ANALYSIS OF EARTHQUAKE LOADING, WIND LOADING AND ICE LOADING EFFECTS ON GUYED MASTS

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ABSTRACT

ANALYSIS OF EARTHQUAKE LOADING, WIND LOADING AND ICE LOADING EFFECTS ON GUYED MASTS

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Guyed masts are special type of structures that are widely used in the telecommunication industry. In the past, there was no guideline for seismic design of these types of structures in the corresponding design codes. On the other hand, in the latest "G" revision of the ANSI/TIA-EIA code [19] there is a comprehensive design criterion for the seismic design of the guyed masts. However, during the design process of these structures the most common approach is to ignore the effect of seismic loading and use only the internal forces developed from the wind load and ice load analysis.

In this study firstly the efficiency and accuracy of the commercial SAP2000 [14] and PLS-TOWER software [15] were investigated, then finite element models of three guyed masts that had been designed in Turkey with the heights 30m, 60m and 100m in the SAP2000 [14] and PLS-TOWER software [15] were analyzed under the effect of earthquake, wind and ice loadings. The most common design code recognized all over the world used for the design of the guyed masts is ANSI/TIA-EIA 222-G "Structural Standards for Steel Antenna Towers and Supporting Structures" [19]. Thus, the corresponding sections of this code were followed during the study. The main objective of this research is to check the correctness of commercial SAP2000 [14] and PLS-TOWER software [15] and to investigate the effect of seismic actions on the guyed masts and also to gain a better understanding of the behavior of guyed masts under the effects of the wind, ice and earthquake loadings.

Keywords: Towers, design of guyed masts, seismic analysis, peripheral actions

ÖZ

LENTELİ KULELERİN DEPREM, RÜZGAR VE BUZ YÜKÜ ETKİSİ ALTINDAKİ ANALİZİ

Yapar, Özgür
Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Lenteli kuleler, telekomünikasyon sektöründe yaygınlıkla kullanılan özel tipte yapılardır. Geçmişte bu tip yapıların sismik tasarımı için dizayn standartlarında hiçbir yönerge bulunmuyordu. Fakat, ANSI/TIA-EIA standardının [19] en son yayımlanan "G" revizyonunda, lenteli kulelerin sismik tasarımına ilişkin kapsamlı bir bölüm bulunmaktadır. Ancak bu tip yapıların tasarımında halen en çok tercih edilen yaklaşım, sismik yüklerden gelen etkileri göz ardı edip, sadece rüzgar ve buz yükleri analizlerinden elde edilen içsel kuvvetlerin kullanılmasıdır.

Bu çalışmada, öncelikle ticari SAP2000 [14] ve PLS-TOWER [15] programlarının yeterliliği ve doğruluğu test edilmiş, daha sonra Türkiye'de tasarlanmış 30m, 60m ve 100m boylarındaki lenteli kulelerin sonlu elemanlar modelleri SAP2000 [14] ve PLS-TOWER [15] programları kullanılarak deprem, rüzgar ve buz yükleri etkisi altında analiz edilmiştir. Lenteli kulelerin tasarımında dünya genelinde en çok kullanılan standart ANSI/TIA-EIA 222-G "Çelik anten kulelerinin ve destek yapılarının tasarım standardı"dır [19]. Bu sebeple, bu çalışmada bu standardın ilgili bölümleri takip edilmiştir. Bu çalışmada ticari SAP2000 [14] ve PLS-TOWER [15] programlarının doğruluğunun test edilmesi ve sismik etkilerin lenteli kuleler üzerindeki etkilerinin araştırılması ve ayrıca lenteli kulelerin rüzgar buz ve deprem yükleri etkisi altındaki davranışlarının daha iyi anlaşılması amaçlanmıştır.

Anahtar Kelimeler: Kuleler, lenteli kulelerin tasarımı, sismik analiz, çevresel etkiler

To My Family

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CHAPTER 1

INTRODUCTION

1.1 RESEARCH SIGNIFICANCE

Cable structures have been used for thousands of years, since the earliest sailing vessels [1]. After the development of modern cable materials, engineering interest in various cable structures were vastly increased. One of the most popular cable structures is guyed masts.

The guyed masts are most commonly used for antenna supporting. In the last decades, the need for tall antenna supporting structures has increased with the requirements for effective communication especially after the development of radios, radars, televisions and cellular phones [2]. Towers and masts are the most economical choices for antenna supporting structures. Lower erection cost, shorter erection time and smaller number of elements, which implies easier logistics and smaller risk of production errors are the main advantages of guyed masts against the other types of towers [3].

A guyed mast is a slender space truss structure which is supported by cables at various levels. Usually, guyed masts are designed as pinned at the base of the mast with a triangular truss and guy cable configuration. Definitely there are many different setups and configurations of guyed masts used for special conditions [4], but for illustration purpose a schematic of guyed mast is shown in Figure 1.1. Forces developed by earthquakes, wind drag and ice storms are the main environmental lateral effects that a guyed mast would be facing during its lifetime.

Since guyed masts body cross-section is narrower than the cantilever towers, some axial load elements should be introduced to the guyed masts for supporting lateral load effects. These axial load elements are the guy cables. To sum up, guyed masts are systems of which lateral loads are supported by guy cables and vertical forces are taken by the mast itself. [5]

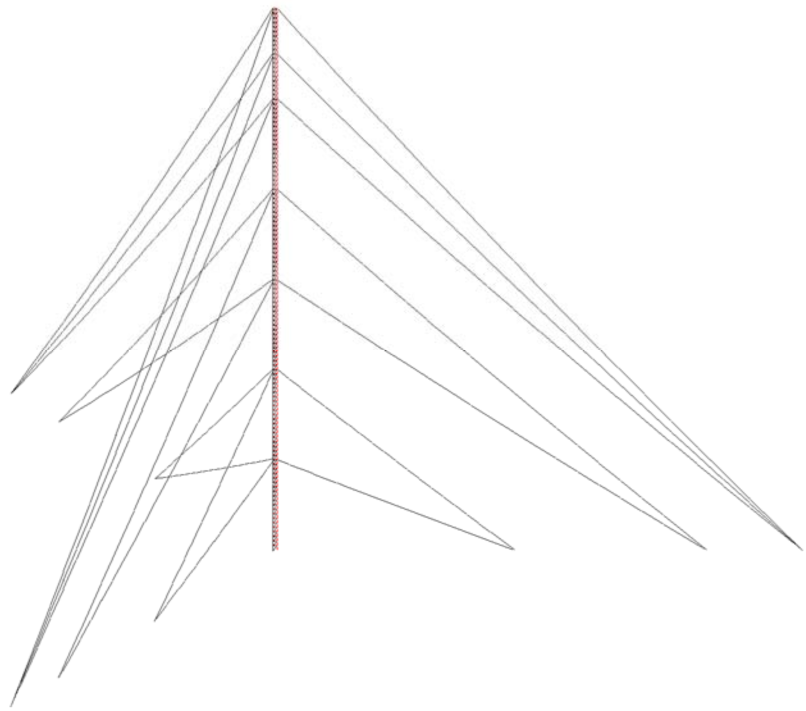


Figure 1.1 A schematic of a guyed mast

Even though guyed masts have a simple geometric configuration, and contain relatively few secondary and non-structural components [6], their behavior is very complex and different from other types of structures. *There are many* *challenges for the engineers associated with this tall and slender structures, and many experts have stated that "a guyed mast is one of the most complicated structures an engineer may be faced with"* [2]. However, during the design process of guyed masts structural engineers use commercial structural analysis software blindfolded. Before using those software, the correctness of the outputs has to be tested. Both the complex nature of the guyed masts and the inexplicitness of the analysis methods of the structural analysis software are the main factors that are making the design of those structures troublesome.

Guyed masts have a relatively high failure rate as compared to more rigid structures [7]. *Since 1959, there have been approximately 100 documented collapses of towers (both guyed and freestanding) in the United States alone* [8]. *The majority of these collapses have been attributed to wind and ice loading during major storms; however a number of permanent deflections and collapses caused by earthquakes have also been reported* [9]. Thus, with the effect of the wind and ice, earthquake effect is an important factor that should be taken into consideration during the design process of guyed masts.

1.2 OBJECT AND SCOPE

From the high rate of failure it can be understood that the behavior of the guyed masts has not yet been fully understood. The main environmental factors causing the failure of the guyed masts are wind, ice and earthquake effects. Although there is enough published research on the wind induced behavior of the guyed masts, the number of researches investigating the seismic behavior of guyed masts is still not satisfactory. In this work, firstly the efficiency of the commercial SAP2000 [14] and PLS-TOWER software [15] were tested, then the effects of the earthquakes, wind drag and ice loading forces on the guyed masts were investigated under different conditions by analyzing the finite element models of three guyed masts by using those software.

The aim of this study is to check the reliability of the results of the commercial software used during the design process of guyed masts and to investigate the effect of seismic actions on the guyed masts to gain a better understanding of the behavior of those structures under the effects of the wind, ice and earthquake loadings. To sum up, the main objective of this research is to investigate the accuracy of the commercial software used for guyed mast design and to be a general guideline for the structural engineers designing guyed mast type of structures according to ANSI/TIA-EIA 222-G [19] code.

Chapter two presents the background and the literature survey. In chapter three, two simple models were analyzed and the results were validated for investigating the consistency of the FEM analyses software that was used during this study. In chapter four accuracy of the structural analysis software used throughout this research were investigated. Chapter five presents the results of the analysis of three guyed masts under the effect of earthquake, wind and ice loading conditions. In chapter six the research findings and the concluding remarks are presented. Recommendations are also given for future studies.

CHAPTER 2

LITERATURE SURVEY

2.1 PREVIOUS STUDIES

The studies conducted by the researchers into the behavior of guyed masts are usually taken the wind loading as a launching point. There are a good number of sources of information about the wind induced behavior of guyed masts, but the number of researches investigating the seismic response of guyed masts is less.

The study of Madugula [10] is a good source of information for the dynamic analysis of guyed masts. Madugula provides a good number of information about many static and dynamic analysis procedures of guyed masts, some of which are summarized as follows:

- Guyed masts can have minimum 20 vibration modes and modes from 0.1 to 10 Hz are generally excited by the earthquakes.

- Since, most of the guyed masts are pinned at their base; the guys with high tension connected to the body of the mast at lower heights can sometimes yields fixed-end boundary conditions.

- For the guyed masts with light tower body and numerous guys with great length, guy cables can account for a fairly large portion of the total mass of the structure.

- During the modeling process of guyed masts, the nonlinear effects must be taken into consideration.

In the study of Amiri [11], eight existing guyed masts of which height is ranging from 150 to 607m were modelled as three dimensional trusses with cable elements consisting of ten prestressed truss elements, using commercial finite element software package ADINA [12]. Those masts were analyzed with the classical El Centro, Parkfield and Taft ground motion data by conducting nonlinear time-history analyses. The three accelerograms were scaled to 0.34g, to match the seismicity level of Victoria, British Columbia, the region of Canada with the highest seismicity level. The main aim of the study was to establish some seismic sensitivity indicators of guyed masts. The proposed seismic sensitivity indicators are as follows:

- For the guyed masts in the range of 150 to 350m, the equation (2.1) is proposed to estimate the fundamental natural period of the tower.

  $$T = 0.0083H - 0.74 \qquad (2.1)$$

  where

  T : fundamental natural period of the tower (s)

  H : tower height (m)

  Also, the Figure 2.1 presents the variation of tower height with respect to fundamental natural period of the tower. For assessing weather or not lateral effects will be important, a simple check was proposed with the design earthquake spectrum. Also, frequencies of 0.5 to 3.0 Hz were considered to be sensitive for horizontal effects in the study.

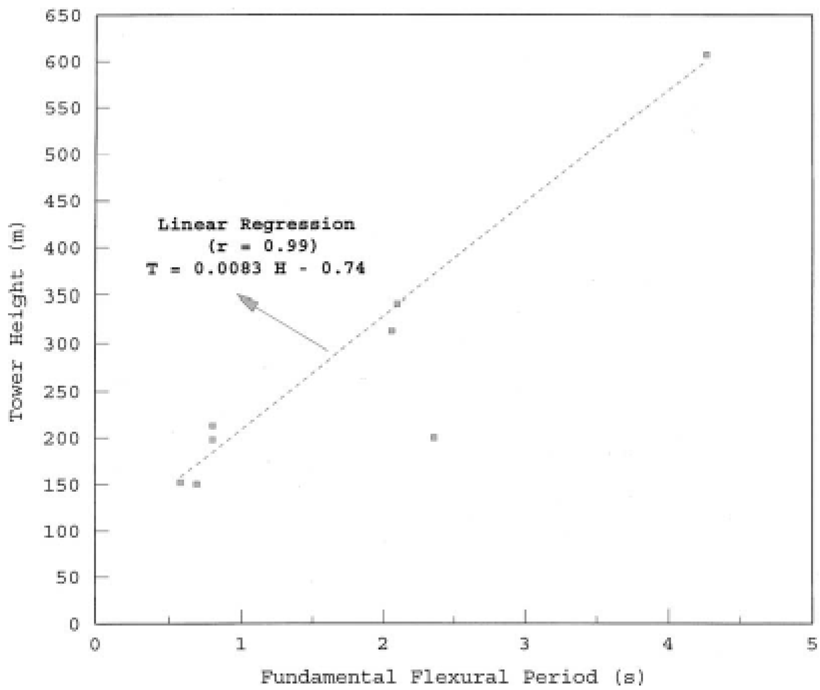


Figure 2.1 Fundamental flexural period versus tower height [11].

- Considerable base shear values may develop in the guyed masts within the height range of 150 to 350m in the order of 40 to 80% of the total tower weight. The total base shear value can be estimated by using the equation (2.2).

$$BS = 28300H^{-1.17} \tag{2.2}$$

where

BS : total base shear (% of weight)

H : tower height (m)

Also, the Figure 2.2 shows the variation of base shear value with respect to the tower height. For towers taller than 350m, the lower natural frequencies of those structures may match with the frequencies of the ground accelerations. As a result, for guyed masts taller than 350m, dynamic amplifications or resonance may occur.

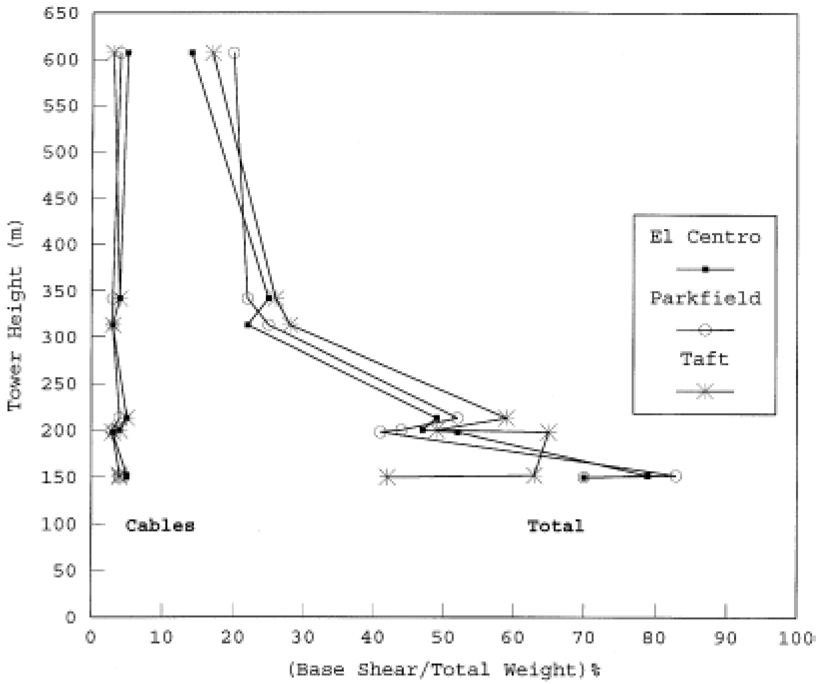


Figure 2.2 Base shear versus tower height [11].

- According to the study, the equation (2.3) is applicable for predicting the maximum axial force in the tower body for the guyed masts within the height range of 150 to 350m.

$$P_{dyn} / BA = 100 - 95(h/H)^2 \tag{2.3}$$

where

P_{dyn} : maximum dynamic component of the axial force in the mast at a section of elevation h (N)

BA : dynamic component of the axial force at the base of the mast (N) h : sectional elevation of the mast (m)

H : tower height (m)

Also, the Figure 2.3 shows the variation of dynamic axial force value in the mast with respect to the tower height.

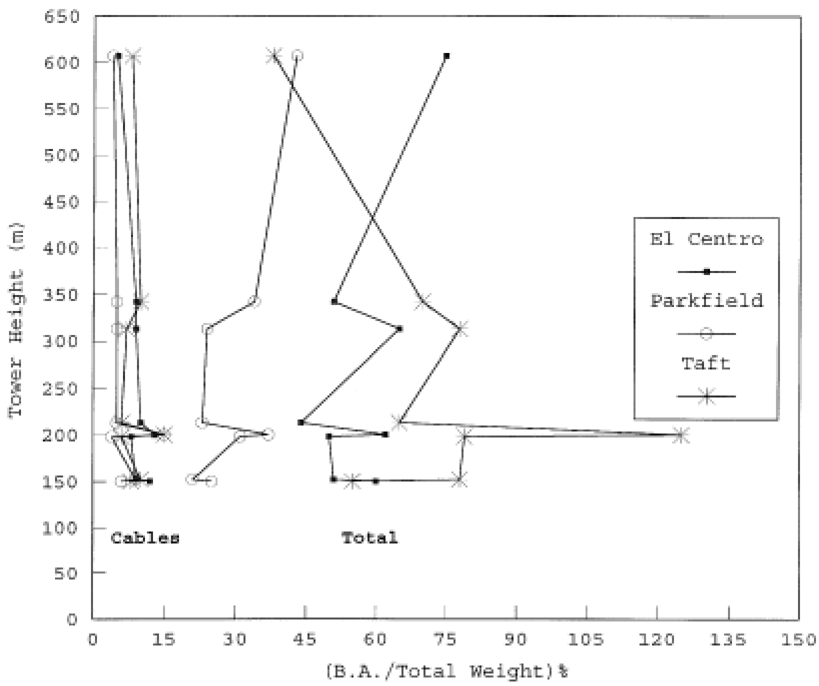


Figure 2.3 Dynamical axial forces at the base of the mast versus tower height [11]

- The dynamic components of the cable tensions vary between 30% and 300% of the initial tension values of the guy cables. For the guyed masts with heights ranging between 150 to 300m, the dynamic component of the cable tensions is in the range of 50% to 200% of the initial tensions.

2.2 DESIGN CODES

The most common building codes that has been used throughout the world for the analysis and design of guyed masts are CSA S37-01 [16], BS 8100-4 [17], Eurocode EN 1993-7 [18] and ANSI/TIA-EIA 222-G [19]. Since, the most common standard used during the analysis and design of guyed masts in the world and in Turkey is ANSI/TIA-EIA 222-G [19], the provisions of that code was followed in the current research.

The subcommittee responsible for preparing the standard ANSI/TIA-EIA-222-G [19], which is Telecommunications Industry Association (TIA) TR-14.7, states [20]:

"The objective of this Standard is to provide recognized literature for antenna supporting structures and antennas pertaining to: (a) minimum load requirements as derived from ASCE 7-02, "Minimum Design Loads for Buildings and Other Structures," [21] and (b) design criteria as derived from AISC-LRFD-99, "Load and Resistance Factor Design Specification for Structural Steel Buildings," [22] and ACI 318-05, "Building Code Requirements for Structural Concrete" [23]." [19]

With the introduction of the "G" revision, the most intense change to the standard ANSI/TIA-EIA 222-G "Structural Standards for Steel Antenna Towers and Supporting Structures" [19] has been represented since its first publication in 1949 [24]. After beginning of the use of the ANSI/TIA-EIA 222-G [19] in 2006, the most abrupt changes that the standard introduced are as follows:

- The standard moves from allowable stress design to load and resistance factor design.
- The G revision of the standard [19] comprehends the wind provisions of ASCE 7 [21].

- The standard contains seismic provisions for the tower type structures.
- The G revision of the standard [19] introduced site specific and structure specific modeling.

Since, the performance of guyed masts under the effect of wind forces, ice loading and seismic effects are in the scope of this research, the wind, ice and seismic provisions of the standard ANSI/TIA-EIA 222-G [19] is explained.

2.2.1 TIA-222-G / Wind Load Provisions

The previous F revision of the ANSI/TIA-EIA standard [30] defines the basic wind speed as the fastest mile wind speed. On the other hand the standard ANSI/TIA-EIA 222-G [19] uses the wind speed definitions of 3-second gust wind speed, which makes it compatible with ASCE 7.02 [21] code. The two wind speed definitions are illustrated in Figure 2.4.

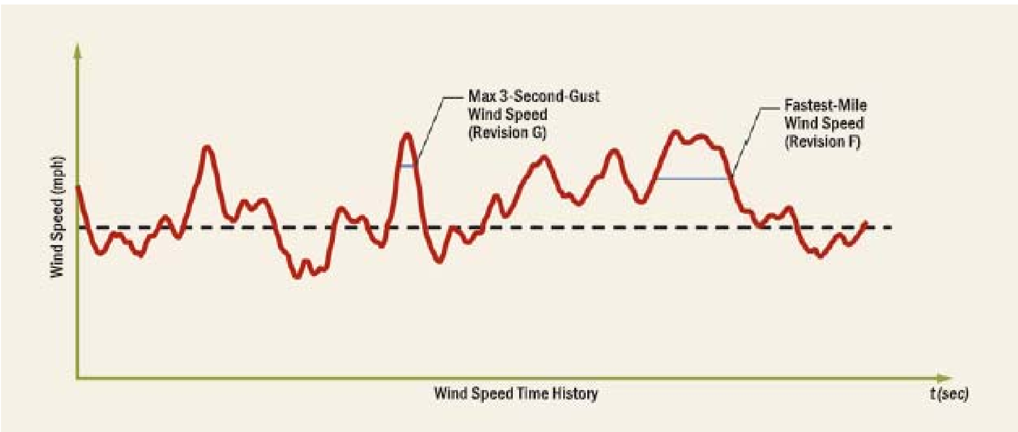


Figure 2.4 Wind speed definitions used in ANSI/TIA-EIA codes [20]

Fastest mile wind speed is defined as the wind speed corresponding to an annual probability of 0.02 at 10m above the ground in ANSI/TIA-EIA 222-F

[30]. 3-second gust wind speed is defined in ANSI/TIA-EIA 222-G [19] as the wind speed at 10m above the ground level for a 50 year mean recurrence interval in exposure category C.

After the field measurements and the determination of the design wind speed in 3-second gust, the wind force is applied to the structure according to the pattern load method defined by the standard. Pattern loading is the simulation of the dynamic wind loading effects on the structure [24]. The pattern loading provisions of the ANSI/TIA-EIA 222-G standard [19] for guyed masts is illustrated in Figure 2.5.

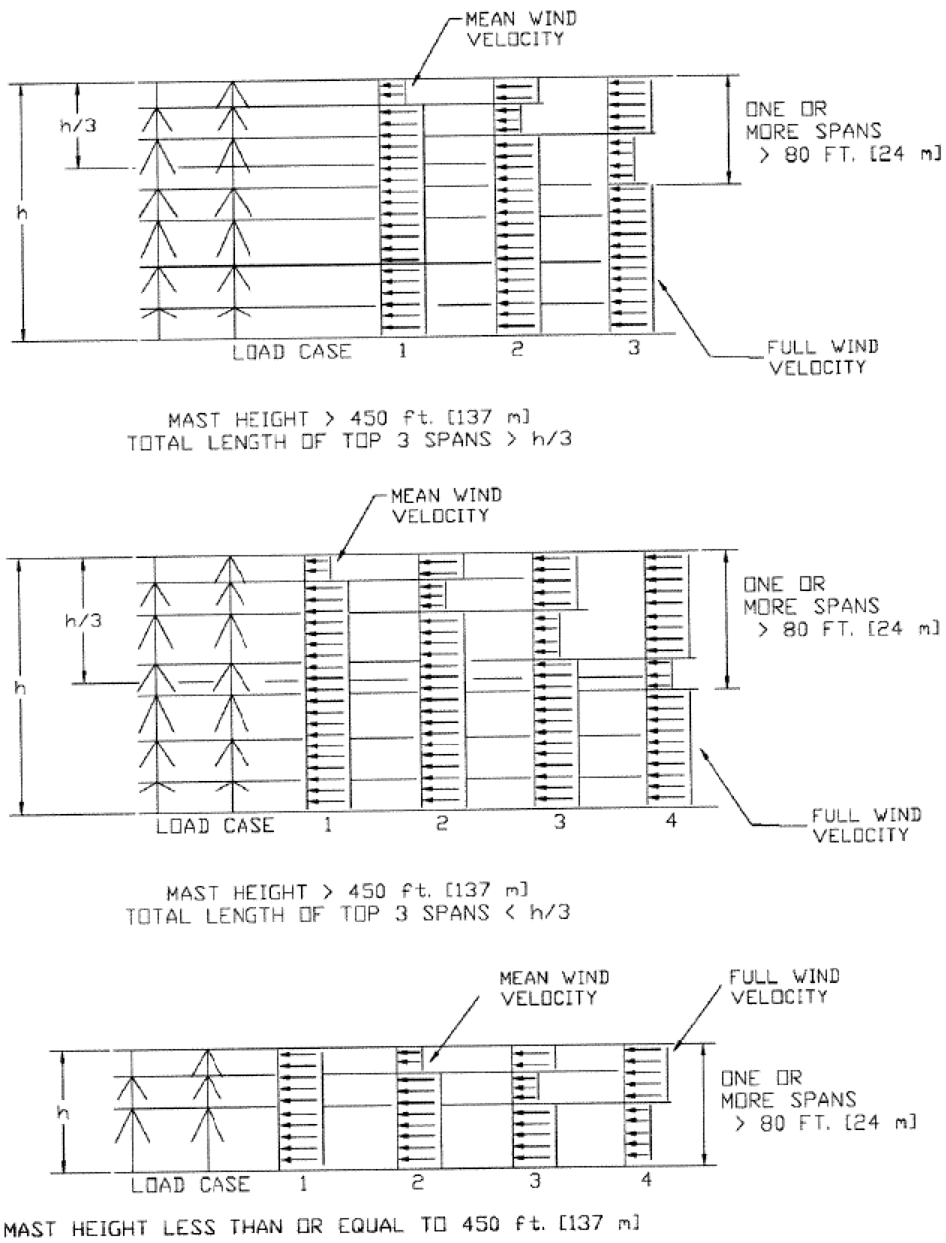


Figure 2.5 Pattern loading in guyed masts in ANSI / TIA-EIA 222-G [19]

The mean wind velocity pressure is determined by multiplying the velocity pressure with the mean wind conversion factor. The values of mean wind conversion factor for exposure categories B, C and D are 0.55, 0.60 and 0.65 respectively.

