.5

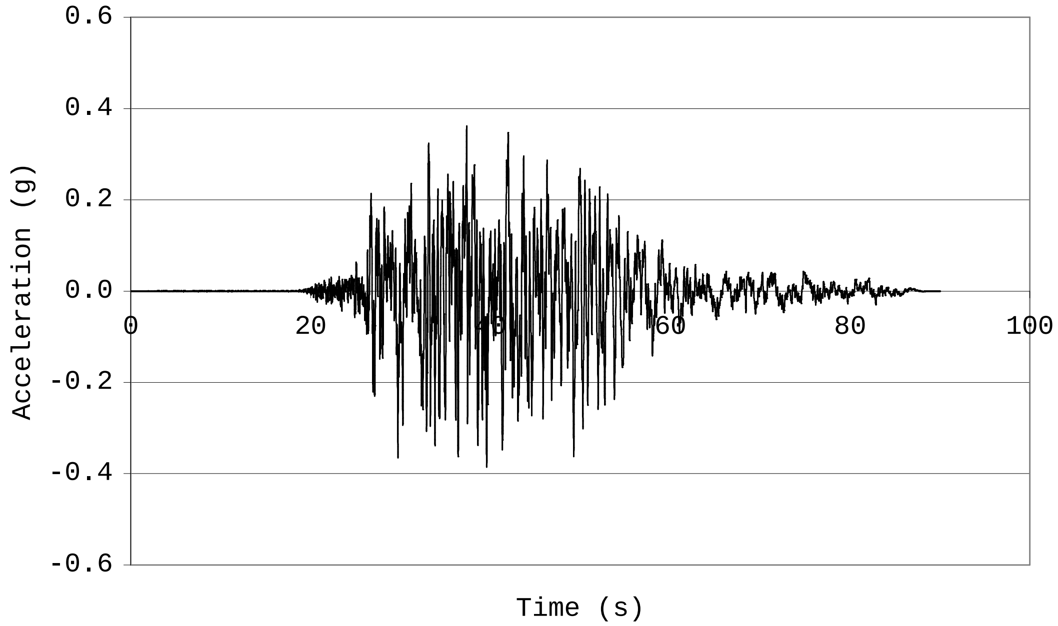


Figure 3.6 Ground motion acceleration records of Chi-Chi 1999

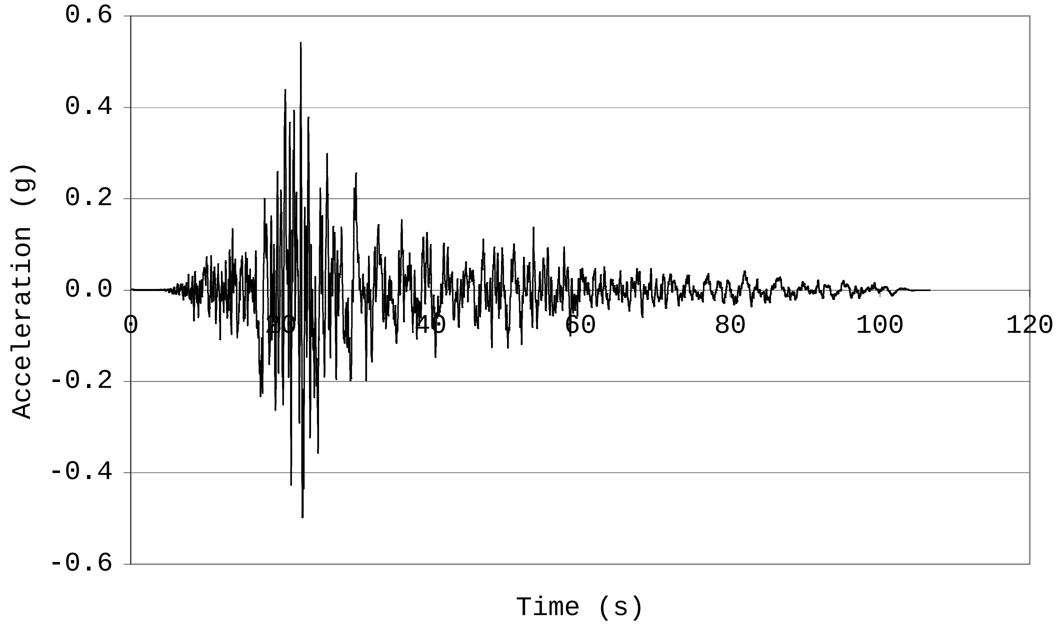


Figure 3.7 Ground motion acceleration records of Kocaeli 1999

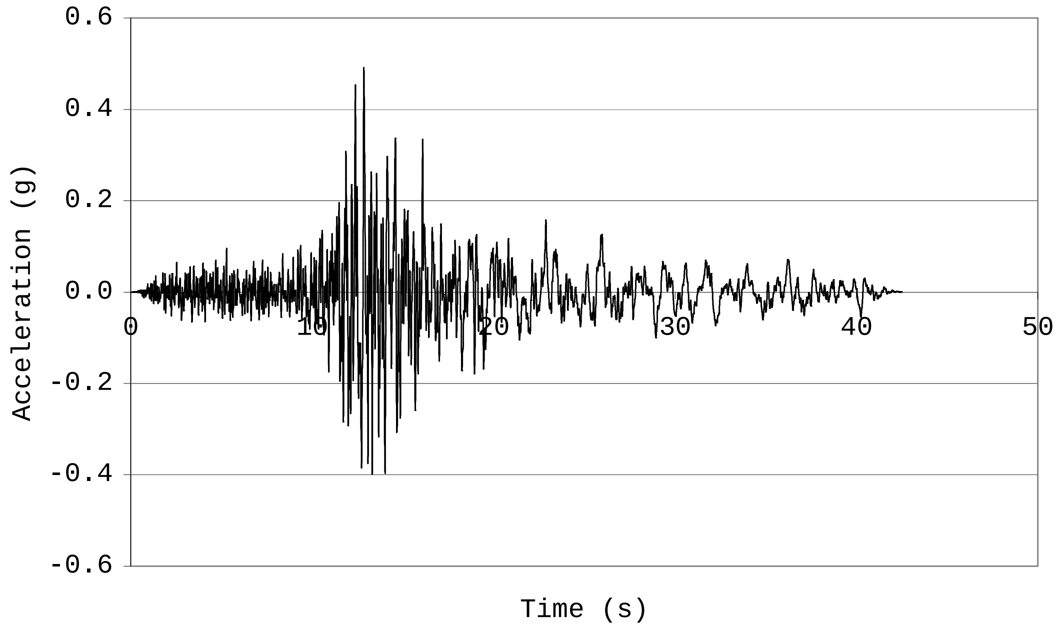


Figure 3.8 Ground motion acceleration records of Hector Mine 1999

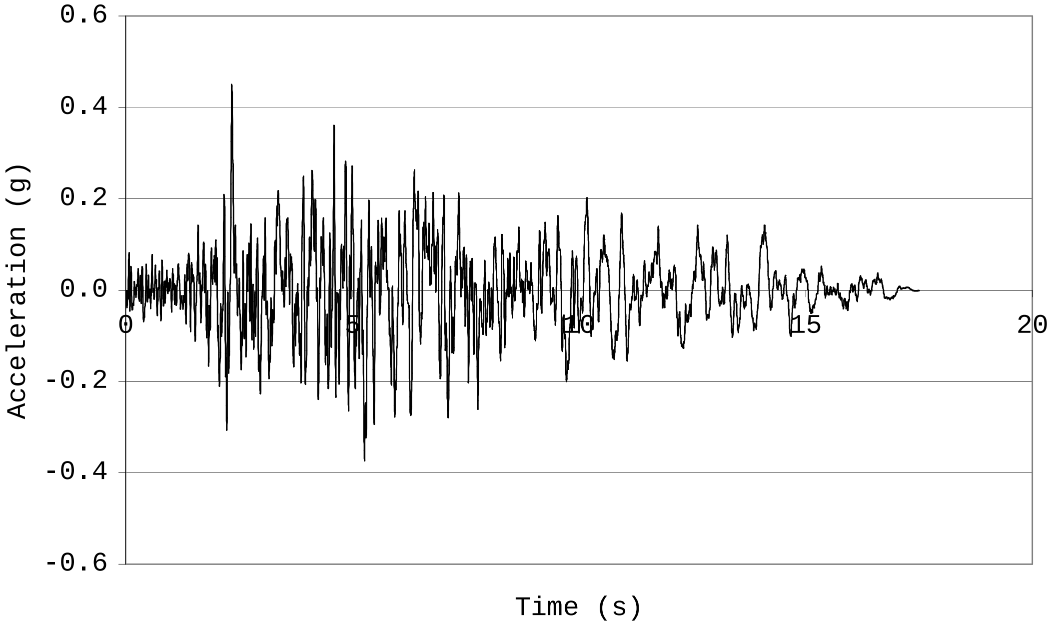


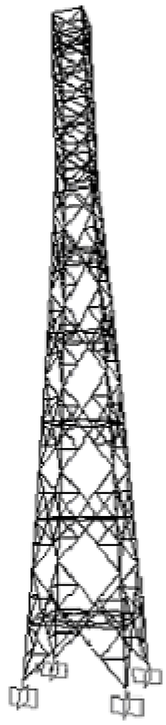
Figure 3.9 Ground motion acceleration records of Landers 1992

CHAPTER 4

DISCUSSION OF THE RESULTS

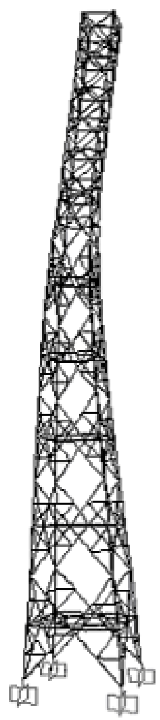
Before conducting the time history analyses and response spectrum analysis to examine the earthquakes response of the sample towers, the structural response characteristics of the case study towers were assessed by the modal analyses. The first and second flexural modes and the first torsional mode for the 40 m tower were obtained as 0.32, 0.11, and 0.056 seconds, respectively. The mode shapes corresponding to the first flexural mode, the second flexural mode, and the first torsional mode for the 40 m steel lattice tower are presented in Figures 4.1, 4.2, and 4.3, respectively.

Similarly, for the 80 m steel lattice tower, the first and second flexural modes and the first torsional mode were obtained as 0.71, 0.33, and 0.17 seconds, respectively. The mode shapes corresponding to the first flexural mode, the second flexural mode, and the first torsional mode for the 80 m