FEBRUARY 2017 M.Sc. in Civil Engineering KAMIRAN MOHAMMED ALSILEVANAI

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COMPARISON OF DIFFERENT BRACING PATTERNS FOR SELF-SUPPORT COMMUNICATION TOWERS

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KAMIRAN MOHAMMED ALSILEVANAI

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Kamiran Mohammed ALSILEVANAI

ABSTRACT

COMPARISON OF DIFFERENT BRACING PATTERNS FOR SELF-SUPPORT COMMUNICATION TOWERS

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The significant system providing lateral load resistance in steel lattice communication towers is the bracing system. There are various kinds of bracing systems for steel lattice towers. This study has focused on identifying the economic bracing system for a given range of tower heights. Towers of height 40 m, 50 m, and 60 m have been designed and analyzed with seven rectangular base and four triangular base towers with different types of bracing systems under design loads. TOWER program is utilized for the design and analysis of towers using the Telecommunication Industry Association (ANSI/TIA-222-G) standard code specification. The TOWER program is capable of performing both linear and nonlinear analyses with more efficiently and reliably. Comparison of the various bracing system is assigned based on the weight of tower, structure fundamental frequency, maximum element usage, maximum overturning moment, sway and twist values. The smallest weight is obtained at KD and YD type bracing systems for rectangular base towers. In the case of triangular base towers, X and XB type bracing systems give the smallest weight design. The best performance according to sway and out of plumb values are obtained at X and K type bracing system for rectangular and triangular base towers. It is worth to mention that Y type bracing system is failed in nonlinear analysis at 60 m height tower.

Keywords: Telecommunication tower, Bracing system, Lattice tower, Linear analysis, Nonlinear analysis, Self-supporting tower.

ÖZET

KENDINDEN DESTEKLİ İLETŞİM KULELERİ İÇİN FARKLI KAFAS BİÇİMLERİNİN KARŞILAŞTIRILMASI

ALSILEVANAI, Kamiran Mohammed
Yüksek Lisans Tezi, İnşaat Mühendisliği
Danışman: Prof. Dr. Mustafa ÖZAKÇA
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Çelik kafes iletişim kulelerinde yanal yük direncini önemli ölçüde sağlayan takviyeli sistemlerdir. Çelik kafes kuleler için çeşitli takviye sistemleri bulunmaktadır. Bu çalışma belirli bir dizi kule yüksekliği için ekonomik takviye sistemini tanımlamaya odaklanmıştır. 40 m 50 m ve 60 m yükseklikteki kuleler, yerçekimi yükleri ve yanal yüklerin etkisi altındaki yedi farklı dikdörtgen tabanlı ve dört farklı üçgen tabanlı takviye sistemli kuleler için farklı ile tasarlanmış ve analiz edilmiştir. Buna ek olarak, TOWER programında Telekomünikasyon Endüstrisi Birliği (ANSI / TIA-222-G) standartı kullanılarak kulelerin tasarımı ve analizi yapılmıştır. TOWER programı hem doğrusal hem de doğrusal olmayan analizleri etkin ve güvenilir bir şekilde yapabilmektedir. Çeşitli takviye sistemlerinin karşılaştırılması kule ağırlığı, yapı temel frekansı, maksimum eleman kullanımı, maksimum devrilme momenti, salınım ve bükme değerleri temel alınarak yapılmıştır. En düşük ağırlık, dikdörtgen tabanlı kuleler için KD ve YD tipi takviye sistemlerinde. Üçgen tabanlı kulelerde X ve XB tipi takviye sistemleri elde edilmiştir. Dikdörtgen ve üçgen tabanlı kuleler için, X ve K tipi takviye sistemleri salınım değerlerine göre en iyi performansı vermektedir. Y tipi takviye sisteminin 60 m yüksekliğindeki kule için doğrusal olmayan analize göre yetersiz olduğu görülmüştür.

Anahtar kelimeler: Telekomünikasyon kulesi, takviye sistemi, kafes kule, doğrusal analiz, doğrusal olmayan analiz, kendinden destekli kule.

To My Family

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LIST OF SYMBOLS/ABBREVIATION

AISC	American Institute of Steel Construction
$(EPA)_A$	Effective projected area of the appurtenance including ice for loading combinations that include ice
$(EPA)_S$	Effective projected area of the structure
F_y	Yield strength
D_g	Dead load of guy assemblies
D_i	Weight of ice due to factored ice thickness
D_o	Wind load without ice
F_A	Design wind force on appurtenances
F_{ST}	Horizontal design wind force on the structure in the direction of the wind
F_{sz}	Lateral seismic force
G_h	Gust effect factor
G_h	Gust effect factor
K_S	1500 for h and W_a in meters
S_{D1}	Design spectral response acceleration at 1 second
S_{DS}	Design spectral response acceleration at short periods
S_{az}	Acceleration coefficient at height z

T_i	Load effects due to temperature
W_0	Face width at base of structure
W_2	Weight of structure and appurtenances within top 5% of the structure height
W_a	Average face width of structure
W_i	Concurrent wind load with factored ice thickness
W_z	Portion of total gravity load (W) assigned to level under consideration
f_1	Fundamental frequency of structure
f_{w1}	Face width
q_z	Velocity pressure
2D	Two Dimensional
3D	Three Dimensional
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
Brac	Bracing
BST	Base Stations Transmitting
Ccap	Compression capacity based on member slenderness column
CD	Drag Coefficient
CDMA	Code Division Multiple Access
CFRP	Carbon Fiber Reinforced Polymer
CSA	Canadian Standards Association

D	Dead load of structure and appurtenances, excluding guy assemblies
D	Diagonal member
DOF	Degree of Freedom
E	Earthquake load
EIA	Electronic Industries Alliance
Ele.	Element
EN	European Standard
Fa	Design axial compressive stress
FA	Design wind force on appurtenances
FE	Finite Element
FG	Design wind force on guys
FST	Design wind force on the structure
Ft	Design axial tensile stress
Fy	Minimum yield stress
G	Gust response factor
GSM	Global System for Mobile
h	Distance between centroid of individual components of the built-up member
H	Height
I	Importance factor
IASS	International Association for Shell and Spatial Structures

IS	Indian Standards
ISO	International Organization for Standardization
JR	Withstand an impact energy
K	Effective length factor
KL/r	Effective slenderness ratio
L	Length of member
L	Laterally unbraced length of member
L/r	Slendemess ratio of member
LC	Load Case
LMR	Land Mobile Radio
MEU	Maxımum Element Usage
Max.	Maximum
MW	Micro Wave
N.G.	Not Given
Ncap	Tension capacity based on net section
PC	Personal Computers
PLS	Power Line System
r	Radius of gyration
R	Response modification coefficient equal to 3.0 for latticed self-supporting structures
Resul.	Resultant

S	Structural steel
SAE	Single Angle Equal
SWR	Standing Wave Ratio
T	Transverse
t	Thickness of element
TIA	Telecommunications Industry Association
TV	Television
UV	Ultraviolet
V	Vertical
V_0	Wind velocity
w	Flat-width of element
W	Weight
w/t	Ratio of width to thickness ratio
WAF	Wind Area Adjustment Factor
z	Number designating the level under consideration
γ	Load factor for wind load
ε	Strain
λ	Slenderness ratio
ν	Poisson's ratio
ρ	Mass density
σ	Normal stress

τ	Shearing stress
W	Total weight of the structure including appurtenances

CHAPTER 1

INTRODUCTION

1.1 General

Communication towers have become a vital point expressly in wireless communication sector with the development of wireless communication technologies such as CDMA (Code Division Multiple Access), GSM (Global System for Mobile), and WAP (Wireless Web Access). In the human civilization at the instant, the telecommunication towers become a fundamental mechanism of communication [1]. Telecommunication towers were used widely since the beginning 1900's [2]. Towers were used for radio transmission since 1940 for small and average frequency, hardly lattice structures including of wood were used. The highest wooden lattice mast was at Mühlacker, Germany. It had a height of 190 m and was constructed in 1934 and destroyed in 1945. Most wood lattice masts were devastated earlier 1960. In Germany, the latest massive radio structure consisting of wood were the overhead towers of the Ismaning and Golm transmitter. They were destroyed in 1979 and 1983 sequentially. In Germany, Gustav-Vietor-Tower was established in Hohe Wurzel (Taunus) city in 1882 as lattice steel tower at 25 m height and demolished in 2006. The tallest lattice tower is the Tokyo Skytree, with a height of 634 m (2,080 ft). Lattice tower structures are typically produced as both a space frame or a hyperboloid structure [3].

Steel towers communication is usually high frames whose objective is to support elevated antennas for radio and TV broadcasting, telecommunication systems. Furthermore, the performance of infrastructures such as dams, electric, gas, and fuel transmission services, depends widely on the data being transmitted using these telecommunication masts. Other fields of application for such towers are an army and defense activities in addition to TV, radio, and telecom industries and thus produces the requirement for more research. For that reason, unexpected serviceability or an even role of the first-aid station infrastructure is the crucially chief affair in the cause of a failure. Telecommunication masts are classifying as slender-high multi-support constructions, Because of their unique geometry. Many industries and communications demand tower for a variety of purposes. Steel towers are normally used for microwave transmission for communication, radio transmissions, television transmission, satellite receptions, air traffic controls, flood light stands in stadiums and at large flyover intersections, power transmission lines, meteorological measurements, derrick and crawler cranes, oil drilling masts and overhead water tanks [1]. For multiple objects, towers height ranges changed between 20 to 500 m. For broadcasting, the range is between 100 to 400 m, 50 to 200 m for radio and telecommunications, 15 to 50 m for flood lights and 10 to 45 m for overhead power transmission lines [4].

The stiffness of the structure, the capability of bearing load and the wind will be a function of tower geometry. Special attention to stiffness will be a need in the instance of the cell to cell radio connections. Indeed little change (> 0.5 of a deg.) could happen in the lack of signal considering the severe line of vision necessity and the constitution of the beam of one to the next antenna [5].

1.2 Cellular Network System

A mobile or cellular network is a wireless system spread over ground zones termed cells, each followed by at concise one sTable-location transceiver and contain some of the cells identified as a base station or cell site. Figure 1.1 present a cellular system network architecture [6].

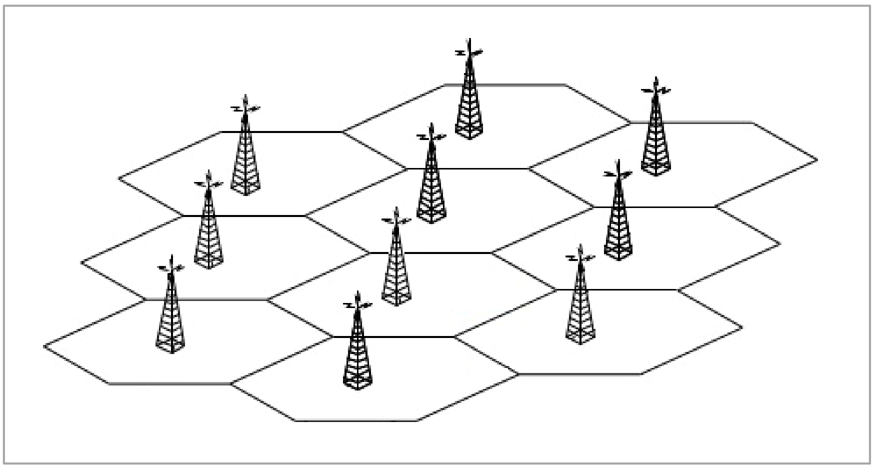


Figure 1.1 Cellular network system architecture

1.2.1 Cell site

A basic geographic unit of a cellular network is a cell site. Cells are Base Stations Transmitting (BST) across little geographic regions that characterized as hexagons. Every cell measurement changes depending on the geographic areas [7]. In fact, cells are not equal and have a different diameter between 0.5 km to 70 km. Bases can be at the crossing of three cells or the center of a base.

In residential surroundings, the cells tend to be very small, and the radio waves reused more frequently. The cause for this is that the capability of the required network in downtown areas is extremely higher than in provincial fields. The antennae linked with small bases frequently placed on low support on buildings or structures. Larger bases commonly applied in provincial regions where coverage slightly than function is the significant condition. Higher towers commonly needed in provincial regions [5].

1.2.2 Base transceiver station

The brief of the term BTS used to indicate a base station terminology in Global System Mobile (GSM). Components of (BTS) includes of GSM, micro wave antennas and the necessary equipment of radio to communicate by radio with a station of a mobile [8].

1.3 Components of Base Station Transmitting

Components of cell site include the active and passive infrastructure. Active component includes the microwave radio equipment, antenna, feeder, transceivers, and coaxial cables. Passive components include the ground, support tower, air conditioning equipment, shelter, electric generator, electrical supply, battery, etc. as present in Figure 1.2 [9].

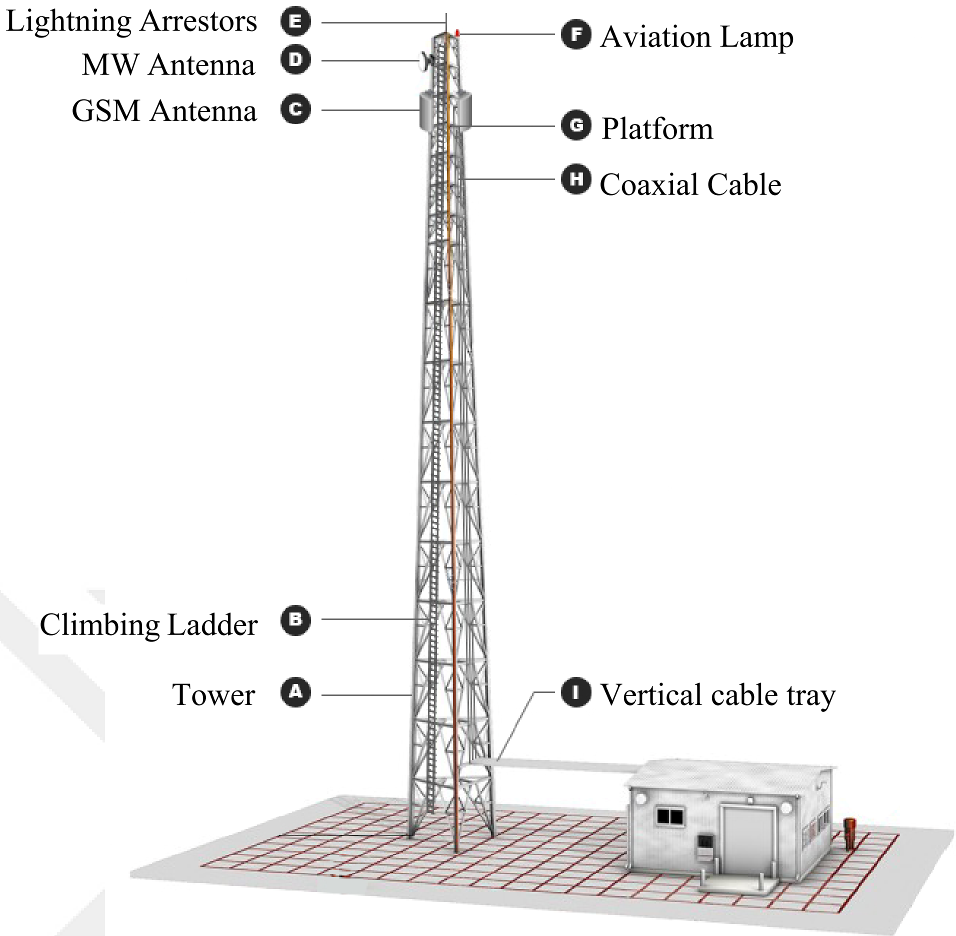


Figure 1.2 Base stations transmitting

(a) Tower

The main supporting unit of communications is a tower. The wave goes as per mounted antennas and microwaves on towers. Tower has to carry the heavy GSM and microwave antenna at an enough, safe height from land. Also to that, all towers have to support all kinds of natural disasters.

(b) Climbing ladder

Climbing equipment involves an element specially implemented to allow access to the supports, like as a mounted ladder, bolted step, etc. Steps need to be from steel or aluminum depending on a support material and must be implemented with protection cages, resting places and guarding terminations.

(c) GSM antenna

GSM term is a Global System for Mobile antenna mostly used in mobiles and cell supports. The GSM antenna permits the mobile phones or devices to communicate with other GSM antenna on a cell support, which then sends the waves to the different cell phone.

(d) Microwave antenna

Microwave or Dish antenna that has a circular shape and different diameter used for receiving or sending electromagnetic waves like as radio or microwave waves.

(e) Lightning arrestors

Which work on discharging unnecessary voltage built upon the tower to earth because of lightning.

(f) Aviation lamp

Lighting on tall towers is made structures more evident to aircraft, during day and night time. Lighting apparatus must have special characteristics and should define by global organizations standards.

(g) Platform

A flat plate with a flat facade established at the level of the identified level of the lattice tower.

(h) Coaxial cable

A coaxial cable connector (radio frequency connector) is a cable connector designed to work at radio frequencies in the multi-megahertz range. Coaxial cable designed to maintain the shielding that the coaxial design offers.

(i) Vertical cable tray

Cable tray is a channel made from galvanized plate are ordinarily utilized for managing the coaxial cables in communication towers.

1.4 Classification of Supports

The various kinds of supports are available and type of support is based on their structural performance, cross-section, type of sections practiced and on the position of support. A short explanation is as provided below [10]:

1.4.1 Based on support performance

Supports are classified into three significant groups as evidence in Figure 1.3 based on the structural behavior. They are self-supporting towers, guyed towers, and monopole.

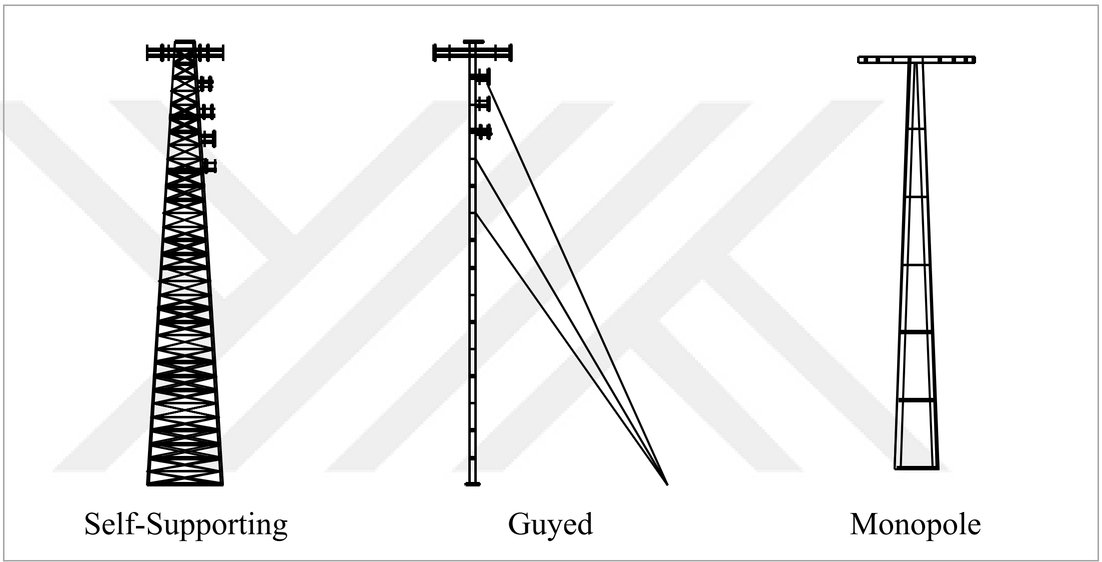


Figure 1.3 Type of communication towers

(a) Self-supporting tower

Or the free-standing towers that are fixed on land or on the building surfaces are termed as self-supporting towers. The mass of these supports is higher, they need limited land area and are proper in various locations. More of the telecommunication, broadcasts, power transmission and floodlight supports are free-standing in other words self supporing towers [10].

(b) Guyed (mast) tower

Guyed supports accommodate tall at a very lower cost and material according to free standing towers due to the effective utilization of high-stiffness steel in the masts. Guyed masts are regularly guyed in three ways above a support radius of typically 2/3 of the support height and have a three-sided frame section. Tubular towers are also provided, strangely where icing is quite high. These masts are lighter than self-supporting towers, but a large free space requires to tie guy cables. There are other limitations to attach dish antenna on these masts and need high support blocks to hold the cables [10].

(c) Monopole tower

It is unique free standing pole and is commonly located over roof tops of high raised structures. Monopole towers are perfect for use when landing availability is difficult. It is preferred when the number of antennae needed is less or height of tower needed is shorter than 9 m [10].

1.4.2 Based on support cross sections

Based on towers cross section, towers classified into triangular, square, rectangular, polygonal, hexagonal and delta towers. Open steel lattice masts obtain the sufficient effective use of material and allow the production of very light-weight and rigid lattice tower by giving a light revealed area to wind and gust loads. Three-sided towers had lighter weight but give lighter rigidity in torsion. With the increase in a side, the weight of tower will increases. The increase is 10 % and 20 % for hexagonal and four-sided cross sections individually. The expenditure incurred for hexagonal towers is somewhat less if the supporting effect of nearby beams is examined [10].

1.4.3 Based on the type of material sections

Towers are distinguished into stiff and composite towers (with tubular and angle bracings) based on the sections used for fabrication. Lattice masts are commonly formed of bolted angles. Tubular legs and bracings can be industrial, particularly if the stresses are low sufficient to provide approximately simplistic connections. Supports with tubular portions can be less than half the weight of angle supports because of the decreased wind load on circular portions. Nevertheless, the additional cost of the tube members and the more complex connection details can pass the increase the weight of steel and foundations [10].

1.4.4 Based on the number of sections

Towers are distinguished based on the number of sections as straight, two, three and single slope towers [10].

1.5 Bracing System

Self-supporting telecommunication lattice towers consist of three-sided or four-sided lattice truss with regular rises among 30 m and 160 m. They concluded of main legs, horizontal and vertical bracing portions. Legs commonly composed of 90° angles in four-sided supports and 60° angles or tube in three-sided supports. Commonly, lattice masts are raised on the stiff land, although because of land limitations in cities and to pretend tremendous and pricey installations; towers are sometimes placed on the rooftop of the buildings. The availability of ground that provides the best installation conditions in residential areas extremely reduced providing no replacement but to perform rooftop towers (with a minor modification in place but not in rising). Therefore, those masts are normally shorter than the built on land [1].

In this study, seven different types of bracings are considering for rectangular base towers consist of (K, KD, Y, YD, D, XB, and X). And also, four different bracings for triangular base towers was provided consist of (K, D, XB, and X) on three different heights consist of 60, 50 and 40 m height. Figure 1.4 illustrate both rectangular and triangular base towers with different bracing patterns.