The structure classification, exposure and terrain categories defined by the standard are the factors which enables accurate site-specific and structure-specific wind modeling [20]. With the constants that obtained from the standard according to the class of the structure, category of the terrain and exposure type of the ground surface, the design wind force F_{ST} on the structure and the design wind force F_G on guys are calculated by the equations (2.4) and (2.5) respectively.

$$F_{ST} = q_Z \times G_h \times (EPA)_S \tag{2.4}$$

$$F_G = C_d \times d \times L_G \times G_h \times q_z \times \sin^2 \theta_g \tag{2.5}$$

where

F_{ST} : design wind force on the structure (N)

F_G : design wind force on guys (N)

q_z : velocity pressure defined by equation (2.6) (N/m^2)

G_h : gust effect factor

$(EPA)_s$: effective projected area of the structure (m^2)

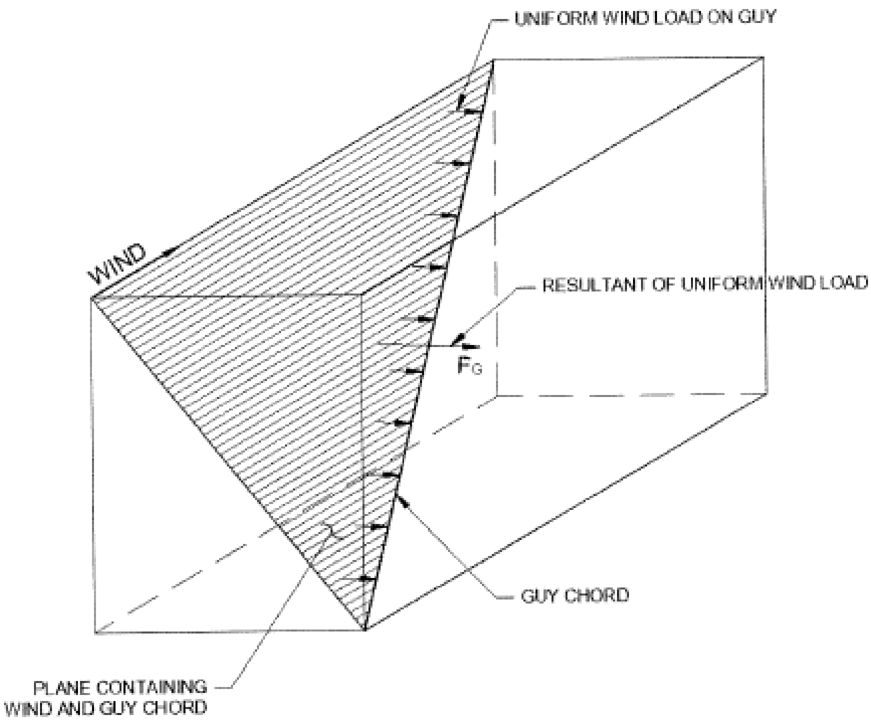
C_d : drag factor for a guy

d : guy diameter (m)

L_G : length of guy (m)

θ_g : angle of wind incidence to a guy chord (°)

The design wind force F_G on guys is shown schematically in Figure 2.6.



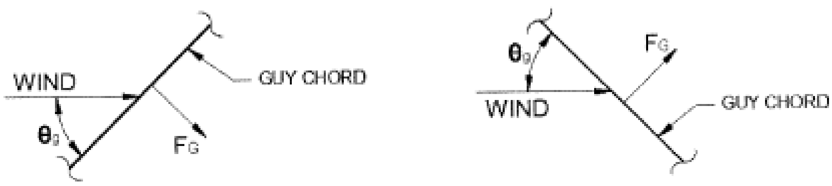


Figure 2.6 Design wind force F_G on guys [19]

where

F_G : design wind force on guys (N)

θ_g : angle of wind incidence to a guy chord (°)

The velocity pressure q_z, evaluated at height z can be calculated by the equation (2.6)

$$q_Z = 0.613 \times K_z \times K_{zt} \times K_d \times V^2 \times I \tag{2.6}$$

where

q_z : velocity pressure (N/m^2)

K_z : velocity pressure coefficient defined by equation (2.7)

K_{zt} : topographic factor defined by equation (2.9)

K_d : wind direction probability factor

V : wind speed (m/s)

I : importance factor

The velocity pressure coefficient K_z shall be determined by the equations (2.7) and (2.8).

$$K_Z = 2.01(z / z_g)^{2/\alpha} \qquad (2.7)$$

$$K_{z\min} \leq K_z \leq 2.01 \qquad (2.8)$$

where

z : height above ground level at the base of the structure (m)

z_g : wind direction probability factor

K_z : velocity pressure coefficient

K_{zmin} : minimum value for K_z

α : 3-second gust wind speed power law exponent

For the calculation of wind speed-up factor K_{zt}, the equation (2.9) shall be used:

$$K_{zt} = \left[1 + \frac{K_e \times K_t}{K_h}\right]^2$$

(2.9)

where

K_{zt} : topographic factor

K_e : terrain constant
K_t : topographic constant
K_h : height reduction factor

2.2.2 TIA-222-G / Ice Load Provisions

Ice loading is a major factor that may govern the design of guyed masts. The increased wind area that an iced mast creates may introduce higher wind load effects. [25]

After the determination of design wind speed with ice and design ice thickness, the design ice thickness escalated with height shall be calculated by the equation (2.10) and (2.11):

$$t_{iz} = 2.0 \times t_i \times I \times K_{iz} \times (K_{zt})^{0.35} \tag{2.10}$$

$$K_{iz} = \left[\frac{z}{10}\right]^{0.10} \leq 1.4 \times z \tag{2.11}$$

where
t_{iz} : nominal thickness of radial glaze ice at height z (cm)
t_i : design ice thickness for site location (cm)
I : importance factor
K_{iz} : height escalation factor for ice thickness
K_{zt} : topographic factor defined by equation (2.9)
z : height above ground (m)

For the purpose of calculating design wind force on the structure for the ice loading condition, ice thickness shall be considered to accumulate with a uniform thickness around the exposed surfaces as illustrated in Figure 2.7. [19]

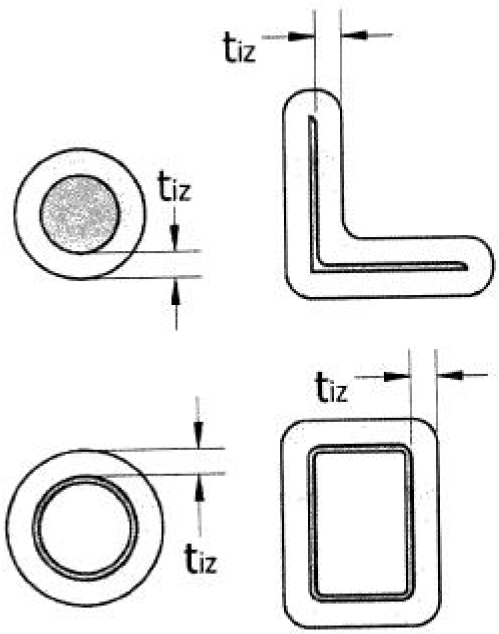


Figure 2.7 Projected area of ice [19]

For the calculation of wind load on ice, the additional projected area of ice shall be determined by considering twice the radial ice thickness of ice. For the purposes of calculating drag factors, the additional projected area of ice shall be considered round [24].

The weight of ice on a member is calculated by considering the ice layer as a cylinder around the member with the thickness equal to the radial thickness of ice. The cross sectional area of ice shall be calculated by the equation (2.12):

$$A_{iz} = \pi \times t_{iz} \times (D_c + t_{iz}) \quad (2.12)$$

where

A_{iz} : cross sectional area of ice at height z (cm^2)

t_{iz} : nominal thickness of radial glaze ice at height z (cm)
D_c : largest out-to-out dimension of a member (cm)

2.2.3 TIA-222-G / Seismic Load Provisions

With their complex and unique behavior characteristics, guyed masts require special considerations under the effect of seismic loading. With the inclusion of seismic provisions, the ANSI/TIA-EIA-G Standard is in compliance with other current standards, codes and guides. *On the other hand, earthquake-resistant design of these structures cannot simply be extrapolated from simple rules available for buildings.* [11]

At the beginning phase of the seismic analysis of a guyed mast, the steps that should be followed are summarized below:

- The classification of the structure shall be determined.
- An appropriate seismic analysis procedure shall be determined for the type of the structure that shall be analyzed. For the guyed masts, the applicable earthquake analysis methods are Equivalent Lateral Force Procedure (Method 1) and Time-History Analysis (Method 4).
- The maximum considered earthquake spectral response acceleration at short periods and at one second shall be determined.
- The class of the site shall be determined based on the soil properties at the site. [19]

With the introduction of ANSI/TIA-EIA-G Standard, seismic approaches and factors considering the uniqueness of towers and sites are included and considered as a part of the design [20].

2.2.3.1 Seismic Procedure Analysis Methods

The seismic analysis procedure methods defined by the standard are summarized in the rest of the chapter.

2.2.3.1.1 Equivalent Lateral Force Procedure (Method 1)

First of all, the total weight of the structure is determined. Then, the total seismic shear is calculated and distributed as specified in the ANSI/TIA-EIA-G Standard [19]. Lastly, the structure is analyzed statically applying the seismic forces as external loads [20].

2.2.3.1.2 Equivalent Modal Analysis Procedure (Method 2)

Along with the seismic forces for each level, the fundamental frequency of the structure is determined. The structure is analyzed statically applying the seismic forces as external loads. [20]

2.2.3.1.3 Modal Analysis Procedure (Method 3)

The mathematical model of the structure is created that describes the distribution of the mass and stiffness. The mode shapes are determined using acceptable structural engineering methods. The design response spectrum shown in Figure 2.8 is developed in accordance with the ANSI/TIA-EIA-G Standard [19] with the help of the equations (2.13), (2.14), (2.15) and (2.16). The determined base shears and forces at each level of the structure for each mode are then combined by the square root of the sum of the squares of the modal values. [20]

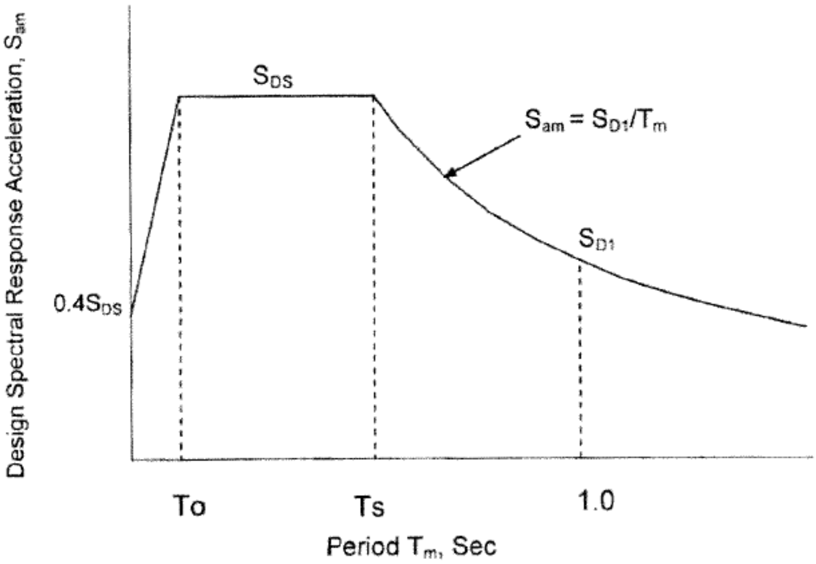


Figure 2.8 ANSI/TIA-EIA-G Design Response Spectrum [19]

When $T_m \geq T_s$

$$S_{am} = S_{D1}/T_m \tag{2.13}$$

When $T_m \geq 4.0$ sec.

$$S_{am} = 4S_{D1}/{T_m}^2 \tag{2.14}$$

$$T_O = 0.2S_{D1}/S_{DS} \tag{2.15}$$

$$T_S = S_{D1}/S_{DS} \tag{2.16}$$

where

T_m : period of the structure (s)

T_S, T_O : periods used to define the design spectral response (s)

S_{am} : design spectral response acceleration at period T_m (g)

S_{D1} : design spectral response acceleration at a period of 1.0 second (g)

S_{DS} : design spectral response acceleration at short periods (g)

2.2.3.1.4 Time - History Analysis (Method 4)

Firstly, a mathematical model of the structure is created that describes the distribution of the mass and stiffness considering structural damping to be five percent of critical damping. Two orthogonal ground motion time histories are selected from not less than three recorded events. For each horizontal component of the events, a five percent horizontal response spectrum is constructed, and the foregoing time histories are combined using the square root of sum squares. The average of those time-histories shall be scaled according to the rules specified in the standard. The scaled time-histories are used to perform a structural analysis. By performing the time history analysis with acceptable methods of structural engineering, load effects for design are determined by selecting the maximum values. [20]

CHAPTER 3

VALIDATION STUDIES

3.1 OVERVIEW OF THE SOFTWARE

SAP2000 [14] is finite element analysis software, which can conduct static and dynamic analysis, with linear and nonlinear capabilities. Since PLS-TOWER [15], the other software that is used, is not capable of conducting dynamic analysis, SAP2000 [14] is used for the dynamic analysis of the guyed mast systems throughout this research.

PLS-TOWER [15] deals with the static analysis of lattice towers [26]. PLS-TOWER [15] can calculate the wind loading and ice loading forces on the structure according to the provisions of ANSI/TIA-EIA-G Standard [19] automatically. Calculating the wind loading and ice loading forces on every structural member of the structure would be very time consuming. Thus wind and ice loading analysis of the three guyed masts are conducted with PLS-TOWER [15].

3.1 PRELIMINARY MODELS

Since the consistency of the modeling and analyzing methods of both software directly affects the results of the research, the results produced by the two modeling software will be compared using two preliminary models. According to the results obtained from this comparison, both SAP2000 [14] and PLS-TOWER [15] software were produced consistent results.

3.1.1 Preliminary Model 1: Cable Modeling

The consistency of the cable elements of both SAP2000 [14] and PLS-TOWER [15] software were investigated using a preliminary model.

After going through the SAP2000 software's verification examples [27] and PLS-TOWER software's engine manual [26], it is demonstrated that both software's cable element can be represented by the cable algorithm introduced by Peyrot [13]. Thus, for investigating the consistency of the outputs generated by SAP2000 [14] and PLS-TOWER [15] software, an example solved in the study of Peyrot [13] was solved by using both software. Also the same example is present in verification examples of the SAP2000 software [27].

The cable with an undeformed length of 30.48 m was subjected to a 14593 N/m uniformly distributed vertical load and a 37.78 °C temperature load. The modulus of elasticity of the cable is 47880 N/m^2, the cross sectional area is 2787091 m^2 and the coefficient of thermal expansion is 1.117e-5 /°C. The material properties and the loading details of the cable are summarized in Table 3.1.

Table 3-1 Material properties and loading details of the preliminary model 1

Cable Properties	
Undeformed Length	30.48m
E.A	1.334E+11 N
E.T	0.535N/m^2°C
Cable Loading	
Uniform Vertical Load	14593 N/m
Temperature Change	37.78 °C

The preliminary cable element was analyzed both with SAP2000 [14] and PLS-TOWER [15] software by displacing its lower support horizontally with the configurations described in Figure 3.1.

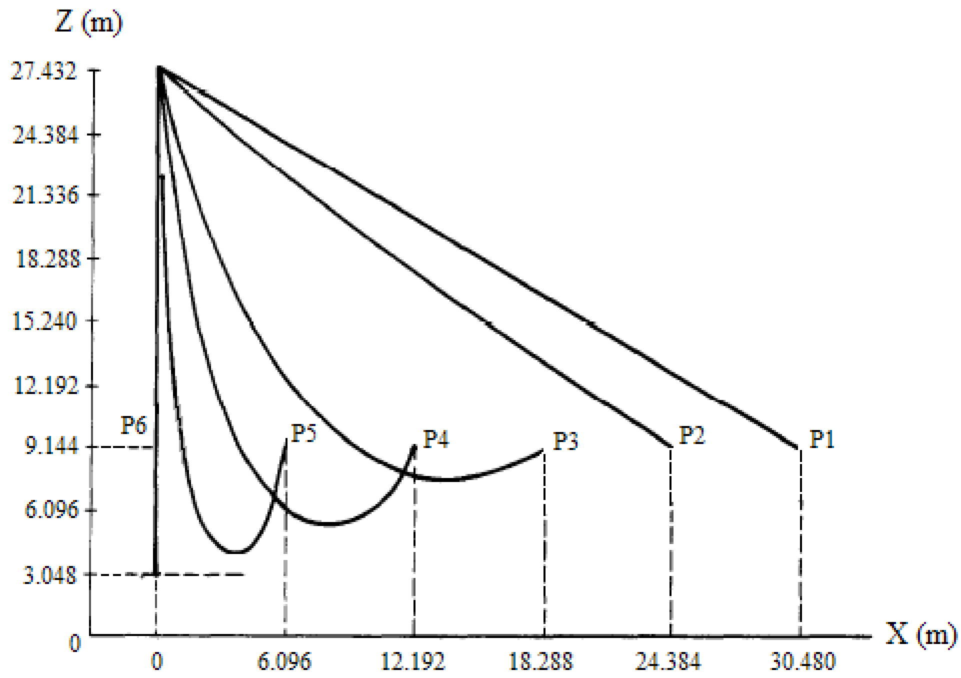


Figure 3.1 Configurations of the preliminary model 1 [13]

The lower support reactions obtained from the analysis of the SAP2000 [14] model, PLS-TOWER [15] model and the results presented in Peyrot [13] are listed in Table 3.2.

Table 3-2 Summary of the analysis results of preliminary model 1

Position	Output	SAP2000 (N)	PLS-TOWER (N)	Peyrot [13] (N)
P1	X Reaction	1.894E+10	1.896E+10	1.855E+10
	Z Reaction	-1.137E+10	-1.138E+10	-1.117E+10
P2	X Reaction	2.242E+6	2.242E+6	2.242E+6
	Z Reaction	-1.463E+6	-1.463E+6	-1.463E+6
P3	X Reaction	9.853E+4	9.853E+4	9.853E+4
	Z Reaction	6.997E+4	6.997E+4	6.997E+4
P4	X Reaction	4.079E+4	4.079E+4	4.079E+4
	Z Reaction	8.558E+4	8.558E+4	8.558E+4
P5	X Reaction	1.361E+4	1.361E+4	1.361E+4
	Z Reaction	8.865E+4	8.865E+4	8.865E+4
P6	X Reaction	0.000	0.000	0.000
	Z Reaction	8.905E+4	8.905E+4	8.905E+4

From the results obtained, it can be clearly understood that both SAP2000 [14] and PLS-TOWER [15] cable models responses show good agreement with each other and the method described in the study of Peyrot [13].

3.1.2 Preliminary Model 2: Guyed Mast

Since the main subject of this study is the analysis of guyed masts, the consistency of SAP2000 [14] and PLS-TOWER [15] software were investigated using a guyed mast as a preliminary model of which schematic is shown in Figure 3.2.

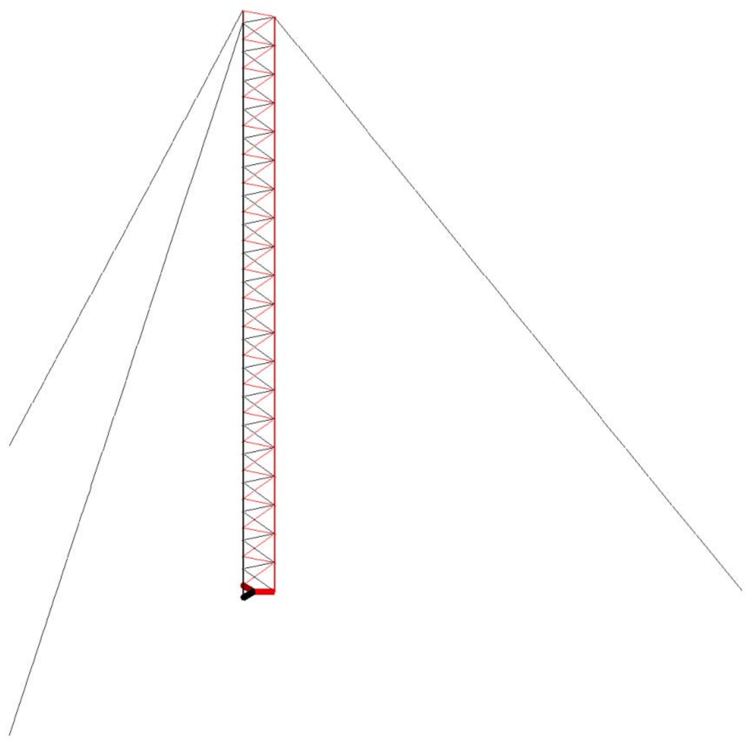


Figure 3.2 Schematic of the preliminary model 2

The model is triangular lattice in form, pinned at the base with the height of 10 meters. The mast has cables with 15.88 millimeters diameter as guys, Φ38x3.6 pipes as legs and Φ16 solid rods as horizontal and diagonal members. The guys are attached to the tower at the top mounted on the corners. The horizontal distance from the base of the structure to the guy anchor points is 8 meters. The initial tension of the guy cables is the 10% of the ultimate tension capacity of the guys which is 17.88kN.

The guyed mast was modeled using both SAP2000 [14] and PLS-TOWER [15] software as three dimensional truss models where the members of the structure are modeled as frame elements hinged at the joints, only producing axial forces. The guys were modeled with the cable elements of the both software. A 1kN force was applied to the top 4 joints of both of the models as described in Figure 3.3.

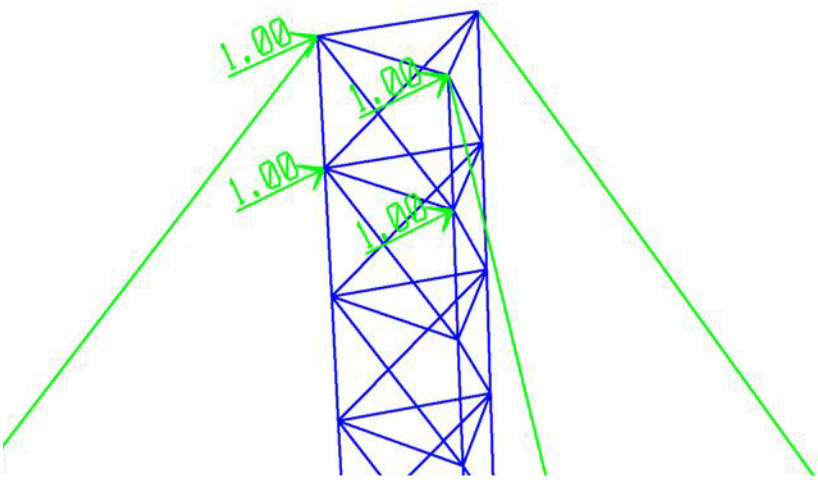


Figure 3.3 Preliminary Model 2 with four 1kN point loads

The support reactions obtained from the analysis of the SAP2000 [14] model and PLS-TOWER [15] model are listed in Table 3.3.

Table 3-3 Summary of the support reactions of preliminary model 2

Support	Output	SAP2000 (N)	PLS-TOWER (N)	% Difference
Base	X Reaction	-2.591E+2	-2.600E+2	0.35%
	Y Reaction	0.000	0.000	0.00%
	Z Reaction	-3.832E+4	-3.839E+4	0.18%
Guy Anchor 1	X Reaction	-5.193E+3	-5.210E+3	0.33%
	Y Reaction	-8.985E+3	-9.010E+3	0.28%
	Z Reaction	1.348E+4	1.352E+4	0.30%
Guy Anchor 2	X Reaction	-5.193E+3	-5.210E+3	0.33%
	Y Reaction	8.985E+3	9.010E+3	0.28%
	Z Reaction	1.348E+4	1.352E+4	0.30%
Guy Anchor 3	X Reaction	6.641E+3	6.670E+3	0.43%
	Y Reaction	0.000	0.000	0.00%
	Z Reaction	8.607E+3	8.640E+3	0.38%

The maximum percent difference between the support reactions of the two preliminary guyed mast models of SAP2000 [14] software and PLS-TOWER [15] software is 0.43%.