tower are given in Figures 4.4, 4.5, and 4.6, respectively. Comparison of the mode of these two sample towers indicated that the values of flexural and torsional modes were depended on the tower height. As expected, higher height resulted in higher modal period.



First flexural mode

Figure 4.1 First flexural mode shape of the 40 m tower



Second flexural mode

Figure 4.2 Second flexural mode shape of the 40 m tower

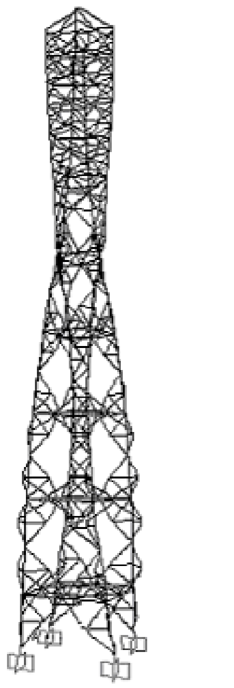


Figure 4.3 First torsional mode shape of the 40 m tower

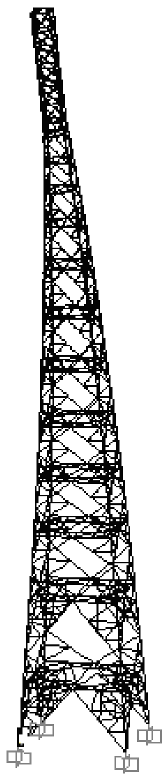


Figure 4.4 First flexural mode shape of the 80 m tower

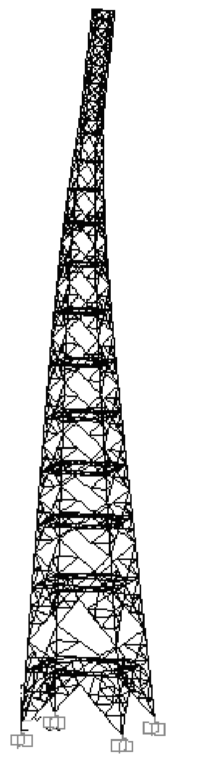


Figure 4.5 Second flexural mode shape of the 80 m tower

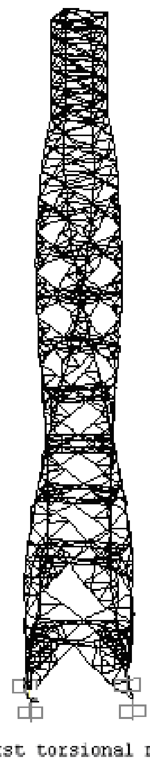


Figure 4.6 First torsional mode shape of the 80 m tower

Firstly, the earthquake ground motions were considered to act in one orthogonal direction (X) and simultaneously in two orthogonal directions (both X and Y directions) with load factors of 1 and 0.3, respectively. Secondly, the earthquake ground motions were applied in one diagonal direction (X’) and simultaneously in two diagonal directions (both X’ and Y’ directions) with the same load factors of 1 and 0.3, respectively. Therefore, for four different loading combinations, the maximum axial forces seen in the leg members, horizontal and diagonal members were evaluated by performing the time history analysis.