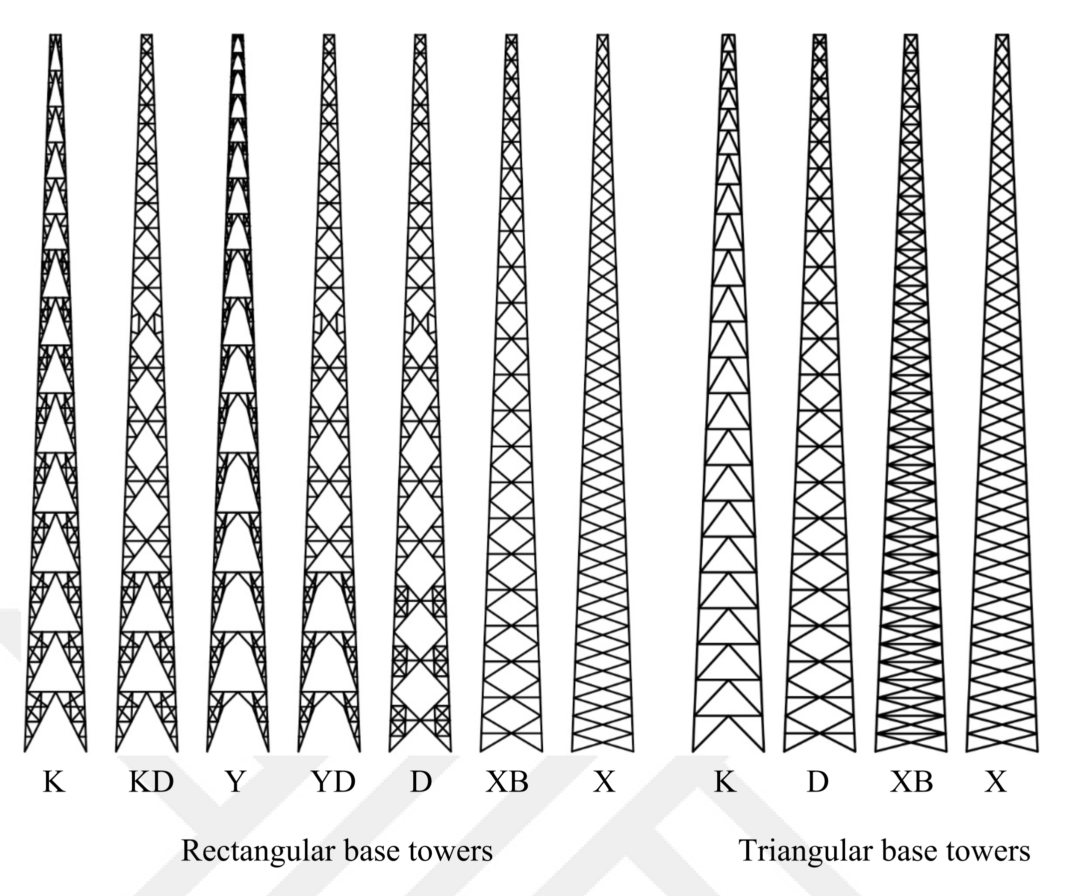


Figure 1.4 Rectangular and triangular base towers with different bracing systems

1.6 Objectives and Aims of the Thesis

Designing the best systems tend to be the main aim of every designer. However, due to the practical restrictions, this has been obtained via intuition, experience and repeated trials, a process that worked well.

The objective of this study deals with the best type of bracing system for both rectangular and triangular base towers with different bracing patterns and variation heights bay comparing both types of towers according to heights and type of bracings. To be able to rank the towers with regard to structural integrity, it is important to develop a procedure for modeling and analyzing the towers using the latest codes and also under earthquake loadings.

Once these objectives have been attained, by comparing output results of towers according to the type of towers, the bracing systems of towers will be compared with each other according to type and height of towers to identify the best bracing system.

1.7 Layout of the Thesis

The contents and arrangement of this thesis are expressed as:

Chapter 1 includes general, types of towers, lattice structures, aim and objectives of the thesis.

Chapter 2 is the literature review about analysis and design of the tower, bracing system, computer program and design codes.

Chapter 3 deals with design and analysis of the various component of communication towers by Using TOWER program.

In Chapter 4 discussion of results are presented.

In Chapter 5 conclusions based on the present results is given at the end and future works that preferred to be done.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Communication masts and broadcasting towers are regularly tall structures specifically built to support microwave and GSM antennas for communication purposes involves radio, TV, GSM, and internet traffic.

Communications support are used in different utilization in wireless systems of a broadband cell to cell systems to LMR (Land Mobile Radio) systems. Supports are often needed to raise antennas over tree lines and rooftops for a line of sight links [10].

The terms "tower" and "mast" commonly used conversely, the two words are different from each other [12]. A tower or mast is a high lattice structure with an approximately small cross-section, include a high ratio between the height of the tower and its maximum width. A tower is a free-standing, self-support structure consisted of a free top and fixed base pinned to the base of foundation and braced with guys, etc. [13].

Telecommunication towers commonly include two main groups: self-supporting towers, include monopoles, and mast towers. The primary manufacturing metal for mast towers is high-strength steel for the wire cables and regular structural steel for the mast members. Self-supporting towers are usually high lattice structures [12].

The purpose of this literature review is to survey previous researches completed in this area.

2.2 Lattice Steel Towers

Steel lattice supports utilized for different objectives, notably, telecommunications, radio and broadcasting, observation, transmission power lines and light supports, etc. [14]. Lattice structures are ideally satisfied for situations requiring a large load bearing capability, a low self-weight, a cost-effective use of supplies and fast fabrication and construction. For these reasons, self-supporting lattice towers are most commonly utilized in the field of telecommunication and power line system. Because one lattice tower design may be used for huge number towers on a communication power transmission purposes, it is vital to find an economical and highly efficient design. The design of the tower members must keep the tower geometry simple by using as few members as possible and they should be sufficiently stressed under more than one loading condition. The goal is to produce an economic structure that is well proportioned and attractive [15].

Steel lattice towers are commonly manufactured utilizing angles for the main legs and the bracing members. Bolts were used to connect the members together, each directly or by gusset plates. Using secondary bracings or redundant reduce the buckling strength of unsupported length, bracing members and main legs of the structures.

The lattice towers are more often than not analyzed assuming the components to be associated via hinged joints. Therefore, the loads in the angles are only axial. Taking this principle, the loads are exceedingly small to be neglected and for this reason, are not considered in the linear analysis models. Nevertheless, the major legs and the bracing components are not axially forced, and the redundant loads are not significantly small. It can be referred to below reasons [16]:

- The major supports are frequently continuous all the way through the joint.
- The joints are semi-rigid as a consequence of using more than one bolt in the links.
- The angle components are typically bolted throughout only one of the supports, and for this reason, the load transfer in the components is unconventional.
- AttribuTable to the support deformation of the angles caused by the determined bolt loads, the joints are flexible.

- A structure designed for high electric levels are usually flexible, and consequently, stability in the deformed design is highly recommended to be taken into consideration.
- Bending moments are increased by the compression member deformation.

As a result, the angle components of the tower come into contact with axial load and bending moments.

There are several published studies examining the influence of the combination of the wind and ice on the stability, strength, and serviceability of telecommunication towers in both a quantitative and qualitative way. Also, several reports on structural tower failures due to extreme wind and/or ice have been made; while there have been only isolated reports and research was done in connection with earthquake-related damage [12].

The requirement to invent a lattice support for reverberating dynamic response due to wind load occurs when the standard fundamental frequency of the structure is low enough to be sufficient by the turbulence in the regular wind [14].

While mainly the wind speeds design obtained by multiplying mean wind speed with gust speed portions to provide as variations in the wind velocity. Neglect the size of the structure and dynamic properties cause an unstable structure or conventional structure design. Loads of the structural formed by wind gusts depend on the damping, size and natural frequency of the structure. The failure of the structural which is straightly attribuTable to gust progress indicates the significance of these parameters in coming at a load of gust wind [14].

To decrease the excessive loading conditions due to icing and wind load, a study on retrofitting of tower structures is of importance and necessity. In the construction of towers, Steel angles are mainly used as members. Due to the asymmetry of member cross sections, the stability of these angles would be a complex argument [17].

Self-supporting lattice steel towers are the most common type of towers used for communication purposes efficient and have the largest resistance to wind and ice loads mostly used when there is a need for narrow tower locations and environmental conditions. Their designs can be modifying according to selected and a number of mounted antennas. Lattice steel structures are cost-effective also in the case of tall towers [18].

2.3 Reasons for Strengthening Towers

With the expanding of the telecommunication systems, the usage of freestanding latticed steel towers to support antennas has been intensive in the last few years. Due to the lightweight of these structures, wind forces are the primary concern in the design [19]. Furthermore, atmospheric icing is a design consideration for communication towers, especially in cold areas. Ice storms are a natural hazard that causes towers to collapse. Atmospheric icing frequently causes tower failures in cold regions [20]. In addition, self-supporting telecommunication towers are usually designed for the effect of wind and ice loads, without considering sesmic loads. These supports can be very significant structures in a telecommunication system, and the designer must warrant that they will carry out adequately in a critical seismic occurring, notably for masts placed in high-risk earthquake zones [21]. Other cases cause latticed communication towers damages corrosion and bolts slip in the joints.

The literature review indicates that the best type of communication towers are self-support and mast towers with having large face widths, face width of towers greater than or equal to the diameter of the mounted dishes. On the other hand, the worst towers are self-supporting towers with small face widths causing the top of towers yields large way and twist values. Otherwise, mast towers with small face widths often not built for large dishes may result in large structural failures and may result in a large twist and sway values. Monopoles frequently yield high sway and twist values, high to stiffen, usually further far away from tolerance to turn into for sway and twist to be workable to fix [22]. Figure 2.1 illustrate twist and sway effect of dish antenna on lattice towers.

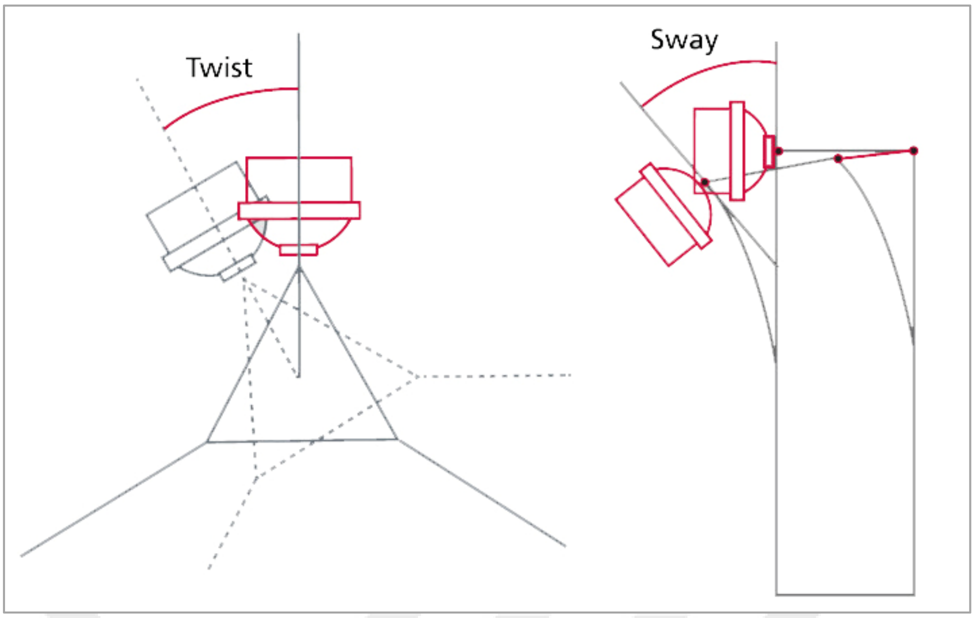


Figure 2.1 Twist and sway effect of dish antenna on lattice towers

2.3.1 Wind force

Chiu and Taoka [23] studied the dynamic response of self-supporting lattice towers under actual and simulated wind forces. A three-legged, 46 m tall self-supported lattice tower was instrumented to study its dynamic response to wind forces. The tower was then idealized as a space truss with masses lumped at the horizontal panel points. The analysis of the field measurements indicated that the measured dynamic properties of the tower agreed with the calculated values. It was also found that the assumption of uncoupled motion in the two principal horizontal directions is valid. The study showed that for free-standing structures the fundamental mode of vibration is predominant. The average damping for the fundamental period was found to be 0.5% of the critical viscous damping value.

Venkateswarlu and Kumar [24] studied the response of microwave lattice towers to random wind loads. The dynamic response was predicted using a stochastic approach. A spectral approach was proposed for calculating the along-wind response and the gust response factor. The gust response factor is defined as the ratio of the maximum expecting the wind load effect in a specific period to the identical mean value in the similarly period. A free standing four-legged tower of 101 m height was used in this study. The variation of the gust response factor along the height of the tower was calculated with and without the contribution of the second and higher modes. It was found that the maximum contribution of the higher modes of vibration to the gust response factor is only of 2%. The gust response factor obtained using the proposed stochastic method varied between 1.547 and 1.584 along the height. Using the formulae recommended by the Indian standard (IS:875-1987), Australian standard (AS 1 170-2-1989), British standard (BS 8100-1986) and American standard (ASCE 7-88-1990), the values of the gust response factor were found to be 2.03, 2.21, 1.93 and 1.89 respectively. Comparing these results, it was concluded that the standards values were 20% to 40% higher than the values obtained using the spectral method.

Agarwal and Garg [25] have evaluated self-supporting steel lattice supports for studying load effect of wind. It was observed that high changes have transpired in wind loads on supports and there are some hollows in the instant references which are to be clarified by further stringent wind drift fulfillment.

Savory et al. [26] researched on the latticed tower by ABAQUS modeling. High intensity winds may have resulted in tower member buckling, and the buckling produces large horizontal deformation. The wind loading models for the tower presented in this paper contained many simplifications. Nevertheless, wind load failure appears to concur with evidence from rehabilitating of the tower members.

Siddesha [16] studied the wind analysis on antenna towers with static and gust factor method and matched the lattice supports using square and angle hollow sections. The displacement was assessed at the peak of the tower as the prime factor. The different configuration of towers has analyzed by removing one member in the regular tower in the lower section of towers. Compared to the angle portions, noticed that using square portions most efficient for legs. Square hollow portions utilized in bracing simultaneously with the leg portions did not expose any significant modification of displacement. X-kind and M-kind bracings in square hollow portions for bracings and legs in the lower first panel of towers noted a maximum modification in displacement as corresponded to the regular towers having angle sections.

Richa Bhatt et al. [27] has studied a reaserch on the influence of modeling in lattice mobile towers in wind loading. Towers are analyzed for wind gust factor. For finding out the impact on supports, member forces, displacements, and maximum stress has been compared.

2.3.2 Ice load

Sundin and Makkonen [28] reported 13 TV towers that collapsed due to ice loads in the US alone during the last 20 years. Ice can accumulate on the towers of the liquid precipitation. Moreover, the build-up of ice can increase the portion area of the structure which will raise the wind loads. Ice load also cause signal interference, structural fatigue, wire stretch, ice load damage when the ice sheds, or complete tower failure. Figure 2.2 show the typical icing situation of the tower.

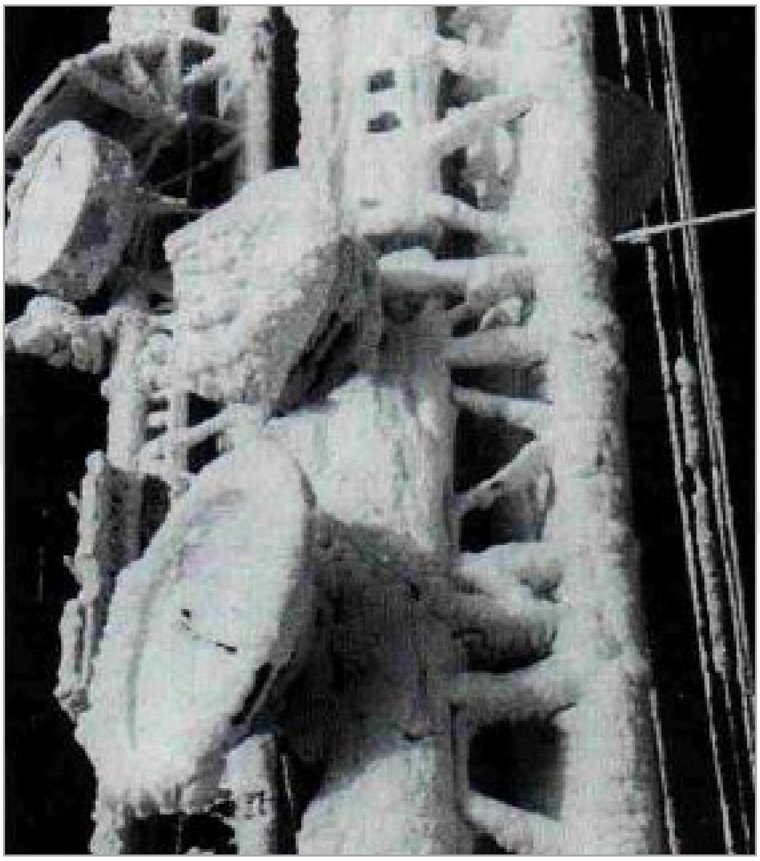


Figure 2.2 Ice load [28]

2.3.3 Seismic load

One of the first studies discussing earthquake effects on antenna lattice towers was presented by Konno and Kimura [29]. The study aimed at collecting information on tower mode shapes, natural frequencies, and damping properties. One of the towers used in this study was instrumented when Off-Tokachi earthquake occurred and the data collected was analyzed and compared with simulated results. The numerical simulation used a stick model of the tower with lumped masses and a viscous damping ratio of 1%. In some members, the forces due to earthquake loading were found to exceed those due to wind. In factl, some local damage and permanent deformations were observed at the base of the tower after the earthquake.

Mikus [30]studied the seismic response of self-supporting telecommunication towers. The aim of this study was to improve the understanding of the response of these towers to earthquakes. Six towers with height from 20 m to 90 m were used in the study: bare towers only, no antennas, attachments, ancillary components, etc. Three earthquake records were selected as the base excitation. A detailed dynamic analysis was performed using modal superposition, and it was concluded that the use of the lowest four modes of vibration provided sufficient accuracy. The frequency of the first axial mode of the towers was found to be in the range of 11 to 43 Hz. This range was either not included in the frequency content of the earthquake records used or corresponded to small amplitudes of input accelerations. It was therefore concluded that the vertical component of the earthquakes has a negligible effect on the towers studied.

A first attempt to find an equivalent static method for the analysis of latticed self-supporting telecommunication towers was made by Galvez [31]. The method was based on modal superposition taking the effect of the lowest three flexural modes of vibration into consideration. As self-supporting towers behave essentially as cantilever beams, he suggested the use of natural frequencies and mode shapes expressions developed for prismatic cantilever. The effects of taper ratio and shear deformations were included by means of correction factors to the classical solution for prismatic Euler cantilevers.

Latticed communication tower design usually considered wind effects as the sole source of lateral loads but rarely the influence of earthquake. Khedr and McClure [32] studied a simplified static method for estimating the member forces in self-supporting lattice telecommunication towers due to both horizontal and vertical earthquake excitations. The used method was modal superposition analysis. These results completely show that for high risk seismic areas, earthquake effects on towers may not be negligible compared to wind effects and it should be considered as a design check.

Two years later, Fatma and Jasbir [33] studied the optimal design of latticed tower subjected to earthquake loading. This research provided the continuous and discrete variable optimization methods to predict earthquake influence when design lattice tower.

(Amiri 2002) a detailed analysis must be performed on a model of the tower in question to analyze whether seismic effects are important and whether a more in-depth analysis is required [34].

Ghodrati and Massah [35] have examined the seismic sensitivity of 4-legged telecommunication towers based on modal superposition analysis. For this case, ten of the available 4-sided self-supporting communication supports in Iran were considered under the effect of earthquake and wind loadings. Some practical similarities have been performed that can encourage designers to approach the dynamic response of supports under earthquake loadings.

Rajasekharan et al. [36] studied the three different of lattice towers for 30 m, 40 m and 50 m heights with different kinds of bracing systems to consider the wind load impact on 4- sided lattice tower for wind region V and VI using the method of gust factor. They also investigated the earthquake impact on the lattice tower by carrying out the modal analysis and response spectrum analysis for zone 2 to zone 5 and consummated that the member stresses in the lower leg of XX bracing type tower are higher as matched to another bracing tower types. The frequency of the Y bracing pattern tower with presented the smallest natural frequency after its stiffness was determined to be greater due to increased weight of the towers as correlated to other types. It was perceived that from 30 m to 40 m height tower, the increase in displacement is approximately linear yet as the increases in height from 40 m to 50 m, there is a sharp increase in the displacement in all zones.

2.3.4 Bolt slip

Ju et al. [37] have studied the structural behaviour of the butt-type steel bolted joints by using three-dimensional elastoplastic finite element methods. The bolt clearance, the bolt head, the washer, the deformable bolt and the friction were included to simulate the actual structural behaviour of bolted connection. Studies are conducted to evaluate the plastic strain field in the local area, the nominal applied force of the bolted connection and the crack behaviour of the A36 steel plate. The numerical results are compared with AISC specification and found to be satisfying notwithstanding the difficulty of stress and strain ranges throughout the loading steps. When the steel was achieved the nonlinear behavior, the bolt nominal forces received from the finite element analyses is essentially linearly proportional to the bolt number ordered in the connection. To calculate the ultimate load for the bolt shear failure of the connection with appropriate bolt spacing and end distance, it is appropriate to neglect the plate deformation and the bolt bending effects, which are major assumptions in the AISC specification for the bolted joints. Linear elastic fracture mechanics can yet be implemented to the bolted joint problem for the main section of the loading, yet despite this problem admits highly non-linear structural behavior.

Latticed communication towers consist of legs, bracings, and secondary bracings and cross members. A small rotation occurring to bolt slip in the joint will generate an additional deformation in the support, which is complex to predict and cannot be considered for in the analysis. The slippage of bolts may have some effects on a deflection, but it does not significantly influence the ultimate strength of lattice structures. In the analysis and design of towers, deflection effects are frequently neglected. Nevertheless, if the tower is adequately high and flexible, then deflections will be enormous and cannot be neglected because they can generate secondary stresses [38].

2.3.5 Corrosion

Corrosion is a common damage on lattice tower, which may result from several reasons including temperature fluctuations, extensive use of deicing salts and improper coating. The load capacity of latticed tower reduced significantly when this tower corroded. In addition, it was found that coating on the latticed tower gave a relation with the tower corrosion. Improper painting system and maintenance may also cause corrosion of lattice tower. The most common and conventional method to repair corrosion was to add plates. Recently, Carbon Fiber Reinforced Polymer (CFRP) and other fiber material were applied in corrosion repair, the material which had a lighter weight than conventional material [39].

2.4 Bracing System

Bracing systems are a very common form of construction, being economical to construct and simple to analyze. Economy reaches of the low-priced, nominally pinned connections between members. Bracing, which provides stability and resists lateral loads [41]. For telecommunication supports, diagonal bracing is the commonly bracing systems used for towers, produced for both axial forces of compression and tension. K, KD, Y, YD, D, XB and X bracing systems are providing for modeling the four-sided and three-sided in this study.

Lattice frames act as vertical trusses, and many different forms commonly used which support the wind loads by cantilever action. The bracing portions can be providing in a variation of forms produced to carry individually tension, or alternatively tension and compression. While designed to take only tension, the bracing is built up of crossed diagonals. Depending on the wind direction, one diagonal will take all the tension though the rest is supposed to settle inactive. Tensile bracing is less in cross-section than the comparable strut and is normally built up of a back-to-back channel or angle sections. While intended to resist compression, the bracings become struts, and the most typical arrangement is the `V' brace [10]. Figure 2.3 shows Tension and compression system of the bracing system. The bracing K type is necessarily designed for compression and tension bracing members. The main benefit of this bracing type is the horizontal struts supported at mid-length by the diagonals; it is chiefly useful for heavy river-crossing towers [40].

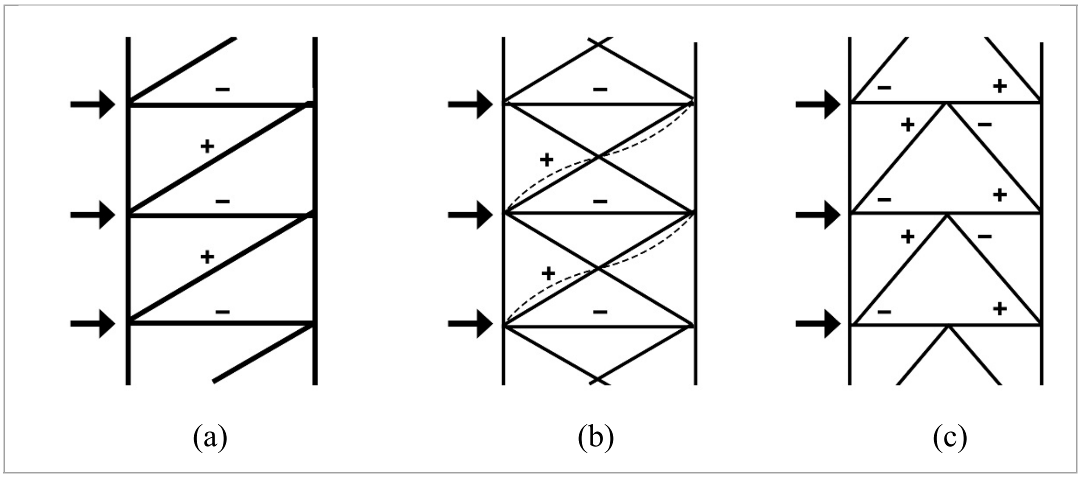


Figure 2.3 Tension/compression system of the bracing system (a) and (b) tension only system (c) tension and compression system.