Since both of the preliminary models of SAP2000 [14] and PLS-TOWER [15] software are three dimensional trusses where the structural members were modeled as frame elements only producing axial forces, compression and tension forces of leg members, diagonal members, horizontal members and guys are compared in Table 3.4, Table 3.5, Table 3.6 and Table 3.7.

Table 3-4 Summary of the leg member forces of preliminary model 2

Member	SAP2000 (N)	PLS-TOWER (N)	% Difference
M1	-1.300E+04	-1.299E+04	0.08%
M2	-1.320E+04	-1.320E+04	0.00%
M3	-1.339E+04	-1.340E+04	0.07%
M4	-1.358E+04	-1.359E+04	0.07%
M5	-1.377E+04	-1.378E+04	0.07%
M6	-1.395E+04	-1.397E+04	0.14%
M7	-1.413E+04	-1.416E+04	0.21%
M8	-1.430E+04	-1.433E+04	0.21%
M9	-1.447E+04	-1.450E+04	0.21%
M10	-1.455E+04	-1.459E+04	0.27%

The maximum percent difference between the compression forces of the leg members of preliminary guyed mast models of SAP2000 [14] and PLS-TOWER [15] software is 0.27%.

Table 3-5 Summary of the diagonal member forces of preliminary model 2

Member	SAP2000 (N)	PLS-TOWER (N)	% Difference
D1	-2.970E+02	-2.990E+02	0.67%
D2	-2.248E+02	-2.230E+02	0.80%
D3	-2.237E+02	-2.220E+02	0.76%
D4	-2.220E+02	-2.200E+02	0.90%
D5	-2.198E+02	-2.180E+02	0.82%
D6	-2.171E+02	-2.160E+02	0.51%
D7	-2.139E+02	-2.130E+02	0.42%
D8	-2.101E+02	-2.100E+02	0.05%
D9	-2.060E+02	-2.060E+02	0.00%
D10	-1.330E+03	-1.339E+03	0.67%

The maximum percent difference between the compression forces of the diagonal members of two preliminary guyed mast models of SAP2000 [14] and PLS-TOWER [15] software is 0.90%.

Table 3-6 Summary of the horizontal member forces of preliminary model 2

Member	SAP2000 (N)	PLS-TOWER (N)	% Difference
H1	-1.909E+02	-1.900E+02	0.47%
H2	-1.693E+02	-1.690E+02	0.18%
H3	-1.684E+02	-1.680E+02	0.24%
H4	-1.671E+02	-1.660E+02	0.66%
H5	-1.654E+02	-1.640E+02	0.85%
H6	-1.632E+02	-1.620E+02	0.74%
H7	-1.607E+02	-1.600E+02	0.44%
H8	-1.579E+02	-1.570E+02	0.57%
H9	-1.546E+02	-1.540E+02	0.39%
H10	6.558E+03	6.577E+03	0.29%

The maximum percent difference between the maximum axial forces of the horizontal members of two preliminary guyed mast models of SAP2000 [14] and PLS-TOWER [15] software is 0.85%.

Table 3-7 Summary of the guy cable forces

Member	SAP2000 (N)	PLS-TOWER (N)	% Difference
Guy 1	1.713E+04	1.717E+04	0.23%
Guy 2	1.713E+04	1.717E+04	0.23%
Guy 3	1.099E+04	1.103E+04	0.36%

The maximum percent difference between the tension forces of the guy cables of two preliminary guyed mast models of SAP2000 [14] and PLS-TOWER [15] software is 0.36%.

According to the results obtained from the analysis of the preliminary model 2, it can be concluded that the two software, SAP2000 [14] and PLS-TOWER [15] produce the same results for the guyed masts with an error in acceptable limits.

3.1.3 Validation Studies Conclusion

In order to verify the consistency of the results of both SAP2000 [14] and PLS-TOWER [15] software, two simple preliminary models were examined with both analysis tools which were used throughout this research. The consistency of the results were validated for both SAP2000 [14] and PLS-TOWER [15] programs by using a cable and a guyed mast as preliminary models. Thus more complex analyses could be performed using both software as analysis tools with enough confidence in the consistency of their outputs.

CHAPTER 4

EFFICIENCY CHECK OF ANALYSIS SOFTWARE

4.1 NUMERICAL RESULTS OF TEST MODELS

The nonlinear and complex behavior of the guyed masts must be modeled correctly for obtaining useful results from the analysis. Therefore the accuracy of the analysis methods of SAP2000 [14] and PLS-TOWER [15] software were investigated. In order to check the efficiency of the software used throughout this research, some simple test models solved in the study of Polat [5] were analyzed by both software and the results were compared.

Since in Chapter three it was validated that the analysis results of both SAP2000 [14] and PLS-TOWER [15] software are consistent, during the efficiency check of the analysis methods of the software in this chapter only the SAP2000 [14] software was used.

4.1.1 Test Model 1: Cable Example

By analyzing an example solved by Polat [5], the nonlinear force deflection relationship of the cable elements of structural analysis software used throughout this research were investigated.

The cable shown in Figure 4.1 with an undeformed length of 381m is subjected to its own weight which is 23.34 N/m. The axial stiffness of the cable is 3.56e7 N. The summary of the material properties and the loading details of the test model 1 can be found in Table 4.1.

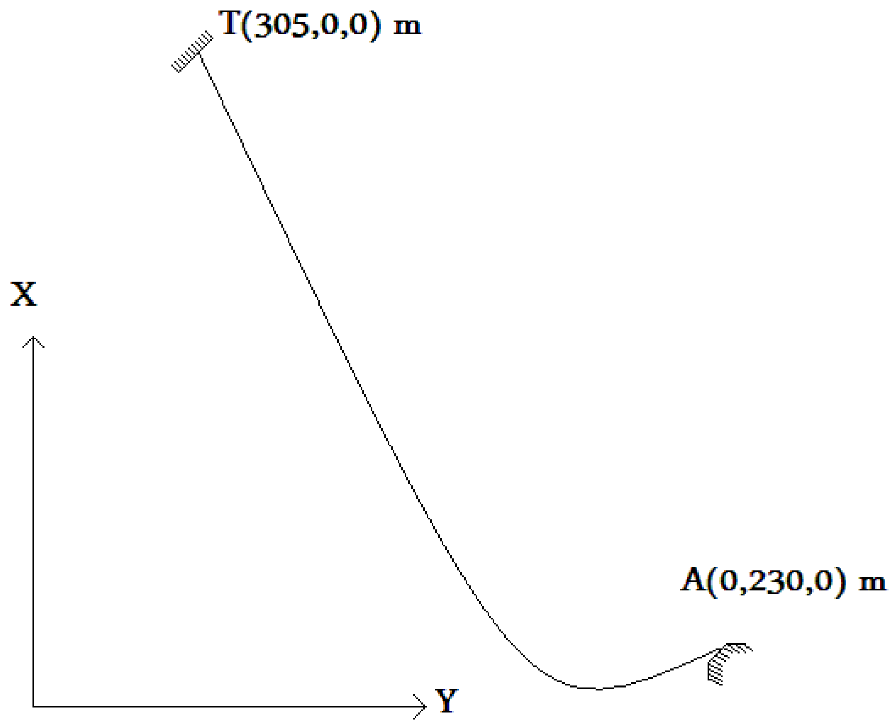


Figure 4.1 Test model 1

Table 4-1 Material properties and loading details of the test model 1

Cable Properties	
Undeformed Length	381 m
E.A	3.56E+7 N
Cable Loading	
Self Weight	23.34 N/m

The support reactions obtained at point “A” by analyzing the test cable both with SAP2000 [14] software were compared with the results obtained by Polat [5] in Table 4.2.

Table 4-2 Support reactions of the test model 1 at anchor point “A”

Force	Polat [5] (N)	SAP2000 [14] (N)	% Difference
Fx	45600	45628	0.06%
Fy	-56097	-56136	0.07%
Fz	0	0	0.00%

The results obtained clearly show that the outputs of the analysis software show good agreement with the results obtained by Polat [5].

4.1.2 Test Model 2: Cable Array

The second test model is a cable array. The purpose of analyzing this example is to investigate the modeling of a cable array in the analysis software used throughout this research.

The array shown in Figure 4.2 is composed of two identical cables with modulus of elasticity of 1.0e10 kg/m^2, dead weight of 10.0 kg/m and cross sectional area of 0.0002 m^2. The unstressed length of both of the cables is 100.0 m. The initial pretension of the cables is 10 000 kg. The properties of the cable array system are summarized in Table 4.3.

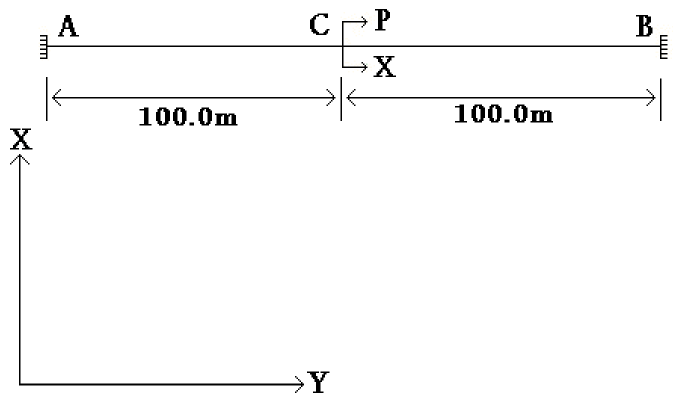


Figure 4.2 Test model 2

Table 4-3 Material properties and loading details of the test model 2

Cable Properties	
Modulus of Elasticity	1.0E+10 kg/m^2
Sectional Area	0.0002 m^2
Unstressed Length	100.0 m
Cable Loading	
Self Weight	10.0 kg/m
Initial Pretension	10 000 kg

Comparison of results of the analysis software with the results found by Polat [5] is presented in Figure 4.3.

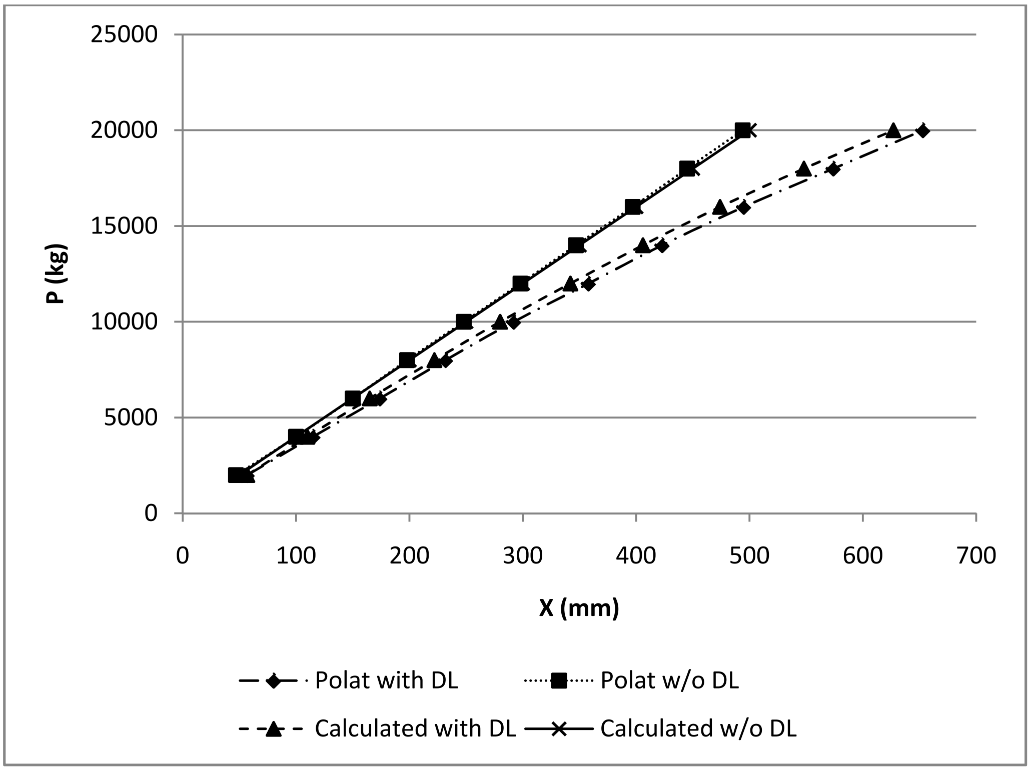


Figure 4.3 Comparison of the results of test model 2

When the self weight of the cables is not taken into account the outputs of the analysis software shows good agreement with the results calculated by Polat [5]. However, when the dead load of the cables are considered there is an upwards shift in the force deflection graph of the cable array.

4.1.3 Test Model 3: Guyed Mast

For testing the guyed mast modeling capability of the analysis software an example which was a rough two dimensional simulation of a guyed mast was solved. The same example is also solved by Polat [5].

The test model 3 is a two dimensional guyed mast with 457 m height. It is pinned at the base and supported by two guy cables on either side with guy radius of 792 m. Diameter of the cables were assumed to be 180 mm. The schematic of the test model 3 is shown in Figure 4.4.

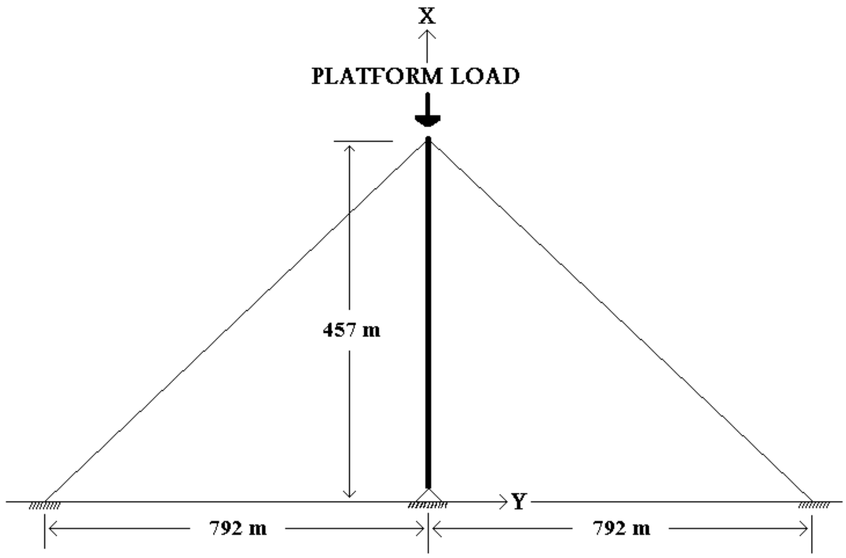


Figure 4.4 The schematic of test model 3

For simulating a platform, the top of the mast is loaded by two point loads with 67e6 N in the negative x direction and 1.1e6 N in the positive y direction which are gravitational load and wind drag acting on the platform respectively. Also the gravitational load per unit length of the mast and the cables are 2918 N/m and 729 N/m respectively. The stiffness properties of the mast and the cables were summarized in Table 4.4.

Table 4-4 Stiffness properties of test model 3

Mast Properties	
Axial Stiffness	5.10E+11 N/m
Shear Stiffness	1.46E+11 N/m
Bending Stiffness	5.10E+11 N/m
Cable Properties	
Axial Stiffness	0.70E+10 N/m
Shear Stiffness	0 N/m
Bending Stiffness	0 N/m

Comparison of results of the analysis software with the results found by Polat [5] is presented in Table 4.5.

Table 4-5 Comparison of the results of test model 3 with Polat [5]

Moment (N/m) and Force (N)	**Polat [5]**	**Calc.**	**% Difference**
Max. Mast moment with first order eff.	12.0937	12.0938	0.08%
Max. Mast moment with total eff.	14.4600	13.2200	8.57%
Max. Tension in the Cables with total eff.	0.7570	0.9280	18.43%

As can be seen in the results although the outputs of the analysis software is efficient when first order effects are taken into account, the results of the analysis software shows slight differences from the results obtained by Polat [5] when total effects are considered during the analysis. Also the calculated maximum tension values in the nonlinear cable elements are different from the tension values calculated by Polat [5].

4.1.4 Efficiency Check of Analysis Software Conclusion

The analysis software used throughout this study were tested in order to check the accuracy and efficiency of the modeling and analysis methods of those software by analyzing three incomplex test models which were also solved by Polat [5]. According to the results obtained, with simple test models outputs of the analysis software showed good agreement with the results obtained in literature. Also, during the analysis process when only the first order effects were taken into account the results obtained by analysis software were consistent with the results in literature.

However, when the total effects were taken into account the outputs of the analysis software showed some differences from the results in literature. This mainly stems from the algorithms used by analysis programs. According to the results obtained from three primitive incomplex test models, the behavior generated by the analysis software does not represent the nonlinear cable behavior. SAP2000 [14] and PLS-TOWER [15] software use approximate approaches for nonlinear problems.

CHAPTER 5

DESCRIPTION AND ANALYSIS OF MASTS

5.1 PROPERTIES OF MASTS USED FOR ANALYSIS

Rather than using fictitious models, for a more realistic representation three guyed masts designed according to the current standards were used for the analysis. The details of the structures were supplied by a firm that has designed and manufactured a great number of steel structures for different sectors including guyed masts for telecommunication and broadcasting industry in all around the world. For confidentiality reasons, detailed information of the masts was not presented.

5.1.1 Geometrical Properties of Masts

All three models are triangular lattice in form with the heights of 30 meters, 60 meters and 100 meters. The general geometrical information of the masts is summarized in Table 5.1. Also, the schematics of the masts are shown in Figure A-1, Figure A-2 and Figure A-3 of Appendix A-1.

Table 5-1 Geometrical information of masts under investigation

Mast	No. Of Guy levels	No. Of Guy Anchors	Maximum Span length (m)	Maximum Support Distance (m)
30m Mast	3	3	10	20
60m Mast	3	6	20	50
100m Mast	5	6	20	90

5.1.2 Mass Properties of Masts

The total mass and mass distribution properties of the masts are important in determination of the total earthquake load that the structures will be exposed to and the vibration modes of the masts. The mass distribution information of the masts is summarized in Table 5.2.

Table 5-2 Mass distribution information of masts under investigation

Mast	Body Mass		Cable Mass		Total Mass (kg)
	kg	%	kg	%	
30m Mast	696	69.74%	302	30.26%	998
60m Mast	4289	88.74%	544	11.26%	4833
100m Mast	10087	87.72%	1412	12.28%	11499

5.1.3 Guy Pretensions

For all of the masts, the pretensile force in the guy cables are 10% of the ultimate tension. According to the ANSI/TIA-EIA-G Standard [19], the upper and lower limits of the initial tension in the guy cables are 15% and 7% of the breaking strength of the guys respectively. All the guy cables with initial tension below the 7% limit are considered as slack. Similarly, guy cables with an initial tension higher than the 15% limit are under the risk of fatigue failure when subjected to cyclic loading. The initial tension information of the guy cables are summarized in Table 5.3.

Table 5-3 Initial pretension information of guy cables of masts under investigation

Mast		30m Mast	60m Mast	100m Mast
Guy Level 1	Slope (deg)	26.97	34.21	22.03
	Initial tension (kN)	17.88	18.15	9.27
Guy Level 2	Slope (deg)	45.50	53.66	38.98
	Initial tension (kN)	17.88	18.15	13.34
Guy Level 3	Slope (deg)	55.87	50.52	33.86
	Initial tension (kN)	17.88	23.74	13.34
Guy Level 4	Slope (deg)	-	-	41.82
	Initial tension (kN)	-	-	23.74
Guy Level 5	Slope (deg)	-	-	48.20
	Initial tension (kN)	-	-	23.74

5.1.4 Structure and Site Specific Properties of Masts

The structure and site specific properties of masts under investigation were set to the defaults according to the ANSI/TIA-EIA-G Standard [19]. The structure classification, exposure category and topographic categories of the guyed masts were assumed to be class II, exposure C and topographic category I respectively.

5.2 ANALYSIS UNDER EARTHQUAKE LOADING

5.2.1 Modeling Assumptions

The structural analysis program SAP2000 is a software package developed by Computers and Structures Inc. [14]. Modeling and analysis procedure of the software is based on the finite element method.

The bodies of the guyed masts were modeled as three dimensional truss models where the members of the structure are modeled as frame elements hinged at the joints, only producing axial forces.

The guy wires were modeled with the cable element of the SAP2000 [14] software. The initial pretension forces on the cables were introduced by applying a temperature load to the guy elements. For obtaining the correct pretension force on the cables, the temperature loads on the guys were adjusted iteratively.

ANSI/TIA-EIA-G Standard [19] states that for nonlinear time history analysis structural damping of 5% shall be used. Thus, all guyed mast models damping ratio was set to 5%.

Since base of the all the masts are pin connected, the mast bases were modeled as resistant to motion, but free to rotate about the X, Y and Z axes. The motions of the guy anchors were assumed to be synchronized with the base of the mast. For very tall guyed masts, the distance between the base of the tower and the guy anchor points increases, which results in the out of phase excitations. However, according to the ANSI/TIA-EIA-G Standard [19], out of phase excitation of the guy anchor points shall be included in the analysis, when the distance between the base of the tower to a guy anchor point exceeds 300 meters. Since for the guyed masts under investigation the maximum support distance is 90 meters, the out of phase excitations of the guy anchor points were not taken into consideration.

During the earthquake loading analysis the mass and weight of the 50% of the design ice was taken into account. 50% of the design ice is an ice load with 0.75cm thickness and 8800N/m^3 density. Also the mass and weight of the cables and ladder inside the tower and antennas on the tower were taken into account during the nonlinear time history analysis.

The following loading combinations defined in equations (5.1) and (5.2) were used during the earthquake loading analysis. Those combinations are also defined in ANSI/TIA-EIA-G Standard [19].

$$1.2D + 1.0D_g + 1.0E \tag{5.1}$$

$$0.9D + 1.0D_g + 1.0E \tag{5.2}$$

where

D : Dead loading of structure and appurtenances, excluding guy assemblies

D_g : Dead load of guy assemblies

E : Earthquake load

5.2.2 Time History Records

Three earthquake records were selected from the magnitude range 6.9 to 7.3 and distance range 11.2 to 28.2 km. 1992 Landers earthquake recorded at Joshua tree, 1989 Loma Prieta earthquake recorded at Gilroy array and Hollister city hall were used in the analysis. Two orthogonal horizontal time history records of each of those events were downloaded from the Pacific Engineering and Earthquake Council's website [28]. Figures 5.1 to 5.6 show the orthogonal horizontal accelograms used for the analysis.