The results obtained from the time history analysis of the 40 m tower under Hector Mine acceleration record for the loading combinations mentioned are illustrated in Figures 4.7, 4.8, and 4.9 for the maximum axial forces occurred in the leg members, horizontal and diagonal members, respectively.

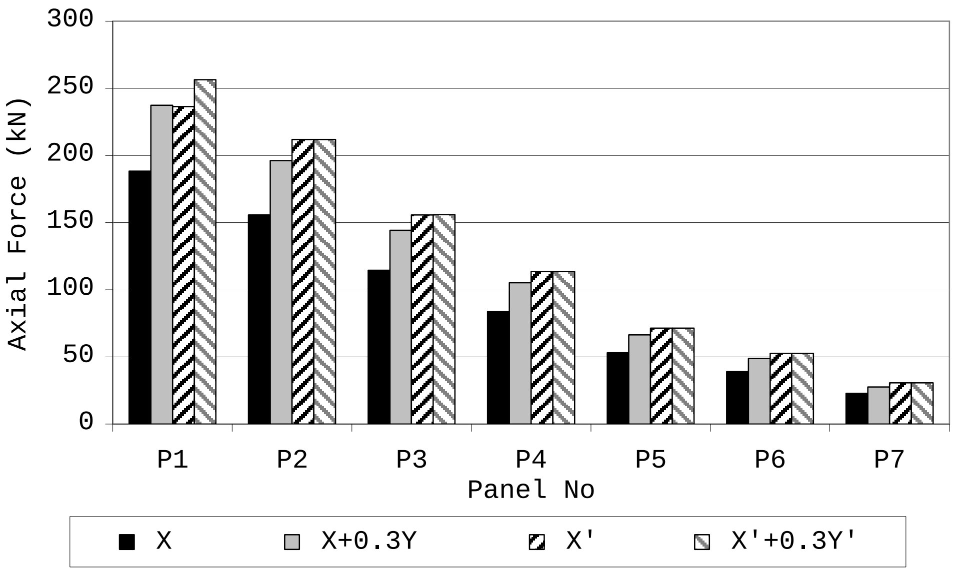


Figure 4.7 Axial member forces under four different loading combinations for leg members of the 40 m tower

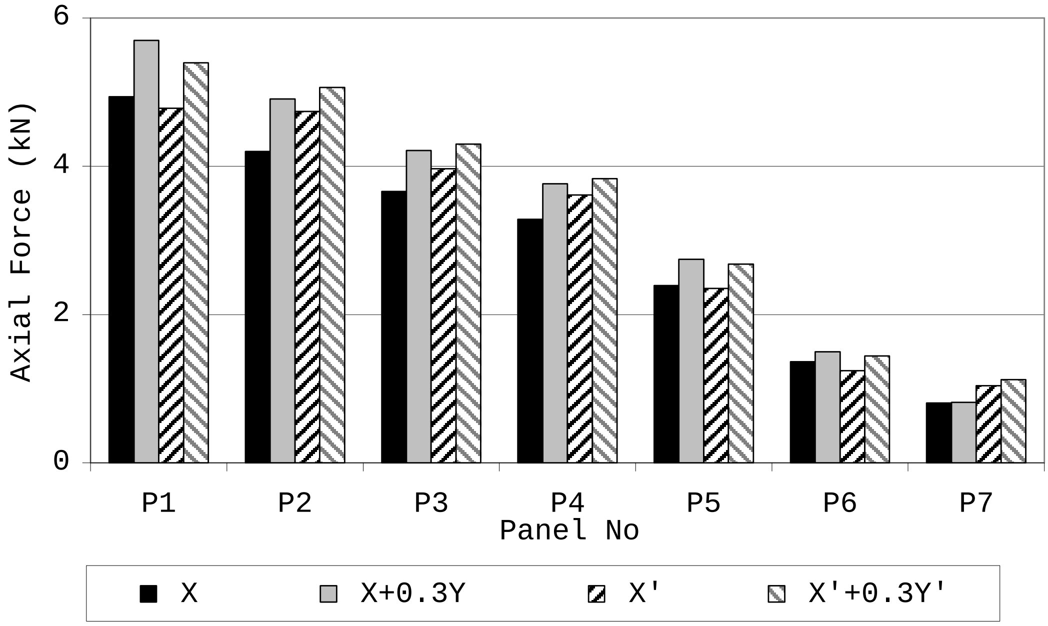


Figure 4.8 Axial member forces under four different loading combinations for horizontal members of the 40 m tower