2.4.1 Truss elements

Lattice towers are often modeled as linear-elastic truss portions because the angle members of the tower originally carry axial loading with minimum bending stability. The end of trusses member's joints is exemplified as frictionless pins, free to transpose in each direction except externally restrained by a particularized boundary condition. In actuality, nevertheless, the exemplified pin connection hardly happens. Where the bending stresses are negligible, truss portions utilized in positions matched to the axial stresses [15].

Krishnamurthy [42] explains their formulation particularly. Angle members cannot resist lateral loading If it is modeled as truss portions. Any dead or wind load is acting on the truss portion necessity be divided to the two connecting joints. It is standard usage to combine half of the self-weight of the portion to each of the two joints the connect member.

2.4.2 Beam elements

The linear-elastic truss assumption is an accurate model for tower members that are subjected to only axial tension or compression. In real communication towers, eccentrically applied loads, lateral wind, and dead loads, initially crooked members, connection rigidity, members not connecting at a single point, and the continuity of the main members can generate bending moments and shear forces to develop. The bending moments in support members generated by joint rigidity and member continuity, widely ignored despite the experimental evidence, can be as significant as axial stresses in certain cases [43].

The conventional linear-elastic truss model should be replaced with a beam element model when significant bending stresses are present in tower members. Some structural analysis programs allow several different types of elements to model the same problem. The stiffer main leg members, subjected to the largest bending stresses, could be modeled as continuous beam elements while the smaller angle members could use the conventional truss model. Some researchers assume that the multiple-bolted end connections offer enough restraint to regard the connection as rigid, and model the entire transmission tower as an assembly of beam elements [44].

Krishnamoorthy [42] describes his formulation in detail. The beam elements considered here do not include the effects of shear deformation or secondary moments (the beam-column effect). In the more advanced inelastic analysis, tower members may be modeled as general thin-walled beam-column elements, capable of yielding and buckling.

Over the past several decades, considerable studies have been carried out to capture the structural behavior of angle members. Kemp et al. [45] performed a series of 13 tests to investigate the property of cross-bracing systems in the tower structure. It could be concluded that the end eccentricities caused by bolting one leg of each bracing to the main legs would significantly influence the displacement within the bracing system. The intersection joints of tension and compression bracing system deflected along the out-of-plane direction even at low loads and bending moments were then induced.

At global structure level, Albermani et al. [46, 47] established an analysis model which took the influence of both geometric and material nonlinearities into account. The tower was modeled with beam-column and truss elements. The analysis results were found to coincide well the corresponding test results.

Based on the experimental works, some retrofitting strategies have been proposed by researchers to improve the wind-resistant performance of towers. Albermani et al. [46] and Xie et al. [48] considered the feasibility of adding diaphragm to strengthening tower structures. They found that adding diaphragm significantly improved the structural performance and thus increased load-carrying capacity of the structure under strong wind.

The survey study shows that the manageable design to increase the compression capability of the leg members of towers is to add added bracing to the tower to decrease the slenderness ratio of the crucial member. The effectiveness of this approach is dependent on the type of bracing and the slenderness of the original leg member. Added bracing can also be presented as horizontal diaphragms, which was shown to be effective. Based on the aforementioned findings, some important design codes have been developed [49].

Jesumi and Rajendran [50] Studied five lattice steel towers with various bracing patterns like as the single diagonal, X-B, K, Y and X-X bracing systems for 40 m and

50 m height with a width of the base of 2 m and 5 m sequentially. The tower has a height 40 m has 13 panels, and 50 m height tower has 16 panels. 70-72% of the height as long as for the tapered section, and 28-30% of the height is provided for the straight section of the tower. From the results obtained, Y bracing has been found to be the most economical bracing system up to a height of 50m.

In lattice towers, the performance of cross-bracings has examined by Kemp and Behncke [45]. The cross bracings have presented complicated performance, and the bolt number in the connection of the bracing to the main leg members have been visible in the results.

Silva et.al. [51] presented a study on comparative analysis of steel telecommunication tower subjected to seismic and wind loading. Article about a structural option analysis modeling approach for the lattice tower design concerning all the substantive structural forces and moments combining three- dimensional beam and truss finite elements. Comparisons of the two design as mentioned above methods with a third method be based on the utility of spatial beam finite elements to design the main structure and the bracing system on two built steel communication support 40 m and 75 m high has been studied. Commonly, in all the states analyzed the maximum stress amount for the lattice tower designed to support on the three reviewed methodologies were considerably modified. The lateral displacement amounts were not considerably modified during the common truss model, the beam model or the beam and truss together model was supposed.

Al-Mashary et al. [52] presented a paper on the effective bracing of trussed towers against secondary moments. The research determined that unsuitable bracing configuration of the primary and secondary braces produced high secondary moments. Therefore, it is important to distinguish the economic bracing patterns for a given range of tower height. The major purpose of the study is to determine the economic bracing pattern of lattice steel towers. Studies based to analyze masts with various heights, and various bracing forms have been produced. Towers have been analyzed for wind loads using STAAD Pro, to compare the maximum displacement of joint of each tower. For estimating and comparing the weight of each tower, it has been carried out for the optimal design. The results have been utilized to identified the economic bracing patterns a particular group of high towers.

Kemp and Behncke [45] studied the behavior of cross-bracings in lattice steel towers; The cross bracings have shown complicated attitude, and bolt numbers used for connecting bracing with main legs have been visible in the results.

Amiri and Booston [53] studied the dynamic response of antenna- supporting structures. In this view, self-supporting steel communication supports with various heights were evaluated as per the wind and seismic loads. An evaluation made between the wind and earthquake loading results.

There are ten types of bracing systems for a lattice tower configuration. Those ten types are as follows [13]:

1. Single diagonal bracings: which is the simplest form of bracing and tend to be used for towers up to 30 m height. The wind shear at any level is shared by the single diagonal of the panel. Figure 2.4 (a)
2. X-X bracing: which is a double diagonal system without horizontal bracing and astatically determinate structure. X-X bracing is used for towers up to 50 m height. Figure 2.4 (b)
3. X-B bracing: which is a double diagonal system with horizontal bracings and the structure is statically indeterminate. The horizontal components are redundant and carry only nominal stresses. These bracings are rigid to a certain extent and used for towers up to 50 m height. Figure 2.4 (c).
4. K-bracing: which have the ability to provide a large head room, and for this reason, K-bracing can be used in lower panels where the large head room is necessary. The structure is statically determinate, and this type of bracing can be used for towers of 50 to 200 m height. In the majority of the transmission line towers, the lower panels are either K- or Y- braced whereas the upper panels are X-braced or XB-braced. Figure 2.4 (d).
5. X B X bracing: which is a combination of XX and XB bracing as horizontal members are provided only at the level of a crossing of diagonals. The structure is statically indeterminate. However, the length of the diagonal is decreased. The system is appropriate for structures of 50 to 200 m height. Figure 2.4 (e).
6. W-bracing: which uses a number of overlapping diagonals. The system is statically indeterminate. However, the effective length of diagonals is decreased,

and the system is rigid to a certain extent and can be used for structures of 50 to 200 m height. Figure 2.4 (f).

7. Y-bracing: which has the ability to give larger head room and can be used for lower panels. The system is statically determinate. In the majority of transmission line towers, Y-braces or K-braces are used for lower panels, and X-B braces or X-braces are used for upper panels. Figure 2.4 (g).
8. Arch bracing: which can be utilized for wider panels providing greater head room. The system is statically determinate. Figure 2.4 (h).
9. Subdivided V-bracing: which can be used for tall towers of communication systems, radio, and TV transmission, etc., for heights between 50 to 200 m. Figure 2.4 (i).
10. Diamond bracing: which is a typical lattice system used for towers of 100 to 200 m height. The base width is approximately between 1/5 and 1/6 of the height. Rigid horizontal diaphragms are used at the top and at middle sections, rather at intervals of 25 to 30 m, which can help the cross-sections to grow the torsion stiffness. Figure 2.4 (j).

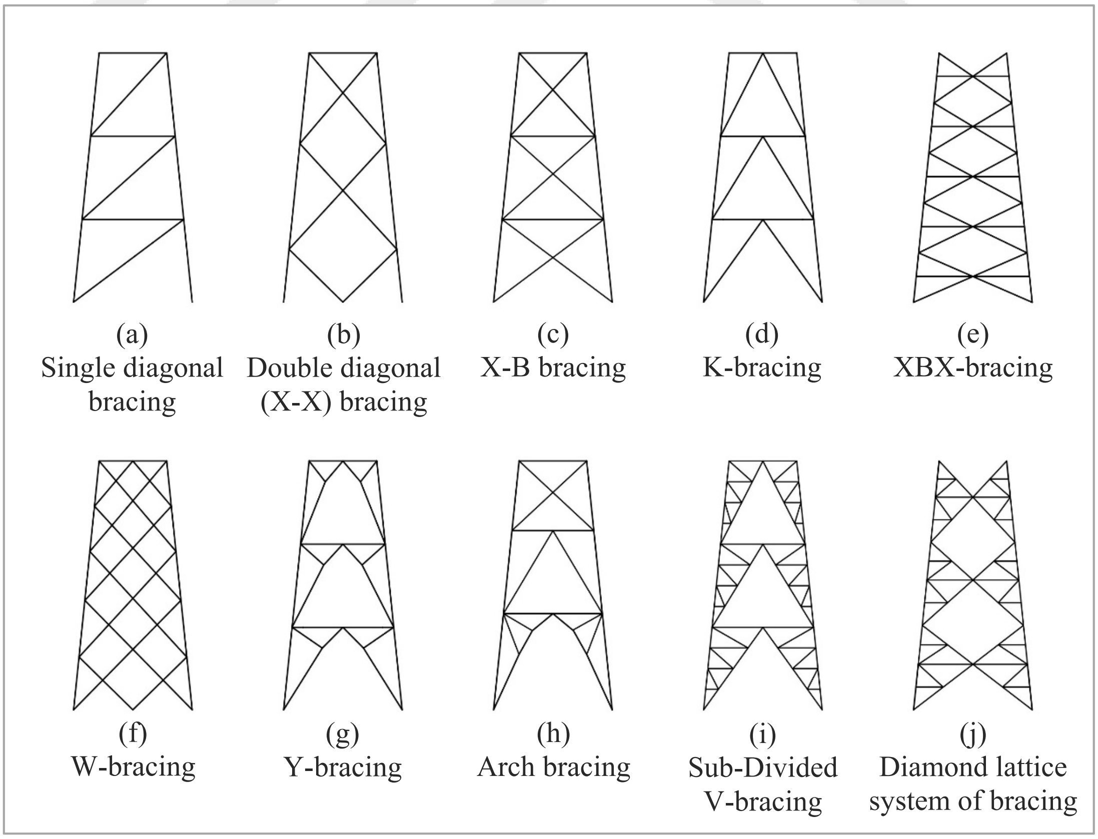


Figure 2.4 Lattice tower configurations with bracing systems

2.5 Analysis of Towers

The finite element technique is a numerical technique, adequate regular computer upheld, which is Figured out how to accomplish relative solutions for the detecting equations of difficult inquiries. In few examples, answers for these issues can not be gotten diagnostically. An analytical solution is a logical expression that can give a correct estimation of the field variable (displacement, temperature) at any area in the body. In the finite element method, the field variable is approximated utilizing interpolation functions sorted out between discrete focuses. Most practical engineering issues include muddled geometry, properties of material, or condition of loadings, and consequently require a numerical solution procedure, for example, the finite element method [15].

The method of the finite element can be theorized as an enlargement or generalization the method of the stiffness (concerning framed structures) to two dimensional and three-dimensional continuum queries, like as shells, plates and solid bodies Ghali [54]. The finite element concepts utilized in continuum queries can be utilized to formulate the stiffness method of analysis using the member of a framed structure as an element Krishnamoorthy [42]. Accordingly, the element stiffness matrices obtained from beam and truss portions utilizing the stiffness scheme of analysis are indistinguishable to the stiffness matrices derived utilizing finite element theories.

For each query using the finite element technique, certain moves must be a follow-up. The physical system must discretize into smaller finite elements. The elements may be one-dimensional, two-dimensional, or three-dimensional depending on the nature of the query. For lattice towers, each angle member usually formed as a one-dimensional portion, or line portion, having one node at each end of the portion. The concealed degrees of freedom or the unidentified primary counted at these nodal positions. An interpolation function needs to chosen that estimated the number of the variable unknown inside an element. The function represented regarding the nodal values of the element. For example, the unidentified quantity within a beam element (the transverse displacements) can be fully explained one the degrees of freedom for all end node identified. The determining constitutive relations and equations are later determined. The equations of the element are formed utilizing the energy methods, direct equilibrium method, or the weighted residuals method [42].

Jesumi and Rajendran [50] modeled five steel lattice towers with different bracing configurations. The towers have been formed utilizing geometric coordinates. The property of member was distributed to all portion of the tower. The diagonal bracings, leg and horizontal bracings are considered with angle portions. The towers were analyzed with STAAD Pro. V8i, supposing it to perform as a space structure including pin joints, was carried out for dynamic wind loading and optimized design. The weights and displacements of the supports achieved were corresponded to arrive at an optimal solution. The results showed that the 50 m height Y bracing system are the most economical bracing type.

Razak et al. [55] considered the analysis of the four legged self-supporting steel telecommunication towers based on different types of seismic zones, peak ground accelerations and soils, in Malaysia using the SAP2000 program, model improvement with evidence to the International Building Code (IBC2000) and Euro Standards (EC8). Linear static analysis concerning displacement of joints and reaction base shears with analysis of axial forces being performed. The research has able to confine the seismic reaction encountered by the shaped towers on the various kinds situations unstable on them.

Sharma et al. [56] studied comparative analysis of steel telecommunication tower subjected to seismic and wind loading. Comparative analysis is being carried out for various bracing systems for wind region I to VI and seismic zones II to V of India for heights of 25 m, 35 m and 45 m of steel communication towers. The towers are designed with 5-different types of bracings: K type, XBX-type, V-type, W-type, XX-type for lower portion and X-Bracing for the upper portion of the tower. STAAD Pro. V8i has been utilized for modeling, analysis, and design of towers. Analysis result complemented that subdivided V-Bracing type provides a satisfying result in an analysis of the wind load, response spectrum analysis and modal analysis for all theorize earthquake and wind zones specified in IS code.

Jatulis et al. [57] studied the discusses rational design, considering the mass criterion, of a lattice steel tower, made up of solid round steel bars affected by the static load. The space between the truss majors and the slope of the bracing portions are set as the principal variables. A numerical model of mast structure was provided utilizing MathCad 14.0 program. The situations of the exposed safety case of the tower's portions were valued. The numerical experiment was used as a basis to present the dependence of the tower's optimum width and the optimum inclination of its struts on the tower's height, wind load intensity, and steel strength. Recommendations were provided as well. It has been distinguished that the optimal slope of bracing portions in a supports of solid round members is essentially sovereign of the height of supports and the wind load concentration; its average amount is 35°. The struts slope in designs of before-mentioned towers must not surpass 39°.

AbdulMuttalib and Ikhlass [58] studied analysis and optimum design of self-supporting steel communication tower. A specific procedure is utilized to describe the tower as an equivalent hollow tapered beam with variable cross section. Four types of towers with X and K bracing patterns, with equal and unequal panels design under the formulation of the problem. Based on the requirements of ANSI/EIA standards codes for allowable stresses and displacement in the members and at the antenna location, the formulations of design are constraints. The unconstrained minimization technique is applied to complete the process. For analyzing of the towers, direct stiffness method provided. The FORTRAN program is produced to design the tower as a uniform beam, and produce the tower joints, members, analysis, design and to find the optimum layout. Four types of the tower are analyzed using various case loads. In two directions the impacts of seismic and wind loadings are taken. Moreover, to attempt the optimum design, two points of the antenna identified in the process. The minimum weight of tower obtained for X-brace type with unequal panels, furthermore, an optimal design is satisfied when the angle of the main stem is equal to (87°).

2.6 Computer Program

Bergstrom [59] earlier than applying the computer to tower analysis concerns, significantly indeterminate transmission structures were separated into determinate planar trusses with forces affecting the same plane as the truss.

Marjerrison, [60] stated that these estimated algebraic or graphical methods had required conventional hypothesis that resulted in overdesigned towers. A number of computer software has been provided to structures analyze regarding the elastic properties of the components and determine the displacements and axial loads using the analysis stiffness way.

Al-Bermani and Kitipornchai [44] have merged some nonlinear effects to one computer software (AK-TOWER) that predicts the answer of steel lattice towers up to the ultimate load. Origins of nonlinearity involve geometric nonlinearity, material nonlinearity, the flexibility of joint, and slippage of joint. Nonlinearity of geometric can be considered by including the impact of primary stresses and variations of geometrical in the structure while loading. For angle members, modeling of material nonlinearity was based on a plasticity parted mode1 coupled with the theory of a yield surface in the space of force. The angle members in the tower used as asymmetrical thin-walled beam column components with stiff conjunctions. The joint flexibility effect of j could be included by modifying the tangent stiffness of an element using a proper moment-rotation kinship for a joint flexibility, given data is recognized. The bolt slippage effect can further be adding. This software further uses the formex formulation to generate the geometry, the loading conditions and the support conditions of towers. Formex algebra is used to greatly decrease the amount of manual input and acts as a preprocessor to the nonlinear AK TOWER program.

At the beginning of tower design, the engineer has used graphical analysis methods, developed to a professional-art for application to 3D space truss towers. With the introduction of the computer, processor computer analysis capabilities using structural matrix algorithms were developed. Tower computer program was developed particularly for transmission line latticed structures. Now with the analytical power of individual computers, the computer programs of tower analysis are numerous, such as Tower, ERITower, RISAtower, Tower Numerics Inc. (TNX), etc. [61].

The structural computer model is based on the behavior of the tension and compression of the entity tower members. These tower analysis programs are based on the linear elastic structural act, whereby elements are considered as axially loaded with pinned connections. The member forces obtained from the computer model are compared to the allowable member capacities.

The first-order linear elastic software supposed the angle members have modeled truss components, pin-linked at the joints. The members have been able of carrying tension and compression, and the tower loaded design has been matching to the unloaded pattern, and the minor influences of the deflected shape have been unnoticed. Minor additional components, used to afford intermediate bracings, can be neglected in such kind of analysis as no consequence on the forces in the load-carrying principal elements [62].

2.7 Design Code

Due to the dependency of communication networks on tall lattice towers, some guidelines are needed for controlling their force and displacement response to applied loads on towers. Under very strong design-level ground motion it is possible that these tall masts would not maintain their serviceability; also, it is important that if minor damages occur, they should not cause any loss of functionality shortly after the earthquake, wind and other applied loads. In order to achieve this, structural engineers require specific design guidelines [12].

2.7.1 American standard ANSI/TIA/EIA-222-G

This standard is dedicated to mandatory minimum design requirements for all types of antenna-supporting structures. It lists four seismic analysis methods of increasing accuracy and complexity with limitations for their applicability to various tower types, geometry, and height. The methods are equivalent static lateral force, equivalent modal analysis, classical modal superposition analysis, and time history analysis. Self-supporting and guyed masts are required to be modeled in three dimensions, as either an elastic truss model or a three-dimensional frame-truss model. Also, the analysis must consider the geometrically linear and non-linear P-Δ effects on the mast.

The ANSI/EIA/TIA-222 is a brief of American National Standards Institute/Electronic Industries Alliance/Telecommunications Industry Association, structural steel standards for antenna and towers steel structures. Work is under way to determine the changes in the next review of ANSI/EIA/TIA-222. Planned changes to align the standard with the newest version of ASCE-7. Moreover, it will incorporate the use of 3 seconds guests instead of the fastest mile wind speed and the use of the design load reduction factor rather than the design of the allowable stress. The expected changes are other considerations involved seismic loading, safety facilities, foundations, and the analysis of available structures.

EIA RS-222 (Revision TR-116 and RS-194) published as a first country standard for structures supporting antennas in 1959. The standard consists of 11 pages identify the nation into three uniform wind pressure zones: A, B, and C considered the wind load on the full length of the structure and was measured in pounds per square foot (PSF). Standard has explained that "coating the ice was not clearly mentioned as icing rarely occurs concurrently with a maximum wind load.

In 1987 EIA 222-D is replacing of wind pressures by the basic wind speed (mph). Wind speed presented in a new map. The value of the speed of the basic wind as a function of the height of the tower was increased to above 10 m.

In 1991 TIA / EIA 222-E first iteration of the code to be determined by the Telecommunications Industry Association (TIA) and Electronics Industries Association (EIA).

In 1996 TIA / EIA 222-F has been provided use of this standard through the largest build out of towers and currently is the most standard used in the country. Increasing the scope of the previous version to include the ice loading effects. Essentially, two methods for the analysis of the ice it provided.

In 2006 TIA-222-G with the standard's changed very radically to the new revision under the letter G adopted by some states and regional authorities. Tower analysis is now compatible with the other structures. Design philosophy to limit the states have changed from the design of the design allowable stress. With the recognition of the International Building Code, most of the states and municipalities ultimately inserted in the G revision [63].

2.7.2 Canadian standard CSA-S37-01 (R2006)

In Canada, guidance on earthquake-resistant design and seismic analysis of communication structures is contained in Appendix M of Canadian standard CSA-S37-01(R2006).

The 1994 edition of this standard was the first to devote an appendix to the seismic analysis of telecommunication towers. In January 1995, the devastating Kobe earthquake in Japan caused extensive damage to structures and led to increasing interest and motivation for establishing earthquake resistance guidelines with a specific focus on telecommunication towers. The Federal Emergency Management Administration (FEMA, 1998 and 2000) of the United States increased the attention of the existing National Earthquake Hazard Reduction Program (NEHRP) to non-structural systems, non-building systems, and lifeline systems such as telecommunication towers. The results of these efforts accounted for the addition of the seismic regulation section to the ANSI/TIA 222-G Standard for the first time in 2006, which was done with the help of the American Electrical and Telecommunication Industries Association (TIA/EIA). In the meantime, the American Society of Civil Engineers (ASCE) had published its guide for the Dynamic Response of Lattice Towers which comprises a special chapter related to seismic response. Coming back to the Canadian Standard CSA-S37, it should be mentioned that a completely revised edition will be issued in 2012 and that seismic design checks will become mandatory for all post-critical towers playing a role in post-earthquake response and rescue. Three safety level categories are defined:

Prevention of injury or loss of life (Life Safety) - Performance Level 1 (PL1): Preservation of life safety is the minimum requirement for all towers located in moderate to high seismicity zones.

Interrupted serviceability- Performance Level 2 (PL2): These towers may become unserviceable during the earthquake but should recover their functionality shortly after. Existing post-critical installation should meet this level of performance.

Continuous serviceability- Performance Level 3 (PL3): This applies to those towers that should maintain serviceability during and after an earthquake as they are essential structure in a telecommunication network or towers that are utilized in the control of electrical utility systems [64].

2.7.3 IASS guidelines (International Association for Shell and Spatial Structures)

IASS-WG4 published its guidelines for the analysis and design of guyed masts in 1981. In fact, one of the objectives of this document was to provide design guidance for National and International Code Drafting Committees, which explains why it has never been updated.

The seismic analysis recommendations for guyed masts now seem simplistic in view of current analysis tools, and essentially pertained to:

1. Simplified quasi-static analysis using a lateral load proportional to the tower weight (as suggested in most building codes at the time).
2. Linear analysis using modal superposition.
3. Representing the guy wires by linear springs.
4. Using lumped masses.

Since seismic design loads represent extreme events, only their combination with dead loads due to self-weight was suggested (that is no ice) with the assumption that earthquakes occur in still air conditions [65].