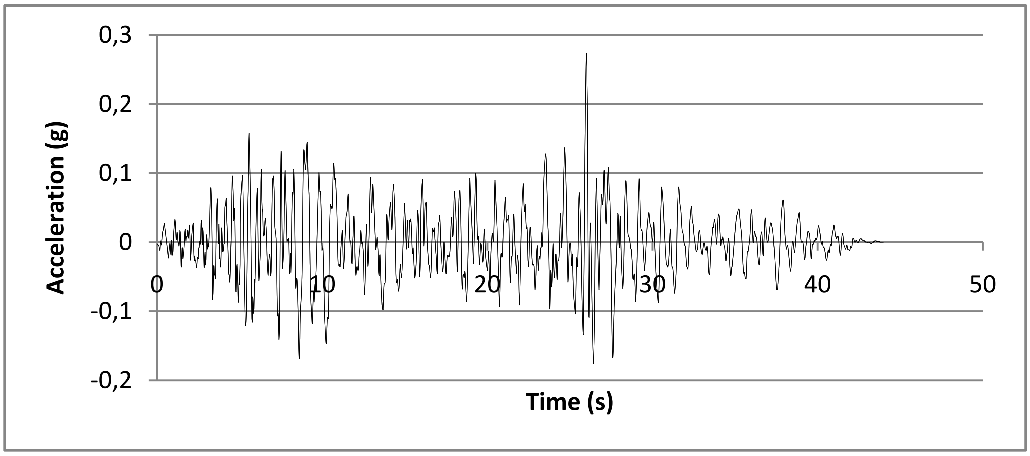


Figure 5.1 Acceleration Time History for Landers at Joshua tree (0 degrees)

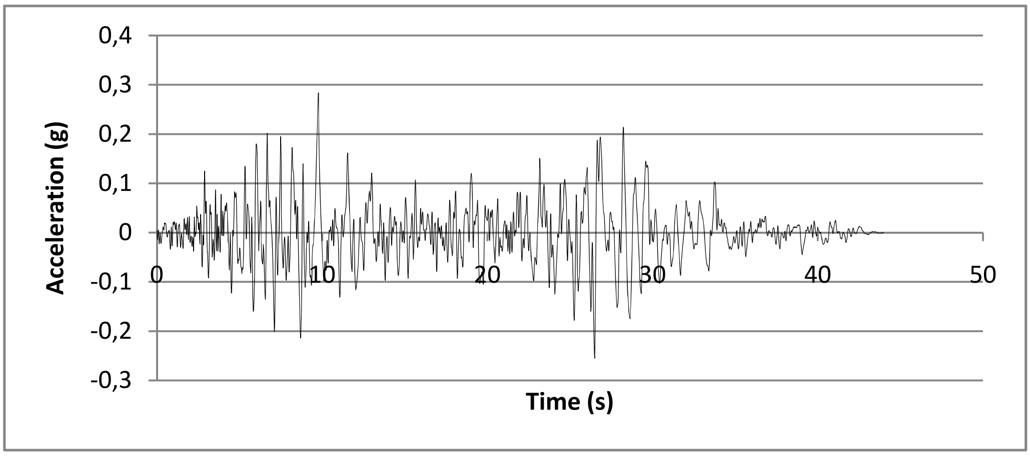


Figure 5.2 Acceleration Time History for Landers at Joshua tree (90 degrees)

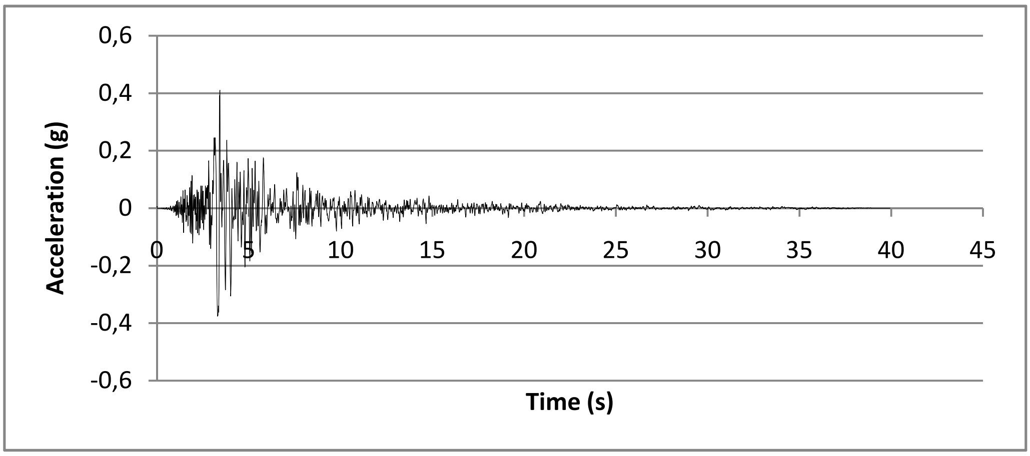


Figure 5.3 Acceleration Time History for Loma Prieta at Gilroy (0 degrees)

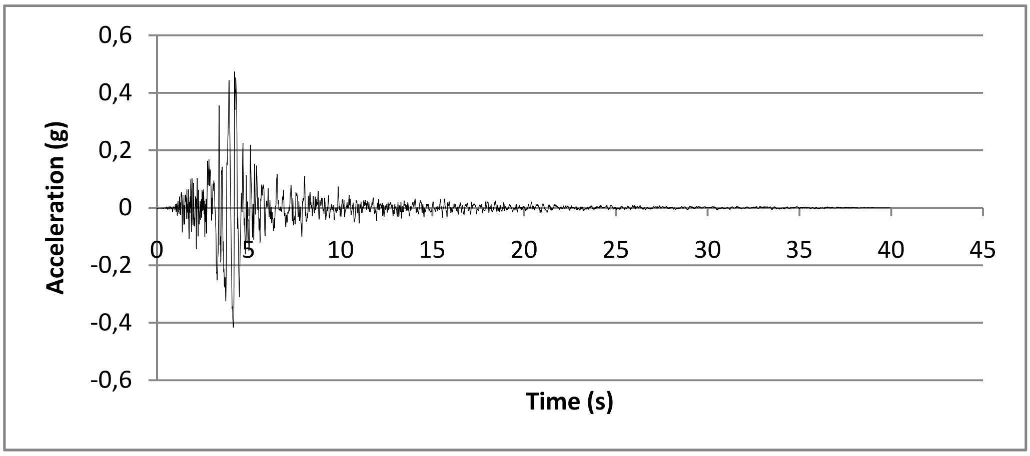


Figure 5.4 Acceleration Time History for Loma Prieta at Gilroy (90 degrees)

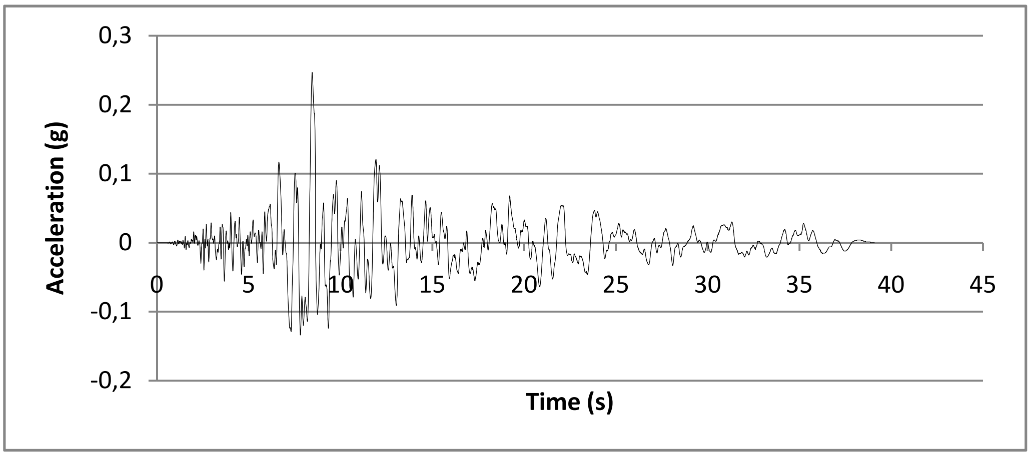


Figure 5.5 Acceleration Time History for Loma Prieta at Hollister (90 degrees)

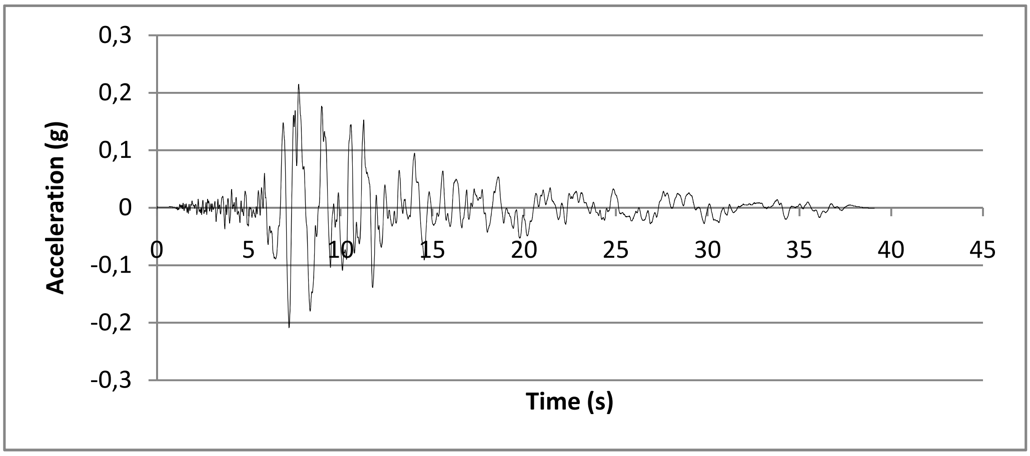


Figure 5.6 Acceleration Time History for Loma Prieta at Hollister (180 degrees)

In order to scale the time history records of the events, a design response spectrum for soil type B was generated following the provisions of ANSI/TIA-EIA-G Standard [19]. The main parameters that are used during the formation of the design response spectrum are spectral response acceleration at short periods and maximum considered earthquake spectral response acceleration at

1 second which were assumed to be as 2.50g and 0.60g respectively. The design response spectrum generated is presented in Figure 5.7.

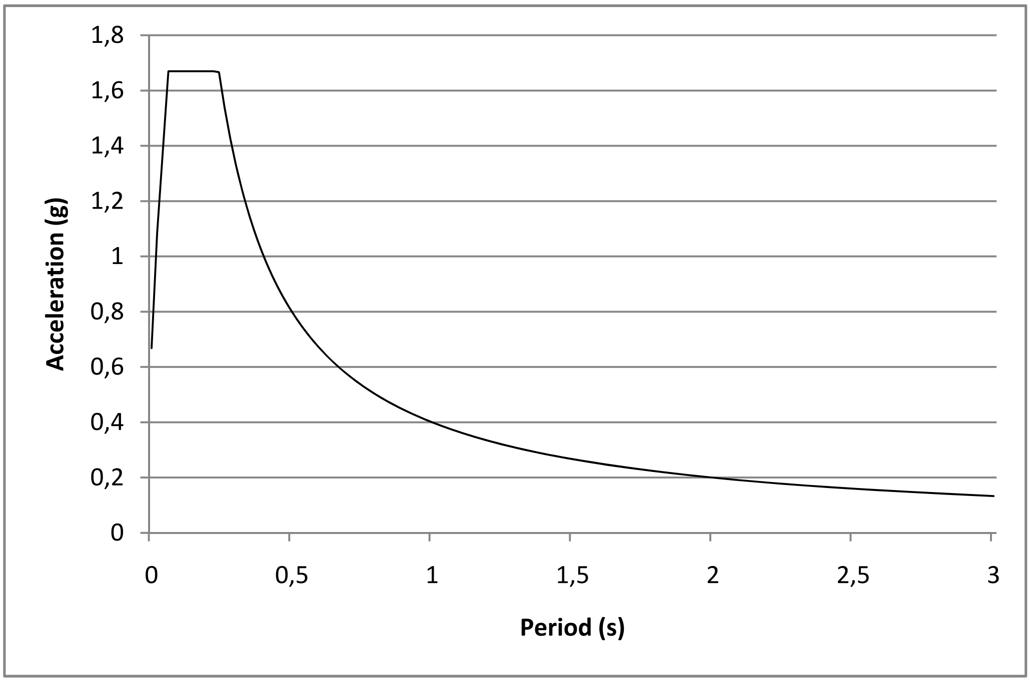


Figure 5.7 ANSI/TIA-EIA-G Design response spectrum for site class B

The time history records response spectrums, for both horizontal components, with 5% damping ratio were constructed with the help of Seismosignal software [29]. For each event, both of the response spectrums were combined using the square root of sum squares (SRSS) method. The combined response spectrums are shown in Figures 5.8, 5.9 and 5.10.

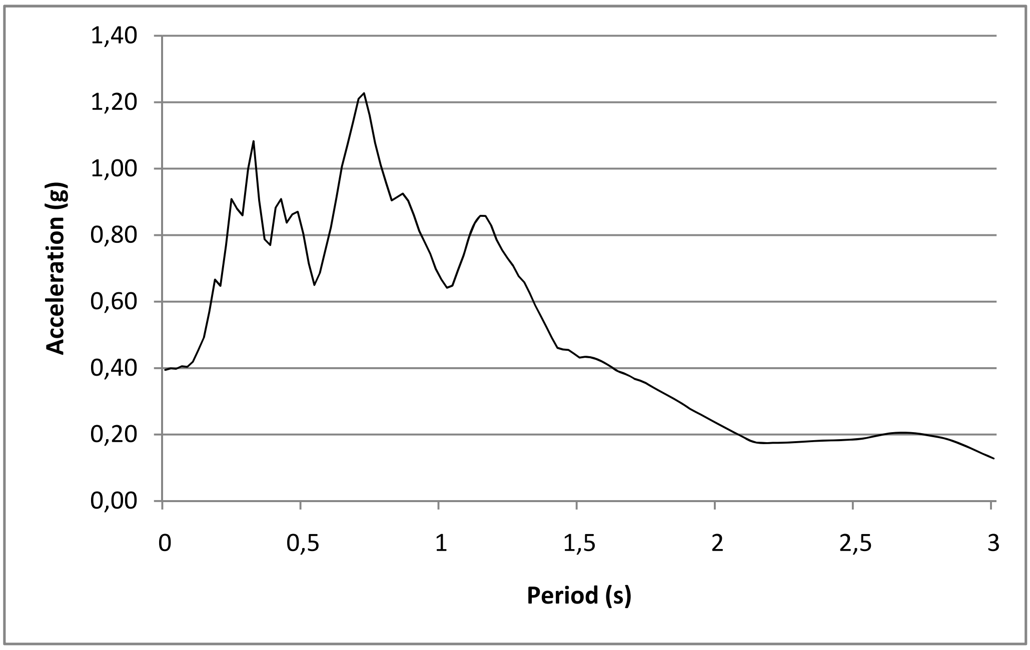


Figure 5.8 Combined response spectrum with 5% damping for Landers at Joshua tree

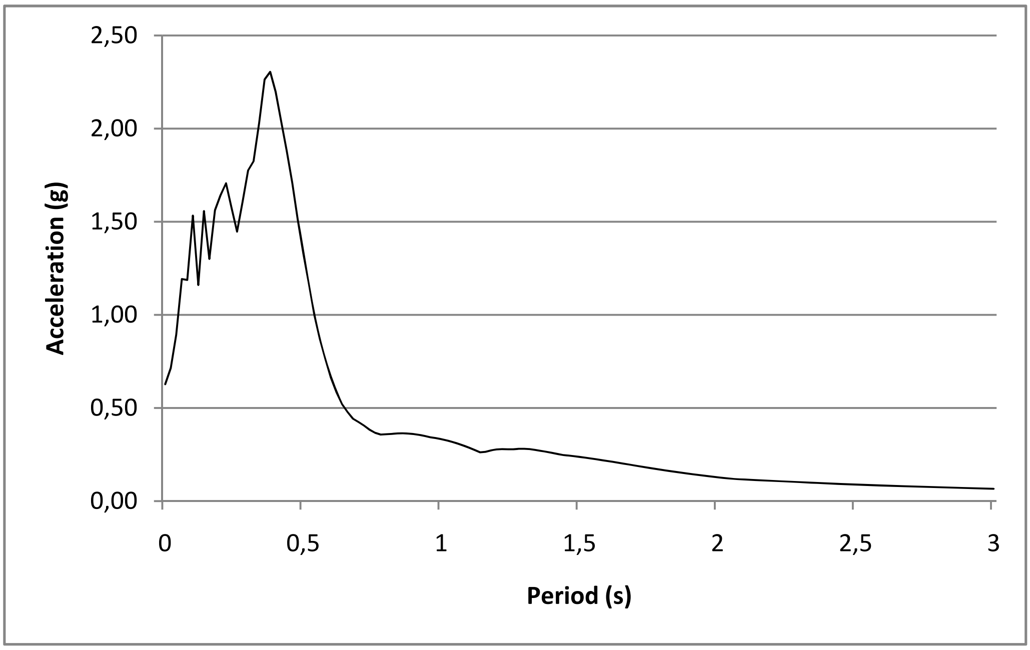


Figure 5.9 Combined response spectrum with 5% damping for Loma Prieta at Gilroy

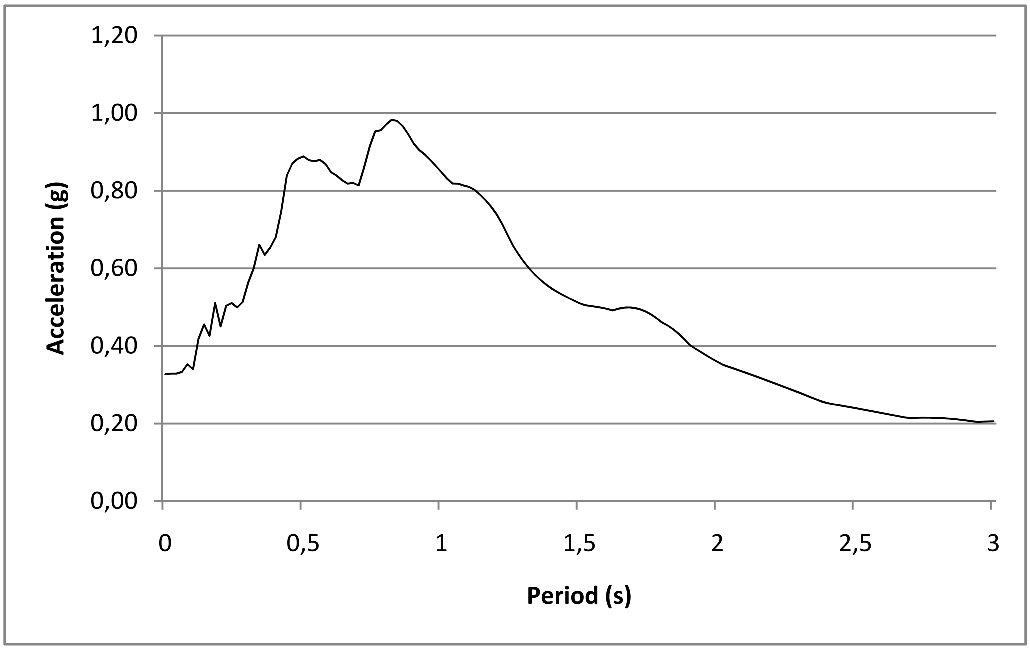


Figure 5.10 Combined response spectrum with 5% damping for Loma Prieta at Hollister

The combined response spectrums were scaled such that the average combined spectrum was not less than 10% of the 1.3 times the design response spectrum for site class B in the range of periods 0.2T and 1.5T, where T is the fundamental period of the structure. The selected scaling factors of each event for each guyed mast are shown in Table 5.4.

Table 5-4 Selected scaling factors for time history records

	Landers at Joshua tree	Loma Prieta at Gilroy	Loma Prieta at Hollister
30m Mast	2.69	3.64	2.83
60m Mast	1.89	2.36	1.73
100m Mast	1.08	0.92	1.32

5.2.3 Nonlinear Time History Analysis

The guyed masts under investigation were analyzed under the effect of earthquake loading by using nonlinear time history analysis method. Based on the stiffness at the end of the temperature load case for pretensioning the guy cables, a linear modal analysis was performed using Ritz vector analysis. The total mass participation percentage to be achieved was set to 99%. For most of the structures, the seismic behavior is governed by the first few modes of the structure. However in the case of guyed masts, the modal mass participation is scattered through many modes which makes the determination of the important modes of the structure extremely difficult. The fundamental natural periods, fundamental natural frequencies and the total calculated number of modes of the guyed masts are presented in Table 5.5.

Table 5-5 Summary of the results of the Modal Analysis

Mast	Fundamental Natural Frequency (Hz)	Fundamental Natural Period (s)	No. Of Modes Calculated
30m Mast	4.55	0.22	42
60m Mast	1.02	0.98	49
100m Mast	0.55	1.83	69

A nonlinear modal time history analysis was performed based on the modes calculated by Ritz vector analysis. Both scaled horizontal components of each acceleration time history were acted to all supports of the structure simultaneously. The resultant load effects are determined by selecting the maximum values from the nonlinear time history analyses.

5.2.4 Results from Earthquake Analysis

Three guyed masts were analyzed under effect of earthquake loading with nonlinear time history analyses using the SAP2000 [14] tool. The response of the structures to nonlinear time history analysis conducted with three sets of accelograms is presented in this section. Since each of the guyed mast models are three dimensional trusses where the structural members were modeled as frame elements only producing axial forces, compression and tension forces of mast body elements and guys is presented.

According to the results of the earthquake analysis, the most critical axial forces generated in the leg members, diagonal members, horizontal members and guys of the masts under investigation are presented in Figure 5.11 to Figure 5.19 and Table 5.6 to Table 5.8.

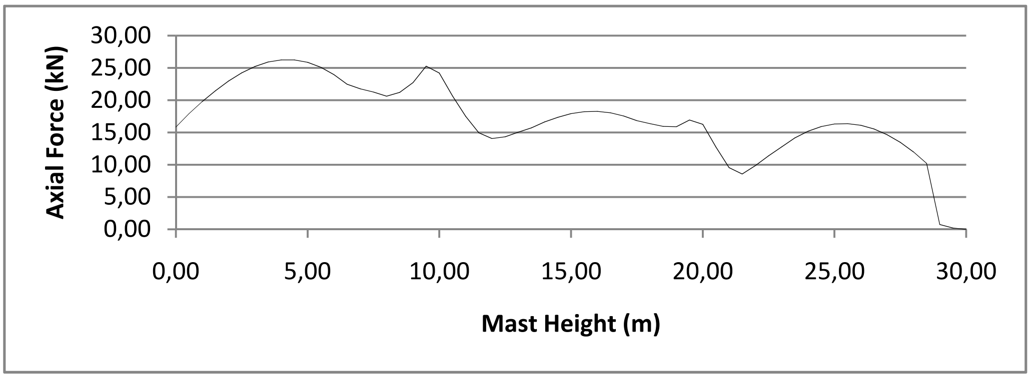


Figure 5.11 Earthquake Loading - Compression forces in the leg members of 30m mast

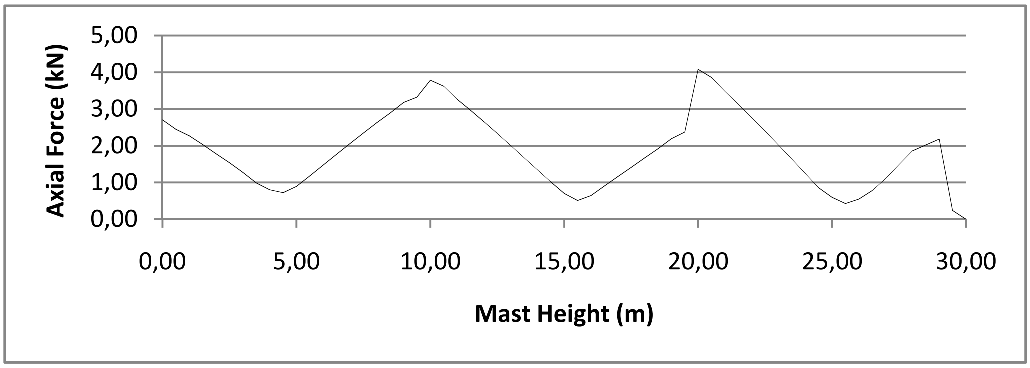


Figure 5.12 Earthquake Loading - Compression forces in the diagonal members of 30m mast

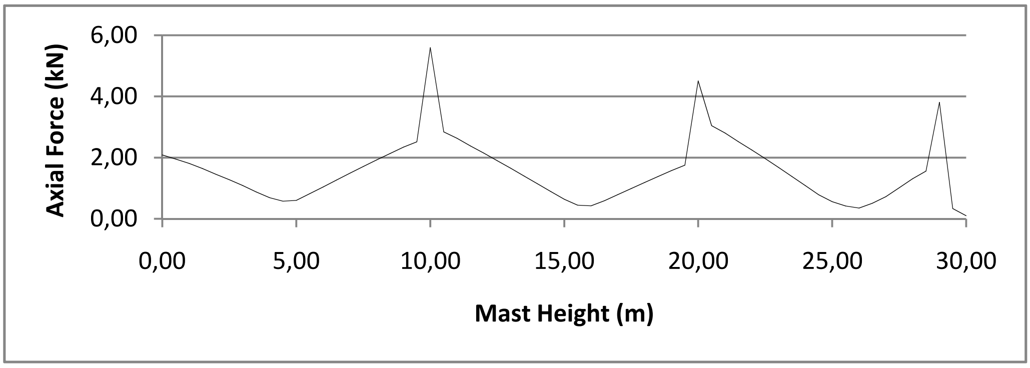


Figure 5.13 Earthquake Loading - Tension forces in the horizontal members of 30m mast

Table 5-6 Earthquake Loading - Tension forces in the guy cables of 30m mast

Guys	Tension Force (kN)
Guy Level 1	1.258E+01
Guy Level 2	1.344E+01
Guy Level 3	1.169E+01

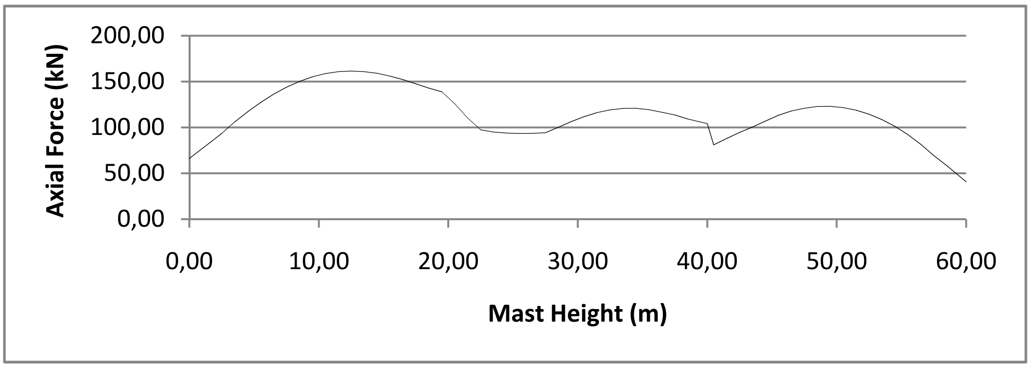


Figure 5.14 Earthquake Loading - Compression forces in the leg members of 60m mast

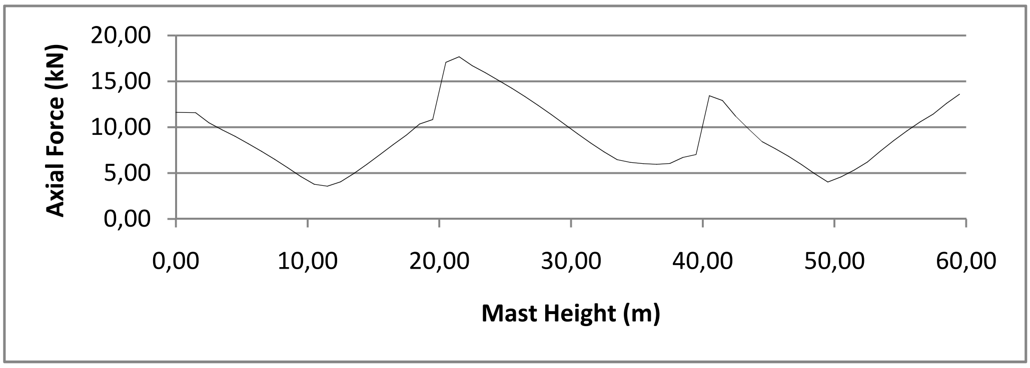


Figure 5.15 Earthquake Loading - Compression forces in the diagonal members of 60m mast