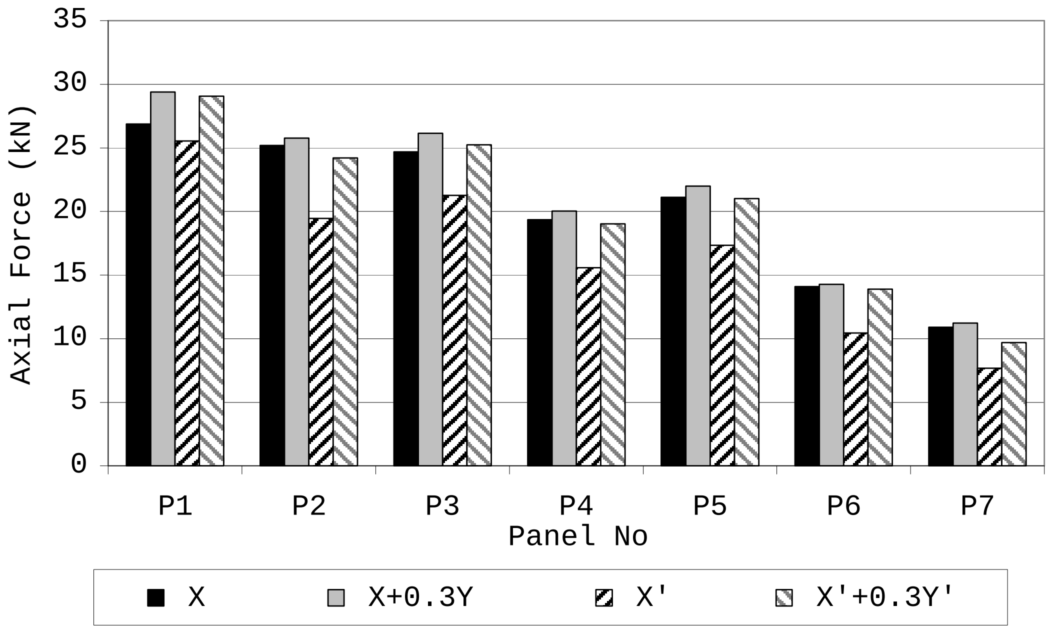


Figure 4.9 Axial member forces under four different loading combinations for diagonal members of the 40 m tower

As seen in Figures 4.7-4.9, for the 40 m tower, the axial forces obtained in the leg members under the effect of seismic load in diagonal direction were greater than the axial forces obtained under the effect of seismic load in orthogonal direction. It was also observed that the axial forces in the leg members attained the highest value for the load combination of X’+0.3Y’ in which the seismic loads were applied in two orthogonal direction simultaneously by using factors of 1 and 0.3 for two diagonal directions. However, when the axial forces in the horizontal and the diagonal members were compared, it was evident that the results obtained from four different loading conditions changed among each other. Since the values of the axial force obtained in these members were small, also the differences in the loading combinations were very small.

The results obtained from the time history analysis of the 80 mm tower under Hector Mine acceleration record for the loading combinations mentioned are also exemplified in Figures 4.10, 4.11, and 4.12 for the maximum axial forces occurred in the leg members, horizontal and diagonal members, respectively.

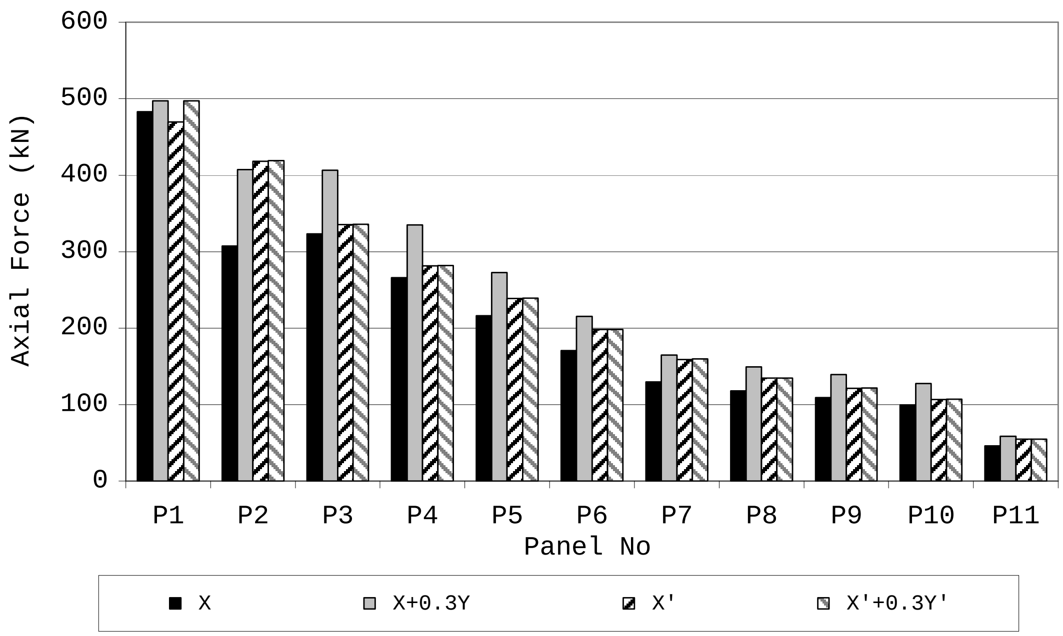


Figure 4.10 Axial member forces under four different loading combinations for leg members of the 80 m tower

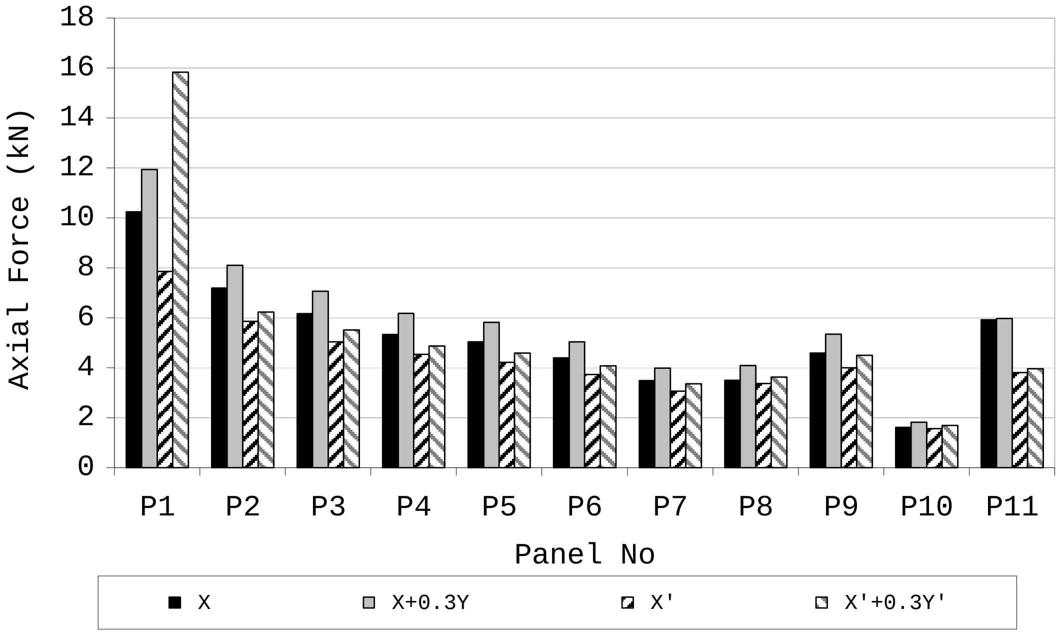


Figure 4.11 Axial member forces under four different loading combinations for horizontal members of the 80 m tower