CHAPTER 3

DESIGN AND ANALYSIS

3.1 Introduction

Communication towers are complex structures with many members. The large of members makes these towers difficult to analyze by hand, due to the many calculations necessary. To make analysis run more quickly and accurately, computer software has been designed to do a finite element and modal analysis. these tools allow for a model to be created fairly quickly for member sizes and connection types to be changed easily. They also allow for multiple loads and load combinations to be applied to the structure at the same time. Figure 3.1 shows typical K bracing system type of rectangular and triangular base lattice tower.

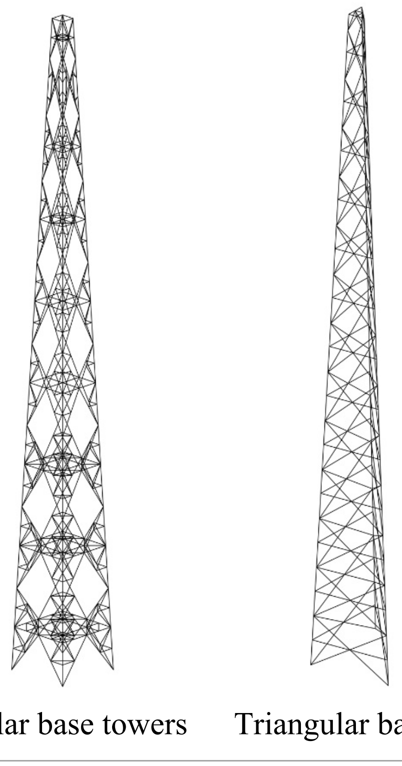


Figure 3.1 Typical K bracing type of rectangular and triangular base lattice tower

Seven types of towers for rectangular base towers with different bracing systems, such as (K, KD, Y, YD, D, XB, and X) as shown in Figure 3.2, and Four types of towers for triangular base towers with different bracing systems, such as (K, D, XB, and X) as shown in Figure 3.3. with using various heights of 60 m, 50 m and 40 m height are analyzed and designed based on the ANSI/TIA standards and using TOWER program (Power Line System, Inc. v. 9.20).

TOWER is a powerful and user-friendly software for analyzing and designing of steel latticed structures which are exploited in both telecommunications and electric power lines. Self-supporting and guyed structures can be designed. The software applied design checks of towers for different specified loads. Linear and nonlinear (p-delta) analyses are performed to analyze both rectangular and triangular base towers for ice, wind, service and temperature loads to determine the displacements and forces in the structure. Once the analysis has been performed, PLS TOWER creates an extensive report consisting of all inputs into the software and results for the tower.

TOWER is Microsoft windows based software for analyzing and designing steel lattice towers. The tower which has K, KD, Y, YD, D, XB, and X bracing systems for both rectangular and triangular base are quickly a common body of the structural tower designed from components.

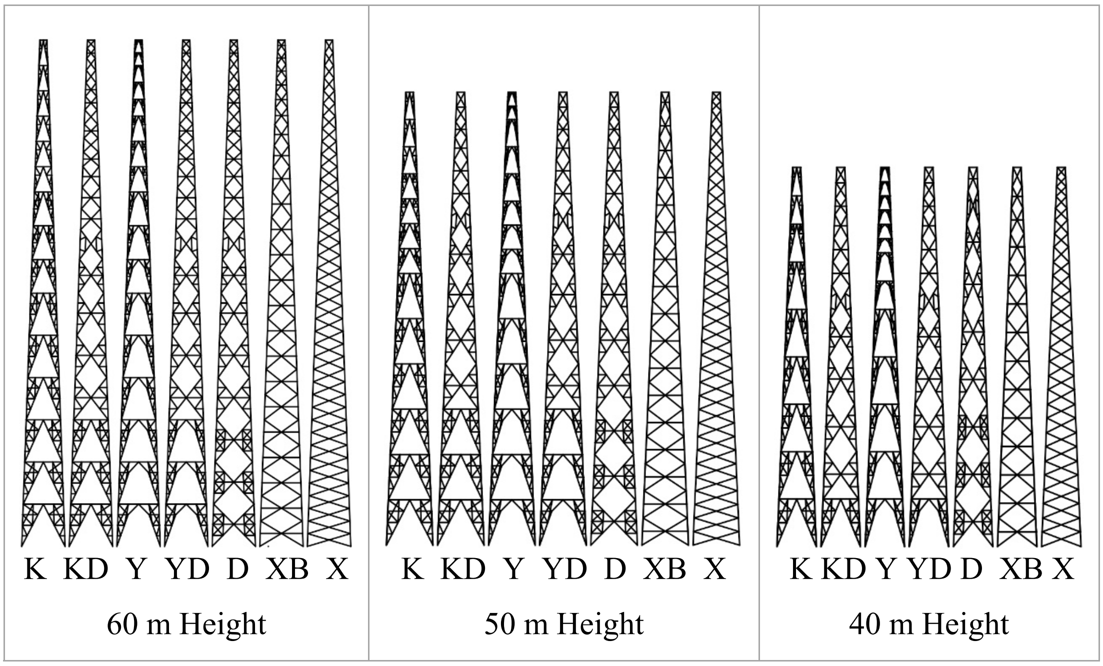


Figure 3.2 Rectangular base towers with different bracing systems

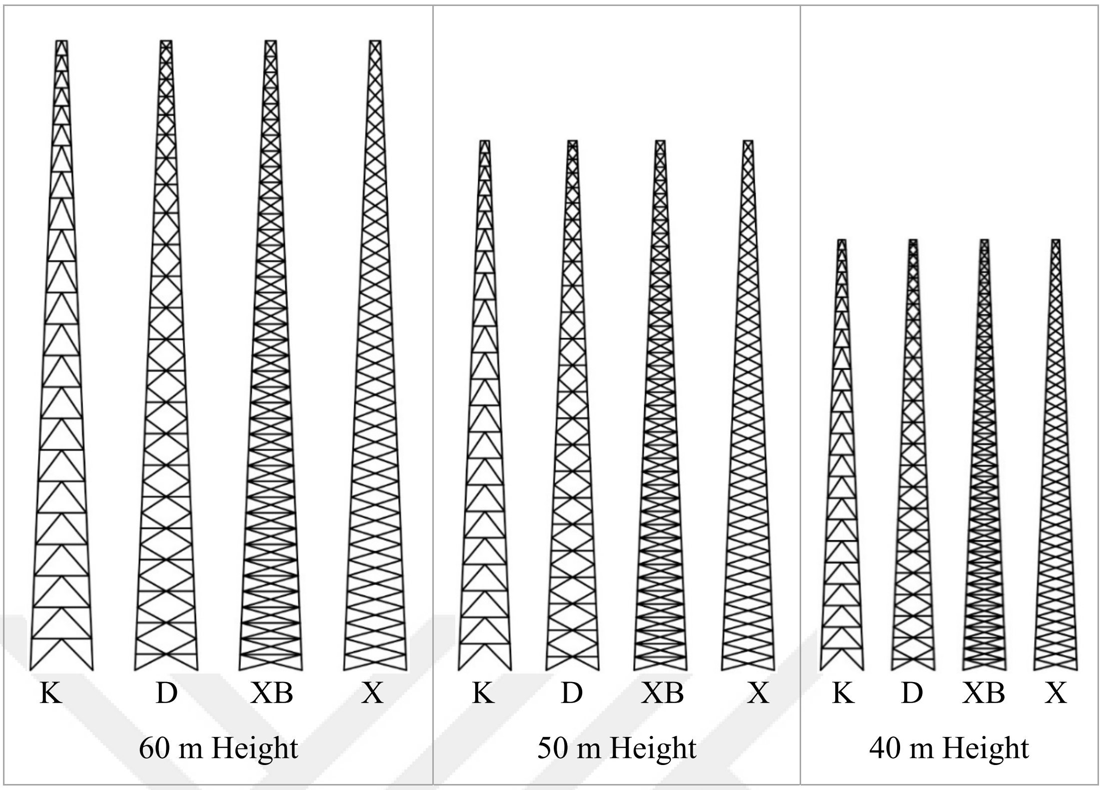


Figure 3.3 Triangular base towers with different bracing systems

Creating a structure is simple where the user can select the needed components from a collection of available elements and telling the software how to gather these elements and how to mix and match the various elements also. This gives the user the ability to construct arbitrarily complex towers. TOWER has the benefit of balancing the structure in joint and member generation, duplicating symmetrical linking points and members about all axes. Even huge and complex structures can also be modeled in this software in less than the day hours [66].

In this study, the (ANSI/TIA) American National Standards Institute/ Telecommunications Industry Association standard is used for the design the both tower members, calculation of the loads, and the combination of loads. These standards consist of general requirements and common specifications for calculating wind, ice, earthquake and service load for self-supporting and guyed towers.

The steps of the tower analysis and design using TOWER program is illustrated in details in subsequent sections.

3.2 Materials

Using a spreadsheets format of *components menu* in TOWER program as shown in Figure 3.4, allows the user to define the steel material, angle, bolt properties and equipment library used for defining selected antennas, tower cables and etc. Using libraries significantly increase user efficiency by mainly decreasing the inputs, which also limits the error probability. European standard steel was used for steel material properties. For leg members of rectangular and triangular base towers, S355 JR low alloy structural steel was used. And for bracing members of towers, S235 JR non alloyed steel was used. Table 3.1 present properties of S355 JR and S235 JR materials in details. The symbol S denotes the fact that it is Structural Steel and associated with the minimum yield stiffness of the steel (tested at a thickness of 16mm).

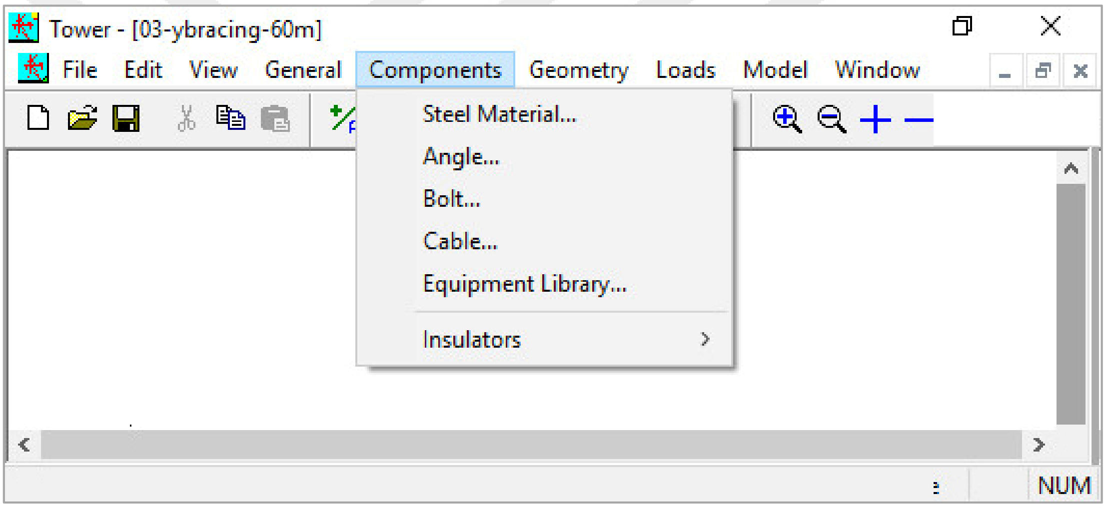


Figure 3.4 Collections of the components menu

Table 3.1 Mechanical Properties of S355 JR and S235 JR Materials [67]

Grade	Elasticity modulus E (MPa)	Poisson's ratio ν	Volumetric mass density ρ (kg/m3	Tensile strength (MPa)	Yield Strength (MPa)
S 235 JR	205,000	0.3	7,850	340	235
S 355 JR	205,000	0.3	7,850	490	355

The connection type mainly used was bolts as two different types M12/8.8 and M16/8.8. In accordance with ISO 898-1 standards, quality 8.8, the allowable shear stress on bolts is 233 N/mm^2. Otherwise, the allowable bearing stress in bolts of mild steel S235 JR is 432 N/mm^2, and for high steel S355, JR is 612 N/mm^2.

Steel that is galvanized, or otherwise protected, shall have a minimum thickness of 3/16 in. (4.8 mm) when exposed to corrosion at the ground level or below. If weathering steel is to be used, consideration should be given to increasing the suggested minimum thickness. According to ISO 898-1 standards.

Minimum thickness of structural steel members:

The main leg angles	8 mm
Bracing members	5 mm
Redundant members	5 mm

Table 3.2 shows Single Angle Equal (SAE), and solid profiles are used for rectangular and triangular base towers, and dimensions are:

Table 3.2 Single Angle Equal (SAE) and solid profiles

Rectangular Base Towers		**Triangular base towers**	
Main Leg	**Bracing, Horizontal Members and Redundants**	**Main Leg**	**Bracing**
120×120×10	60×60×5	2 3/4"	1"
110×110×10	60×60×4	2 1/4"	3/4"
100×100×8	50×50×5	3 1/2"	7/8"
90×90×8	50×50×4		1 1/4"
80×80×8	45×45×4		1 1/8"
80×80×6	40×40×4		1 3/4"
70×70×5			2 1/4"
65×65×5			3 1/4"
60×60×5			
55×55×5			
50×50×4			
40×40×4			

Two AV2069 super gain series antenna and one short backfire antenna AV4802 are provided in this study, mechanical behavior of antennas shown in Table 3.3. HPL50-

1/2 Super Flexible coaxial cable for transferring waves between mounted antennas and equipment shelter provides and its mechanical properties cleared in Table 3.4. The antenna construction includes aluminium framework, power divider, and the radiating elements shielded with an ultraviolet (UV) resistant fiberglass radome. The radome does not behave on equaling, so the antenna maintains its Standing Wave Ratio (SWR) performance even if capped by thick ice and snow [68].

Table 3.3 Mechanical properties of antennas [68]

Type	**AV2069**	**AV4802**
Dimensions (H x W x D) mm	2820 x 300 x 130	(Ø x H) 520x200 mm
Weight (kg)	12	5
Wind area (m^2)	1,3	0,17
Operational wind speed (m/s)	40 (default)	40 (default)
Survival wind speed (m/s)	55 (default)	55 (default)
Polarization	Vertical	Vertical or horizontal
Materials	Aluminium	Aluminium

Table 3.4 Mechanical properties of coaxial cable [68]

Type	**HPL50-1/2 SF cable**
Cable weight (kg/m)	0.227
Diameter ø (mm)	13
Minimum bending radius (mm)	25
Multiple bending radius (mm)	50
Number of bends	20
Bending moment (N.m)	4.8
Tensile strength (N)	700
Installation temperature (°C)	- 25 / + 60
Operating temperature (°C)	- 40 / + 85

3.3 Geometry of Tower

The basic TOWER geometric dimensions are established by describing the locations of its joints. Each joint is given a unique identification label and is located in space with coordinates that are associated with a global 3-Dimensional coordinate system. The TOWER geometry is completed by connecting the joints with members (elements). The selection of the minimum number of joints and members for accuracy and the stability of the TOWER requires some familiarity with structural engineering [69].

In this study, the design of seven and four different types of self-supporting lattice towers with different bracing patterns are provided for rectangular, and triangular base towers with different heights as 60 m, 50 m, and 40 m height. The base width of 60 m and 50 m height rectangular base towers is 5.14 ×5.14 m. The base width of 40 m height rectangular base towers is 4.15 × 4.15 m. Furthermore, triangular base towers are equal triangular, and each side of the base triangle is 5.94 m and 4.26 m for 60 - 50 m height and 40 m height towers respectively. The top width of all type of rectangular and triangular base towers is 0.85m. Towers are supported by reinforced concrete foundation using fix type of supporting. Figure 3.5 illustrate geometry of rectangular and triangular base lattice tower.

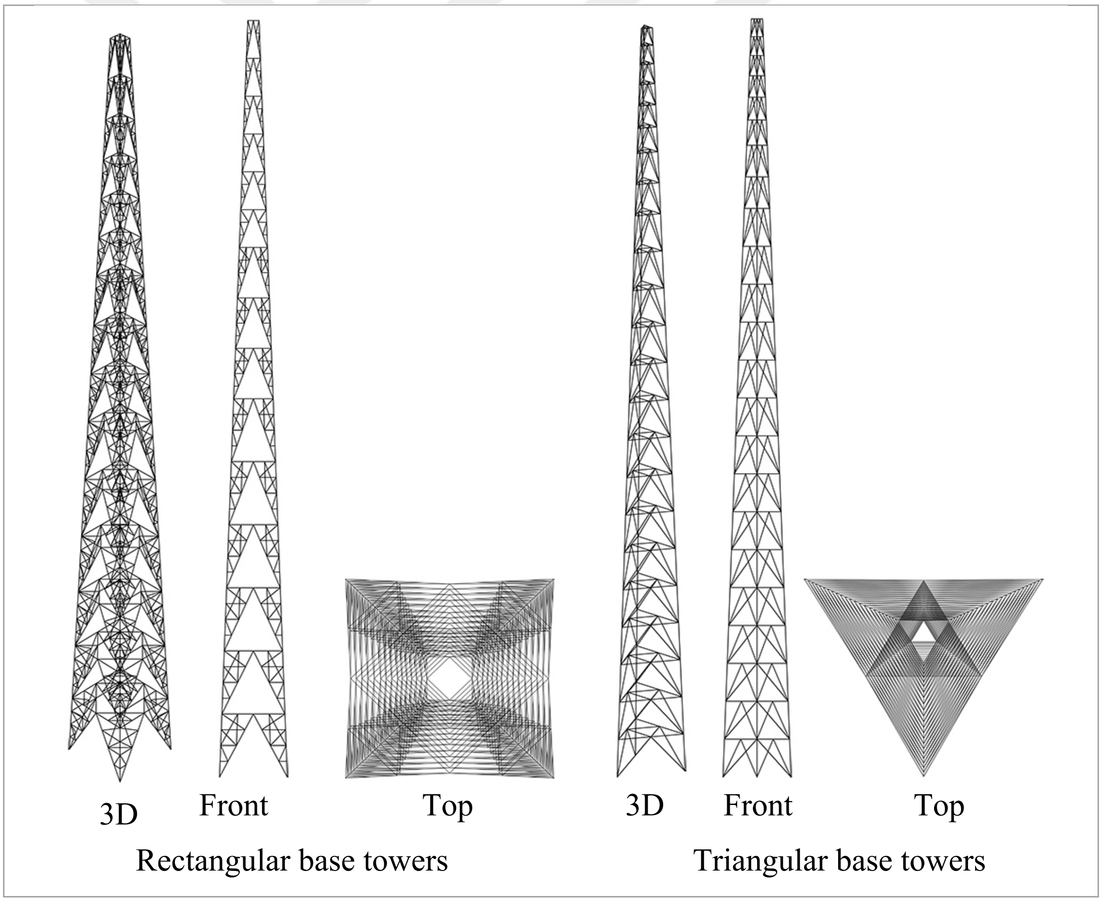


Figure 3.5 Geometry of rectangular and triangular base lattice tower.

Geometry menu in TOWER program provides the user with the interface that allows the tower model to be modeling. The height of the tower, number of sections, angle groups, angle members and linear appurtenances are assigned with the *geometry menu* as presented in Figure 3.6. In addition, linear appurtenances in *geometry menu* is an alternative of equipment library which utilize for defining select antennas, coaxial cables and other equipment attached to the communication tower. In this study, *linear appurtenances* are used instead of *equipment library* for defanging selected antennas and coaxial cables.

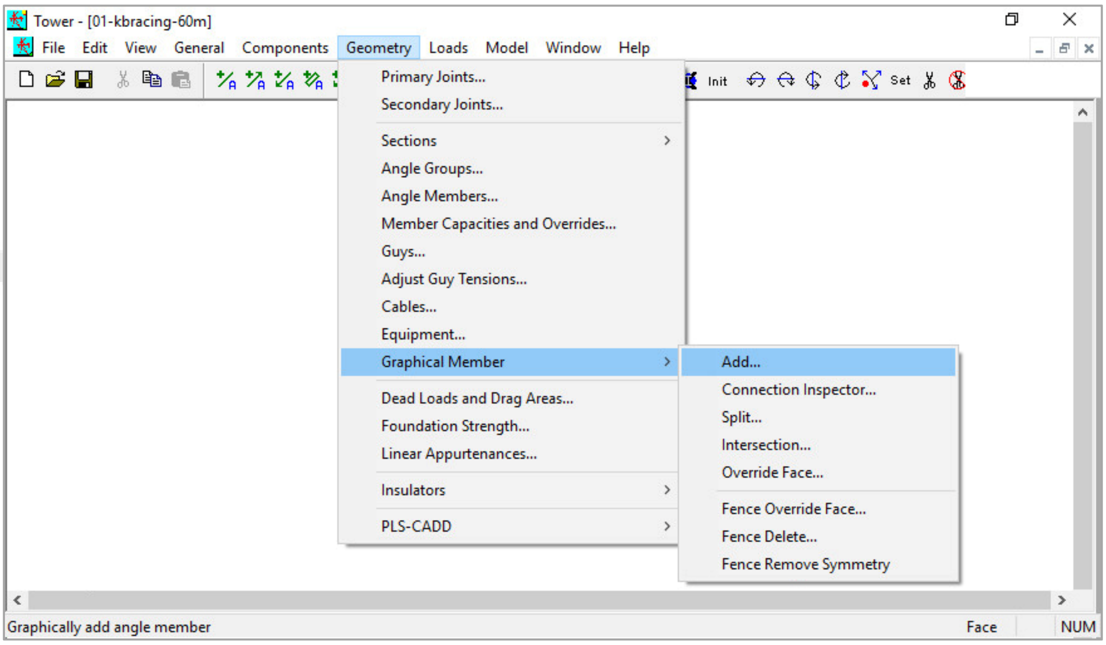


Figure 3.6 Geometry submenu components in TOWER program

Angle groups in *geometry menu* are providing element type as present in Figure 3.7. All design standards show a decrease in the design stress for an increasing overall slenderness ratio. Most of them also impose some limits, λ_{max} on the maximum value of λ depending on the member type. If the limit is exceeded, the user will get a warning. Available member types in TOWER are [69]:

a) Leg: for main leg members generally located at the corners of the TOWER and along the lower chords of the arms. If connected by bolts, such main members must have bolts on both faces of the angle to avoid any eccentricity in its connections. If this is not the case, user will get a warning

b) Redundant: for redundant bracing members, i.e. members which do not carry loads but are used to brace load-carrying members to increase their compression capacity

c) Fictitious: for fictitious members with an artificially small cross section area that were used in the past to solve the problem of unstable mechanisms and planar joints.
d) Crossing diagonals: for pairs of diagonal members where the compression capacity of the most compressed member depends on the force of the other member.
e) Other: for any load-carrying member which is not a main member and which is not a redundant bracing member

Angle Groups

	Group Label	Group Description	Angle Type	Angle Size	Material Type	Element Type	Group Type
1	Dum	Dum	DUM	1*1*1	S235 JR	Truss	Fictitious
2	L1	Leg	SAE	120*120*10	S355 JR	Beam	Leg
3	L2	Leg	SAE	110*110*10	S355 JR	Beam	Leg
4	L3	Leg	SAE	100*100*10	S355 JR	Beam	Leg
5	L4	Leg	SAE	100*100*10	S355 JR	Beam	Leg
6	L5	Leg	SAE	100*100*8	S355 JR	Beam	Leg
7	L6	Leg	SAE	90*90*8	S355 JR	Beam	Leg

Figure 3.7 Angle groups components in TOWER program

A tower model is built by placing in angle or round members between joints and connecting other components to existing joints. *Primary joints* must be located by their coordinates, whereas *secondary joints* are located by straight line interpolation or extrapolation between key joints. The first step in assembling user model is to describe the model joints. Primary joints then secondary joints are described below:

3.3.1 Primary joints

Primary joints are those which much be located by their global coordinates. Data for *primary joints* are entered in the *primary Joints* spreadsheet as shown in Figure 3.8 which is reached with *Geometry/Primary* Joints. In modeling of the rectangular base tower, X, Y, and Z coordinates have a value and its different from triangular base towers by missing Y coordinate. In triangle base tower Y coordinate value must be zero to be able modeling the primary triangles key by selecting Tri-Symmetry code [69].