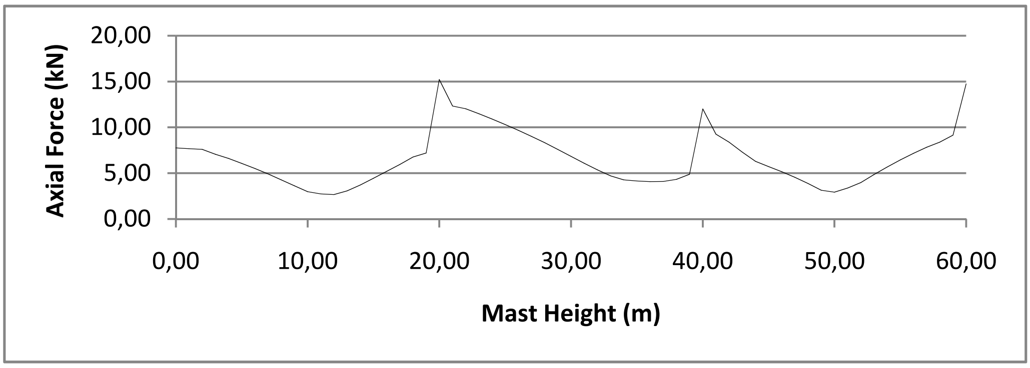


Figure 5.16 Earthquake Loading - Tension forces in the horizontal members of 60m mast

Table 5-7 Earthquake Loading - Tension forces in the guy cables of 60m mast

Guys	Tension Force (kN)
Guy Level 1	3.722E+01
Guy Level 2	4.202E+01
Guy Level 3	4.105E+01

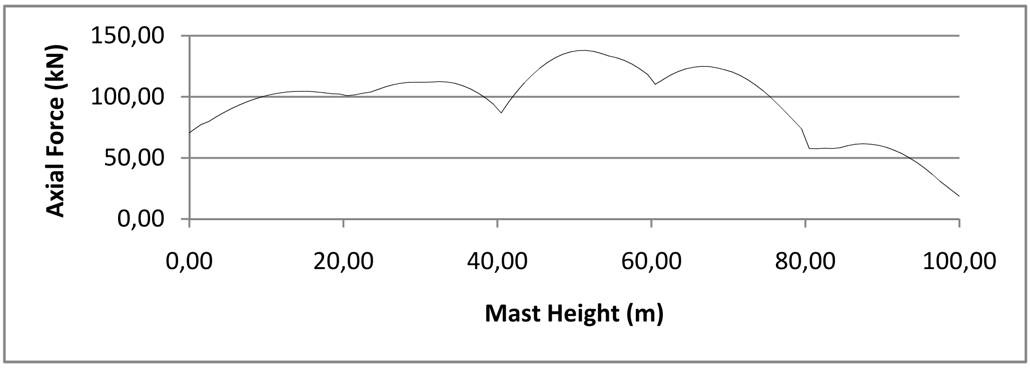


Figure 5.17 Earthquake Loading - Compression forces in the leg members of 100m mast

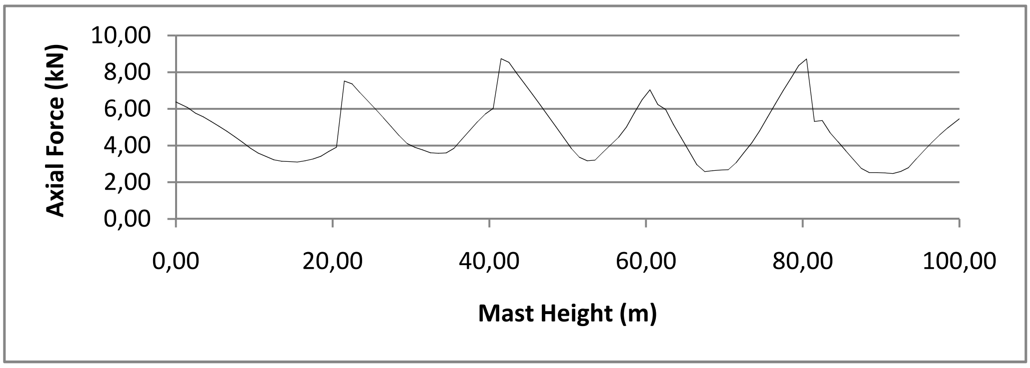


Figure 5.18 Earthquake Loading - Compression forces in the diagonal members of 100m mast

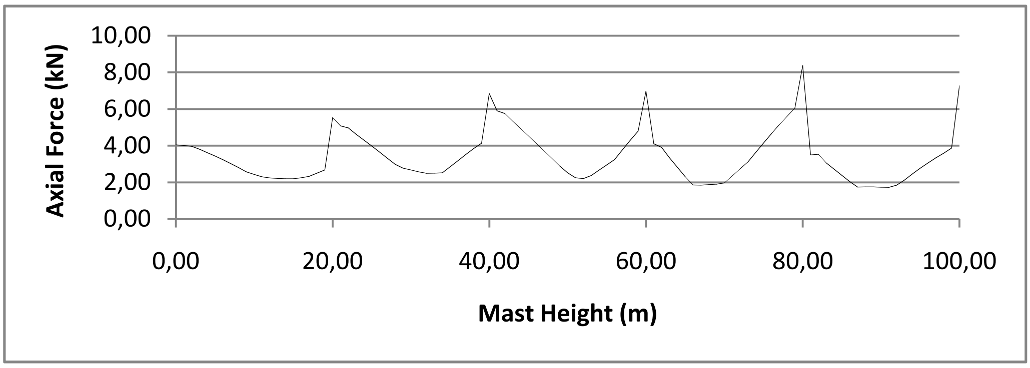


Figure 5.19 Earthquake Loading - Tension forces in the horizontal members of 100m mast

Table 5-8 Earthquake Loading - Tension forces in the guy cables of 100m mast

Guys	Tension Force (kN)
Guy Level 1	9.882E+00
Guy Level 2	1.618E+01
Guy Level 3	1.893E+01
Guy Level 4	2.720E+01
Guy Level 5	2.008E+01

Maximum compression forces in the leg members of the masts under investigation were produced generally in the middle of the spans. However when the horizontal stiffness introduced by the cables were high, the maximum compression forces in the leg members increased sharply in the stay connection points.

On the other hand, diagonal and horizontal member's maximum axial forces were occurred in the cable connection points because of the horizontal forces introduced by the cables at those points.

The maximum axial forces on the leg members of the 30m mast and 60m mast at lower spans were larger than the forces on the leg members at the higher spans. On the other hand, the distribution of leg forces in the highest mast, 100m mast was irregular. The maximum axial forces on the leg members of the highest mast were generated in the middle spans implying that some higher modes were dominant in the response of the 100m mast.

5.3 ANALYSIS UNDER WIND AND ICE LOADING

5.3.1 Modeling Assumptions

PLS-TOWER [15] is structural analysis software used for the design of guyed or free-standing lattice transmission structures [26]. Like SAP2000 [14] software, PLS-TOWER [15] uses finite element method for the analysis of the structures.

Since the PLS-TOWER [15] software is a tool used for analysis and design of lattice structures, the only option to model the bodies of the guyed masts were defining them as three dimensional truss models where all the members of the structure hinged at the joints, only producing axial forces.

The cables of the guyed masts were modeled with PLS-TOWER [15] software's cable element. The pretension forces in the cables were introduced by using the pretensioning option of the PLS-TOWER [15] software.

Since all of the guyed mast bases under investigation are pin connected, the guyed masts were modeled as free to rotate about X, Y and Z axes, but resistant to motion in all directions at the base.

The following loading combinations defined in equations (5.3), (5.4) and (5.5) were used during the wind and ice loading analysis. Those combinations are also defined in ANSI/TIA-EIA-G Standard [19].

$$1.2D + 1.0D_g + 1.6W_o \tag{5.3}$$

$$0.9D + 1.0D_g + 1.6W_o \tag{5.4}$$

$$1.2D + 1.0D_g + 1.0D_i + 1.0W_i + 1.0T_i \tag{5.5}$$

where

D : Dead loading of structure and appurtenances, excluding guy assemblies

D_g : Dead load of guy assemblies

W_o : Wind load without ice

D_i : Weight of ice due to factored ice thickness

W_i : Concurrent wind load with factored ice thickness

T_i : Load effects due to tempreature

5.3.2 Nonlinear Wind Loading and Ice Loading Analysis

For the wind loading condition, the guyed masts under investigation were analyzed nonlinearly under the effect of 120kph, 145kph, 160kph and 180kph 3 second gust wind speeds with the pattern loading method of ANSI/TIA-EIA-G Standard [19]. For the ice loading condition, the thickness of the ice accumulated around the members of the structures was assumed to be 1.5cm and according to the ANSI/TIA-EIA-G Standard [19] density of the ice was taken as 8800N/m^3.

For both wind loading and ice loading conditions the projected area of the cables and ladder inside the tower were taken into account. The projected area of the antennas was not taken into account because of their negligible wind drag area. The accumulated ice on the cables and ladder inside the tower were not taken into account during the nonlinear ice loading analysis.

The equivalent wind speed values of 120kph, 145kph, 160kph and 180kph 3 second gust wind speeds of the wind loading condition were assumed to be 40.0kph, 48.3kph, 53.3kph and 60.0kph 3 second gust wind speeds respectively for the ice loading condition. Like the wind loading condition, for ice loading the wind forces on the structures were applied to the models with the pattern loading method of ANSI/TIA-EIA-G Standard [19]. For both wind loading and ice loading conditions, the wind forces were applied to the structure from the 0°, 60° and 90° directions as presented in Figure 5.20. Also, for ice loading condition according to the ANSI/TIA-EIA-G Standard [19], a temperature drop of 28°C was taken into account for all masts under investigation.

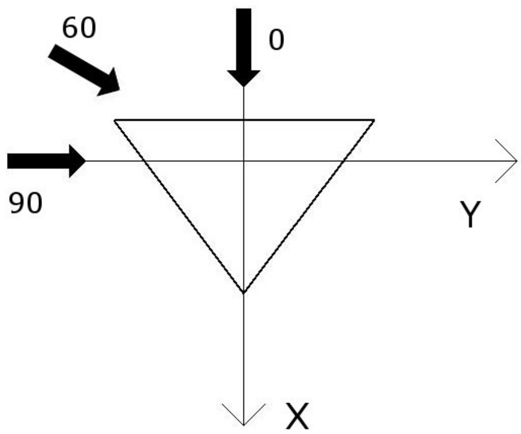


Figure 5.20 Wind loading orientations

5.3.3 Results from Wind Loading Analysis

The three guyed masts under investigation were analyzed under the effect of 120kph, 145kph, 160kph and 180kph wind speeds by using PLS-TOWER [15] software. The resultant load effects were determined by selecting the maximum values from the nonlinear wind loading analyses with different combinations of wind directions and wind loading patterns. Since the PLS-TOWER [15] software is a tool used for analysis and design of lattice structures, the only output of the software is the axial force of the members of the structure.

According to the wind loading analysis results with 120kph, 145kph, 160kph and 180kph wind speeds, the most critical axial forces obtained in the leg members, diagonal members, horizontal members and guys of masts under investigation are presented in Figure 5.21 to Figure 5.29 and Table 5.9 to Table 5.11.

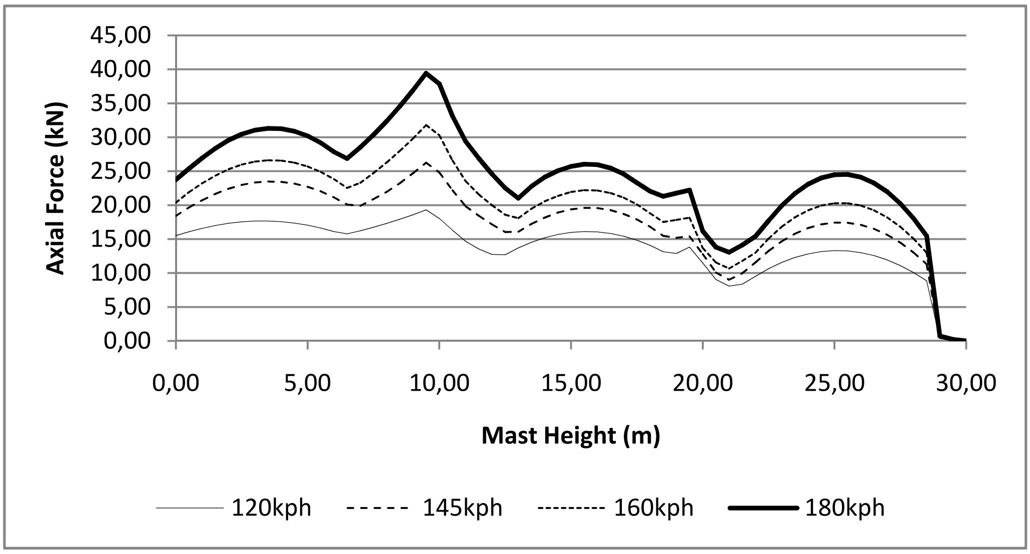


Figure 5.21 Wind Loading - Compression forces in the leg members of 30m mast

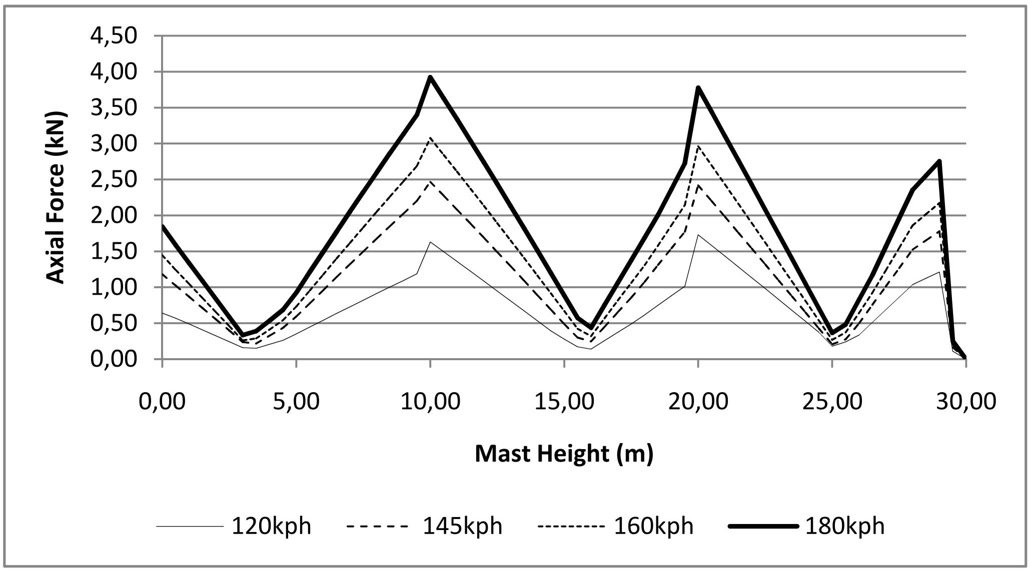


Figure 5.22 Wind Loading - Compression forces in the diagonal members of 30m mast

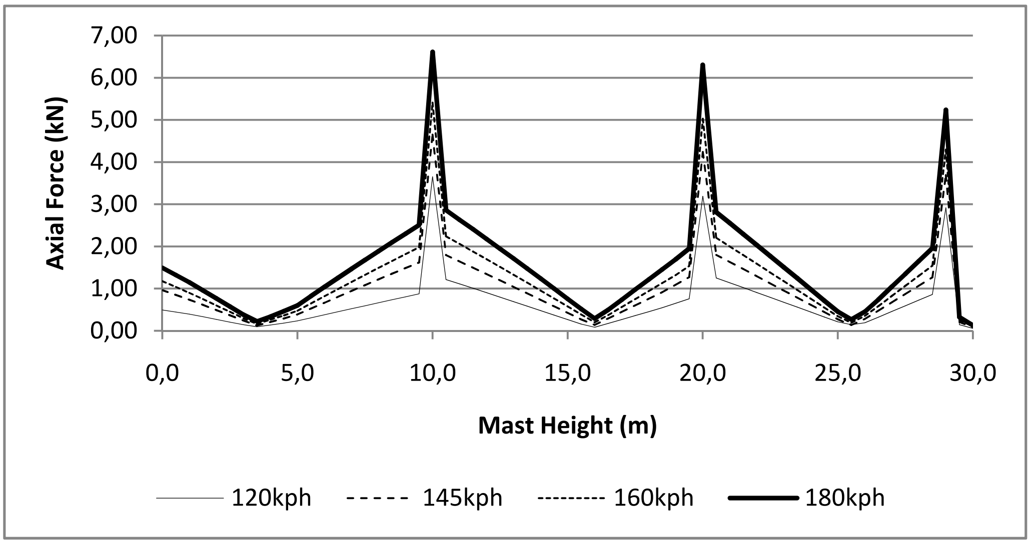


Figure 5.23 Wind Loading - Tension forces in the horizontal members of 30m mast

Table 5-9 Wind Loading - Tension forces in the guy cables of 30m mast

Guys	Tension Force in 120kph Wind Speed (kN)	Tension Force in 145kph Wind Speed (kN)	Tension Force in 160kph Wind Speed (kN)	Tension Force in 180kph Wind Speed (kN)
Guy Level 1	1.011E+01	1.294E+01	1.489E+01	1.775E+01
Guy Level 2	9.880E+00	1.324E+01	1.561E+01	1.915E+01
Guy Level 3	8.415E+00	1.113E+01	1.284E+01	1.542E+01

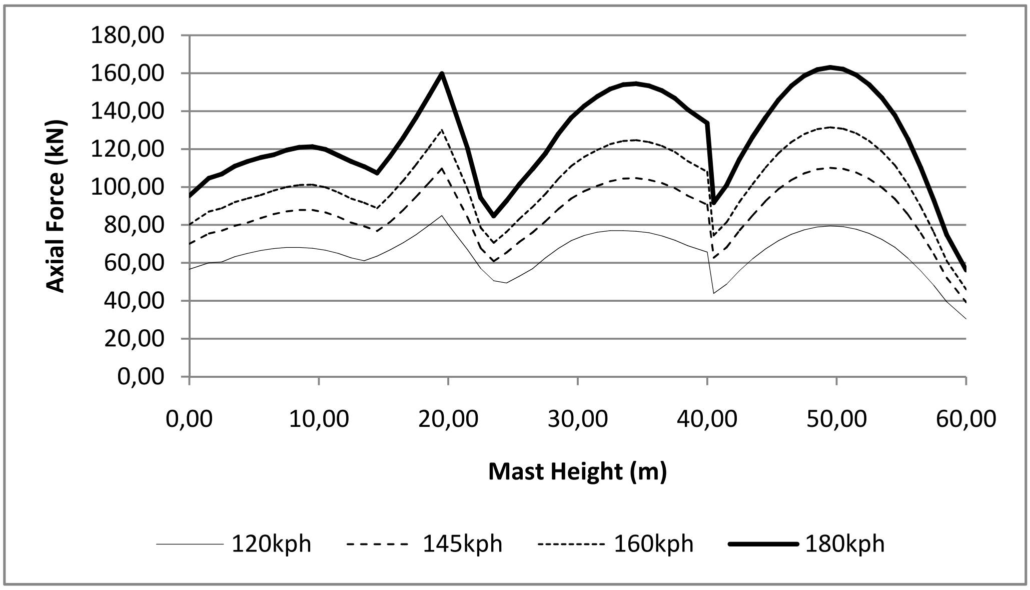


Figure 5.24 Wind Loading - Compression forces in the leg members of 60m mast

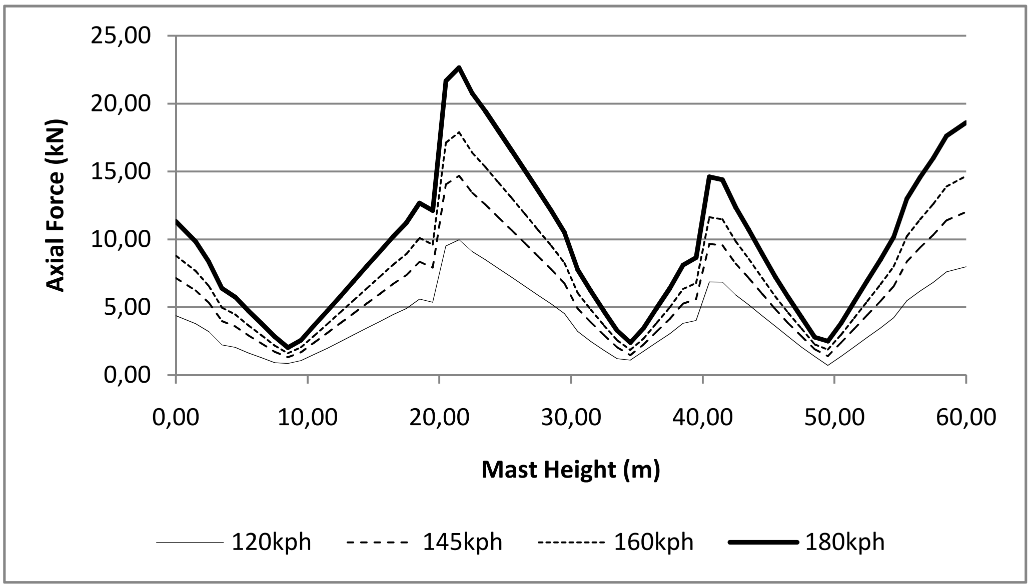


Figure 5.25 Wind Loading - Compression forces in the diagonal members of 60m mast

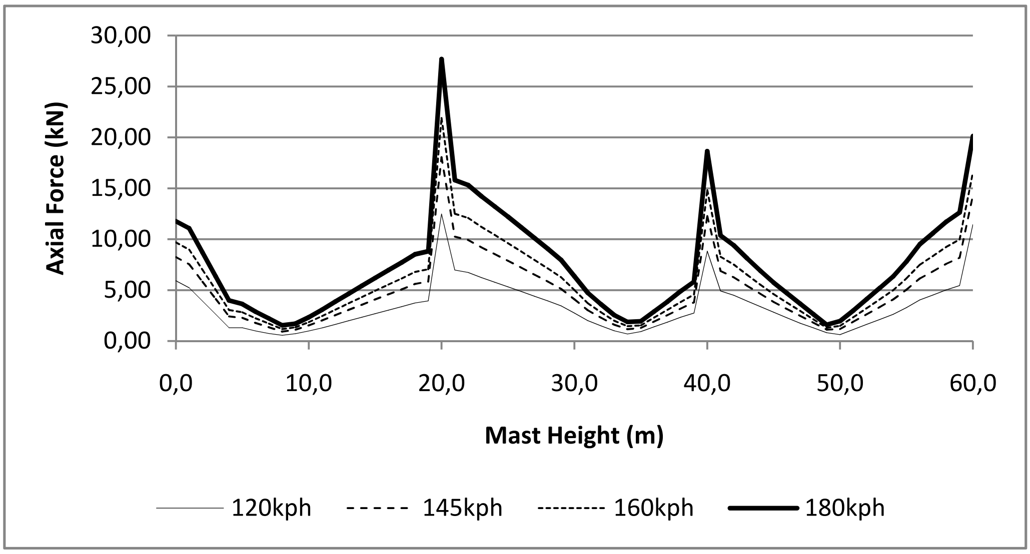


Figure 5.26 Wind Loading - Tension forces in the horizontal members of 60m mast

Table 5-10 Wind Loading - Tension forces in the guy cables of 60m mast

Guys	Tension Force in 120kph Wind Speed (kN)	Tension Force in 145kph Wind Speed (kN)	Tension Force in 160kph Wind Speed (kN)	Tension Force in 180kph Wind Speed (kN)
Guy Level 1	3.241E+01	4.044E+01	4.623E+01	5.492E+01
Guy Level 2	3.106E+01	4.132E+01	4.860E+01	6.033E+01
Guy Level 3	2.926E+01	4.139E+01	4.994E+01	6.269E+01