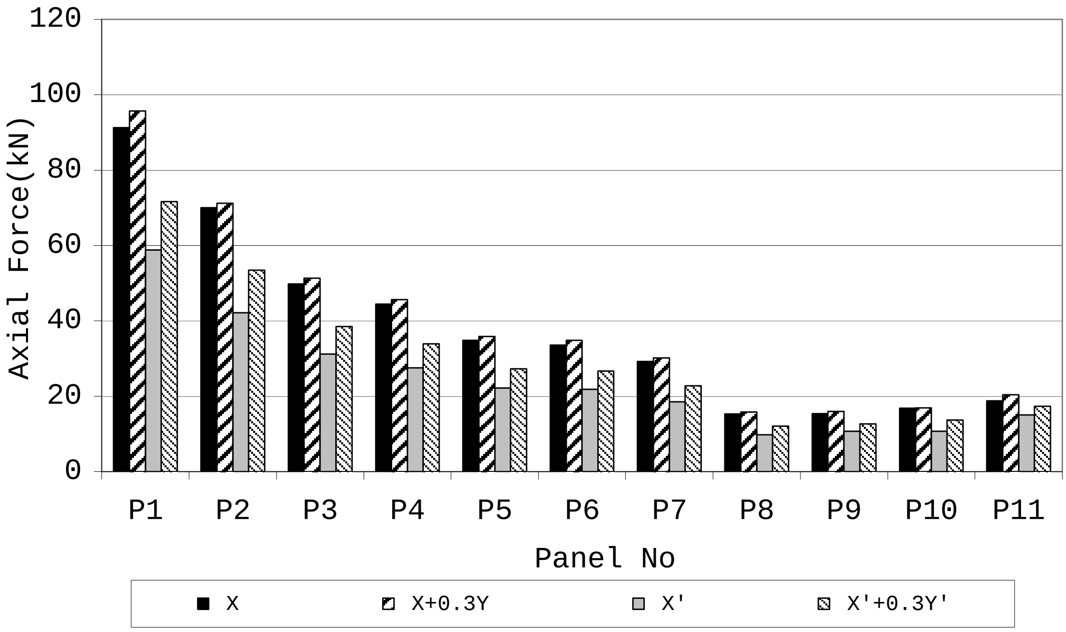


Figure 4.12 Axial member forces under four different loading combinations for diagonal members of the 80 m tower

For the 80 m tower, as shown in Figures 4.10-4.12, the loading conditions were very effective on the magnitude of axial forces occurred in the members of the structure as highlighted in the analysis results of the 40 m tower. It was also observed that in diagonal members and horizontal members, there is a dominant effect due to the load combination of X+0.3Y, namely two orthogonal directions. However, unlike the 40 m tower, the highest value of 80 m tower becomes from the X+0.3Y, namely two orthogonal directions.

The 40 m and 80 m towers were then analyzed under different seismic loads by using response spectrum analysis (RSA) and by using time history analysis (THA) which utilizes four different natural ground motions, namely, Chi-Chi, Kocaeli, Hector Mine, Landers earthquake acceleration records. The results of the axial forces of the leg, horizontal and diagonal members for the 40 m tower subjected to the seismic loading in one orthogonal direction (X) are given in Figures 4.13, 4.14, and 4.15, respectively while those for the 80 m tower are demonstrated in Figures 4.16-4.18.

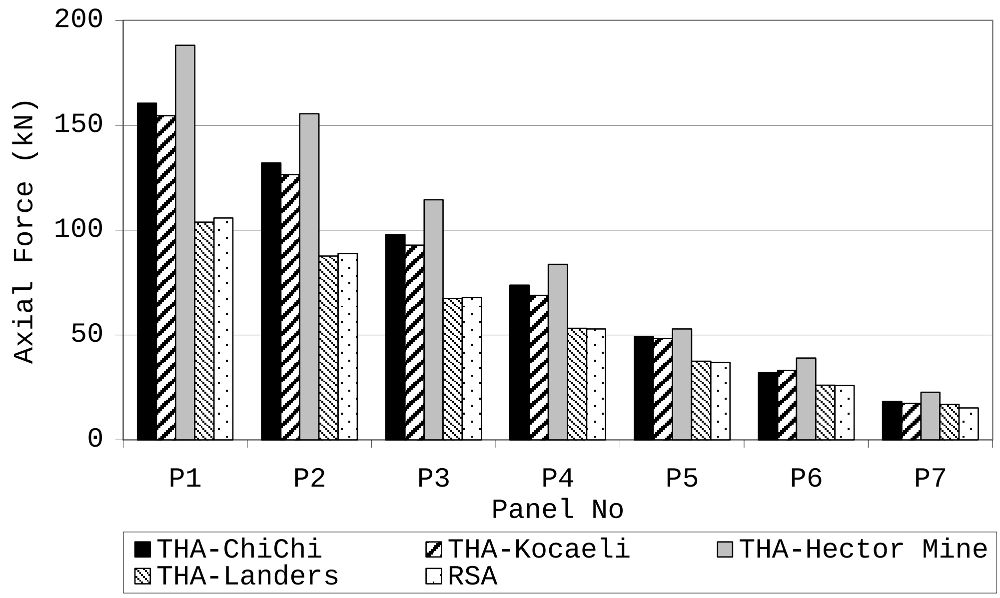


Figure 4.13 Axial member forces obtained using response spectrum analysis and time history analysis for leg members of the 40 m tower

