

ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING



**Design Module for Self-Supporting Steel Latticed Communication
Towers**

A Thesis in Structural Engineering

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UNDERTAKING

I certify that research work titled “***Design Module for Self-Supporting Steel Latticed Communication Towers***” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ABSTRACT

Self-supporting towers are one of the most essential and common steel structures which are being constructed quite frequently to provide a better facility in communication network to the people. Hence, the failure of such a structure is a major concern. Therefore, utmost priority must be given in considering all possible extreme conditions for designing communication towers, by evaluating and studying their wind and seismic behaviors properly.

In this study, concepts behind codes specifications in relation with, equal angle steel section used as structural members in communication towers and communication tower itself, have been investigated thoroughly, so that they can be referred as a principles for analysis and design.

In this research, wind and seismic behavior of self-supporting steel latticed communication towers have been studied for 35 m/s, 40 m/s, 45 m/s, and 50 m/s wind speed, in five different seismic zones, by analyzing different height of bare towers with XX bracing system. Moreover, charts for specified wind speeds and seismic zones which can be used as preliminary design for communication towers has been developed. These charts shows the responses of towers with different heights, wind speeds and seismic loads in different loading directions.

In addition to that, step by step design procedure and design of articulated modular self supporting steel latticed tower is presented with the help of algorithm and flowchart by using recommended allowable strength design codes specifications, in relation with analyzing and designing of steel communication tower.

Finally, the design module is prepared in a way that it can be used as a Guide line in designing and analyzing the effect of wind load and seismic load on the steel latticed communication towers.

Key word: *Self-Supporting Communication Tower, Wind Load, Seismic Zone, Allowable Strength Design*

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LIST OF SYMBOLS AND NOTATIONS

A_F	Projected area of flat structural components in one face of the section
A_G	Gross area of one face as if the face were solid
A_R	Projected area of round structural components in one face of the section
C_F	Force coefficients
D_F	Wind direction factor for flat structural components
D_r	Wind direction factor for round structural components
E	Modulus of elasticity
EPA_A	Effective projected area of the appurtenance
EPA_N	Effective projected area along windward face normal to the azimuth of the appurtenance.
EPA_S	Effective projected area of the structure
EPA_T	Effective projected area associated with the windward side face of the appurtenance
f	Fundamental frequency
F_A	Design wind force on antennas and appurtenances
f_e	Euler's Critical stress
F_S	Design wind force on the structure
F_w	Design wind load
f_y	Yield stress
G_h	Gust effect factor
I	Importance factor
K_D	Wind direction probability factor
K_Z	Velocity pressure coefficient
K_{Zt}	Topographic factor
q_Z	Velocity pressure
R	Response Modification Factor
R_R	Reduction factor for a round element
V	Basic wind speed for the loading condition under investigation
λ	Slenderness ratio
ε	Solidity ratio
σ_r	Rankines allowable compressive stress
σ_e	Elastic critical stress in compression

ACRONYMS

The following acronyms appear in this work. They are written out based on the letters first appear.

AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineering
ASTM	American Society for Testing and Materials
ASD	Allowable Strength Design
EIA	Electronic Industries Alliance
IBC	International Building Code
MCE _R	Maximum Considered Earthquake
RCSC	Research Council on Structural Connections
SEI	Structural Engineering Institute
SSCT	Self Supporting Communication Tower
TIA	Telecommunications Industry Association

CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND OF STUDY

Towers are tall steel framework structures used for different purposes such as communication, radio transmission, satellite receptions, air traffic controls, television transmission, power transmission, flood light stands, oil drilling masts and meteorological measurements, etc. self-supporting Lattice towers act as vertical trusses and resist lateral loads by means of cantilever action. Those bracing members in the tower structure are arranged in many forms together with zero force members, which carry solely tension, or alternatively tension and compression. The bracing is made up of crossed diagonals and Zero force members, but only diagonal bracings resist tension. Depending on the direction of wind, one diagonal takes all the tension while the other diagonal is assumed to remain inactive or compression.

Tensile bracing is smaller in cross-section and is usually made up of a back-to-back channel or angle sections. The bracings behave as struts, when it is designed to take compression. One of the most common arrangements is the cross bracing.

In steel tower the most significant dimension is its height. It is normally several times larger than the horizontal dimensions.

The area that is occupied at the ground level is considerably limited and so, slender structures are commonly used. The tapered part of the tower is advantageous with regard to the bracing, as it reduces the design forces. The greater the height of the tower, greater will be the distance it can transmit radio signals, [1].

The structural engineers face the challenging job of designing and



Figure 1: Communication Tower

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Constructing telecommunication towers to support antenna, appurtenance loads and platform as well as steel ladder loads in open weather with high degree of reliability.