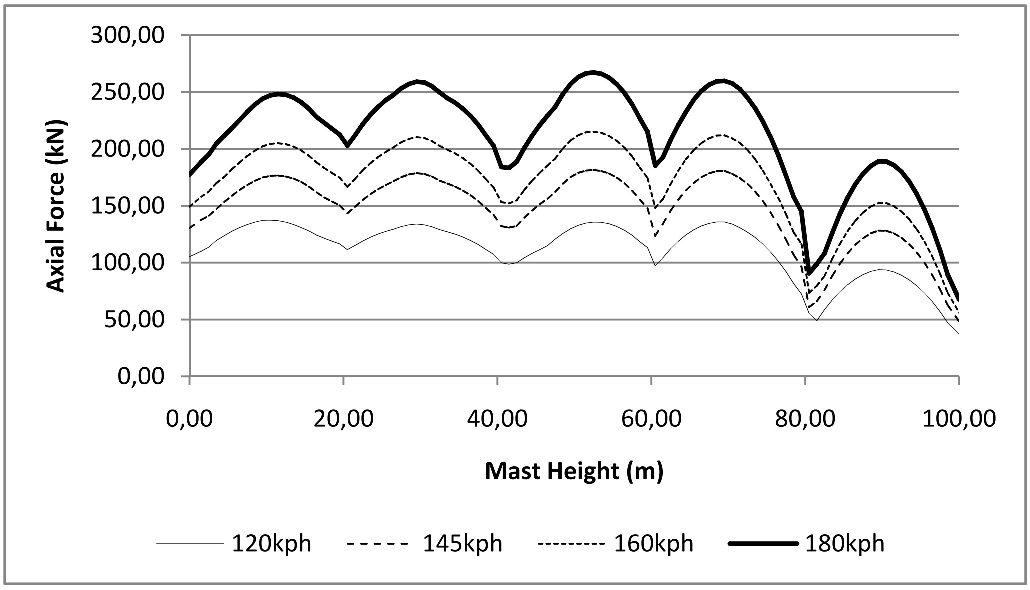


Figure 5.27 Wind Loading - Compression forces in the leg members of 100m mast

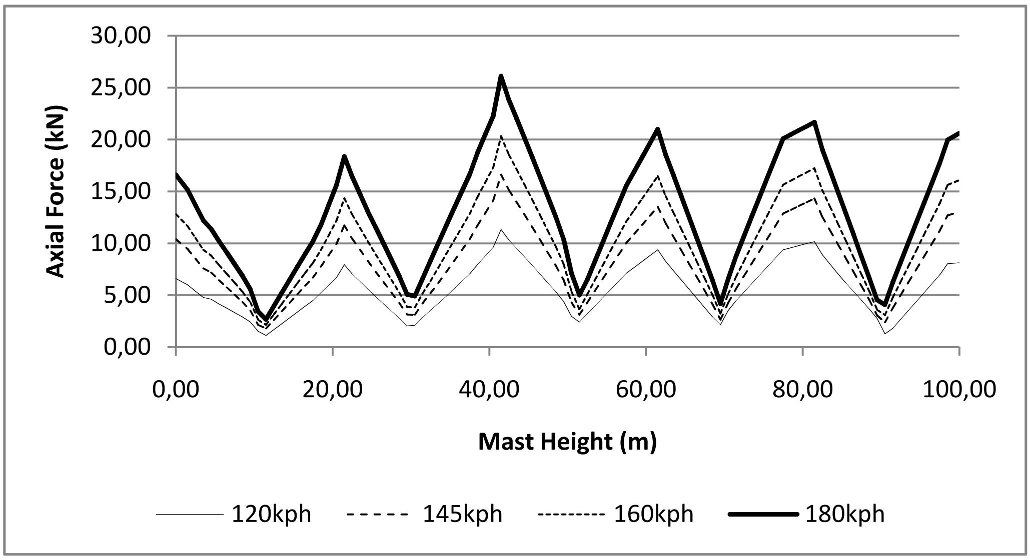


Figure 5.28 Wind Loading - Compression forces in the diagonal members of 100m mast

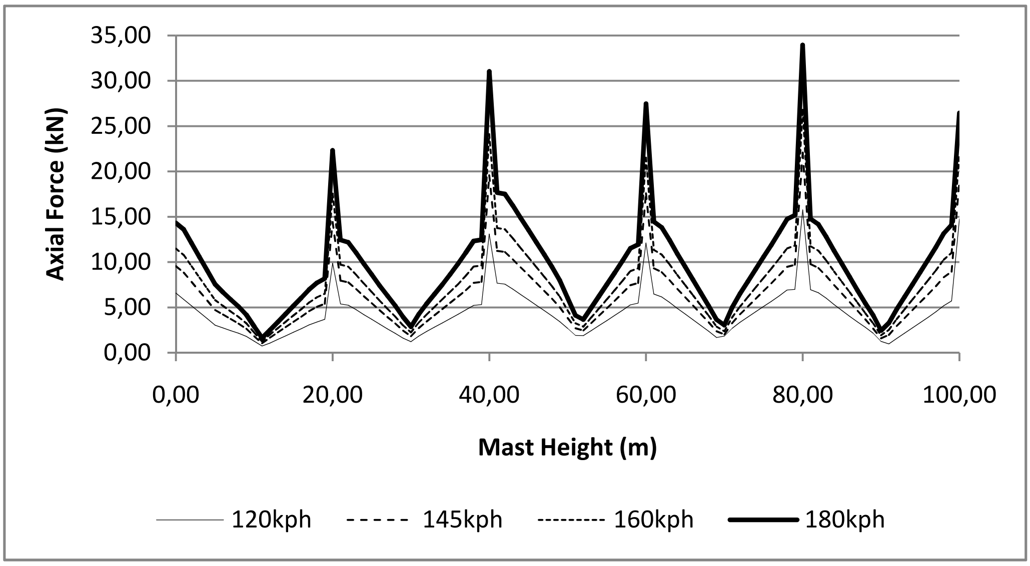


Figure 5.29 Wind Loading - Tension forces in the horizontal members of 100m mast

Table 5-11 Wind Loading - Tension forces in the guy cables of 100m mast

Guys	Tension Force in 120kph Wind Speed (kN)	Tension Force in 145kph Wind Speed (kN)	Tension Force in 160kph Wind Speed (kN)	Tension Force in 180kph Wind Speed (kN)
Guy Level 1	4.077E+01	5.205E+01	5.996E+01	7.177E+01
Guy Level 2	4.930E+01	6.549E+01	7.691E+01	9.413E+01
Guy Level 3	3.121E+01	4.223E+01	5.029E+01	6.332E+01
Guy Level 4	3.523E+01	5.012E+01	6.092E+01	7.748E+01
Guy Level 5	2.141E+01	3.057E+01	3.695E+01	4.671E+01

According to the results obtained from the wind load analysis, the most critical compression forces in the leg members were generated in the middle of the spans and maximum axial forces of the diagonal and horizontal members were generated in the guy connection points, like the results obtained from the earthquake loading analysis.

The maximum axial force on the members of the masts under investigation can be doubled or tripled with an increase in the wind speed from 120kph to 180kph. This is because the wind pressure on the structure is proportional to the square of the wind speed.

5.3.4 Results from Ice Loading Analysis

The three guyed masts under investigation were analyzed under the effect of 40.0kph, 48.3kph, 53.3kph and 60.0kph wind speeds with an accumulated 1.5cm thickness of ice by using PLS-TOWER [15] software. The resultant load effects are determined by selecting the maximum values from the nonlinear ice loading analyses with different combinations of wind directions and wind loading patterns. Since the PLS-TOWER [15] software is a tool used for analysis and design of lattice structures, the only output of the software is the axial force of the members of the structure.

The most critical axial forces obtained in the leg members, diagonal members, horizontal members and guys of the masts under investigation are presented in Figure 5.30 to Figure 5.38 and Table 5.12 to Table 5.14. With an accumulated 1.5cm thickness of ice, wind speeds used in the ice loading analysis were 40.0kph, 48.3kph, 53.3kph and 60.0kph.

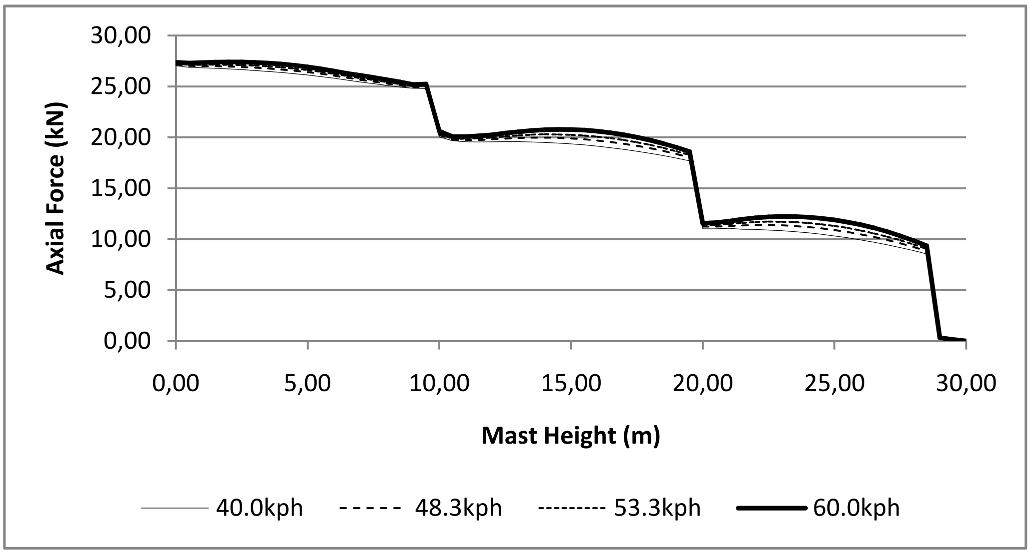


Figure 5.30 Ice Loading - Compression forces in the leg members of 30m mast

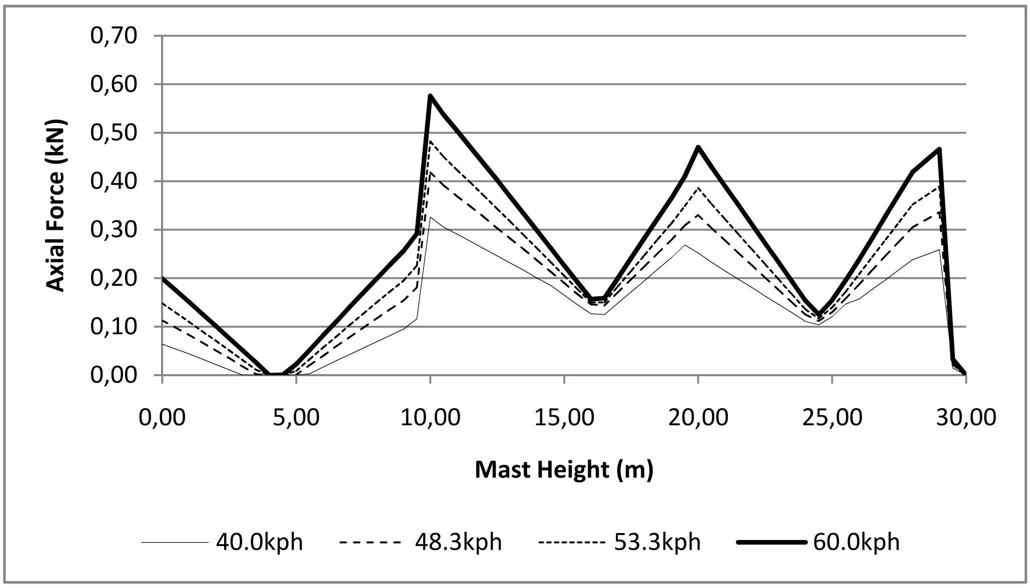


Figure 5.31 Ice Loading - Compression forces in the diagonal members of 30m mast

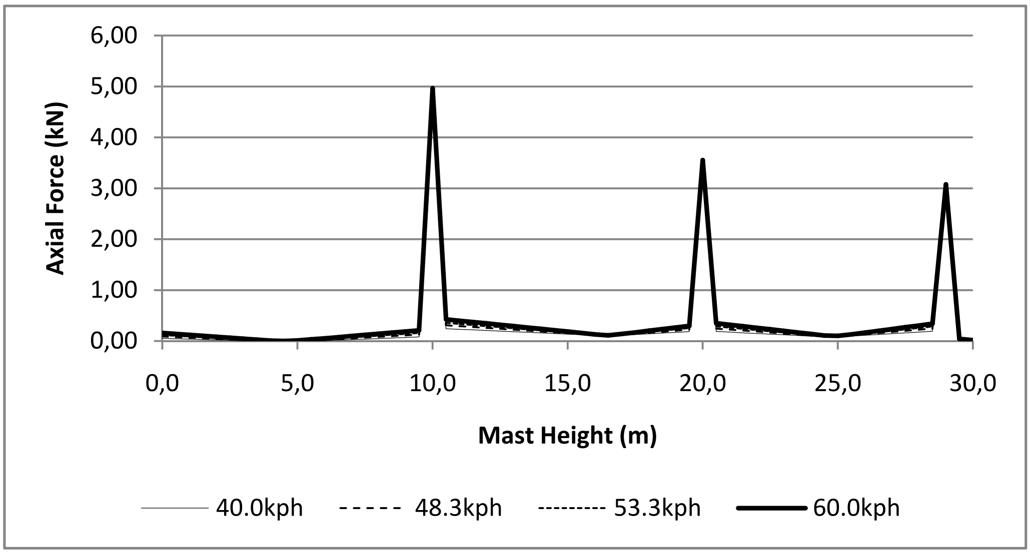


Figure 5.32 Ice Loading - Tension forces in the horizontal members of 30m mast

Table 5-12 Ice Loading - Tension forces in the guy cables of 30m mast

Guys	Tension Force in 40.0kph Wind Speed with ice (kN)	Tension Force in 48.3kph Wind Speed with ice (kN)	Tension Force in 53.3kph Wind Speed with ice (kN)	Tension Force in 60.0kph Wind Speed with ice (kN)
Guy Level 1	9.508E+00	9.961E+00	1.027E+01	1.077E+01
Guy Level 2	8.787E+00	9.229E+00	9.554E+00	1.005E+01
Guy Level 3	9.601E+00	9.984E+00	1.025E+01	1.066E+01

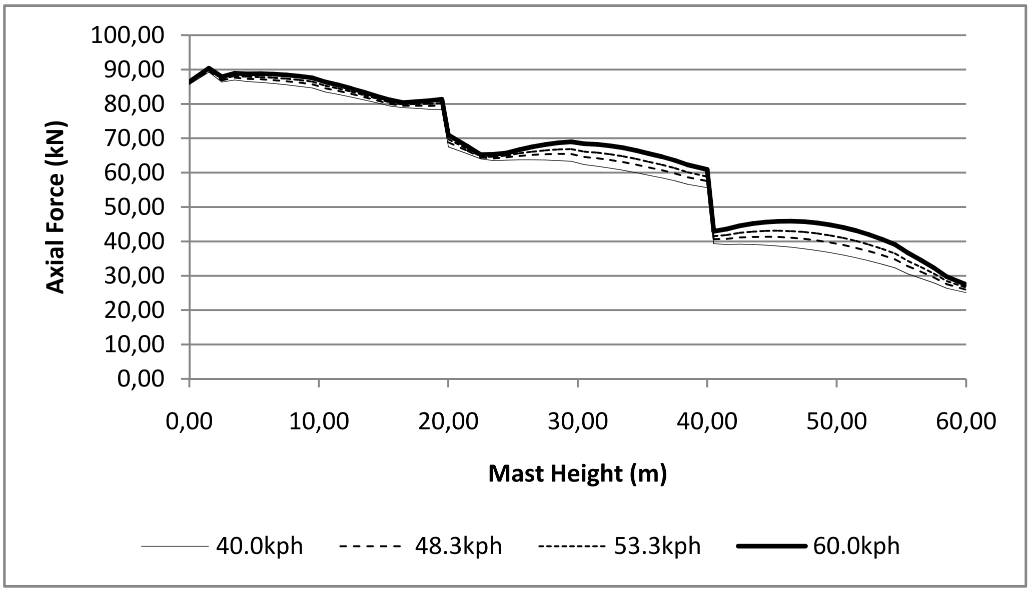


Figure 5.33 Ice Loading - Compression forces in the leg members of 60m mast

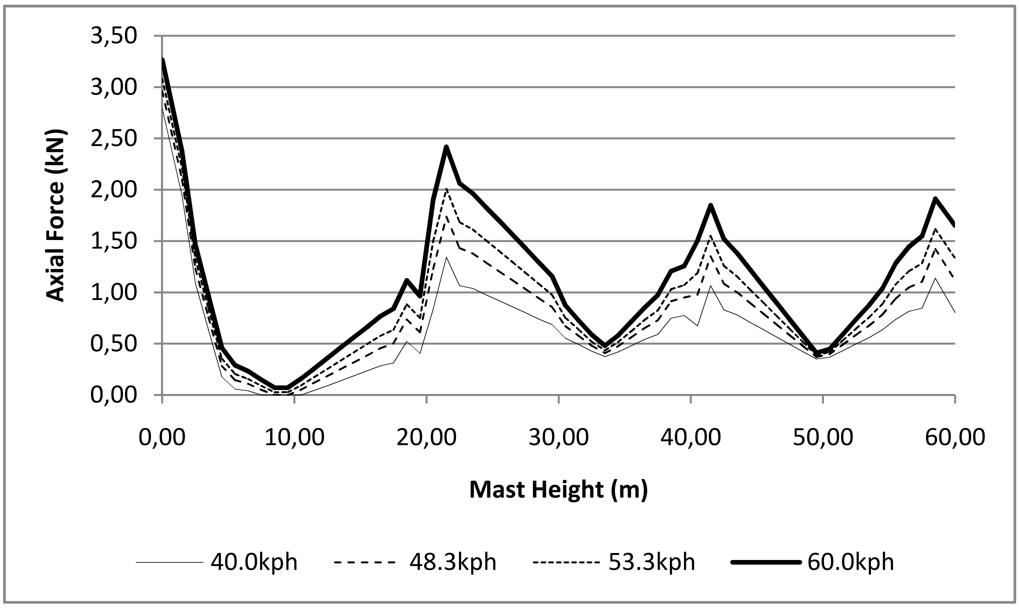


Figure 5.34 Ice Loading - Compression forces in the diagonal members of 60m mast

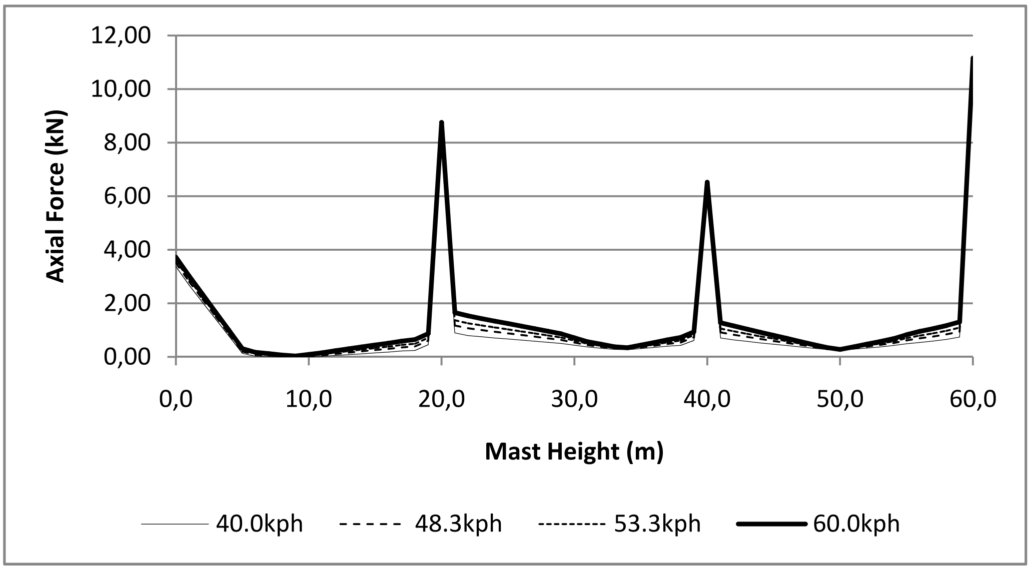


Figure 5.35 Ice Loading - Tension forces in the horizontal members of 60m mast

Table 5-13 Ice Loading - Tension forces in the guy cables of 60m mast

Guys	Tension Force in 40.0kph Wind Speed with ice (kN)	Tension Force in 48.3kph Wind Speed with ice (kN)	Tension Force in 53.3kph Wind Speed with ice (kN)	Tension Force in 60.0kph Wind Speed with ice (kN)
Guy Level 1	3.072E+01	3.072E+01	3.126E+01	3.211E+01
Guy Level 2	1.950E+01	1.950E+01	2.009E+01	2.100E+01
Guy Level 3	1.909E+01	1.909E+01	1.958E+01	2.034E+01

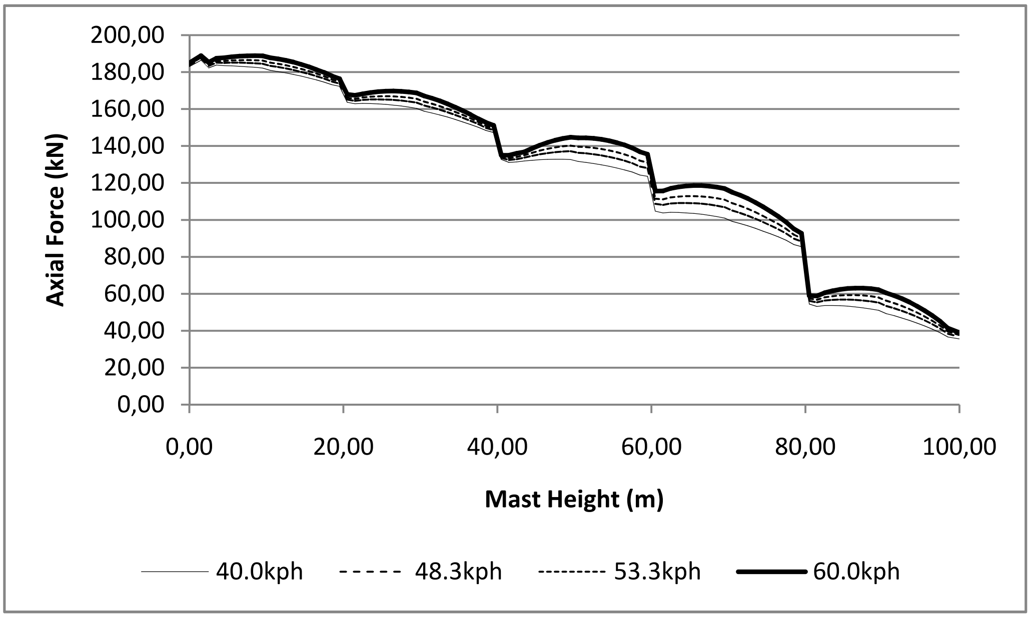


Figure 5.36 Ice Loading - Compression forces in the leg members of 100m mast

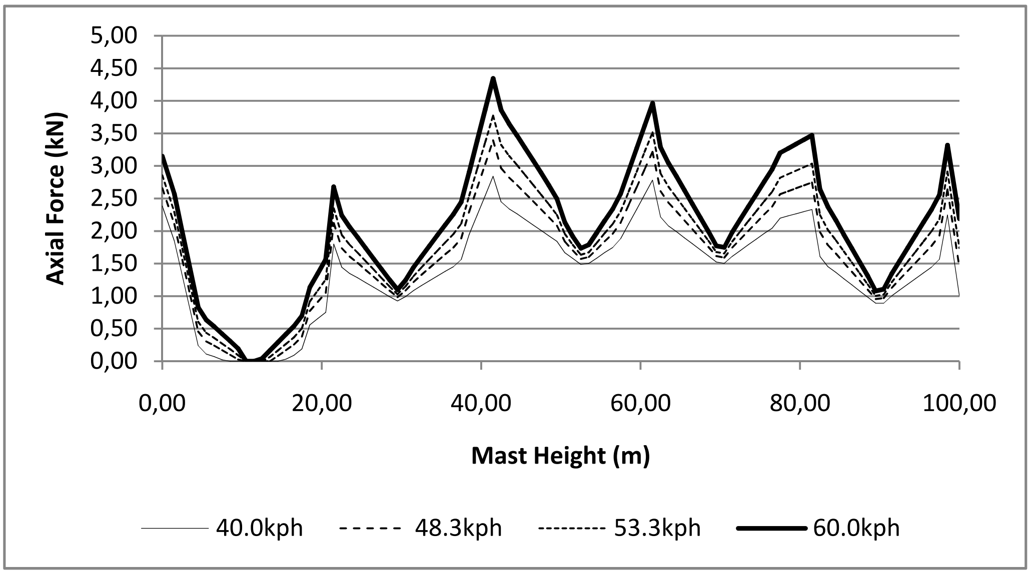


Figure 5.37 Ice Loading - Compression forces in the diagonal members of 100m mast

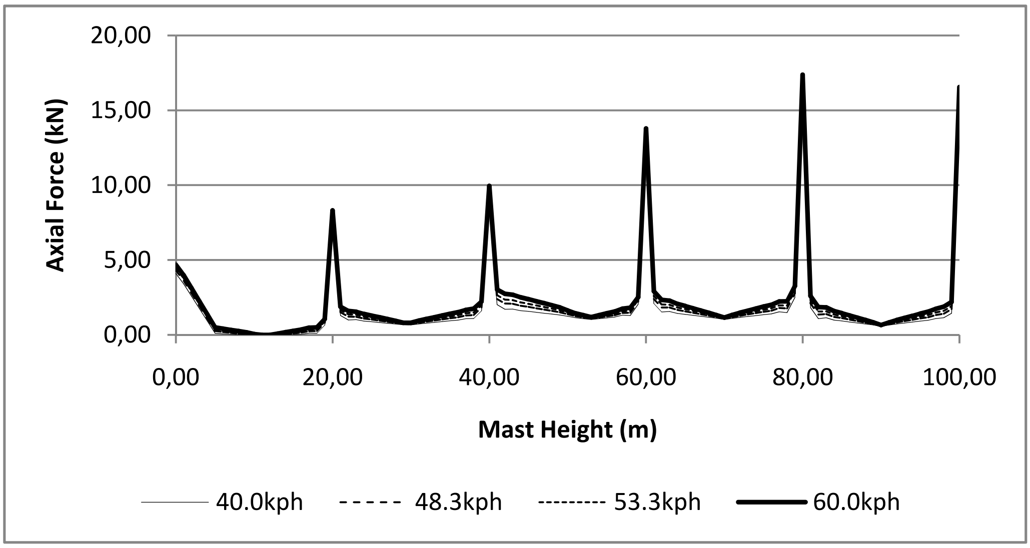


Figure 5.38 Ice Loading - Tension forces in the horizontal members of 100m mast

Table 5-14 Ice Loading - Tension forces in the guy cables of 100m mast

Guys	Tension Force in 40.0kph Wind Speed with ice (kN)	Tension Force in 48.3kph Wind Speed with ice (kN)	Tension Force in 53.3kph Wind Speed with ice (kN)	Tension Force in 60.0kph Wind Speed with ice (kN)
Guy Level 1	4.305E+01	4.419E+01	4.503E+01	4.632E+01
Guy Level 2	4.248E+01	4.381E+01	4.477E+01	4.625E+01
Guy Level 3	2.888E+01	2.975E+01	3.037E+01	3.133E+01
Guy Level 4	2.182E+01	2.295E+01	2.375E+01	2.499E+01
Guy Level 5	1.613E+01	1.672E+01	1.713E+01	1.776E+01

Results obtained from the ice loading analysis are very similar with the results generated by the wind loading analysis. But the difference between the maximum axial forces obtained from the ice loading analysis with minimum wind speed 40kph, and maximum wind speed 60kph is very slim. This mainly stems from the low wind pressures of ice loading analysis compared to the wind pressures of the wind loading analysis and gravitational forces introduced by the weight of the ice accumulated on the structure.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 SUMMARY AND CONCLUSIONS

In this research, firstly it is aimed to test the efficiency of cable structure analysis output generated by the commercial analysis software used throughout this study, then in light of foregoing to understand the behavior of guyed masts under the effect of seismic, wind and ice loadings and aid designers in estimating the response of these complex structures. Three dimensional modeling of the structures was accomplished by PLS-TOWER [15] and SAP2000 [14] finite element analysis programs.