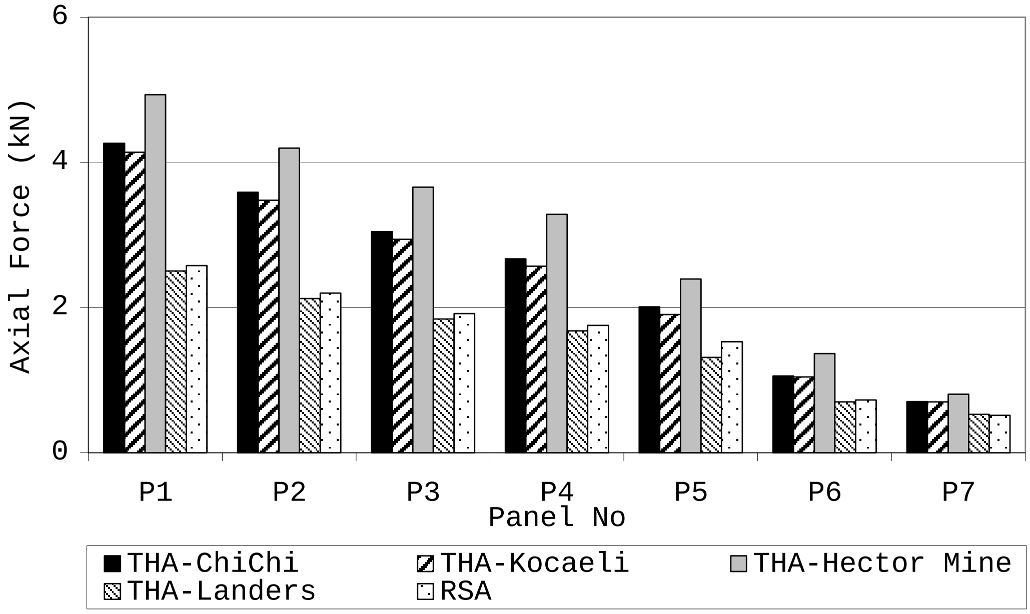


Figure 4.14 Axial member forces obtained using response spectrum analysis and time history analysis for horizontal members of the 40 m tower

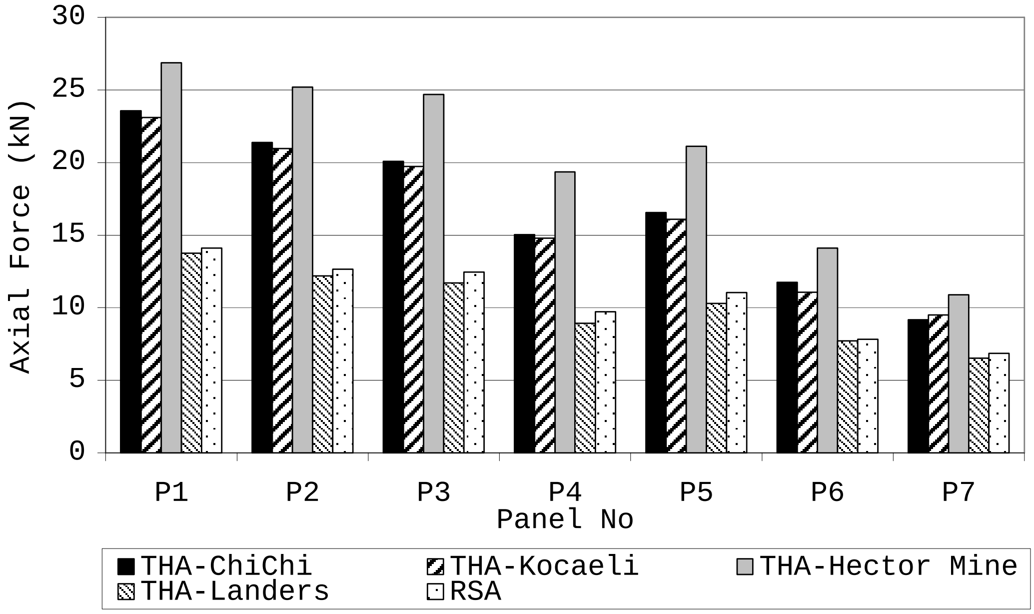


Figure 4.15 Axial member forces obtained using response spectrum analysis and time history analysis for diagonal members of the 40 m tower

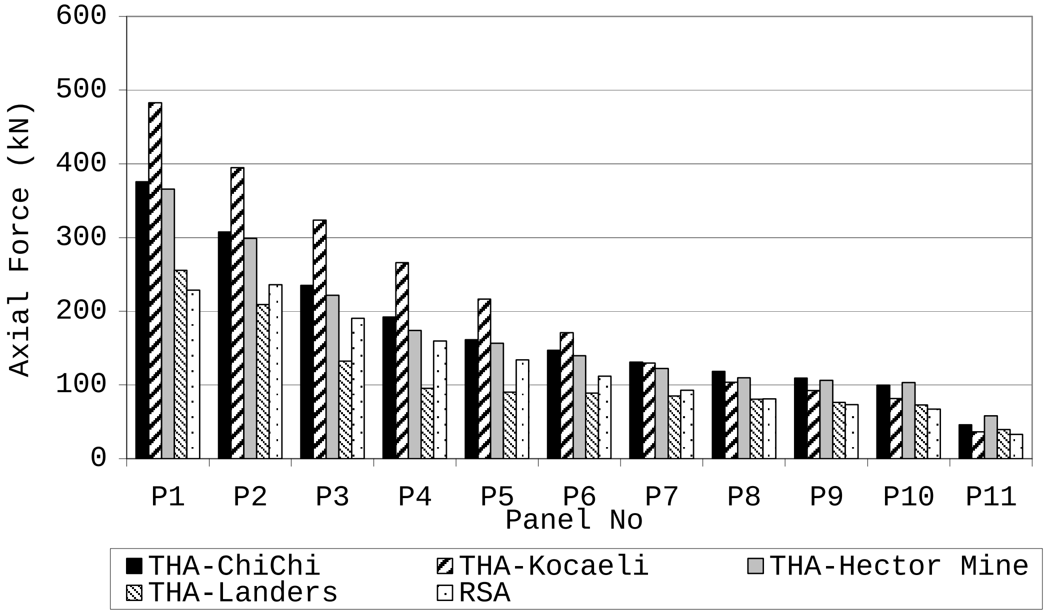


Figure 4.16 Axial member forces obtained using response spectrum analysis and time history analysis for leg members of the 80 m tower