The major cause of failures of telecommunication tower throughout the world though still remains to be high intensity winds and seismic load. The major problem faced is the difficulty in estimating wind and seismic loads as they are based on a probabilistic approach. There have been several studies in telecommunication towers taking into consideration the wind load effect only.

The expansion of the telecommunication systems in the world were the main reasons for the continuous demands for the production of steel telecommunication towers. Lattice type steel structures have been largely utilized in world to support cellular and microwave transmission antennas.

1.2 STATEMENT OF THE PROBLEM

At most all of the national construction company sectors in Ethiopia hesitate to engage in the construction of steel structure towers due to lack published analysis and design codes, and recommendations in accordance with national parameters which focuses on steel tower for wind and seismic load. In this circumstance, client in need of these facility do not consult national companies rather than transferring tasks to the foreign companies. So to overcome those problems, studies which can provides better understanding on the behavior of self-supporting communication tower must be conducted.

Usually, it is alleged that any tower structure designed for wind load can with stand Seismic load. Even if it is true, it is wrong to perceive the statement works for all Seismic Zones because, tower structure within high seismic zone may exhibits Structural response more than response caused by wind load.

Since, Ethiopia Building codes Standards do not give sufficient emphasis on minimum wind load and seismic load consideration for towers, it is tried to refer codes that recommends the minimum design load and specification for the communication trussed tower in relation with wind load and seismic load consideration.

Therefore, in this paper ASCE 7-05, [10] AISC 360-10, [19] ASCE 10-15, [20] and ANSI/TIA-222-G, [9] code of tower designs will be referred accordingly. Because of the above statements of the problem, it is an interesting to provide a design module by studying their behavior, designing and analyzing self-supporting tower.

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1.3 OBJECTIVES

1.3.1 General objectives

The main objective of this thesis work is to provide a module, used for designing and analyzing of self-supporting communication towers structures by carrying out the design and analysis of steel towers for various heights of towers by considering wind and seismic loads.

1.3.2 Specific objectives

- To study the behavior of seismic and wind loads under various heights, bracing system and various conditions
- To compare the effect of wind load and seismic load on communication tower
- To provide tables of values or/and analysis chart which can be used as preliminary design input for the analysis and designing of steel latticed tower
- To provide step by step designing procedures used for self-supporting communication tower, with an example governed by wind load

1.4 SCOPE AND LIMITATION

- Design Module is prepared only for square, crossed braced, four leg self-supporting communication tower
- Allowable Stress Design method is used
- Only Equal angle steel section are used
- Design of the column base plate and concrete foundations are not included in the module
- Design of climbing structures are not included in the module
- Prying action in splice connection is not included
- Economical design is not considered

1.5 ORGANIZATION OF THE THESIS

The thesis is organized in to six chapters in a way that it is easy to understand and follow. The First chapter discusses about communication tower, and followed by statement of the problem and objectives. The Second chapter comes with the literature review regardless of concepts related with design philosophy, types and bracing arrangements of communication tower.

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The Third chapter, briefly discusses concepts behind code specifications that are used in analysis and designing of self-supporting communication tower which are referred from various American Standards accordingly and provides step by step procedures.

The Fourth chapter, provides preliminary input for design, by studying the wind and seismic behavior of self-supporting communication bare tower under various conditions. Modelling of the tower and manual computations are conducted using AutoCAD and Microsoft Excel respectively.

The Fifth and Sixth chapters, provides a step by step design sample of self-supporting communication tower in Addis Ababa as an example to grasp principles that have been discussed in the previous chapters. Finally, conclusion and recommendation are drawn based on the chapter four and five in the seventh chapter.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Once the external loads acting on the tower are determined, one can proceed with an analysis of the forces in various members with a view to fixing up their sizes. Since axial force is the only force for a truss element, the member has to be designed for either compression or tension. When there are multiple load conditions, certain members may be subjected to both compressive and tensile forces under different loading conditions. Reversal of loads may also induce alternate nature of forces; hence these members are to be designed for both compression and tension.

The total force acting on any individual member under the normal condition and also under the broken-wire condition is multiplied by the corresponding factor of safety, and it is ensured that the values are within the permissible ultimate strength of the particular steel used, [2].

Studies have been carried out study on the influence of modeling in lattice mobile towers under wind loading where in the towers are analyzed for gust factor wind. Displacements, Member forces and maximum stress have been compared to find out the effect on towers. The displacements at the top of the tower are determined using the wind analysis on antenna towers with static & gust factor method, the displacements at the top of the tower with angle and square sections are considered, [3].

2.2 TYPES OF COMMUNICATION TOWERS

2.2.1 Based on structural action

Telecommunication towers are classified into three major groups based on the structural action.