To make sure that PLS-TOWER [15] and SAP2000 [14] software produce consistent and accurate results, two preliminary and three test models were analyzed using the structural analysis software used in this work and the outputs were compared with each other and the results presented in literature respectively. Afterward three realistically designed guyed masts were modeled and analyzed under the effect of wind, ice and earthquake loadings using the PLS-TOWER [15] software for wind and ice loading analysis and SAP2000 [14] program for seismic analysis. Comparison of results of the seismic, wind and ice loading analyses is presented in Figure A-2.1 to Figure A-2.9 and Table A-2.1 to Table A-2.3 in Appendix A-2.

The following conclusions were drawn;

- Three incomplex test models were analyzed with commercial software and the outputs were compared with the outcomes presented in literature. Based on the results, the outputs generated by the commercial software showed some differences with literature. This mainly stems from the approximate analysis methods used by the commercial software.
- According to the results obtained from the wind, ice and earthquake loadings, the dominant response of the masts were under the wind effect. However, for earthquake prone regions and cold regions with high atmospheric ice accumulation with low design wind speeds, seismic effects and effect of ice accumulation on the structure should be taken into consideration during the design process.
- At the bottom guy connection point of the 60m mast, the shape of the maximum compression forces in the leg members graph shows a considerable difference for the wind and earthquake loadings. For the wind loading analysis maximum axial forces in the leg members reach a peak with a sharp increase at the bottom stay connection point, on the other hand for the earthquake loading analysis the peak value was generated in the midspan. This mainly stems from the decrease in the cable tensions because of the oscillation of the adjacent spans in the same direction. These simultaneous oscillations cause a decrease in the forces generated by adjacent spans and the decline of stiffness of the cables between those spans.
- The distribution of maximum member forces of 30m and 60m masts are more or less regular for seismic analysis implying a dominant mode of vibration. The maximum axial forces of the members of the shorter masts are at lower spans. However, for 100m mast distribution of maximum member forces are

irregular implying that higher modes were dominant in the seismic response of the structure.

6.2 RECOMENDATIONS

The study can be extended by developing a practical static, pattern loading type, earthquake loading method which simulates the dynamic earthquake analysis. Since it will not be feasible for the structural engineers designing guyed masts to conduct a complex analysis like nonlinear time history analyses with the tight scheduling of the companies and competitive market conditions, this would be a useful method for structural engineers designing guyed masts.

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APPENDIX A

SCHEMATICS OF MASTS AND ANALYSIS RESULTS

A-1 SCHEMATICS OF MASTS

Schematics of masts under investigation are presented in Figure A-1.1, A-1.2 and A-1.3.

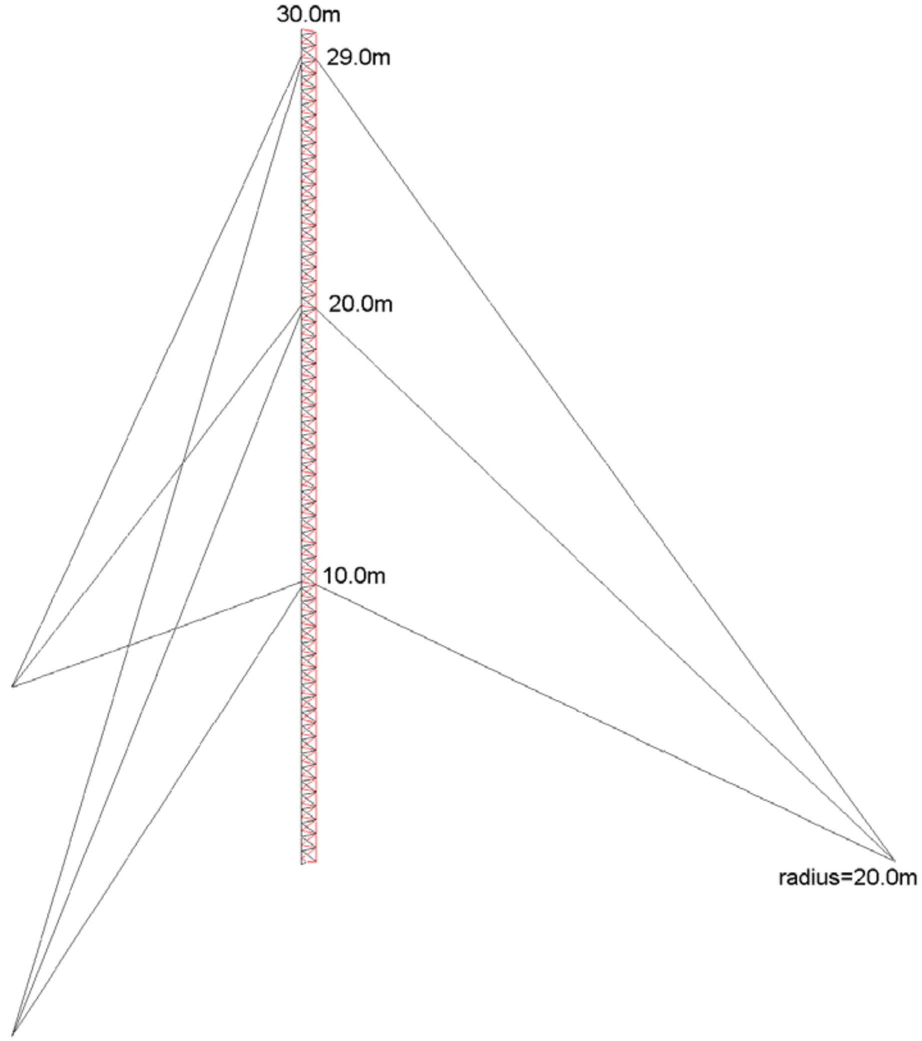

Figure A-1.1 Schematics of 30m mast

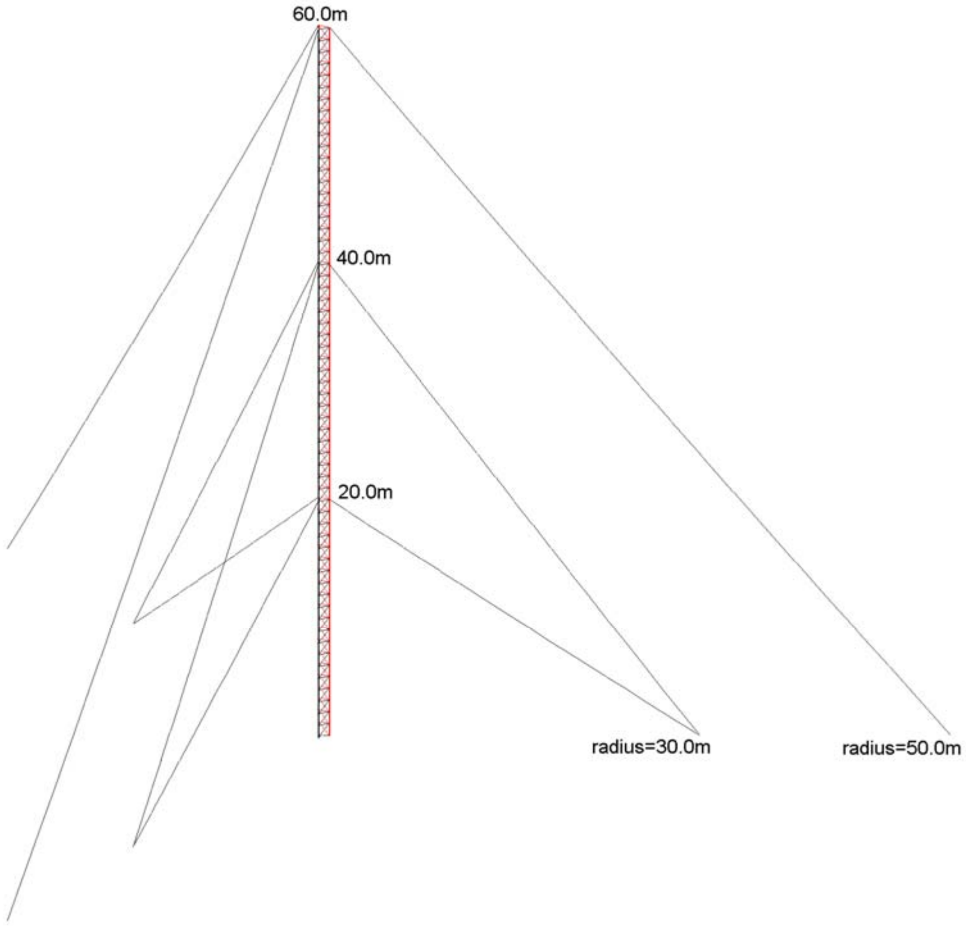

Figure A-1.2 Schematics of 60m mast

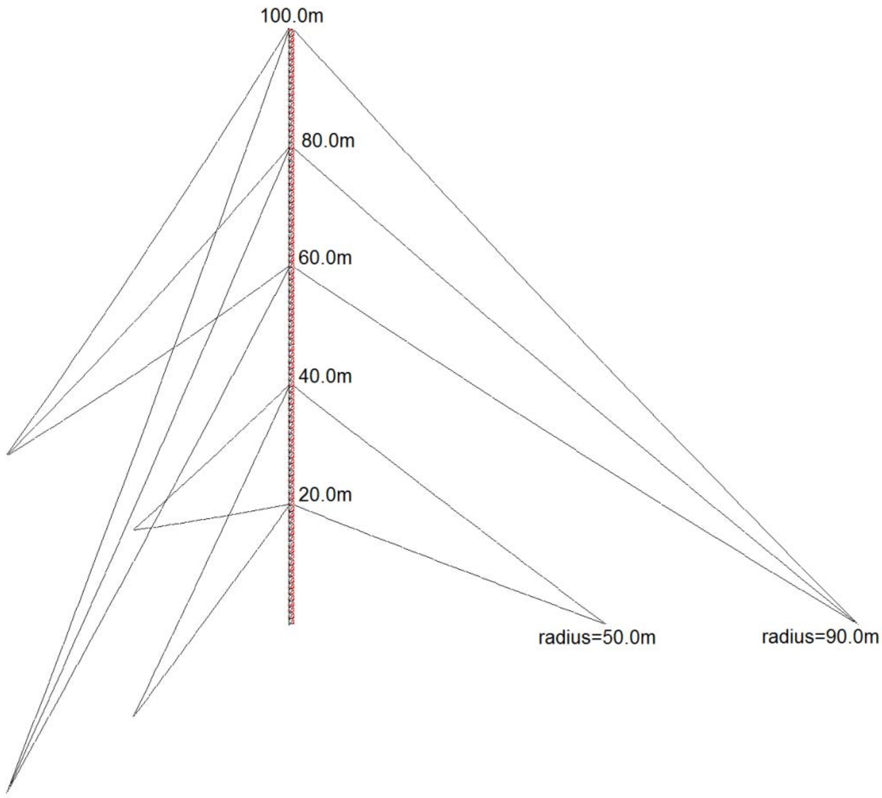

Figure A-1.3 Schematics of 100m mast

A-2 COMPARISON OF RESULTS OF ANALYSIS

Comparison of results of earthquake, wind and ice load analyses is presented in Figure A-2.1 to A-2.9 and Table A-2.1 to A-2.3.

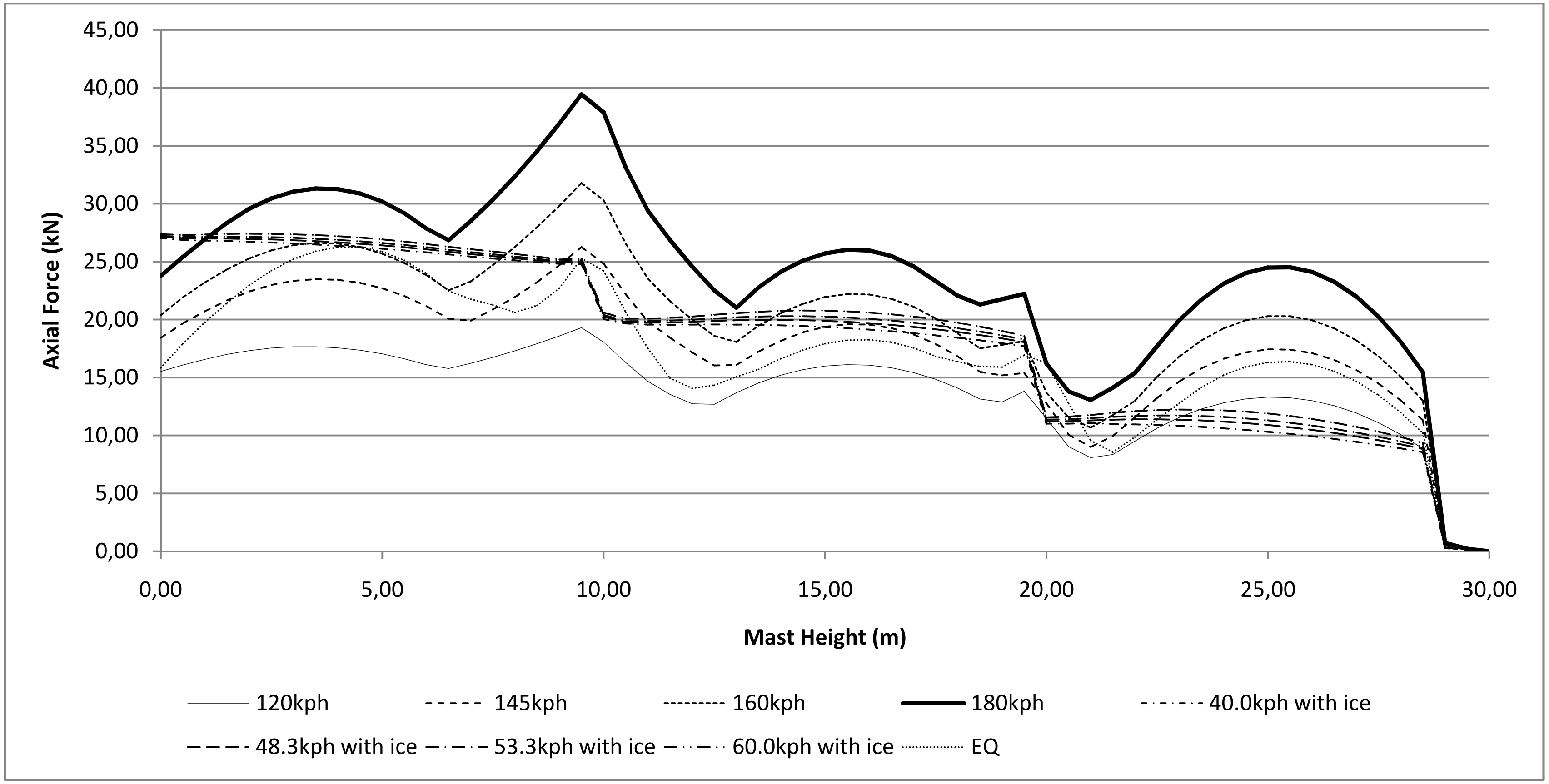

Figure A-2.1 Comparison of compression forces in the leg members of 30m mast

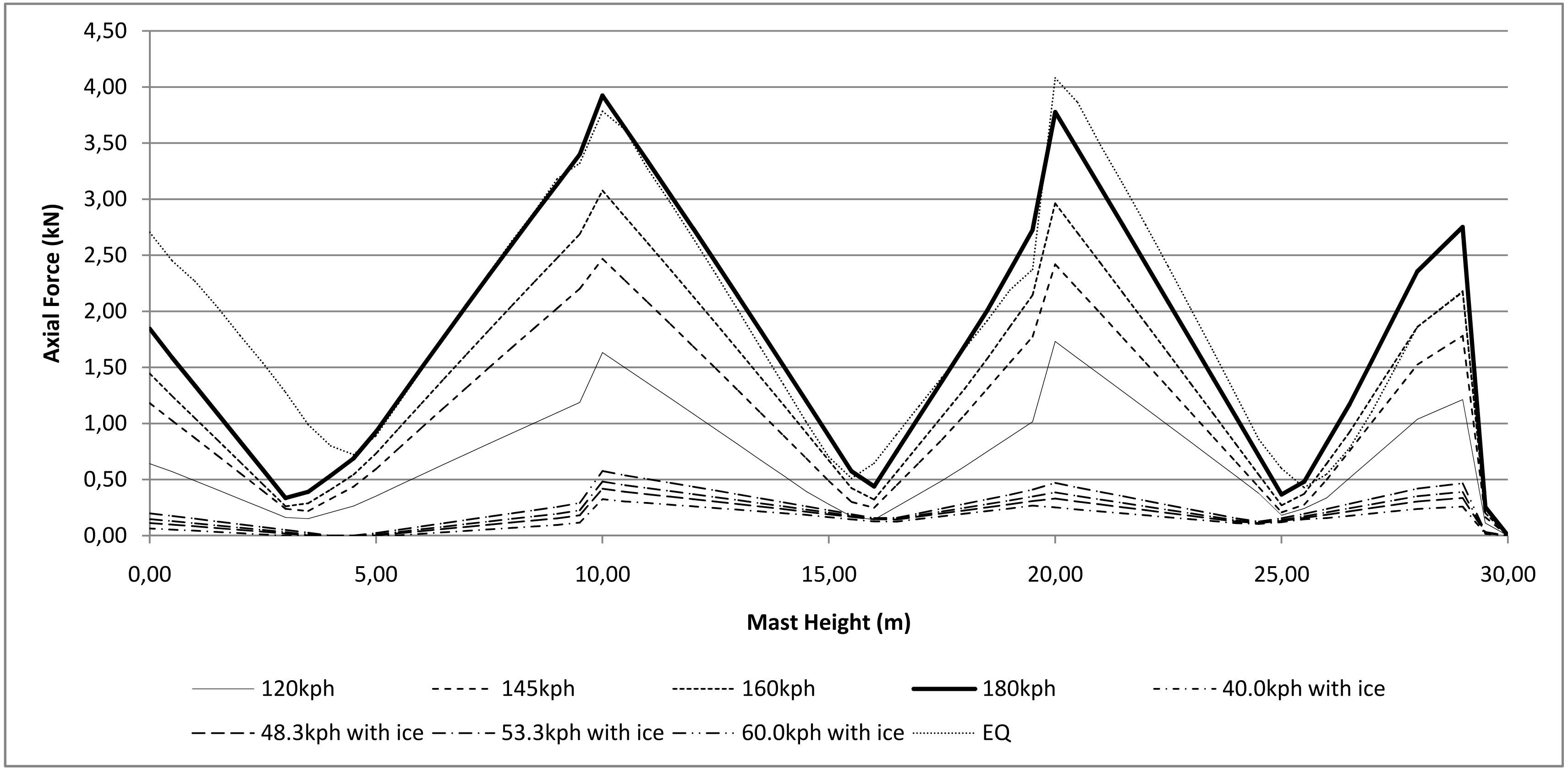

Figure A-2.2 Comparison of compression forces in the diagonal members of 30m mast

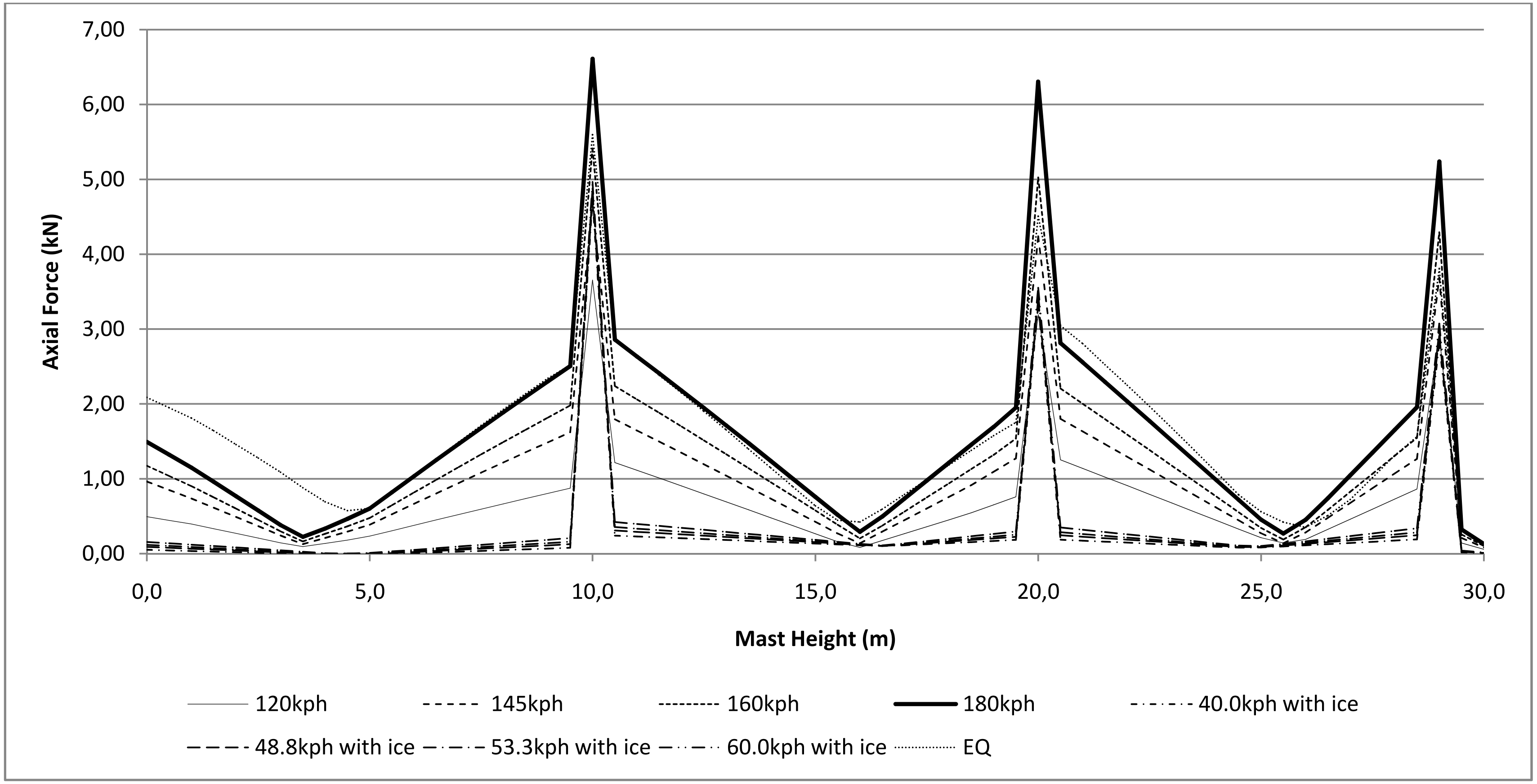

Figure A-2.3 Comparison of tension forces in the horizontal members of 30m mast

Table A-2.1 Comparison of tension forces in the guy cables of 30m mast

Guys	Tension Force in EQ loading (kN)	Tension Force in 120kph Wind Speed (kN)	Tension Force in 145kph Wind Speed (kN)	Tension Force in 160kph Wind Speed (kN)	Tension Force in 180kph Wind Speed (kN)	Tension Force in 40.0kph Wind Speed with ice (kN)	Tension Force in 48.3kph Wind Speed with ice (kN)	Tension Force in 53.3kph Wind Speed with ice (kN)	Tension Force in 60.0kph Wind Speed with ice (kN)
Guy Level 1	1.258E+01	1.011E+01	1.294E+01	1.489E+01	1.775E+01	9.508E+00	9.961E+00	1.027E+01	1.077E+01
Guy Level 2	1.344E+01	9.880E+00	1.324E+01	1.561E+01	1.915E+01	8.787E+00	9.229E+00	9.554E+00	1.005E+01
Guy Level 3	1.169E+01	8.415E+00	1.113E+01	1.284E+01	1.542E+01	9.601E+00	9.984E+00	1.025E+01	1.066E+01