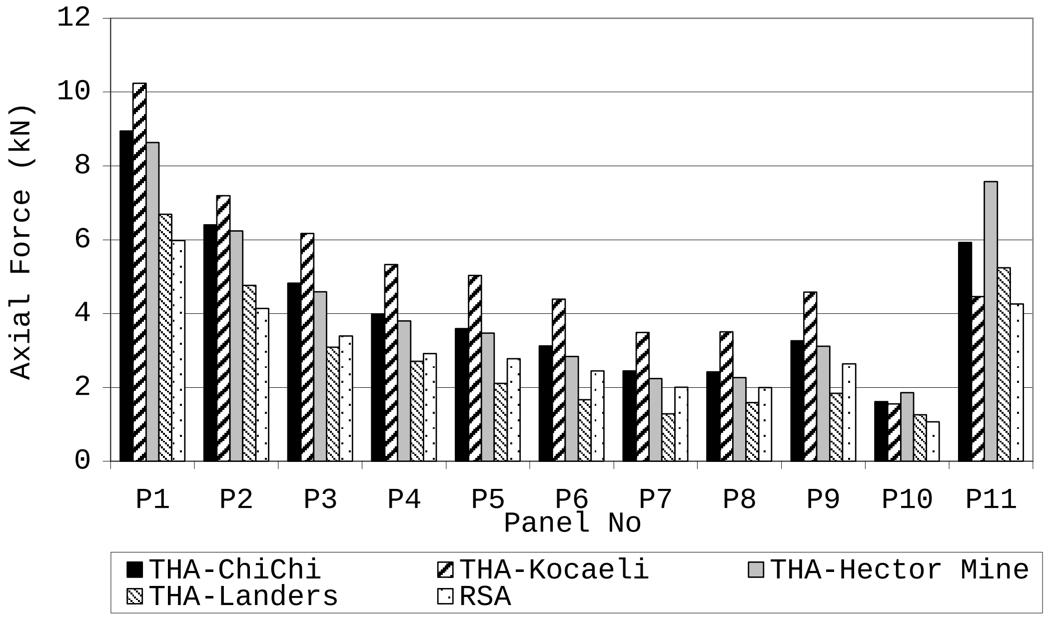


Figure 4.17 Axial member forces obtained using response spectrum analysis and time history analysis for horizontal members of the 80 m tower

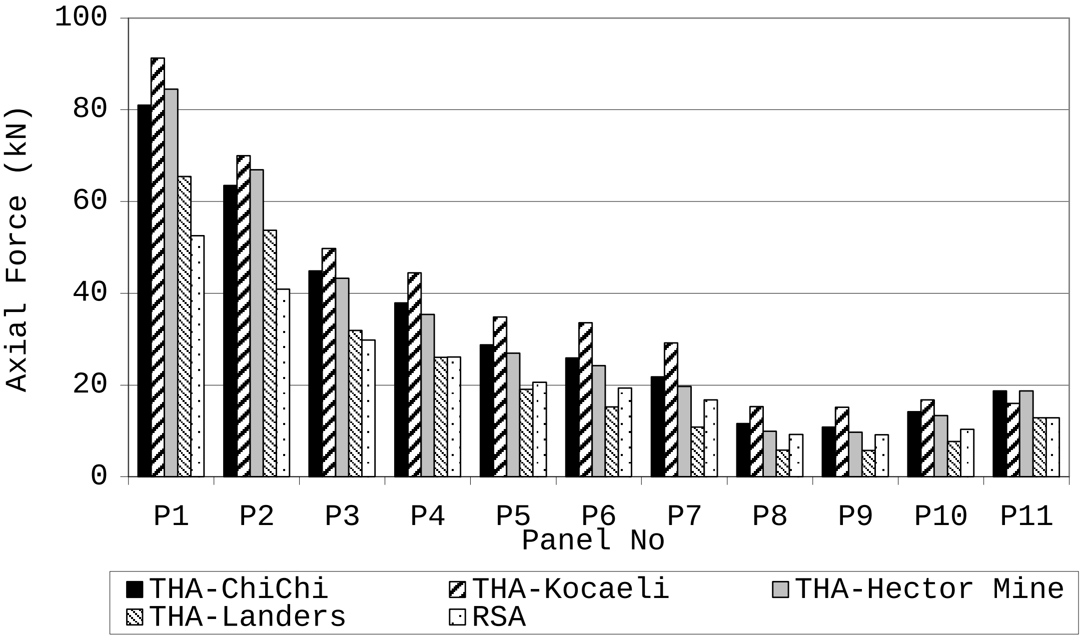


Figure 4.18 Axial member forces obtained using response spectrum analysis and time history analysis for diagonal members of the 80 m tower

As seen from the figures above, the results of the response spectrum analysis and the time history analysis conducted on the 40 m tower indicated that for the leg members, the highest values of the maximum axial forces were obtained under Hector Mine earthquake. The results of time history analysis by using Landers ground motion and the results of response spectrum analysis obtained were similar to each other. Nevertheless, similar to behavior seen under different loading combinations, when the axial forces of the horizontal and diagonal members were compared, the resultant values showed insignificant difference.

The analysis of the results demonstrated that for the 80 m tower, similarly, for the horizontal and diagonal members, the lowest values were observed under the effect of Landers earthquake. However, for the leg members, the highest values of the maximum axial forces were found under Kocaeli earthquake. Moreover, as seen from the figures, the comparisons of the results of analyses obtained from Landers earthquake and the response spectrum analysis indicated almost similar values.

To examine the effect of wind and earthquake loads on the 40 m and 80 m towers, the axial forces in the leg members attained under only wind loads, wind load with ice, and the earthquake loads (based on the most critical values obtained from each earthquake load) are shown in Figures 4.19 and 4.20, respectively. For the 40 m tower, as seen in Figure 4.9, it was found that higher values for the axial forces of leg members were observed under the wind load in comparison to seismic loads in some of the panel groups and in some of panels the trend was reverse. In addition to this, it was observed that the results due to seismic loads were close to those results from wind loads. Similarly, for the 80 m tower, it was pointed out from Figure 4.20 that the wind load was very effective. However, for some panels, the seismic load became more crucial. For both towers, the inclusion of ice load in the analysis resulted in greater axial force values.