Joints Geometry

	Joint Label	Symmetry Code	X Coord. (m)	Y Coord. (m)	Z Coord. (m)
1	A				
2	B				
3					
4					
5					
6					

Figure 3.8 Angle groups components in TOWER program

3.3.2 Secondary joints

TOWER automatically generate secondary joints by interpolation between the positions of two primary joints. The geometric location of a secondary joint is described by its location along the straight line joining of the primary joints. Data for secondary joints are entered in the secondary joints Table which is reached with *geometry/ secondary* joints as shown in Figure 3.9[69].

Secondary Joints

	Joint Label	Symmetry Code	Origin Joint	End Joint	Fraction	Elevation (m)	X Disp. Rest.	Y Disp. Rest.
1	C	XY-Symmetry	AP	BP	0	1.251	Free	Free
2	D	XY-Symmetry	AP	BP	0	2.502	Free	Free
3	E	XY-Symmetry	AP	BP	0	3.7519999	Free	Free
4	F	XY-Symmetry	AP	BP	0	5.003	Free	Free
5	G	XY-Symmetry	AP	BP	0	6.2540001	Free	Free
6	H	XY-Symmetry	AP	BP	0	7.5049999	Free	Free

Figure 3.9 Angle groups components in TOWER program

3.6 Tower Sections

Tower models can be divided into sections as represent in Figure 3.10. This allows you to globally assign loading or other parameters to all the members in one section.

This also allows you to select a section of a basic module which can easily be replicated to build the model of a larger structure made of modular sub-assemblies. For each section, you can assign separate drag or area adjustment factors to take care of various wind models or the effects of redundant members not included in the model. Membership in sections or faces can be visualized by selecting *Section* or *Face* as the color in the 3-D Controls dialog box - pressing the F9 key repeatedly will also let the user cycle between all the color options of the 3-D Controls dialog box as shown in Figure 3.10. Sections limited in length is no longer than 18.288 m or less, and no longer than a span length for guyed masts [69].

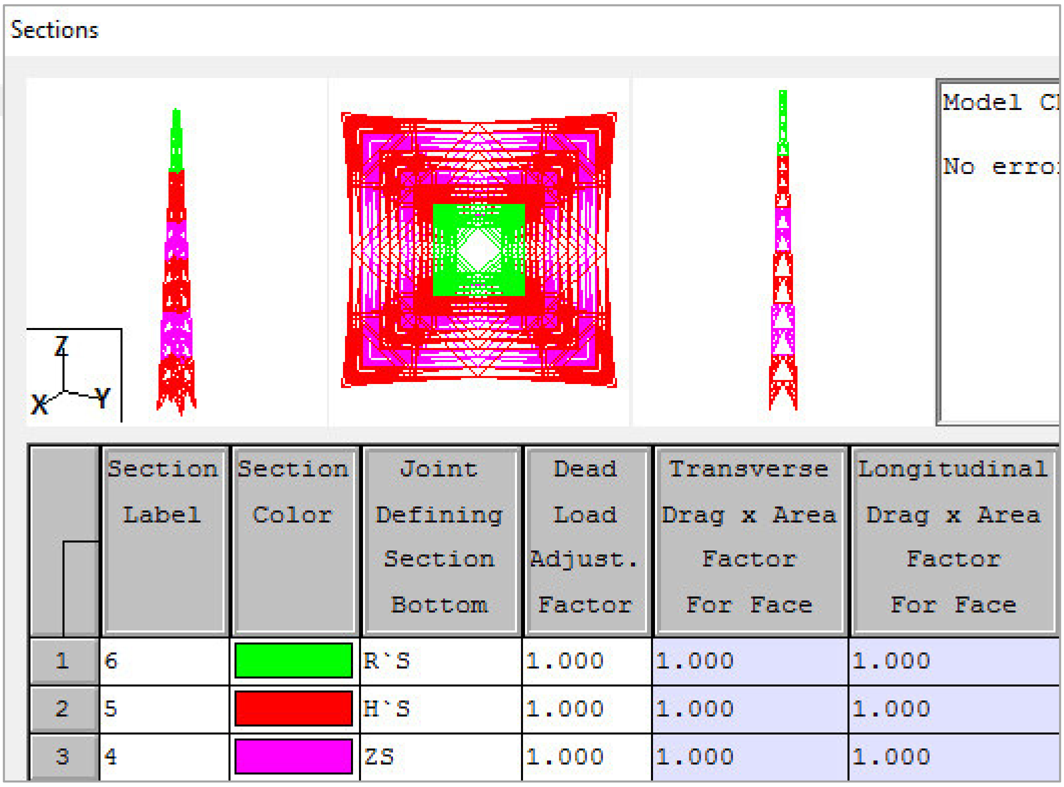
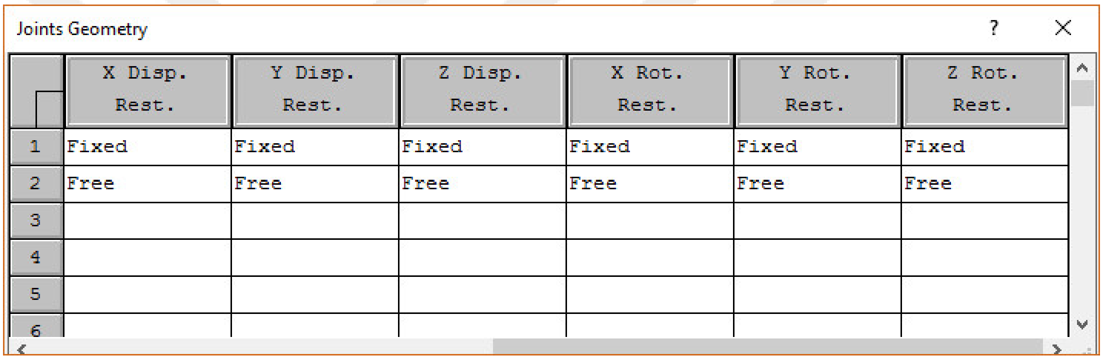


Figure 3.10 Sections properties in TOWER program

3.4 Boundary Elements and Joint Degree of Freedom

Boundary elements are used to specify displacement boundary conditions (zero and non-zero values), to provide artificial restraints at planar nodes, and to compute the values of the support reactions. A boundary element is a spring with axial stiffness to resist translation and torsional stuffiness to resist rotation. In a three-dimensional communication tower problem, a boundary component is connected to any footing joint in each of the global directions X, Y, and Z. If the specified displacements at the footing joints of a tower are input as zero, the footings are prevented from translating or rotating in any of the global directions. If a non-zero displacement is specified (negative translation represents a foundation settlement, and positive adaptation describes a foundation uplift due to frost heave), the quantity of translation or rotation is indicated as a property of the boundary element [15].

The base of both rectangular and triangular base towers are fixed, and the top is free. Boundary condition in TOWER program is provided with *Geometry/ Primary Joints* as shown in Figure 3.11 is used for defining joint displacement or boundary condition. By defining X, Y and Z displacement as fixed in primary joints of towers, the base of the towers will be fixed, and the tops will be free by defining X, Y and Z displacement of top primary joints as free.


Joints Geometry

	X Disp. Rest.	Y Disp. Rest.	Z Disp. Rest.	X Rot. Rest.	Y Rot. Rest.	Z Rot. Rest.
1	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
2	Free	Free	Free	Free	Free	Free
3						
4						
5						
6						

Figure 3.11 Boundary condition in TOWER program

3.5 Loads on Communication Tower

Calculation of tower loading which is most important part of tower design is the first step towards tower design. Any mistake or error in the load assessment will make the tower design erroneous. Various types of loads are to be calculated accurately depending on the design parameters.

The gravity loads are almost fixed since these are dependent on the structural design. In the load calculation, the wind plays a vital role. The correct assessment of wind will lead to proper load assessment and reliable design of tower structure. Maximum wind pressure is the chief criterion for the design of lattice towers. The simultaneous concurrence of the earth quake and maximum wind pressures are unlikely to take place. However, in particular, regions where earthquakes are experienced frequently, as in the North regions of Turkey and other parts of the country, the seismic load is also should be considered in the design of towers in accordance with ANSI/TIA 222-G code. Communication towers are subjected to various loads which are consists of gravity loads, lateral loads and erection loads [13].

3.5.1 Gravity loads

Gravity load of communication towers consists of weights of members, the weight of platforms, railings, ladders, lifts, etc., the weight of antenna, instruments, appliances, etc., the weight of gussets and secondary bracings and live load. For foundation design, the weight of substructure and soil. Dead loads vary from one support to another due to the variation of weight and height of the tower.

Automatically calculated dead loads are based on the following assumptions. The program knows the weight of all the members used in user model in each tower section. However, because the model generally does not include the weight of redundants, gusset plates, etc. the weight of all the members in user model is likely smaller than the true weight of the tower. Therefore, designer can estimate the true weight of the tower by multiplying the weight of all the members in user model by a dead load adjustment factor. Actually, the adjustment is made by section.

Dead load adjustment factors are numbers based on experience. They are usually in the range of 1.00 to 1.4, where 1.00 represents a situation where all members are modeled (including redundants), and 1.4 indicates that several heavy redundants were not included in the model. Dead load adjustment factor in TOWER program which is reached with *geometry/section/defines* as shown in Figure 3.10 is used to estimate the true dead weight of all members [69].

3.5.2 Lateral loads

(a) Wind Load: Communication tower standards have unique and very detailed procedures for specifying wind load on structures with square and triangular cross sections, their appurtenances and antennae. The wind velocity is generally increased with height. However, it can be assumed constant over sections limited in length 18.288 m or less and no longer than a span length for guyed masts. The wind direction is also a factor. Some of the requirements may be subject to interpretation. Therefore, it is important that the TOWER user understands how some of the load calculations are automated for a given standard. The only communication towers standards currently supported by TOWER are the ANSI/EIA standards and the Canadian standard CSA S37-01 [69].

When the conditions for the automatic calculation of loads on the tower are met the following rules are followed as shown in Figure 3.12:

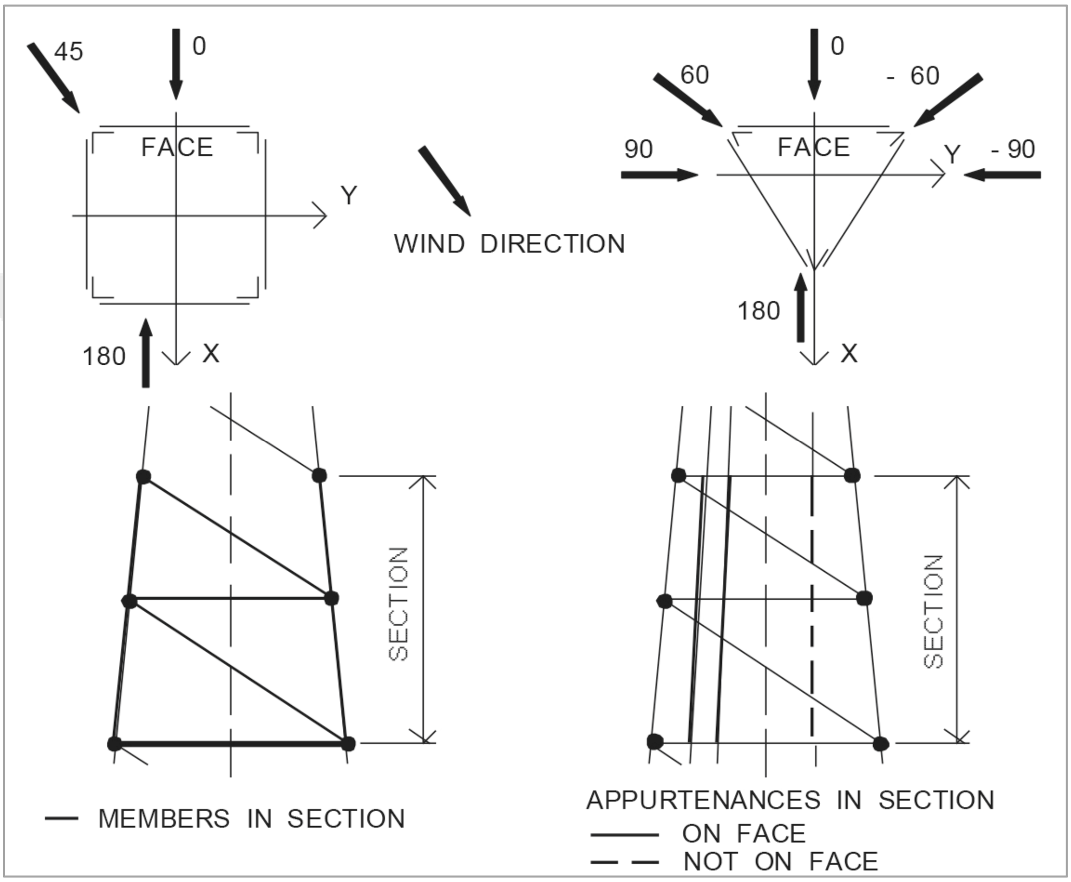


Figure 3.12 Wind direction of rectangular and triangular base towers

The wind load design must accommodate the sum of the horizontal design wind forces applied to the structure in the wind direction and the wind forces design on guys and appurtenances. All appurtenances, containing antennas, mounts, and lines, must be considered to be left whole and mounted to the structure. Stiffness design must be based on the directions of the wind occurring in the maximum replies. The design wind load, F_W, (described in detail in ANSI/TIA-222-G) shall be determined in accordance with the following [70]:

$$F_W = F_{ST} + F_A + F_G \tag{3.1}$$

Where F_{ST} is horizontal design wind force on the structure in the direction of the wind, F_A is design wind force on appurtenances and F_G is design wind force on guys.

Design wind force on tower: The design wind force, F_{ST} applied to each section of a structure shall be determined in accordance with the following [70]:

$$F_{ST} = q_z \, G_h \, (EPA)_S \tag{3.2}$$

Where q_z is velocity pressure, G_h is gust effect factor and $(EPA)_S$ is effective projected area of the structure.

Design of wind force on appurtenances: The design wind force, F_A shall be utilized at the centroid of the effective projected area of the appurtenance in the wind direction. For a linear appurtenance, the estimated length to have uniform velocity pressure and ice thickness shall not pass the length of section [70]. F_A shall be determined by the equation:

$$F_A = q_z \, G_h \, (EPA)_A \tag{3.3}$$

Where F_A is design wind force on appurtenances, q_z is velocity pressure at the centerline height of the appurtenance, G_h is gust effect factor and

$(EPA)_A$ is effective projected area of the appurtenance including ice for loading combinations that include ice.

Equation of F_G is not mentioned in this study, because studied towers is self-supporting towers and F_G is design wind force on guys of guy towers.

(b) Earthquake loads: Antennas supporting structures require specific attention or their answer features in areas of large seismicity. The requirements of this standard present design standards to ensure sufficient strength and resistance to obtain the impacts of seismic ground motions for self-supporting and guyed structures which antenna is supporting. ANSI/TIA-222-G lists four seismic analysis methods of increasing accuracy and complexity with limitations for their applicability to various tower types, geometry, and height. The methods are equivalent static lateral force, equivalent modal analysis, classical modal superposition analysis and time history analysis.

Earthquake impacts can be neglected on Class I structures or for each structure established in a zone where the earthquake spectral response acceleration at short periods (S_{DS}) less than or equal to 1.00. Further, for structures without irregularities, earthquake impacts can be neglected when the total seismic shear is fewer than 50% of the total horizontal wind load without ice. The similar lateral force method shall be utilized for all structures to determine the total seismic shear to compared to wind loading for this association shall be based on the 50 year recurrence period basic wind speed for the place. For self-supporting towers, equivalent modal analysis procedure is provided for estimating earthquake load of towers [70].

The lateral seismic force F_{sz} induced at each level of the structure z described in detail in ANSI/TIA-222-G and shall be determined by the following equation [70]:

$$F_{sz} = \frac{S_{az} W_z I}{R} \tag{3.4}$$

Where z is number designating the level under consideration, W_z = portion of total gravity load W assigned to level under consideration, I is importance factor, R is response modification coefficient equal to 3.0 for latticed self-supporting structures, a, b & c is acceleration coefficients determined from Figure 3.13 and S_{az} is acceleration coefficient at height z. S_{az} can be computed as:

$$S_{az} = \frac{a(S_A)^2 + b(SD_S)^2}{[(S_A)^2 + c(SD_S)^2]^{1/2}} \tag{3.5}$$

Where $S_A = S_{D1} * f_1$ when $f_1 \leq \mathrm{S_{DS}}/\mathrm{S_{D1}}$, otherwise $S_A = S_{DS}$. S_{DS} is design spectral response acceleration at short periods, S_{D1} is design spectral response acceleration at 1 second and $f_1$is fundamental frequency of structure.

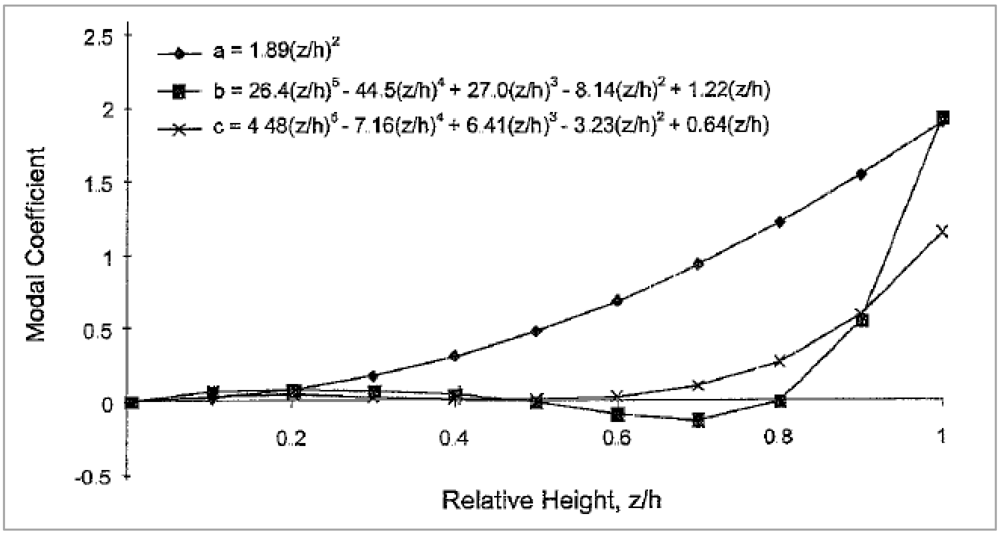


Figure 3.13 Equivalent model coefficients, a, b & c

The structure fundamental frequency of self-supporting towers a formula has been given in ANSI/TIA-222-G

$$f_1 = \frac{K_S(W_a)}{h^2}\sqrt{\frac{W_1}{W_1 + W_2}} \quad Hertz \tag{3.6}$$

Where K_S is 1500 for h and W_a in meters, W is total weight of the structure including appurtenances, W_2 is weight of structure and appurtenances within top 5% of the structure height, $\mathrm{W_a}$ is average face width of structure, $\mathrm{W_0}$ is face width at base of structure and h is height of structure. W_1 can be computed as:

$$\mathrm{W_1} = \mathrm{W}\left[\left(\frac{\mathrm{W_a}}{\mathrm{W_0}}\right)^2 + 0.15\right] \tag{3.7}$$

The design earthquake spectral response acceleration at short periods, S_{DS}. and at 1 second, S_{D1}, shall be determined from the following equations:

$$S_{DS} = 2/3F_aS_s \tag{3.8}$$

$$S_{D1} = 2/3F_yS_1 \tag{3.9}$$

Where F_a is acceleration-based site coefficient, S_s maximum considered earthquake spectral response acceleration at short periods, F_y velocity-based site coefficient and S_1 is maximum considered earthquake spectral response acceleration at one second.

In this study, towers are designed based on the site class A and is hard rock with 3 m or less of soil overburden. The maximum considered earthquake spectral response acceleration at short periods. for site class A at $S_s \geq 1.25$ is 0.8. The value of S_{DS} will be ($\frac{2}{3}1.25 \times 0.8 = 0.67$). The maximum considered earthquake spectral response acceleration at one second. for site class A at $S_1 \geq 0.5$ is 0.8. The value of S_{D1} will be ($\frac{2}{3}0.5 \times 0.8 = 0.268$).

3.5.3 Service Loads

The loading combination used to calculate serviceability limit state deformations. Service loads shall be defined by the loading combination for a 27 m/s basic wind speed.

3.5.4 Erection loads

Erection services build of communication masts, set up microwave dishes and mounted similar communications apparatus. They further present design engineering, structural analysis, and tower testing services. Any communication mast is erecting services present purchase, agreement, ultimate load, and supplementary load inquiries. Purchase inspections confirm the propriety of subsisting structures, designs, and accessories. Approval investigations guarantee that a new mast is plumb and that guyed wires are correctly tensioned. Communication tower erecting services should also investigate support arrangements, bolt twists, and tower shaft elements as a component of an agreement investigation. High load investigations showed after a tower subjected to ultimate loading due to huge winds or ultimate weather. Supplementary load investigations are utilized to define whether a current tower can handle a supplementary load. Following, communication tower erecting services offer periodical investigations to comply with state and regional arrangements [71].

3.6 Load Cases and Combination of Loads

Differing external loads acting simultaneously on the supports of towers are combined to load cases in an adequate manner. These combinations of actions need to comply with the requirements concerning reliability, security, and safety. The load cases should take care of all loading conditions to be expected during construction and during the whole life period of towers such that damage will be unlikely. In many standards for power, telecommunication towers are distinguished between normal and exceptional loads, whereby differing stability requirements or differing permissible stresses apply. The classification of those specific loads may occur to be chosen arbitrarily in a specific case. The load cases according to TIA/EIA-222-G for telecommunication are used in the present study [18].

The combination of loads of communication towers according to ANSI/TIA standard which is reached with *Loads/EIA loads (file)*. This menu will appear when user design and analysis towers using ANSI/TIA standards. Load cases in TOWER program consist of EIA 222-G information and combination of the load's spreadsheet as shown in Figure 3.13.

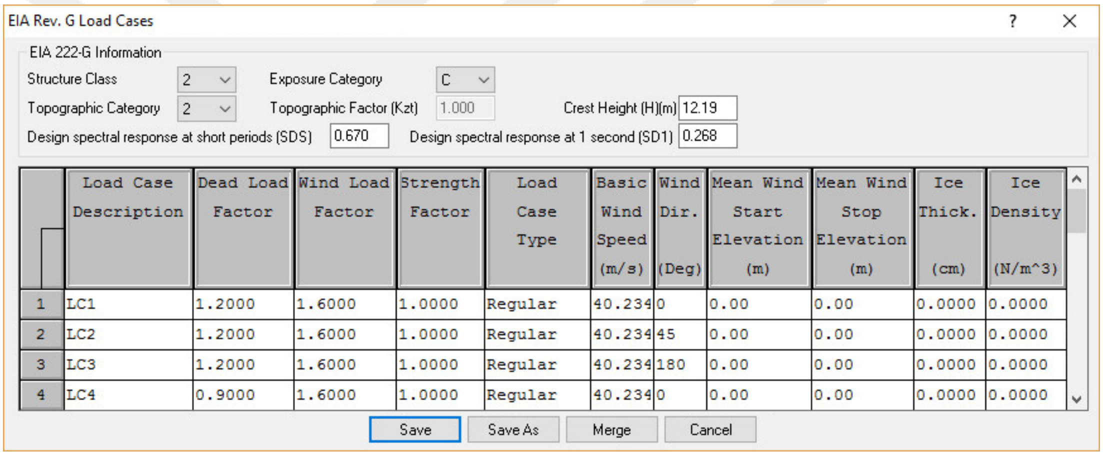


Figure 3.14 Load cases and combination of loads menu

3.6.1 EIA 222-G information

(a) Structure Class: The standard establishes three classifications of structures based on reliability criteria. The default Structure Classification is Class II. Class II: Structures utilized for services that can be produced by another means like wireless communications; TV and radio broadcasting; cellular, and microwave communications [69].

(b) Exposure Category: The default Exposure Category is Exposure C. Open terrain with scattered obstructions having heights generally less than 9.1 m. This section covers flat, open country, fields, and shorelines in storm-prone regions [70].

(c) Topographic Category: The default Topographic Category 1. For Topographic Categories 2, 3 and 4, the height of the topographic characteristic is to be involved in the acquirement specifications. Specific site notice obliged for the utility of Topographic Category 5 [70].