- Self-supporting towers
- Guyed towers
- Monopole

Self-Supporting Towers

A self-supporting tower is constructed without guy wires. Self-supporting towers have a larger footprint than monopoles, but still requires a much smaller area than guyed masts due to its relatively small footprint, this kind of tower is commonly seen in cities or other

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places where it is short of free space. Self-supporting towers are categorized in to two groups

- Four legged towers and
- Three legged towers.

Guyed Tower

Guyed towers are normally guyed in three directions. These towers are much lighter than self-supporting type but require a large free space to anchor guy wires. Whenever large space is available, guyed tower can be provided. There are other restrictions to mount dish antennae on these towers and require large anchor blocks to hold the ropes, [4].

Monopole Towers

Monopoles are hollow tapered poles made of galvanized steel. They are constructed of slip jointed welded tubes and can be up to 60m. Due to its construction, they are expensive to manufacture but simple to erect.

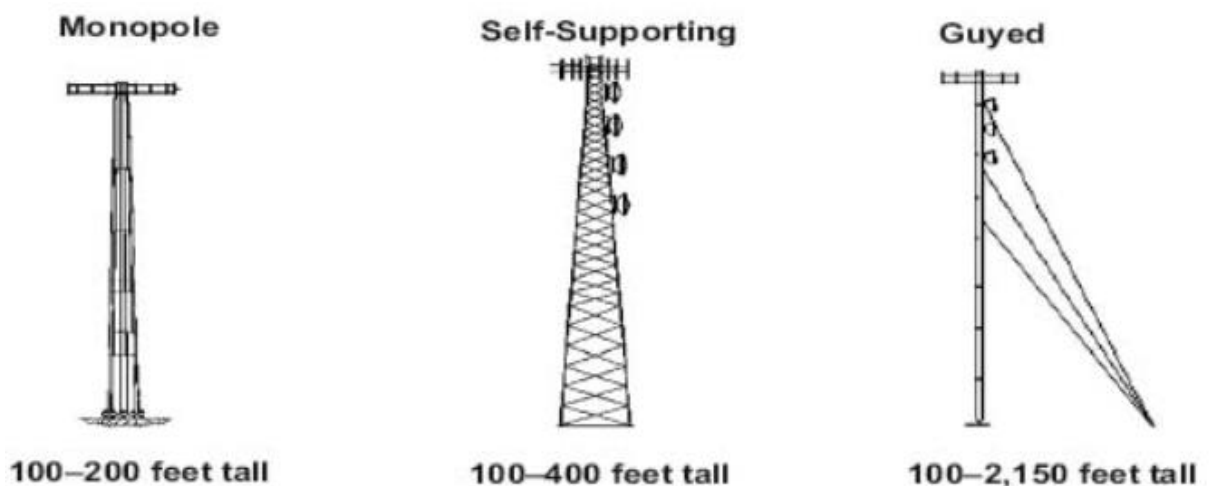


Figure 2: Type of tower

2.2.2 Based on cross section of tower

Towers can be classified, based on their cross section, into square, rectangular, triangular, delta, hexagonal and polygonal towers. An open steel lattice tower make the most efficient use of material and enables the construction of extremely light-weight and stiff structures by offering less exposed area to wind loads. Most of the power transmission, telecommunication and broadcasting towers are lattice towers.