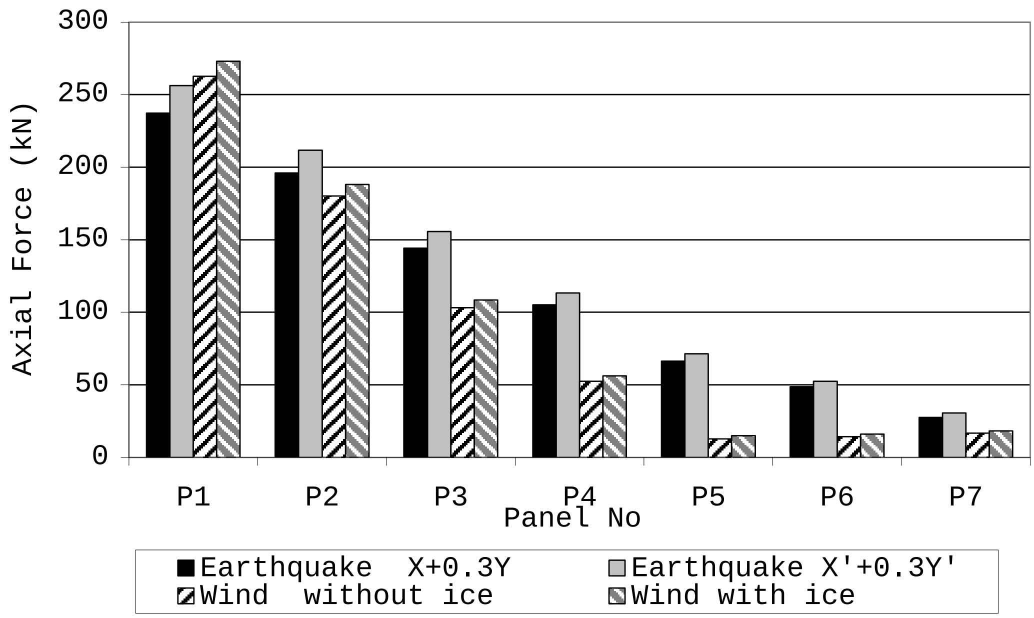


Figure 4.19 Maximum axial forces attained in the leg members of the 40 m tower under wind, wind with ice and earthquake loads

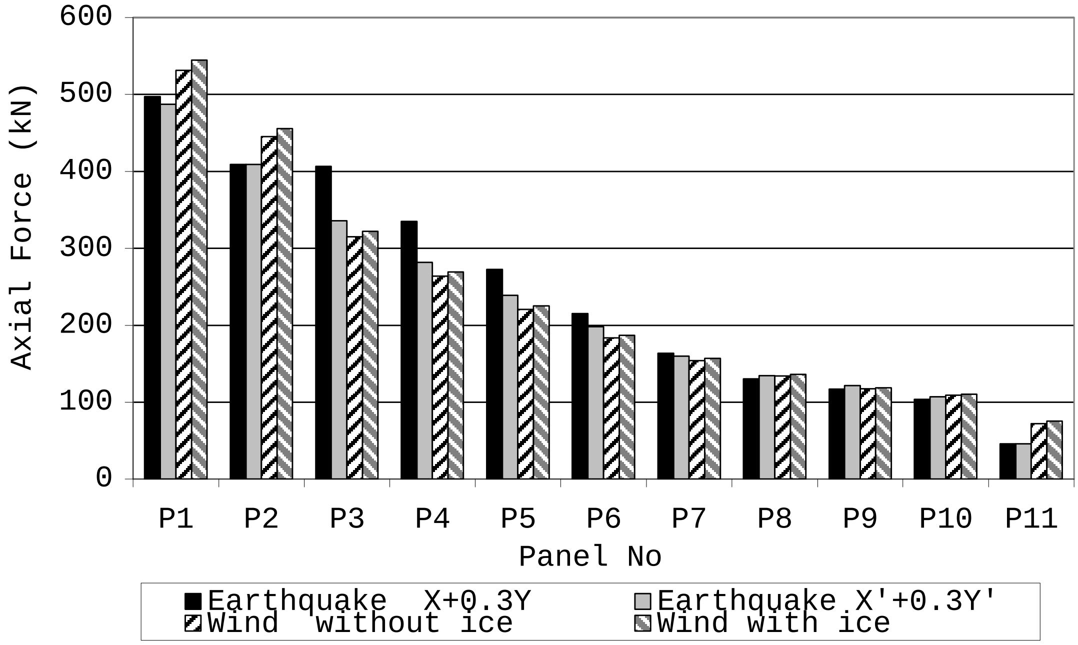


Figure 4.20 Maximum axial forces attained in the leg members of the 80 m tower under wind, wind with ice and earthquake loads

CHAPTER 5

CONCLUSIONS

In this thesis, 4-legged steel telecommunication towers having two different heights of 40 m and 80 m were investigated as a case study under the effects of earthquakes with other actions. In the structural analysis, the earthquake ground motion records, namely Chi-Chi 1999, Kocaeli 1999, Landers 1992, and Hector Mine 1999 were utilized to identify the behavior of the steel telecommunication tower under seismic excitations. Moreover, vertical loads as well as wind effects over the steel towers were taken into consideration. All computer simulations were performed using a finite element modeling software of SAP 2000. The responses of the towers were evaluated by using both response spectrum analysis and time history analysis methods. The results of analyses conducted on the towers are presented in terms of the mode shapes, axial forces occurred in the leg, horizontal, and diagonal members in line with different load combinations, and then discussed. Based on the results obtained in this study, the following conclusions could be drawn:

- Analysis of the results showed that the first and second flexural modes and the first torsional mode for the 40 m tower were obtained as 0.32, 0.11, and 0.056 seconds, respectively while those for the 80 m towers were 0.71, 0.33, and 0.17 seconds.

- It was observed that the axial forces obtained in the leg members of the towers under the effect of seismic load in diagonal direction were greater than the axial forces obtained under the effect of seismic load in orthogonal direction. It was also pointed out that axial forces in the leg members attained the highest value for the load combination of X’+0.3Y’ in which the seismic loads were applied in two orthogonal direction simultaneously by using factors of 1 and 0.3 for two diagonal directions.

- When the axial forces in the horizontal and the diagonal members were compared, it was seen that results obtained from four different loading conditions changed among each other. However, the differences in the loading combinations were small.

- The results of the response spectrum analysis and the time history analysis conducted on the towers revealed that for the leg members of the 40 m tower, the highest values of the maximum axial forces were obtained under Hector Mine earthquake. However, for 80 m tower, it was observed for Kocaeli earthquake. Moreover, for both of the towers, the lowest values were observed under the effect of Landers earthquake.

- The results of the time history analysis by using Landers ground motion and that of the response spectrum analysis obtained were similar to each other. However, similar to behavior seen under different loading combinations, when the axial forces of the horizontal and diagonal members were compared, the resultant values showed insignificant difference.

- It was observed that regardless of the height of the towers, higher values for the axial forces of leg members were obtained under the effect of wind load in comparison to the seismic loads in some of the panel groups. However, this trend was reverse for some of panels of the case study towers.

- The axial forces in the structural members under the effect of wind and earthquake loads were compared. It was pointed out that the dominant force in the design of the tower could be seismic forces or the wind forces, depending on the magnitude of the wind and seismic load. Thus, both types of lateral forces should be criticized in the design of the towers to improve the reliability of the tower structures.

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