(d) Topographic Factor: These apply to significant geographical features e.g. escarpment, hills, and ridge. If there are no significant geographical features nearby, the value of topographic factor will be 1.0. The topographic factor is described in detail in ANSI/TIA 222-G standards [70].

(e) Crest Height: In the case of Topography Category 2, 3 or higher, the height of the crest of the escarpment should be specifying which provided tower structure is located at nearby. The wind speed-up effect has an obviously different impact on the velocity pressure which can be described and calculated by the Topography Factor K_{zt} in TIA-222-G. There are several cloud services for topography/terrain assessments. One of them is Google Earth where the user needs to create some path for elevation profiles. Then the user can get a better overview about the height of the crest based on design location [72].

(f) SDS and SD1: The design earthquake spectral response acceleration at short periods, SDS. and at 1 second, SD1 are described in details in ANSI/TIA 222-G standards [70].

According to ANSI/TIA standards, structures and foundations intend to be designed in a way that their strength of design approaches or passes the load effects of the factored loads in every of the following boundary element combinations [70]:

1. $1.2D + 1.0D_g + 1.6W_o$
2. $0.9D + 1.0D_g + 1.6W_o$
3. $1.2D + 1.0\,D_g + 1.0\,D_i + 1.0\,W_i + 1.0\,T_i$
4. $1.2D + 1.0Dg + 1.0E$
5. $0.9D + 1.0Dg + 1.0E$
6. $1.0D + 1.0D_g + 1.0W_o$ (Service load)

Where D is dead load of structure and appurtenances, excluding guy assemblies, D_g is dead load of guy assemblies, D_i is weight of ice due to factored ice thickness, E is earthquake load, T_i is load effects due to temperature, D_o is wind load without ice and W_i is concurrent wind load with factored ice thickness.

As described in ANSI/TIA 222-G, temperature effects need not be considered for self-supporting structures, ice and sesmic loading require not be an estimate for structures Class I, no load factor must be used to the initial tension of guys, a load combinations 2 and 5 apply to self-supporting structures only [70].

In this project, eleven load case type are considered for rectangular base towers as a combination of loads is described in Table 3.5, and twenty combinations of loads are considered for triangular base towers are described in Table 3.6.

Table 3.5 Combination of loads of rectangular base towers

Load Case Description	Dead Load Factor	Wind Load Factor	Load Case Type	Basic Wind Speed (m/s)	Wind Direction (Deg)
LC1	1.2	1.6	Regular	40.234	0
LC2	1.2	1.6	Regular	40.234	45
LC3	1.2	1.6	Regular	40.234	180
LC4	0.9	1.6	Regular	40.234	0
LC5	0.9	1.6	Regular	40.234	45
LC6	0.9	1.6	Regular	40.234	180
LC7	1.2	1.0	Earthquake	0	0
LC8	0.9	1.0	Earthquake	0	0
LC9	1.0	1.0	Service	26.822	0
LC10	1.0	1.0	Service	26.822	45
LC11	1.0	1.0	Service	26.822	180

Table 3.6 Combination of loads of triangular base towers

Load Case Description	Dead Load Factor	Wind Load Factor	Load Case Type	Basic Wind Speed (m/s)	Wind Direction (Deg)
LC1	1.2	1.6	Regular	40.234	0
LC2	1.2	1.6	Regular	40.234	90
LC3	1.2	1.6	Regular	40.234	180
LC4	0.9	1.6	Regular	40.234	0
LC5	0.9	1.6	Regular	40.234	90
LC6	0.9	1.6	Regular	40.234	180
LC7	1.2	1.0	Earthquake	0	0
LC8	0.9	1.0	Earthquake	0	0
LC9	1.0	1.0	Service	26.822	0
LC10	1.0	1.0	Service	26.822	90
LC11	1.0	1.0	Service	26.822	180
LC12	1.2	1.6	Regular	40.234	-90
LC13	0.9	1.6	Regular	40.234	-90
LC14	1.0	1.0	Service	26.822	-90
LC15	1.2	1.6	Regular	40.234	-60
LC16	0.9	1.6	Regular	40.234	-60
LC17	1.0	1.0	Service	26.822	-60
LC18	1.2	1.6	Regular	40.234	60
LC19	0.9	1.6	Regular	40.234	60
LC20	1.0	1.0	Service	26.822	60

3.7 Analysis

Linear or nonlinear analysis can be carried out for towers. Using linear analysis, the minor effects of the structure displacements, called P-Delta effects, are ignored. With the other option, all forces and moments are in the balance in the structure deformed state, including P-Delta effects. Despite the fact that the nonlinear option considers all geometric nonlinearities, linear material properties are still used and preferable in all programs, which is consistent with all current code-based design procedures for transmission structures [69].

As described in ANS/TIA-222-G standards. P-Δ effects need not be considered for self-supporting latticed towers with height less than 137 m provided that the height to face width ratios, $\frac{h_i}{f_{wi}}$ are less than 10 [70]. Figure 3.14 show height to width ratios. $\frac{h_i}{f_{wi}}$ can be computed as:

$$\text{Max}\frac{\text{height}}{\text{face width}}\text{ ratio} = \text{max of}\frac{\mathrm{h_1}}{\mathrm{f_{w1}}}\text{ or }\frac{\mathrm{h_2}}{\mathrm{f_{w2}}} \tag{3.10}$$

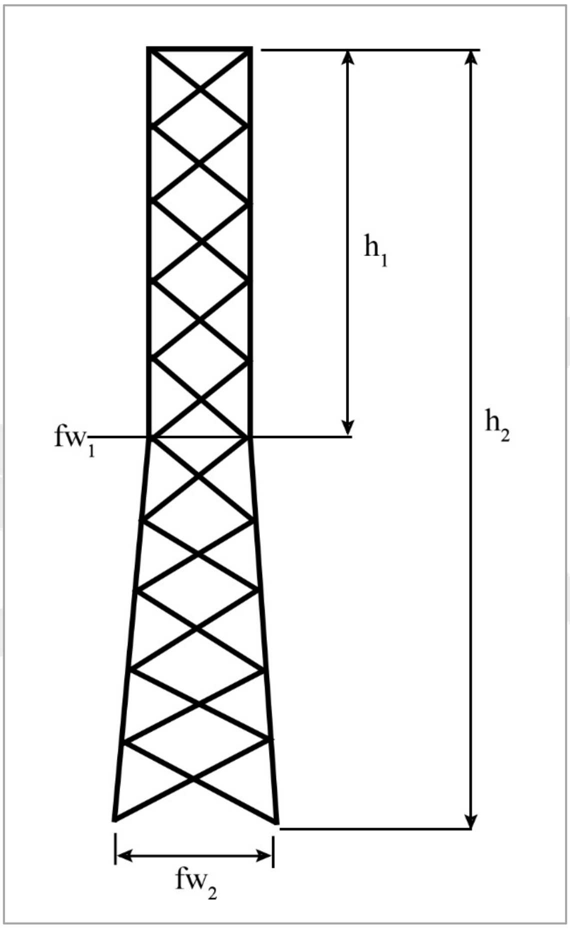


Figure 3.15 Height to width ratios

According to the geometry of towers, base width and top width of rectangular base towers, provided that the height to face width ratios for 60, 50 and 40 m is 11.68, 9.73 and 9.63 and 10.1, 8.42 and 9.38 for triangular base towers. 50 and 40 m rectangular and triangular base towers is less than 10 as described in ANS/TIA 222-G standards P-Δ effects need not be considered for 50 and 40 meter height towers. For studding purposes, all towers analyzed in both analysis type linear and non-linear analysis.

Once the tower is modeled, it can be checked from model menu before running process, if the model contains any error the program will detect the type and location of error automatically, if not, then from model menu by clicking *Run* the analysis of tower will proceed with respect to the defined code. TOWER program will run an analysis on the tower and create an output report. In addition, before running model, the user is able to set analysis report from general data menu by selecting *Output Option* as shown in Figure 3.15. The output report consists of the inputs of the tower, member stresses due to loading, maximum usage of elements, the weight of the tower and all checked options that user ticked before analysis process from *Output Option* menu.

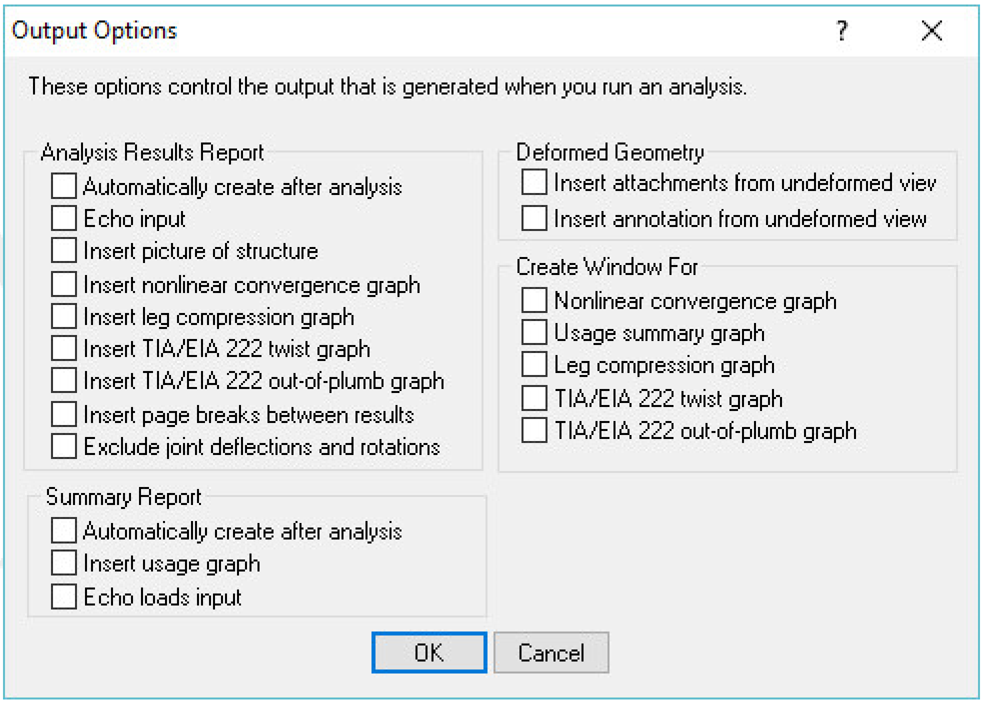


Figure 3.16 Output option menu

3.7.1 Linear and nonlinear analysis

The linear analysis defines the deformations of structures when linear combinations of loads are applied. The aim of the linear analysis is to recognize the transformation and inverse transformation between these two set of quantities. This transformation is called the structure stiffness matrix, while nonlinear relationships between loads and displacements might take place as a result of many reasons, including related material or geometric non-linearity of the structure. Regular solution technique regards locally linear relationships between loads and deformations. Consequently, nonlinear is more difficult than linear analysis [15].

For analysis of towers, small displacements and a linear elastic behavior of the material are assumed. The calculation yields the structure deformation state, and the deformations of nodes are achieved by solving this system of linear equations. The displacements attended from the stress-strain relation can then be used to determine the member forces. To the extent that the programming is concerned, the displacement method of analysis is identical to the finite element method with regard to the system. The results are precise, the theory of linear static analysis provided. All types of truss structures can be analyzed via the method which has been systematically developed to such this level. Several computer programs designed for framework systems are available for practical applications. The computation effort is significantly high to the level that it might be essential for an economic execution of the calculation to adopt programs specially designed for lattice towers with the purpose of limiting the input effort to a satisfactory degree. The geometry should be identified through a few input data only, forming the basis for creating nodes and member relations by the computer program. The node loads related to the regarded load cases are obtained by the computer program itself. The rating of the member cross sections and connections relies on the standards to be applied and performed additionally by the computer program [18].

3.7.2 Numerical analysis

The steel telecommunication tower design is not a straightforward process, but an interactive compromise between many factors, which must ultimately satisfy basic strength requirements. Generally, in structural analysis, the actual complex structure and loading are modelled numerically, using several simplifying assumptions. On the other hand, the most widely used tower geometries, when the truss solution is adopted, possess structural mechanisms that compromise the assumed structural behavior. The linear elastic analysis of communication tower, nonlinear effects at member and system level (geometric) are taken into consideration, and the tower is modeled and analyzed using column-beam and truss elements. Thus moments produced by the continuity of members are generally not considered since each leg member is assumed pinned between two joints [74].

In the present study, structural analysis based on a less conservative solution, for the steel tower design considering all the actual structural forces and moments. A modeling method combining three-dimensional beam and truss finite elements is proposed. In tower models, the main members such as legs use beam elements while the bracing system utilizes truss elements.

The linear and nonlinear analyses of the tower are carried out for obtaining the performance of bracing systems. The TOWER program is used in this study to evaluate the structural performance of the bracing system. The towers have been modelled in 3D using TOWER program. This program capable of carrying out linear and nonlinear analysis and also provide a chance for checking design such structures under user specified loads and can also calculate maximum allowable wind and weight spans.

CHAPTER 4

ANALYSIS AND RESULTS

4.1 Introduction

The analysis of communication steel lattice towers is usually achieved with obvious linear numerical methods. Nevertheless, the exact behavior of bolted lattice masts is complicated and may be modified by various circumstances such as rotational stiffness of connections, bolt slippage, eccentricities at the connection, initial deformations of the members, etc. For this reason, most utility company perform full-scale tests for the requirement of comparison various type of bracing systems or new design of steel lattice communication towers. These tests are costly and add postponements in the preparation of the construction of new communication towers. Advanced numerical programs were updated over the years, and they may efficiently accommodate numerous perspicacity into the force distribution system, and loading condition of specific members found in the steel structure and therefore, help to optimize the use of full-scale laboratory tests. The purpose of this thesis is to determine the best type of bracing system for different heights of both rectangular and triangular base towers to model the performance of steel lattice towers perfectly [75].

While a varies kinds of towers with several bracing systems of the structure body are designed, it is essential for the user to be qualified to determine strength usages and maximum forces for each group of members. TOWER program allows the user to link multiples analysis runs and summarize all results in easily understandable reports and Tables. The percent usage of all member and each foundation for each specified load case are also possible to be represented. One of the biggest qualified methods to resemble at the analysis results is in the several results Tables which user can reach with *model/ results*. Using both linear and nonlinear analysis type based on ANSI/TIA-

222-G standard code. TOWER program used for comparing different types of rectangular and triangular base towers with various heights has been designed and analyzed then the obtained results have been and analyzed then the obtained results have been compared with each other

4.2 Linear and Nonlinear Analysis

Many types of loading on communication towers such as wind, ice, earthquake loads are in actuality dynamic loads, but their impact is frequently assessed in static analyses using quasi-static methods. It should be noted that the method used for the evaluation of internal forces should be consistent with the method used for estimating member resistance. The standard approach to design is to use resistance equations that take only the axial load into account. This leads to being consistent with a truss model where all internal forces are in tension/compression. Hence, designers should estimate accurately the magnitude of bending moments that are predicted by a numerical model made of beam elements. Advanced numerical studies will frequently aim at estimating resistance directly with the model, therefore be avoiding this kind of consideration. In design, linear analyses are commonly used. Though, in the context of advanced numerical studies, a nonlinear analysis is most likely to be provided. One aim to choose a nonlinear analysis is to take into account geometric nonlinearity, i.e. huge displacements of the structure. A nonlinear analysis will perform iterations to ensure that the stiffness matrix used for the resolution of the system is representative of the structure in its deformed state and that the forces are in equilibrium. Depending on the tower stiffness and the applied load, not considering large displacements could lead to neoconservative evaluation of internal forces. Regarding high displacements is in particular required to simulate the buckling of a tower member numerically because second order effect causes the instability. A nonlinear analysis also provides integrating material non-linearity, i.e. the change in the stress-strain curve followed while yielding of steel elements. When material non-linearity is considered, an incremental static analysis is needed to preserve track of the variations in the material properties [75].

4.3 Specific Load Cases

The various load cases, considered for the design and check of the structure according to the user standards, will be obtaining the required checking for the tower structures. Required Loading cases and the combination of loads on both rectangular and triangular base towers are described in Chapter 3 in details.

4.4 Results and Discussion of Rectangular Base Towers

Comparing the outcomes will be described according to the type of bracings, the height of towers and analysis type. In the present study, linear and nonlinear simulation is used, compared the output data between the linear and nonlinear simulation, it was detected that the tower members are less prone to the effects during the comparison between them.

4.4.1 Weight of towers

The weight of towers designed based on linear and nonlinear analyses are the same. According to Figure 4.1, the minimum weight of tower for 60 m, 50 m, and 40 m heights are obtained for the KD and YD bracing system is 60868.1 N, 46352.5 N and 30876 N, respectively, On the other hand, the maximum weight of towers for 60 m, 50 m, and 40 m heights is obtained for the XB, and Y bracing system is 145852.6 N, 131852.3 N and 43606.4 N as present in Figure 4.1.

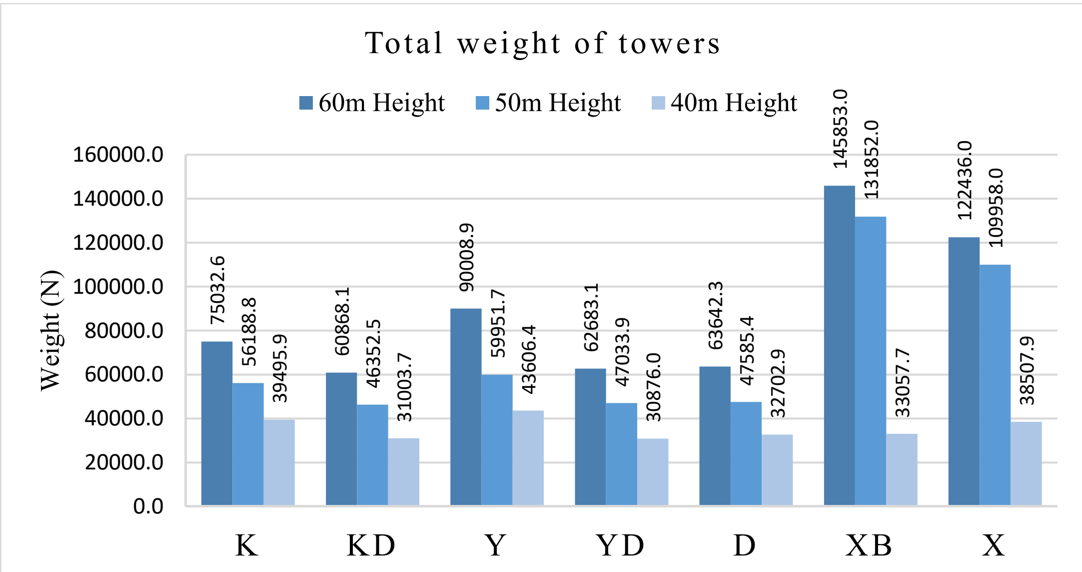


Figure 4.1 Total weight

4.4.2 Structure fundamental frequency

Results of structure fundamental frequency of all towers are nears to each other. The best achievement fundamental frequency for 60m, 50 m, and 40 m heights returns to KD, Y, and D bracing system, respectively, is 1.1339, 1.6064 and 2.0626 Hz. The worst achievement is for 60m, 50 m, and 40 m heights are concerned for K, D and KD are 1.2262, 1.7847 and 2.2804 Hz, respectively. Figure 4.2 show structure fundamental frequency of towers.

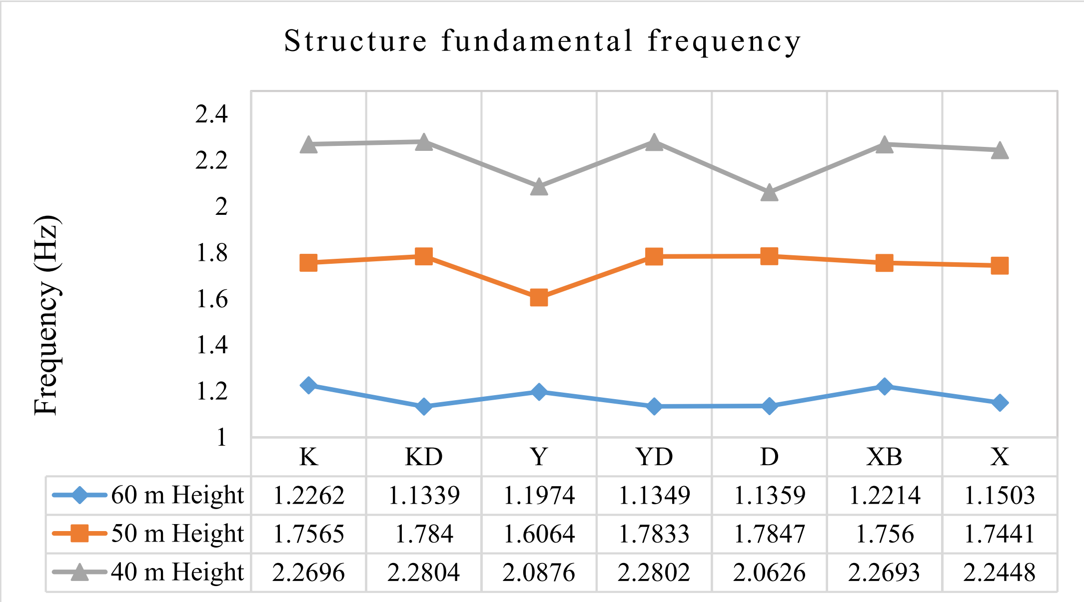


Figure 4.2 Structure fundamental frequency

4.4.3 Maximum element usage

Maximum Element Usage (MEU) of all members considering the individual members are mainly loaded by axial compression and tension forces, considering all load cases. The member forces are calculated based on three-dimensional linear and nonlinear analysis. The maximum elements usage is listed in Table 4.1 In both linear and nonlinear analyses, the critical loading is obtained for X and XB structure type for 60, 50 and 40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to the tower with an angle of 45°) is critical. For XB and YD the failure containment load case 1.2D (dead load) + 1.6Wo (wind load applied to the tower with an angle of 45°) is critical. The critical members which are given in Table 4.1 are the same in linear and nonlinear analyses. The all critical members are in compression. The difference percentage of maximum element usage between linear and nonlinear analysis type are given also in Table 4.1. Figure 4.3 and 4.4 shows linear and nonlinear results of maximum elements usage values of rectangular base towers in percentage.