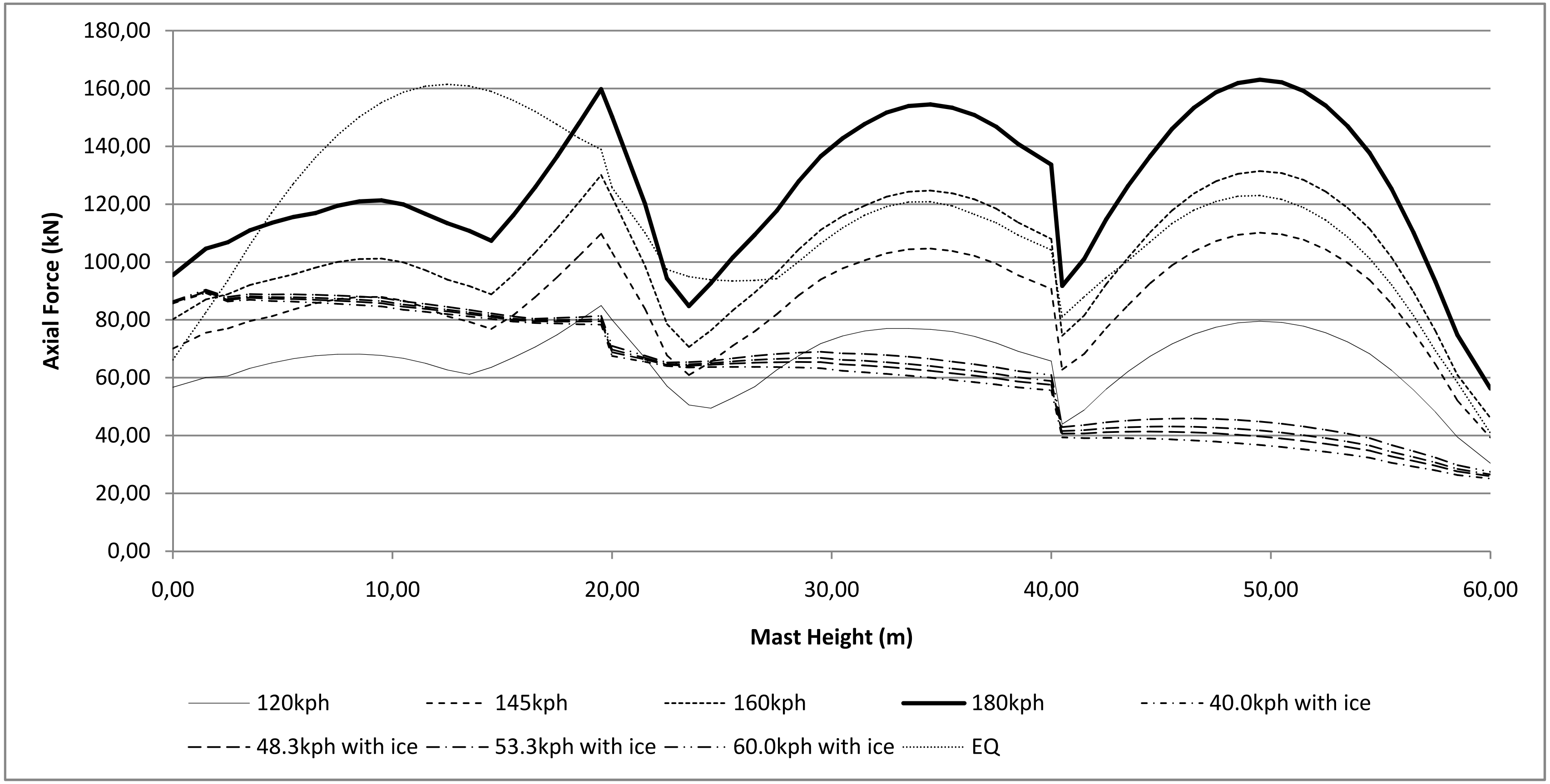

Figure A-2.4 Comparison of compression forces in the leg members of 60m mast

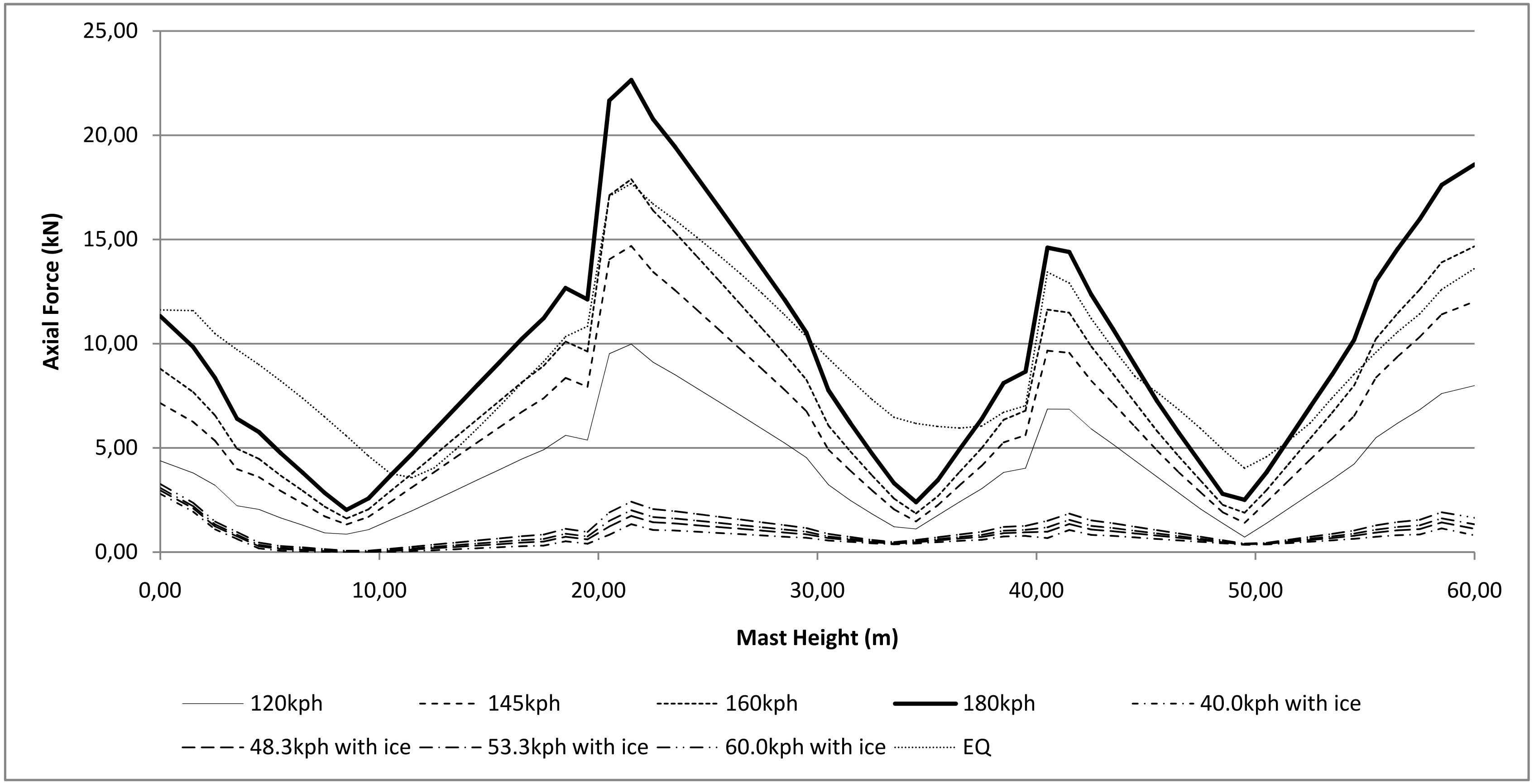

Figure A-2.5 Comparison of compression forces in the diagonal members of 60m mast

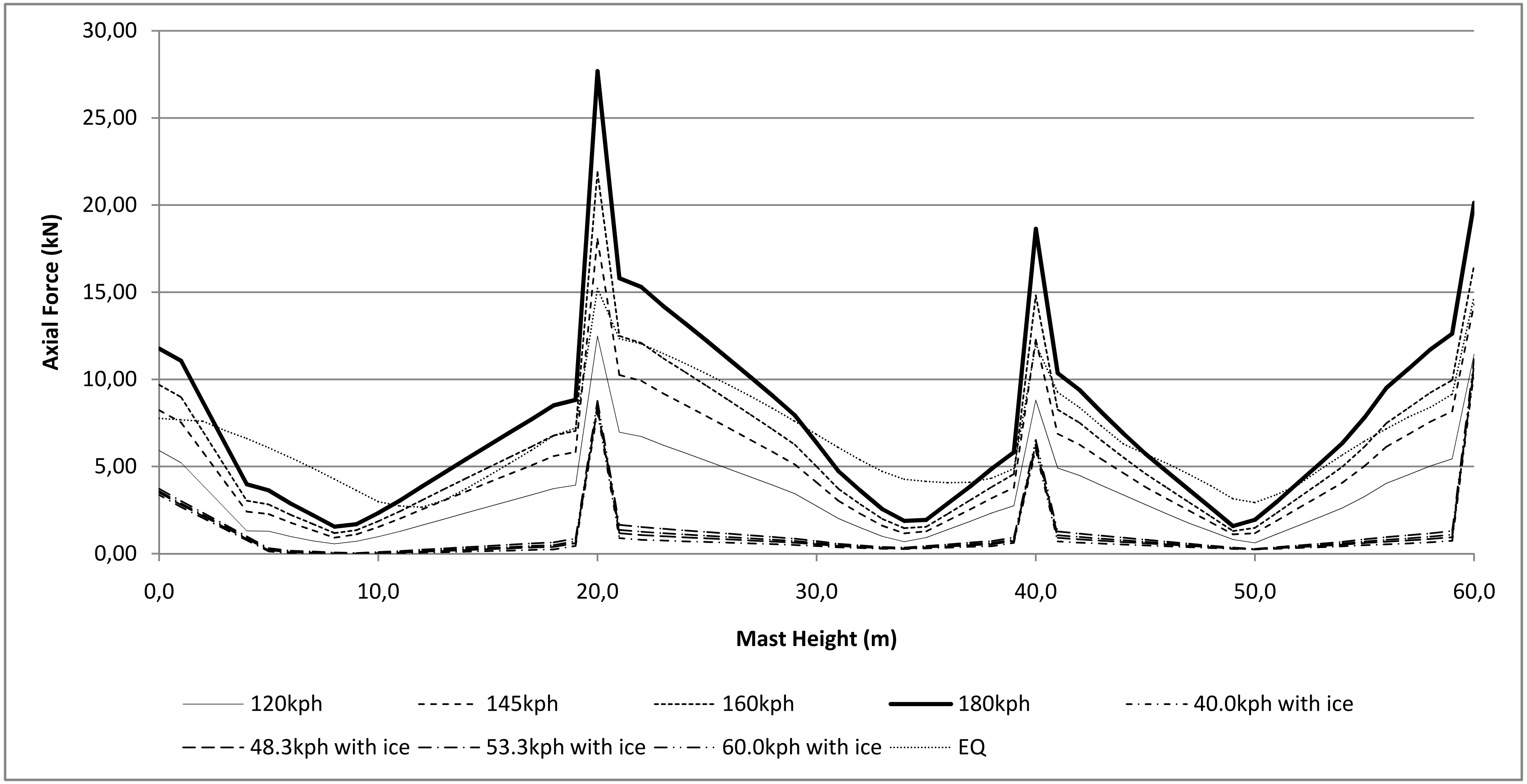

Figure A-2.6 Comparison of tension forces in the horizontal members of 60m mast

Table A-2.2 Comparison of tension forces in the guy cables of 60m mast

Guys	Tension Force in EQ loading (kN)	Tension Force in 120kph Wind Speed (kN)	Tension Force in 145kph Wind Speed (kN)	Tension Force in 160kph Wind Speed (kN)	Tension Force in 180kph Wind Speed (kN)	Tension Force in 40.0kph Wind Speed with ice (kN)	Tension Force in 48.3kph Wind Speed with ice (kN)	Tension Force in 53.3kph Wind Speed with ice (kN)	Tension Force in 60.0kph Wind Speed with ice (kN)
Guy Level 1	3.722E+01	3.241E+01	4.044E+01	4.623E+01	5.492E+01	3.072E+01	3.072E+01	3.126E+01	3.211E+01
Guy Level 2	4.202E+01	3.106E+01	4.132E+01	4.860E+01	6.033E+01	1.950E+01	1.950E+01	2.009E+01	2.100E+01
Guy Level 3	4.105E+01	2.926E+01	4.139E+01	4.994E+01	6.269E+01	1.909E+01	1.909E+01	1.958E+01	2.034E+01

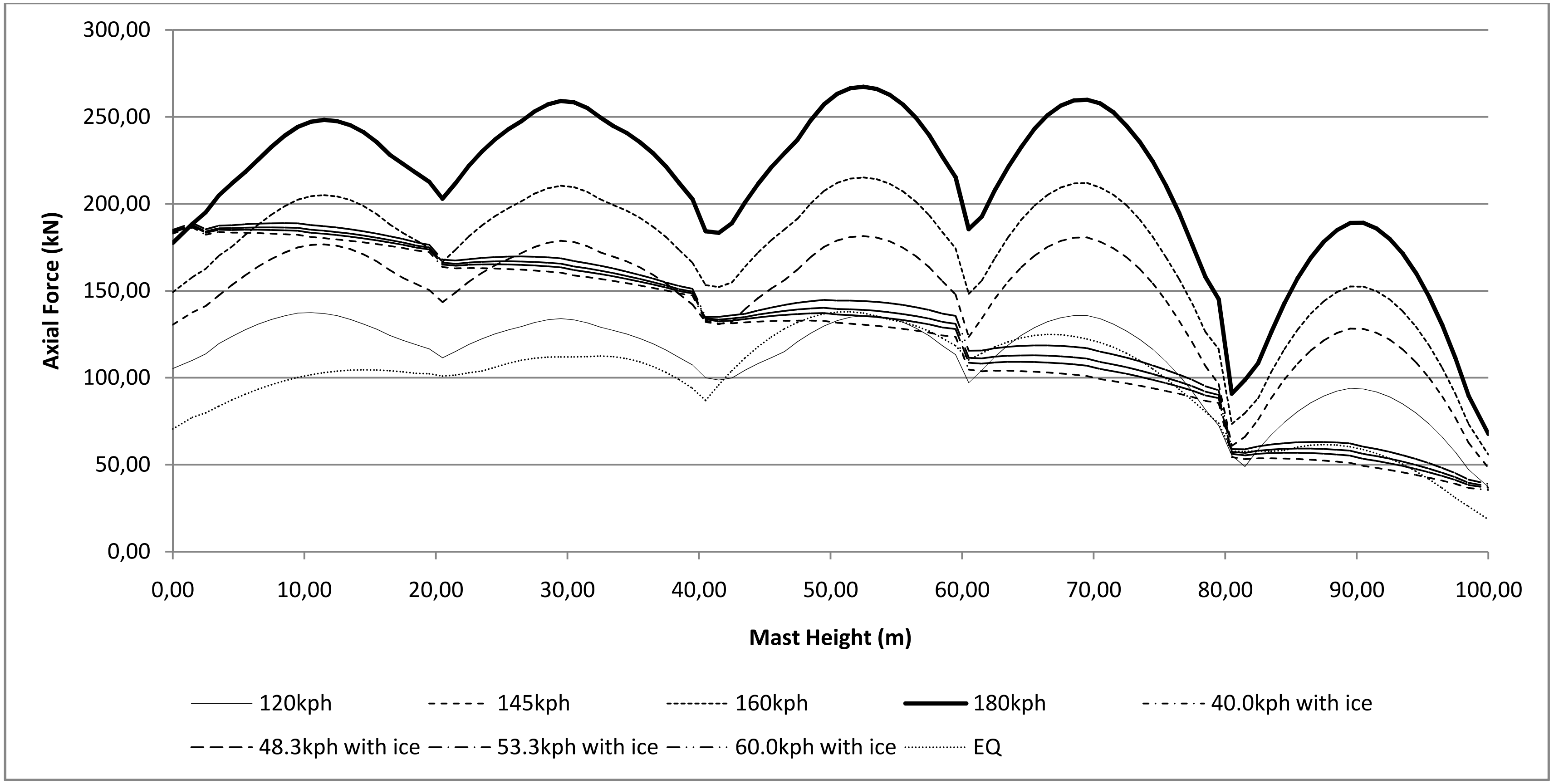

Figure A-2.7 Comparison of compression forces in the leg members of 100m mast

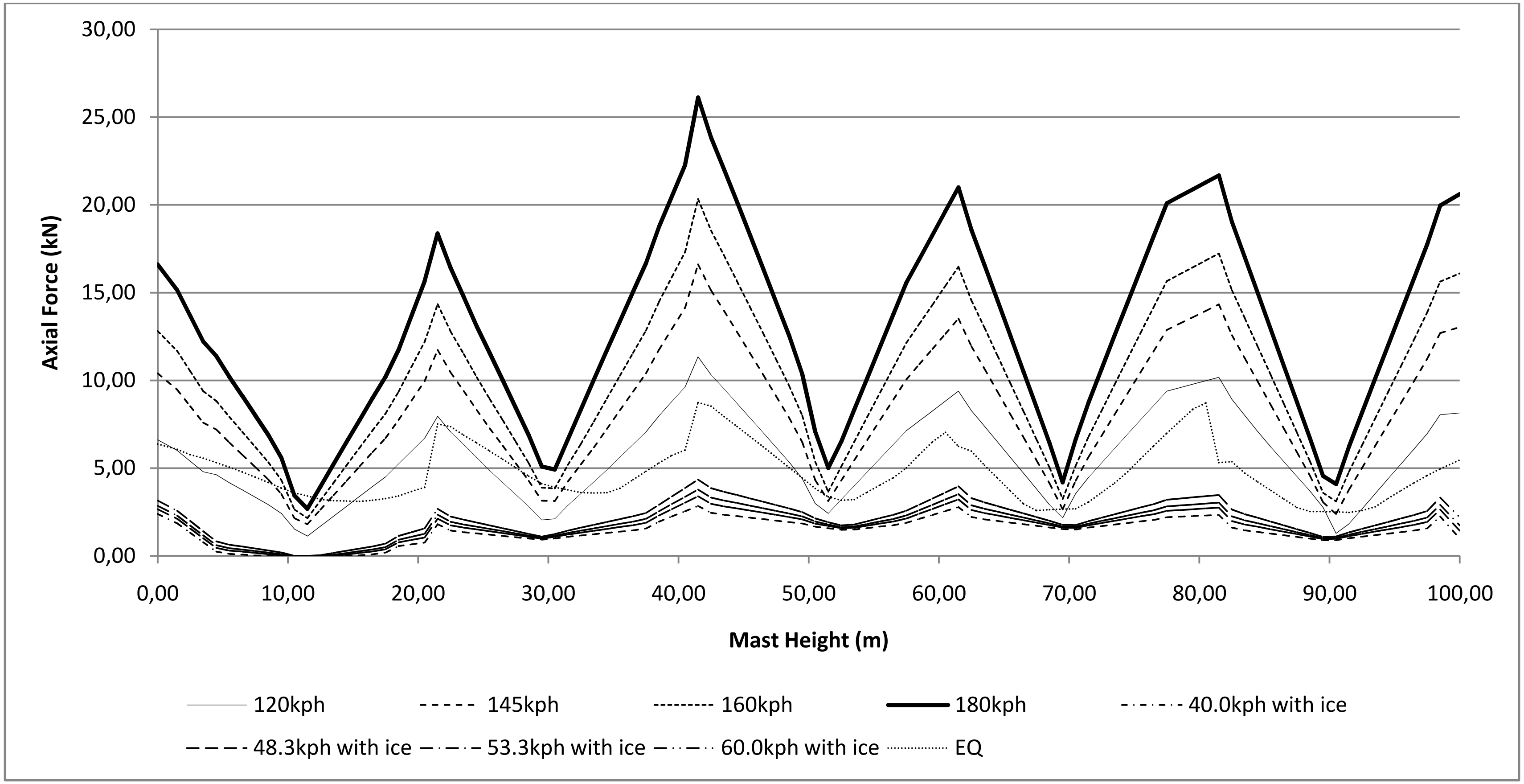

Figure A-2.8 Comparison of compression forces in the diagonal members of 100m mast

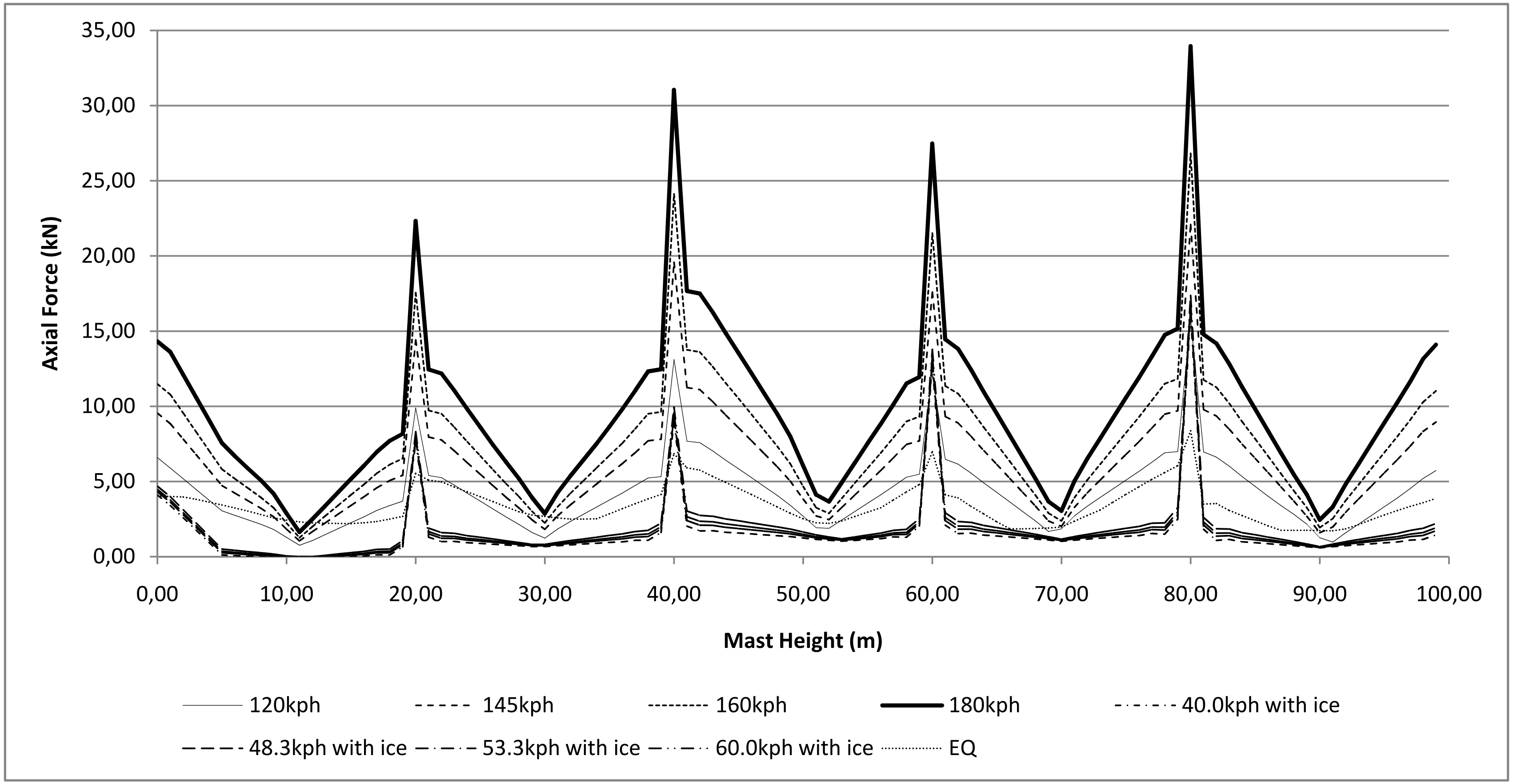

Figure A-2.9 Comparison of tension forces in the horizontal members of 100m mast

Table A-2.3 Comparison of tension forces in the guy cables of 100m mast

Guys	Tension Force in EQ loading (kN)	Tension Force in 120kph Wind Speed (kN)	Tension Force in 145kph Wind Speed (kN)	Tension Force in 160kph Wind Speed (kN)	Tension Force in 180kph Wind Speed (kN)	Tension Force in 40.0kph Wind Speed with ice (kN)	Tension Force in 48.3kph Wind Speed with ice (kN)	Tension Force in 53.3kph Wind Speed with ice (kN)	Tension Force in 60.0kph Wind Speed with ice (kN)
Guy Level 1	9.882E+00	4.077E+01	5.205E+01	5.996E+01	7.177E+01	4.305E+01	4.419E+01	4.503E+01	4.632E+01
Guy Level 2	1.618E+01	4.930E+01	6.549E+01	7.691E+01	9.413E+01	4.248E+01	4.381E+01	4.477E+01	4.625E+01
Guy Level 3	1.893E+01	3.121E+01	4.223E+01	5.029E+01	6.332E+01	2.888E+01	2.975E+01	3.037E+01	3.133E+01
Guy Level 4	2.720E+01	3.523E+01	5.012E+01	6.092E+01	7.748E+01	2.182E+01	2.295E+01	2.375E+01	2.499E+01
Guy Level 5	2.008E+01	2.141E+01	3.057E+01	3.695E+01	4.671E+01	1.613E+01	1.672E+01	1.713E+01	1.776E+01