Table 4.1 Maximum element usage of linear and nonlinear analysis results

Type	H. (m)	Linear Analysis			Nonlinear Analysis			%Difference between linear and nonlinear analysis
		Load case	% MEU	Element Type	Load case	% MEU	Element Type	
K	60	LC2	97.58	Leg	LC2	98.01	Leg	0.43
	50	LC1	93.93	Bracing	LC2	95.0	Leg	1.07
	40	LC2	94.71	Leg	LC2	94.65	Leg	0.06
KD	60	LC2	99.01	Bracing	LC2	99.34	Leg	0.33
	50	LC3	96.08	Bracing	LC1	95.67	Bracing	0.41
	40	LC3	96.19	Bracing	LC3	95.5	Bracing	0.69
Y	60	LC2	97.12	Bracing	N.G.	N.G.	N.G.	N.G.
	50	LC2	97.15	Leg	LC2	96.98	Leg	0.17
	40	LC2	95.58	Leg	LC2	95.42	Leg	0.16
YD	60	LC1	97.4	Bracing	LC1	97.62	Leg	0.22
	50	LC2	98.9	Leg	LC2	98.9	Leg	0
	40	LC1	98.06	Bracing	LC1	97.33	Bracing	0.73
D	60	LC1	98.79	Bracing	LC3	98.44	Bracing	0.35
	50	LC2	97.05	Leg	LC2	97.35	Leg	0.30
	40	LC2	94.22	Leg	LC2	95.38	Leg	1.16
XB	60	LC2	99.06	Leg	LC2	99.55	Leg	0.49
	50	LC2	93.86	Leg	LC2	94.22	Leg	0.36
	40	LC2	93.42	Leg	LC2	93.74	Leg	0.32
X	60	LC2	93.6	Leg	LC2	91.91	Leg	1.69
	50	LC2	89.26	Leg	LC2	88.58	Leg	0.68
	40	LC2	96.74	Leg	LC2	97.32	Leg	0.58

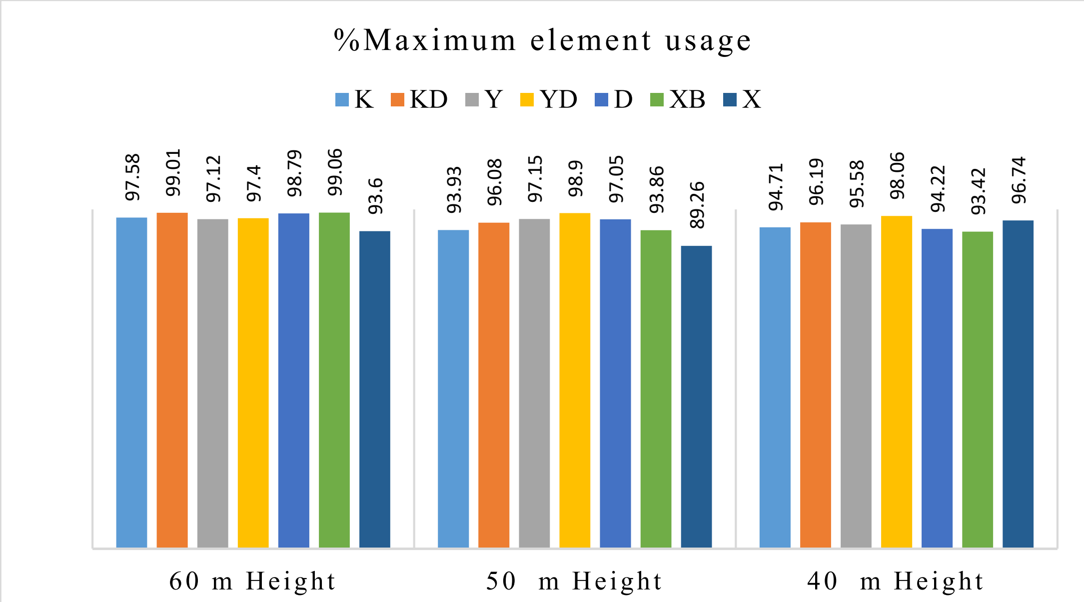


Figure 4.3 Linear analysis results of maximum element usage

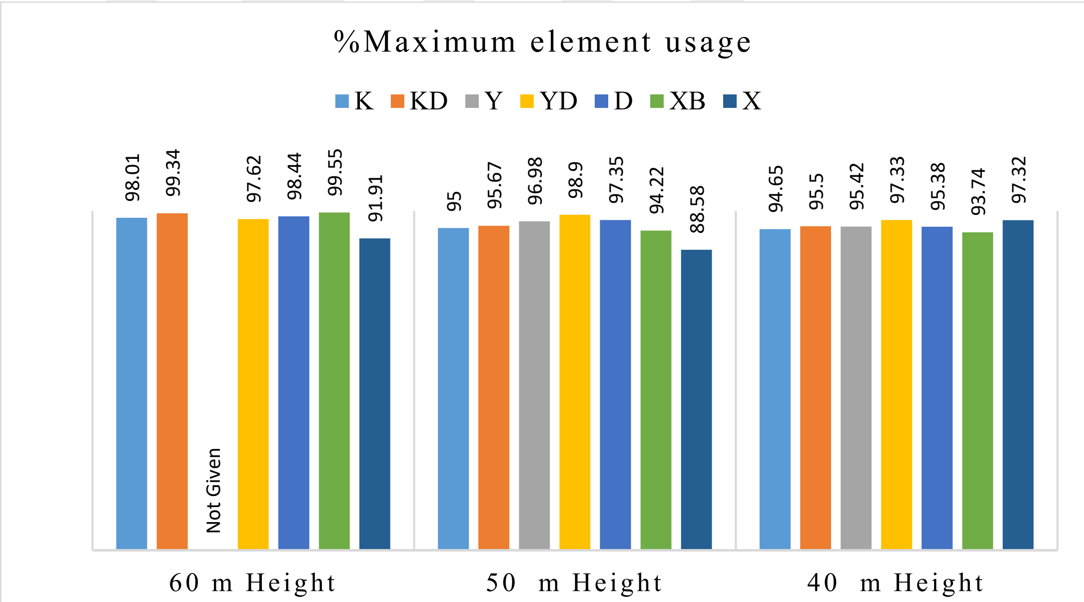
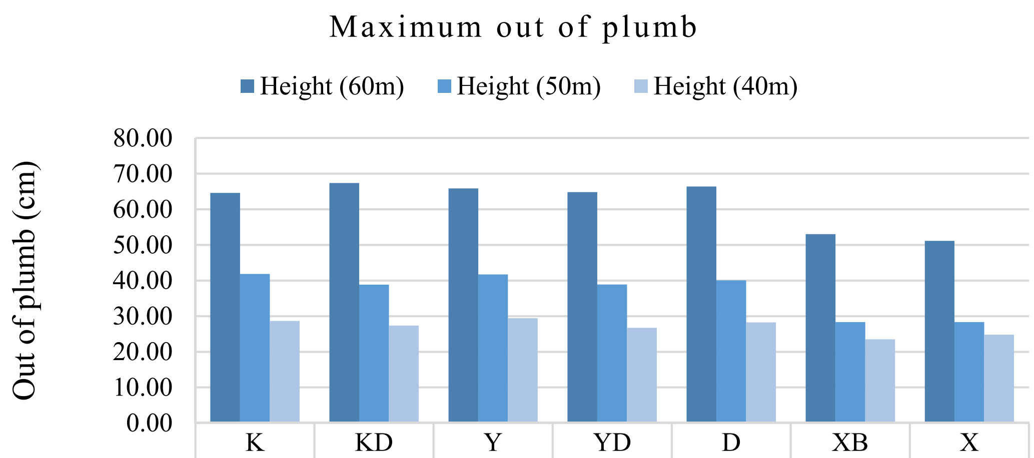


Figure 4.4 Nonlinear analysis results of maximum element usage

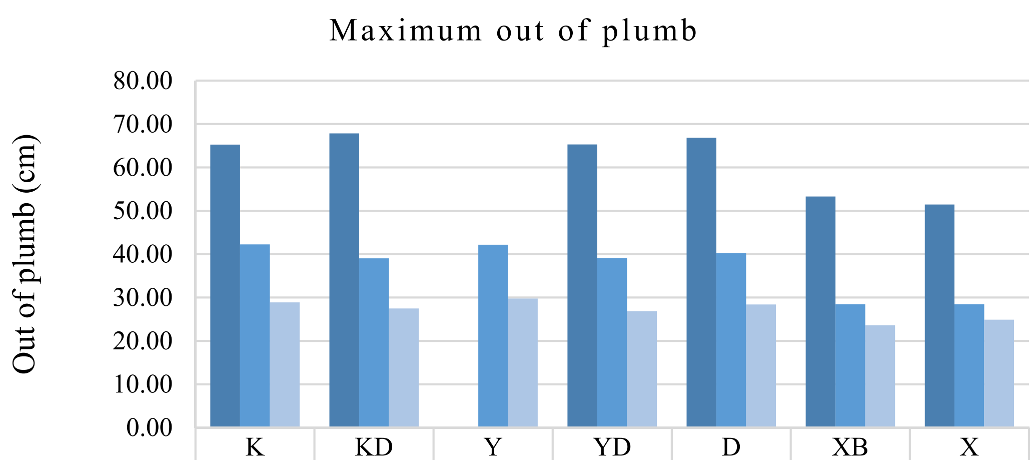
4.4.4 Out of plumb

In all bracing systems, the critical combination of load for maximum and minimum value of out of plumb and sway values for both linear and nonlinear analysis is obtained for LC2 which is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°) expect K bracing system in the height of 50 m which is LC1 is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°). The minimum value of out of plumb obtained for X and XB bracings is 51.12 cm, 28.34 cm and 23.52 cm was obtained for 60 m, 50 m, and 40 m height. The maximum values of out of plumb for 60 m, 50 m, and 40 m heights is 67.37, 41.87 and 29.41 cm, respectively, obtained for KD, K and Y bracings. Figure 4.5 and Figure 4.6 demonstrate linear and nonlinear analysis results of maximum out of plumb value of rectangular base towers, respectively.



	K	KD	Y	YD	D	XB	X
Height (60m)	64.60	67.37	65.83	64.81	66.40	53.00	51.12
Height (50m)	41.87	38.82	41.73	38.87	40.05	28.34	28.34
Height (40m)	28.64	27.35	29.41	26.71	28.25	23.52	24.79

Figure 4.5 Maximum linear analysis results of out of plumb value

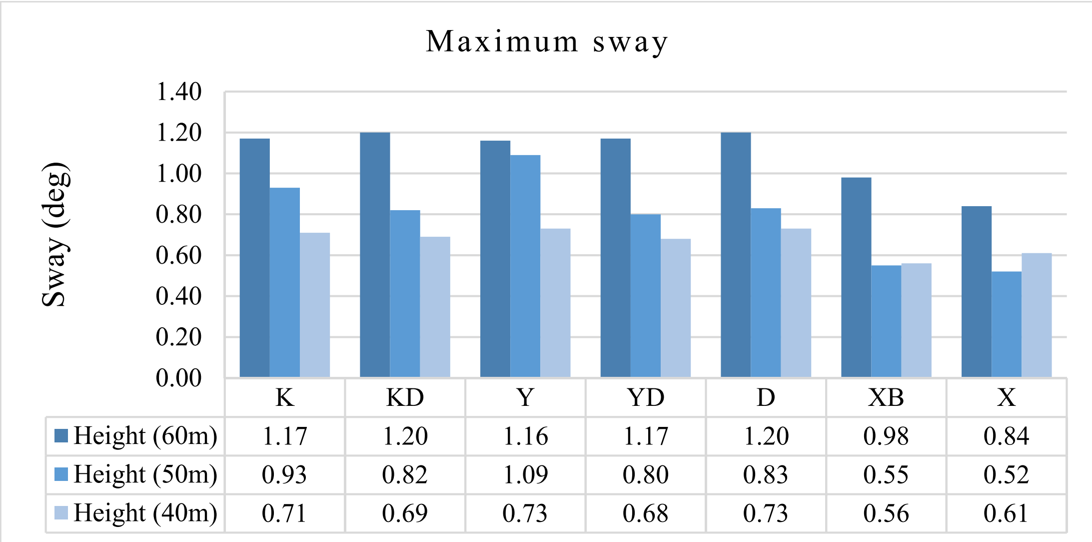


	K	KD	Y	YD	D	XB	X
Height (60m)	65.26	67.85	0.00	65.30	66.83	53.29	51.45
Height (50m)	42.26	39.02	42.17	39.1	40.23	28.43	28.43
Height (40m)	28.88	27.47	29.75	26.83	28.39	23.6	24.87

Figure 4.6 Maximum nonlinear analysis results of out of plumb

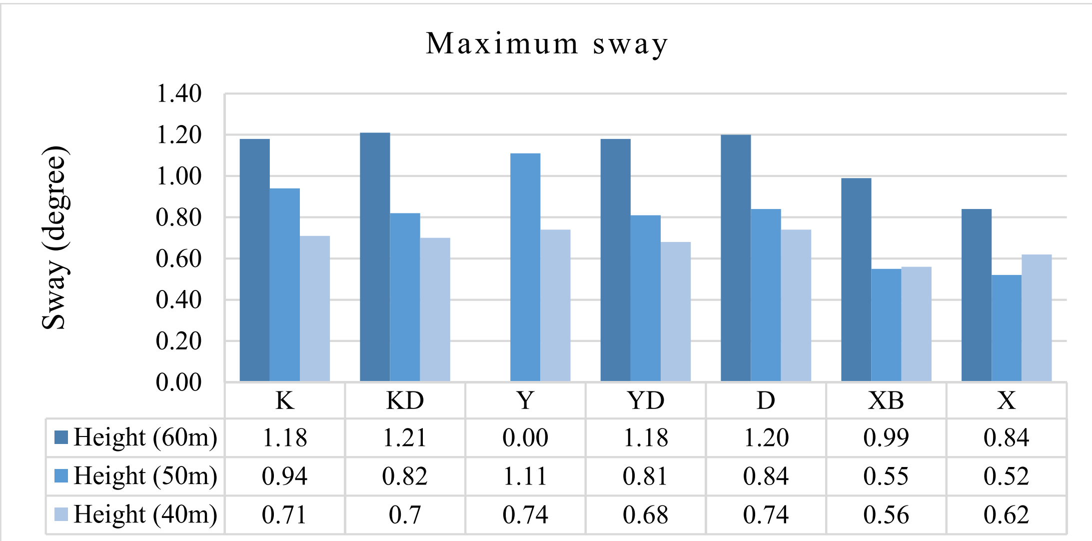
4.4.5 Sway

The best performance of linear analysis of sway values in degree is obtained for X and XB bracing for 60 m, 50 m, and 40 m heights are 0.84, 0.52 and 0.56 degrees. The worst performance returns to KD and D bracing with value 1.2 degrees for 60 m and for 50 m height is obtained for Y bracing system is 1.09 degree. Y and D bracings have a value of 0.73 degrees for 40 m height. The maximum difference between linear and nonlinear analysis results is about 0.01 degree for all type of bracings. Figure 4.7 and Figure 4.8 presents linear and nonlinear analysis results of maximum sway value of rectangular base towers, respectively.



	K	KD	Y	YD	D	XB	X
Height (60m)	1.17	1.20	1.16	1.17	1.20	0.98	0.84
Height (50m)	0.93	0.82	1.09	0.80	0.83	0.55	0.52
Height (40m)	0.71	0.69	0.73	0.68	0.73	0.56	0.61

Figure 4.7 Linear analysis results of maximum sway value



	K	KD	Y	YD	D	XB	X
Height (60m)	1.18	1.21	0.00	1.18	1.20	0.99	0.84
Height (50m)	0.94	0.82	1.11	0.81	0.84	0.55	0.52
Height (40m)	0.71	0.7	0.74	0.68	0.74	0.56	0.62

Figure 4.8 Nonlinear analysis results of maximum sway value

4.4.6 Maximum overturning moment

In all bracing systems, the critical load combination of the maximum overturning moment is 1.2D (dead load) + 1.6Wo (the wind load applied to the tower with an angle of 45°). For linear and nonlinear analyses, as present in Table 4.2, the maximum resultant moment is 5401.62 kN.m, 3565.06 kN.m, and 1507.96 kN.m occurred at the bracings Y, XB, and YD for 60 m, 50 m, and 40 m heights, respectively. The minimum resultant is 2721.10 kN.m, 2555.72 kN.m and 1507.96 kN.m occurred, respectively, at the bracings KD, D, and YD for linear analysis. Maximum and minimum linear and nonlinear analysis result returns to the same bracing types. Failure has occurred in Y bracing system for 60 m height in case of nonlinear analysis. There are 10.42 kN.m and 3.33 kN.m difference between linear and nonlinear analysis results for each 50 m and 40 m height of tower as listed in Table 4.2. Figure 4.9 and Figure 4.10 shows linear and nonlinear analysis results of maximum overturning moments of rectangular base towers.

Table 4.2 Maximum overturning moment results of linear and nonlinear analysis

Type	H. (m)	Linear Analysis			Non-linear Analysis		
		Trans. Moment (kN.m)	Longitud. Moment (kN.m)	Resul. Moment (kN.m)	Trans. Moment (kN.m)	Longitud. Moment (kN.m)	Resul. Moment (kN.m)
K	60	3306.65	3306.67	4676.32	3321.92	3321.94	4697.92
	50	2275.46	2275.48	3218.00	2283.07	2283.08	3228.75
	40	1370.87	1370.88	1938.71	1374.79	1374.79	1944.24
KD	60	2721.08	2721.10	3848.20	2731.77	2731.79	3863.32
	50	1807.65	1807.66	2556.42	1812.28	1812.28	2562.95
	40	1068.59	1068.60	1511.22	1070.93	1070.94	1514.53
Y	60	3819.49	3819.56	5401.62	N.G.	N.G.	N.G.
	50	2389.88	2389.92	3379.83	2399.39	2399.42	3393.27
	40	1511.26	1511.28	2137.25	1516.58	1516.59	2144.78
YD	60	2722.85	2722.91	3850.74	2733.28	2733.33	3865.48
	50	1831.44	1831.46	2590.06	1836.27	1836.29	2596.89
	40	1066.28	1066.30	1507.96	1068.63	1068.65	1511.29
D	60	2740.12	2740.11	3875.11	2750.54	2750.54	3889.85
	50	1807.16	1807.16	2555.72	1811.91	1811.91	2562.42
	40	1117.82	1117.82	1580.83	1120.50	1120.50	1584.62

XB	60	3440.89	3440.89	4866.16	3454.00	3454.00	4884.70
	50	2520.88	2520.88	3565.06	2528.25	2528.25	3575.48
	40	1137.95	1137.95	1609.30	1140.34	1140.34	1612.69
X	60	3241.66	3241.66	4584.40	3251.69	3251.69	4598.59
	50	2399.53	2399.53	3393.45	2405.03	2405.03	3401.23
	40	1300.57	1300.57	1839.29	1303.23	1303.23	1843.05

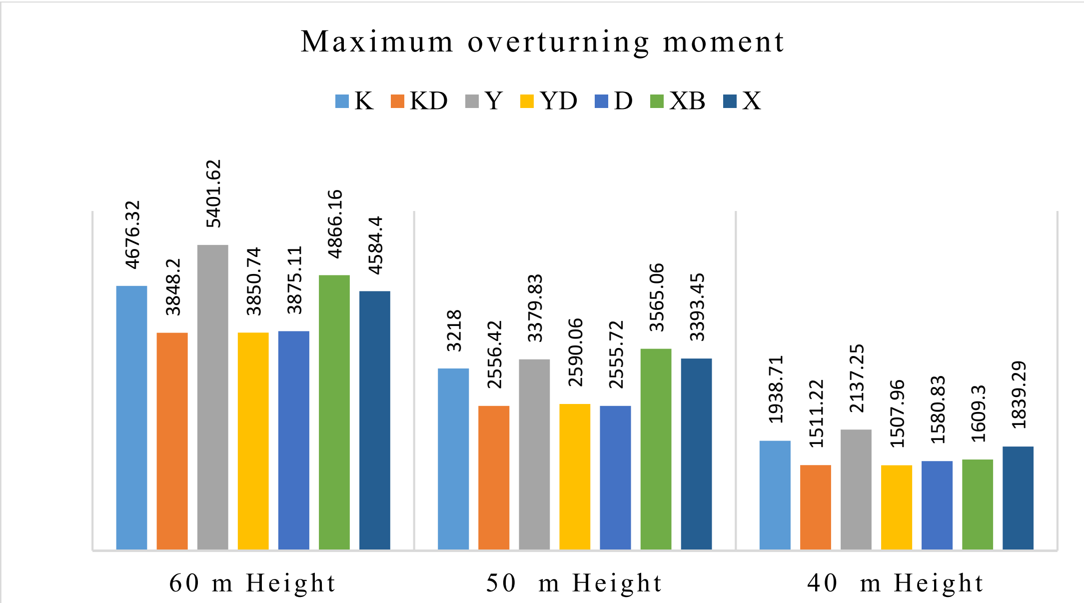


Figure 4.9 Linear analysis results of maximum overturning moment

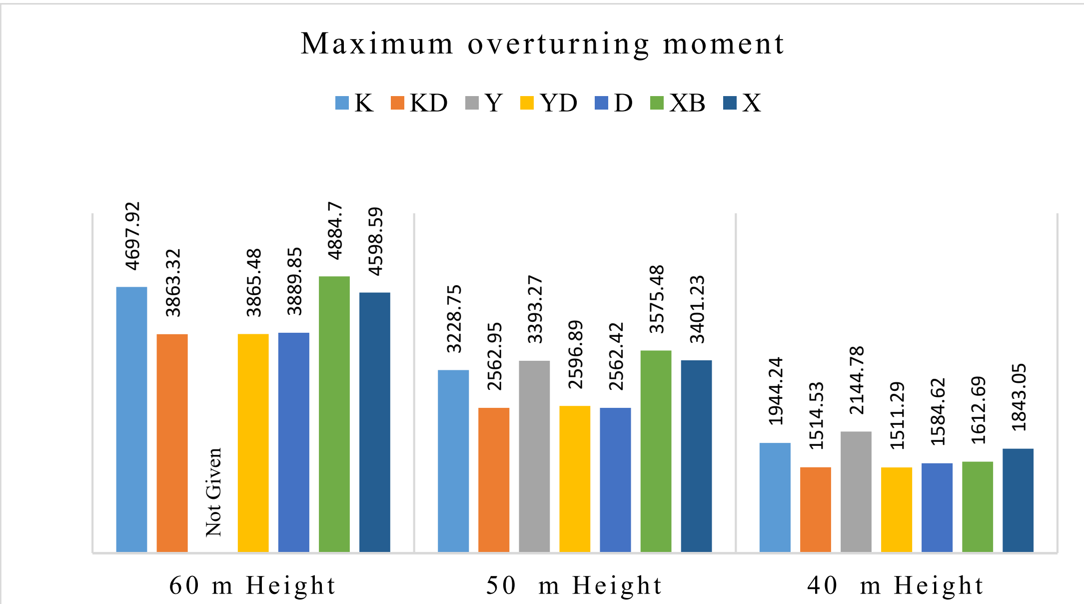


Figure 4.10 Nonlinear analysis results of maximum overturning moment

4.5 Results and Discussion of Triangular Base Towers

Results of triangular base towers will be discussion in this section in accordance to the height of towers and analysis type. Based on linear and nonlinear simulation, outcomes were compared to determine the best performance type of bracing. According to outcomes, it was detected that the tower members less prone to the effects during the comparison between them.

4.5.1 Weight of towers

The weight of towers designed based on linear and nonlinear analyses are the same. According to Figure 4.7, the minimum weight of tower for 60 m and 40 m heights are obtained for X bracings are 73716.2 N and 34669 N, respectively, and for 50 m height is obtained for XB bracing system is 56489.1 N. On the other hand, the heaviest tower for 60 m, 50 m, and 40 m heights obtained for the K bracing system is 242048.3 N, 207310.9 N and 78979.2 N as present in Figure 4.11.

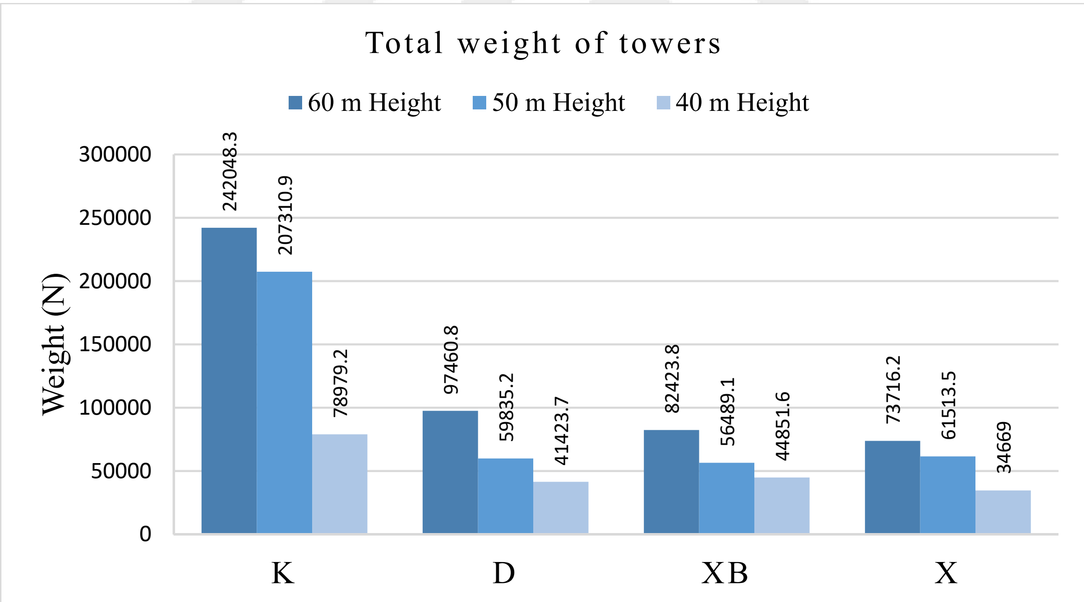
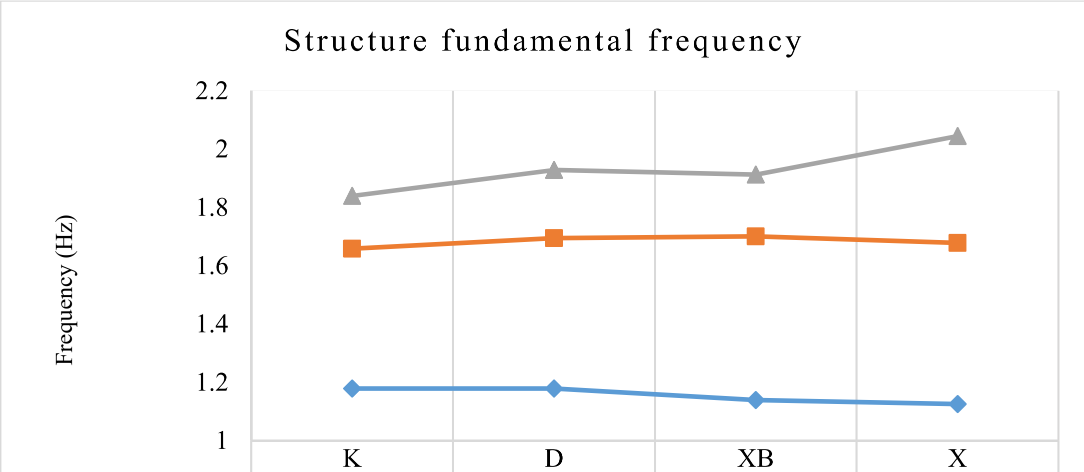


Figure 4.11 Total weight of 60, 50 and 40 m height

4.5.2 Structure fundamental frequency

Figure 4.12 illustrate the results of structure fundamental frequency of different bracing systems and heights. The best performance fundamental frequency for 60m, 50 m, and 40 m heights is obtained for X and K bracing systems, respectively, was

1.1255 Hz, 1.6582 Hz, and 1.8397 Hz. The worst achievement for 60m, 50 m, and 40 m heights is concerned for D, XB and X are 1.1786 Hz, 1.7006 Hz, and 2.0448 Hz, respectively.



	K	D	XB	X
60 m Height	1.1785	1.1786	1.1391	1.1255
50 m Height	1.6582	1.6947	1.7006	1.678
40 m Height	1.8397	1.9285	1.9123	2.0448

Figure 4.12 Structure fundamental frequency of triangle base towers

4.5.3 Maximum element usage

The maximum elements usage is listed in Table 4.3. In both linear and nonlinear analyses, the critical loading is obtained for D and XB bracing types for 60, 50 and 40 m height. Critical containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to the tower with an angle of -90°) is critical for D bracing system at height 60 m. For 50 m height, critical containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 90°) is critical at D bracing system and for 40 m height, critical containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) for XB bracing system. X and K the failure containment load case for 60 and 50 – 40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 90°) is critical for 60 m height X bracing system and failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) are critical for both 50 and 40 m height of K bracing system. The critical members which are given in Table 4.3, are the same in linear and nonlinear analyses. The all critical members are in compression. Difference percentage of maximum element usage between linear and nonlinear analysis type are described in Table 4.3. Figure 4.13 and 4.14 shows linear and nonlinear results of maximum elements usage values of triangular base towers in percentage.

Table 4.3 Linear and nonlinear analysis results of maximum element usage

Type	H. (m)	Linear Analysis			Non-linear Analysis			%Difference between linear and nonlinear analysis
		Load case	% MEU	Element Type	Load case	% MEU	Element Type	
K	60	LC1	95.99	Leg	LC1	96.63	Leg	0.64
	50	LC1	96.37	Leg	LC1	96.84	Leg	0.47
	40	LC1	96.27	Leg	LC1	96.57	Leg	0.30
D	60	LC12	85.85	Bracing	LC12	85.14	Bracing	0.71
	50	LC2	87.69	Bracing	LC2	85.99	Bracing	1.70
	40	LC1	79.21	Leg	LC1	79.63	Leg	0.42
XB	60	LC12	94.13	Bracing	LC12	90.08	Bracing	4.05
	50	LC2	82.43	Bracing	LC1	82.87	Bracing	0.44
	40	LC1	77.32	Leg	LC1	72.94	Leg	4.38
X	60	LC2	97.64	Bracing	LC2	97.77	Bracing	0.13
	50	LC2	83.00	Bracing	LC2	82.78	Bracing	0.22
	40	LC1	94.43	Leg	LC1	94.78	Leg	0.35

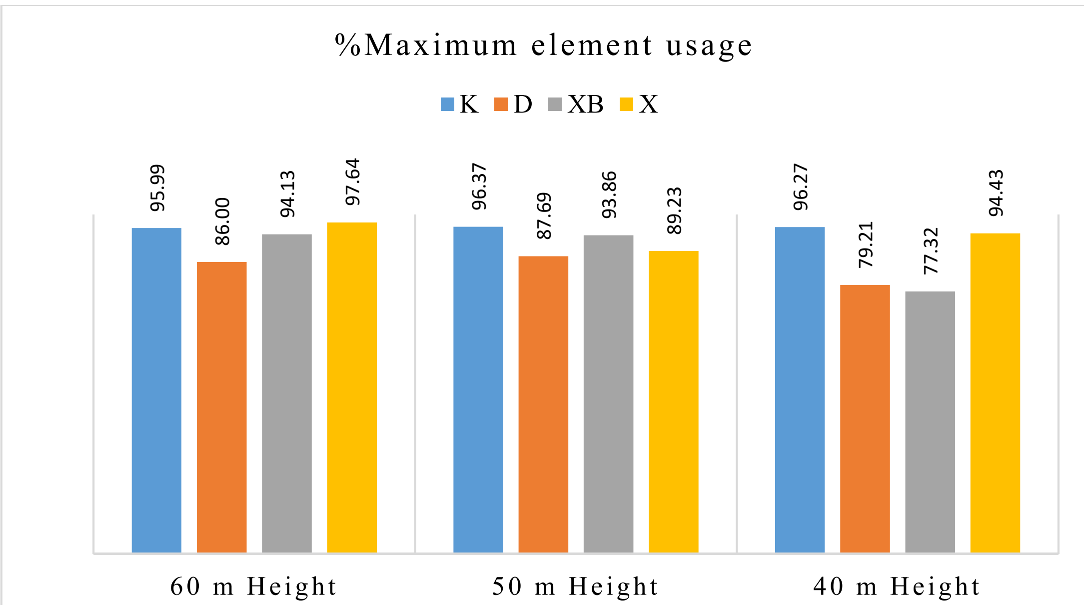


Figure 4.13 Linear analysis results of maximum element usage

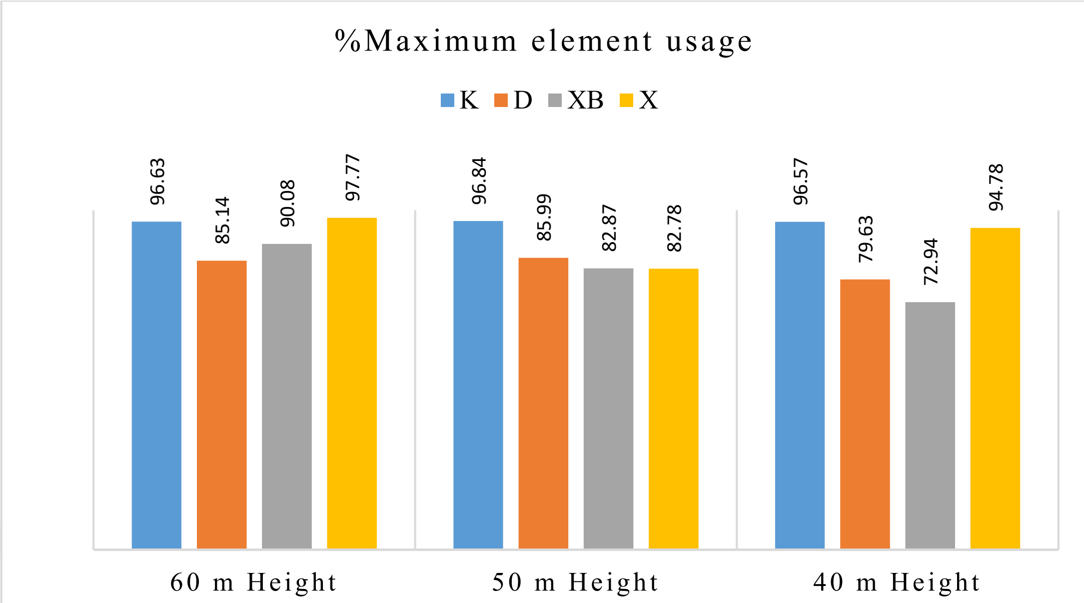
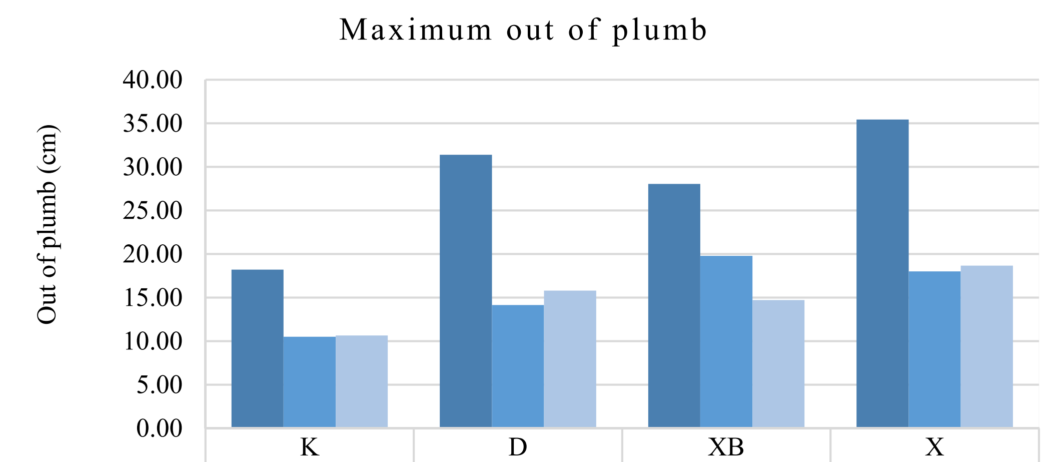


Figure 4.14 Nonlinear analysis results of maximum element usage

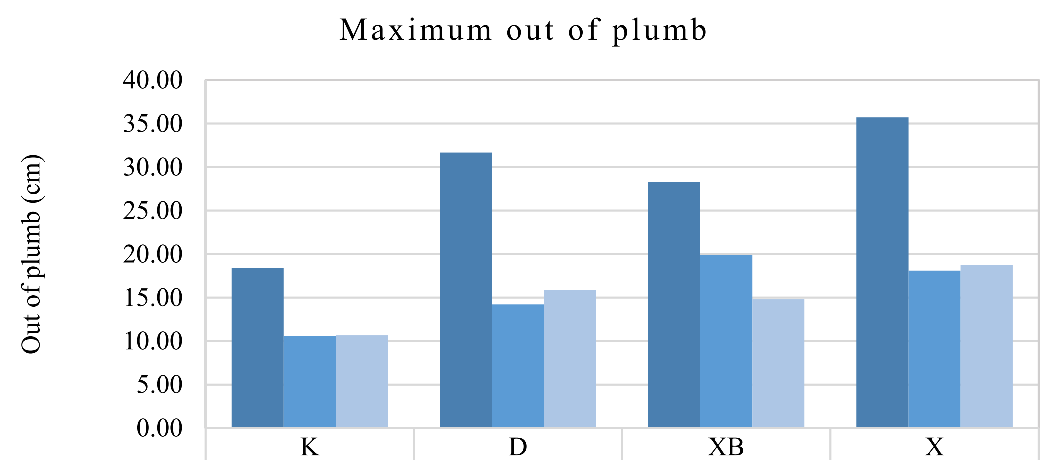
4.5.4 Out of plumb

In all bracing systems, the critical combination of load for best and worst values of out of plumb and sway for linear and nonlinear analysis is obtained for LC1 which is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) expect X bracing system in the height of 60 m which is LC2 is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°). The best performance value of out of plumb is 18.22 cm, 10.51 cm, and 10.66 cm obtains for 60 m, 50 and 40 m height, all attained for K bracing system. The worst value of out of plumb for 60 m and 40 m is 35.45 cm, and 18.68 cm is returned to X bracing system. For 50 m heights is 19.79 cm obtained for XB bracing. Difference between linear and nonlinear analysis results is 0.18 cm,0.08 cm, 0.00 cm, 0.27 cm, 0.07 cm, 0.08 cm, 0.09 cm, 0.07 cm, 0.25 cm and 0.08 cm, respectively for K, D, XB and X bracings for 60 m, 50 m and 40 m heights. Figure 4.15 and Figure 4.16 shows linear and nonlinear analysis results of maximum out of the plumb value of triangular base towers respectively.



	K	D	XB	X
Height (60m)	18.22	31.39	28.05	35.45
Height (50m)	10.51	14.15	19.79	18.01
Height (40m)	10.66	15.80	14.73	18.68

Figure 4.15 Maximum linear analysis results of out of plumb



	K	D	XB	X
Height (60m)	18.40	31.66	28.25	35.70
Height (50m)	10.59	14.22	19.88	18.09
Height (40m)	10.66	15.88	14.80	18.75

Figure 4.16 Maximum nonlinear analysis results of out of plumb

4.5.5 Sway

The best performance of linear analysis of sway values in degree is obtained for K bracings for 60 m, 50 m, and 40 m heights are 0.31, 0.2 and 0.24 deg. The worst performance is for X bracing with value 0.52 and 0.42 degree for 60 m and 40 m heights. XB bracing has a value of 0.4 degrees for 50 m height. About 0.01 degree is the maximum different value between linear and nonlinear analysis for K, D, XB and

X bracing systems. Figure 4.17 and Figure 4.18 presents linear and nonlinear analysis results of maximum sway value of rectangular base towers, respectively.

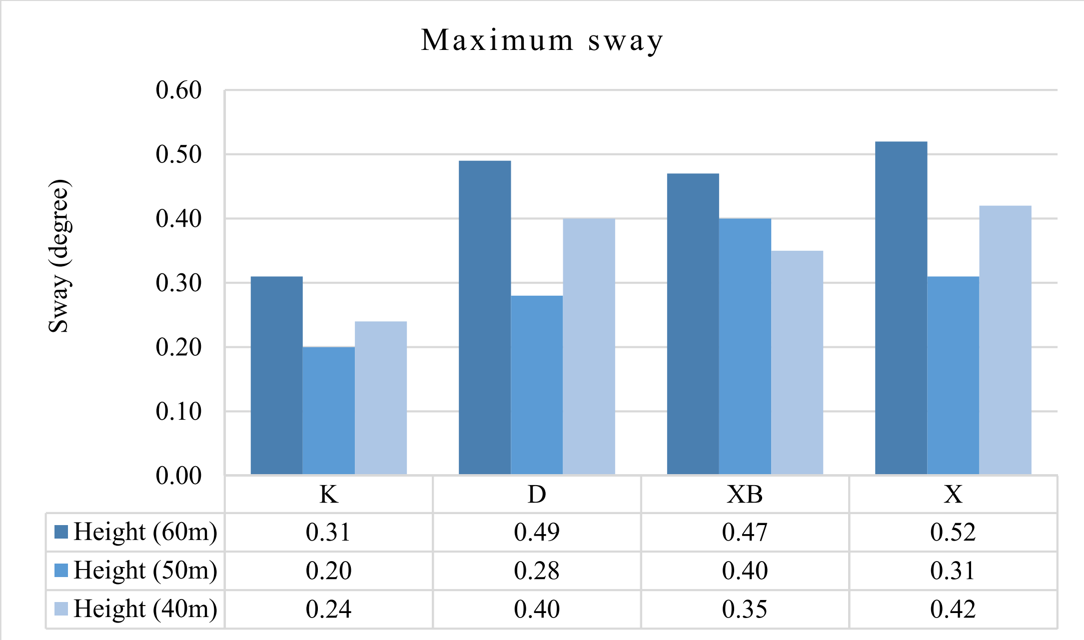


Figure 4.17 Linear analysis results of maximum sway

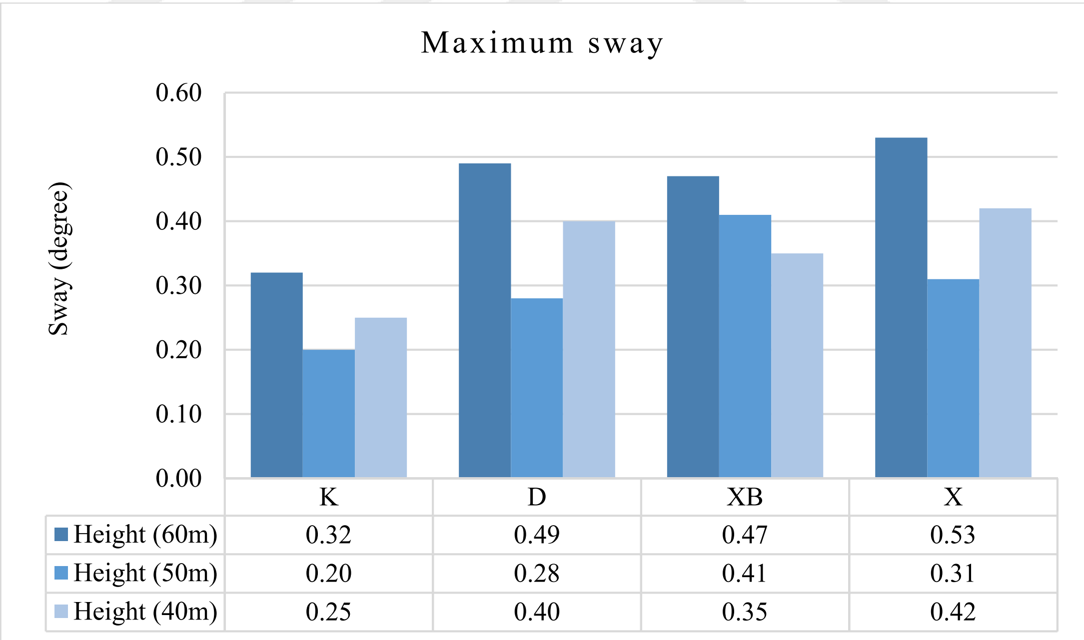


Figure 4.18 Nonlinear analysis results of maximum sway

CHAPTER 5

CONCLUSIONS AND FURTHER WORK

5.1 Conclusions

As objective of this thesis, the performance of the existing towers was analyzed considering different loading as per equivalent static method is given in AN SI/TIA-222-G for selected rectangular and triangular base green field towers.

This thesis encompassed the modeling of thirty-three communication steel lattice towers. Twenty-one of towers are rectangular base towers, and twelve are triangular base towers designed on three different heights of 60, 50 and 40 m heights using various type of bracing patterns. K, KD, Y, YD, D, XB and X bracing patterns have been used for rectangular base towers, and K, D, XB and X bracings are utilized for triangular base towers.

Numerical tools to perform advanced numerical analyses of lattice towers are now widely available and well-documented. However, one needs to choose wisely the modeling strategy and hypotheses to suit the problem studied. Unnecessary complexity of a model normally avoided because it increases the number of assumptions and could lead to additional model uncertainty. When complex models are required, they should always be validated with simpler numerical models, and if possible, with experimental tests. Also, fabrication and erection tolerances could have an impact on the tower behavior, and these needs to be considered in the interpretation of results. Nevertheless, in many cases, advanced numerical models represent a powerful tool for the analysis of lattice towers and could lead to a reduction or an optimization of the need for full-scale testing.

Both towers were modeled in TOWER program. TOWER is an analysis tool consist of linear and nonlinear analysis type that utilizes the ANSI/TIA 222 standard code.

If the height of the tower is less than 50 m the difference between linear and nonlinear analysis negligible.

The minimum weight of tower for 60, 50 and 40 m height is obtained at KD and YD bracing system, respectively, for rectangular base towers. In the case of triangular base towers, X and XB type bracing systems give the smallest weight design for the same height. The maximum weight of towers is obtained for the XB and Y bracing systems for rectangular base towers. Otherwise, K bracing system has a maximum weight of triangular base towers for all heights.

The best achievement fundamental frequency for 60 m, 50 m, and 40 m heights of rectangular base towers returns to KD, Y, and D bracing system. Furthermore, the worst result is obtained for K, D, and KD bracing systems. On the other hand, for triangular base towers, the best achievements returns to X and K bracing systems. Otherwise, the worst achievement at D, XB, and X bracing systems.

In both linear and nonlinear analyses; the critical loading is obtained for X and XB structure type for 60, 50 and 40 m height rectangular base towers, respectively. XB and YD the failure containment load case 1.2D (dead load) + 1.6Wo (wind load applied to the tower with an angle of 45°) is critical. For triangular base towers, the critical loading is obtained for D and XB bracing types. X and K the failure containment load case for 60 and 50 – 40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 90°) is critical for 60 m height X bracing system and Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) are critical for both 50 and 40 m height of K bracing system.

The minimum value of out of plumb obtained for X and XB bracings and the maximum value of out of plumb obtained for KD, K and Y bracing systems, respectively, for rectangular base towers for 60, 50 and 40 m height. The minimum value of out of plumb obtained for K bracings for the same heights. Otherwise, the maximum value at X bracing system for 60 and 40 m height and for 50 m height is obtained for XB bracing system for triangular base towers.

The best performance of linear and nonlinear analysis of sway values in degree for rectangular base towers is obtained for X and XB bracing systems for the heights of 60, 50 and 40 m. The worst performance returns to KD and D bracings for 60 m height and for 50 m height is obtained for Y bracing. For 40 m height is obtained for Y and D bracings. For triangular base towers, the best performance is obtained for K bracing, and the worst performance return to X bracing for 60 - 40 m heights and for 50 m height is obtained for XB bracing.

In all bracing systems, the critical load combination of the maximum overturning moment is 1.2D (dead load) + 1.6Wo (the wind load applied to the tower with an angle of 45°). For linear and nonlinear analyses, the maximum resultant moment occurs at the bracings Y, XB, and YD for 60 m, 50 m, and 40 m heights, respectively. The minimum resultant occurs, respectively, at the bracings KD, D, and YD for linear analysis. Maximum and minimum linear and nonlinear analysis result returns to the same bracing types.

Failure has occurred in Y bracing system for 60 m height in case of nonlinear analysis.

5.2 Future Work

As with any study, there are further opportunities for the investigation to improve and refine knowledge. Some activities which would further refine the studies described here are:

The spot of this study was on the design and analysis of single slope towers. In the current work, a single slope segment towers utilized. The present methodology can be extended to straight, two slopes, three slope towers;

TOWER is special program produced to design and analysis guy, monopole, and self-supporting towers, in current study self-supporting towers utilized. It would be useful to design and analysis another type of towers by using TOWER program to determine the behavior of the towers;

In particular, region where ice load available, future studies can be computed ice load beside wind, earthquake and service loads as computed in current work;

ANSI/TIA 222-G standards is provided for computing load cases and combination of loads on both rectangular and triangular base towers in this study, it would be more accurate to design, and analysis same type of towers and bracings by using other standards are available in TOWER program for comparing purposes between standards;

Since a tower can be used for more than hundreds of times, a future work can be focus on an optimization of lattice steel towers to determine the safe weight to the least;

Design and analysis of the steel lattice towers provided using TOWER program in current comparison. By utilizing another engineering program, it would be more accurate to compare output results of same bracings, cross-sections, and heights of towers to determine the best performance of bracing;

Present work contains some selected type of bracing systems; potential future studies can be done on other types of bracing systems with referring to the main similarities and differences;

Current work based on comparing 60, 50 and 40 m heights of rectangular and triangular base towers, a future work can be utilized smaller or larger heights of towers for the same bracing systems and compare it.

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