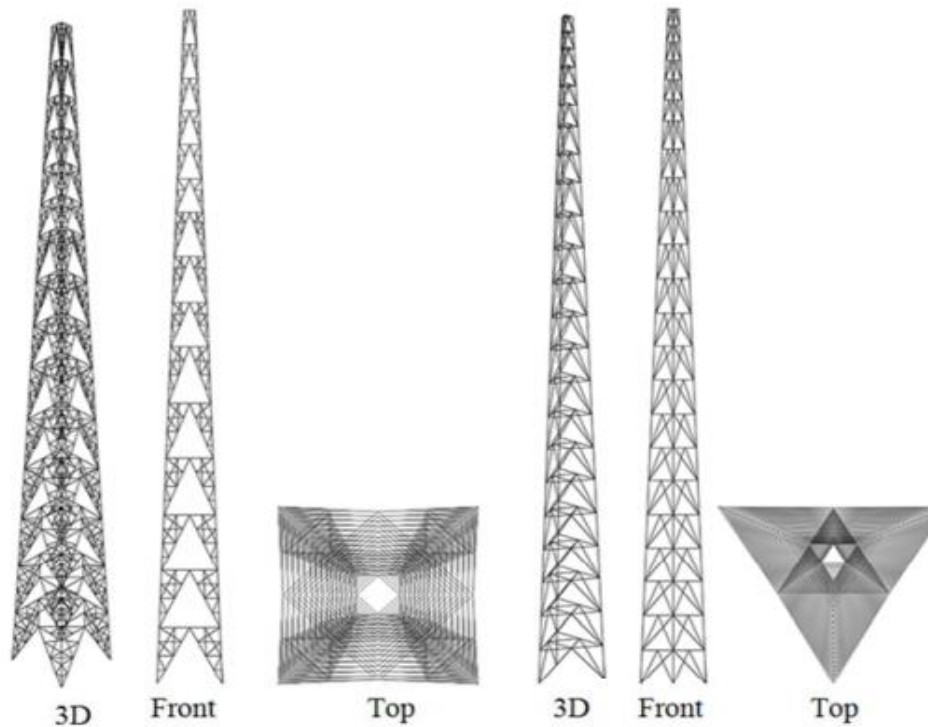


Figure 3: Type of cross section of communication tower

2.2.3 Based on the type of material sections

Based on the sections used for fabrication, towers are classified into angular and hybrid towers (with tubular and angle bracings). Lattice towers are usually made of bolted angles. Tubular legs and bracings can be economic, especially when the stresses are low enough to allow relatively simple connections. Towers with tubular members may be less than half the weight of angle towers because of the reduced wind load on circular sections. However the extra cost of the tube and the more complicated connection details can exceed the saving of steel weight and foundations.

2.3 HEIGHT AND PANELING

Towers of height 40m and 50m have been analyzed with different types of bracing systems under wind loads. The diagonal wind has been found maximum for towers. The optimal bracing system has been identified and reported. The bracing is made up of crossed diagonals, when it is designed to resist only tension. Based on the direction of wind, one diagonal takes all the tension while the other diagonal is assumed to remain inactive, [1].

The steel lattice towers have been analyzed, idealizing it as a 3D truss as per Indian Standard: 800-2007, with various bracing configurations. The dead loads acting on the tower are self-weight of the tower and self-weight of antenna. The wind loads are considered acting both normal and diagonal to the face of the tower. Wind loads have been

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computed by MS Excel and have been incorporated in the analysis done by STAAD Pro.V8i. The joint displacement with respect to normal wind and diagonal wind has been obtained from the analysis for each tower from the results. It has been found that Y bracing is the most appropriate arrangement of bracing system that resists lateral loads for the given range of heights of the towers, as it shows comparatively lesser weight than the other bracing systems.

The Steel Communication tower is designed for heights of 25 m, 35 m and 45 m. The towers are provided with 5-different types of bracings: K type, XBX-type, V-type, W-type, XX-type for lower portion and X-Bracing for upper portion of tower. STAAD Pro. V8i has been used for modeling, analysis and design of towers. Comparative analysis for different heights of towers using different bracing patterns for Earthquake zones II to V of India has been carried out. Gust factor method is used for wind load analysis; modal analysis and response spectrum analysis are used for earthquake loading.

- i. Displacement increases with the increase in speed of the wind. Results displayed that the increase in the displacement from wind zone I to wind zone VI is maximum for W-Bracing and it is minimum for K-Bracing.
- ii. For all wind zones tower height between 25m to 35m with different bracing patterns does not show much difference in displacement.
 - For wind zone I to IV, tower height between 35m to 45m having K Bracing or W-Bracing gives maximum value of displacement and V-Bracing gives minimum value of displacement.
 - For wind zone V and VI tower height between 35m to 45m having W-Bracing gives maximum value of displacement and V-Bracing or XBX -Bracing gives minimum value of displacement
- iii. Stress increases with the increase in speed of the wind. Results show that the increase in stress in the bottom leg members of the tower from wind zone I to wind zone VI is maximum for K Bracing and it is minimum for XX-Bracing.
- iv. Stress increases with the increase in the height of the Tower. Results show that the increase in stress is maximum for K-Bracing and it is minimum for XX-Bracing, [6].

2.4 DIAGONAL AND ZERO FORCE MEMBERS IN BRACING SYSTEMS

2.4.1 Diagonal bracings

Bracings hold the structure stable by transferring the loads sideways (not gravity, but wind loads) down to the ground and are used to resist lateral loads, thereby preventing sway of the structure. Bracing increases the resistance of the structure against side sway or drift. The higher the structure, the more it is exposed to lateral loads such as wind load and earthquake load, since it has higher tendency to sway. If the bracing is weak, the compression member would buckle which leads to failure of the tower. Diagonal braces are efficient elements for developing stiffness and resistance to wind loads. There are different types of bracing systems in common use such as Single diagonal bracing, double diagonal (XX) bracing, and XB bracing, etc. In the current study, commonly provided with five different types of bracing systems are considered in the telecommunication towers such as KX, K, D, XD, and KD as shown in Figure 4. These towers are modeled, analyzed and designed accordance to the ASCE 10 code. Figure 4 illustrates both rectangular and triangular base towers with different bracing patterns. Load, load combination and other design parameters specified in ANSI/TIA-222-G, [9] code have been used for analysis and designed of towers [7].

The analysis of the towers for lateral load with different heights and configurations showed the optimal solution. Hence, this research project is aimed to suggest the efficient bracing

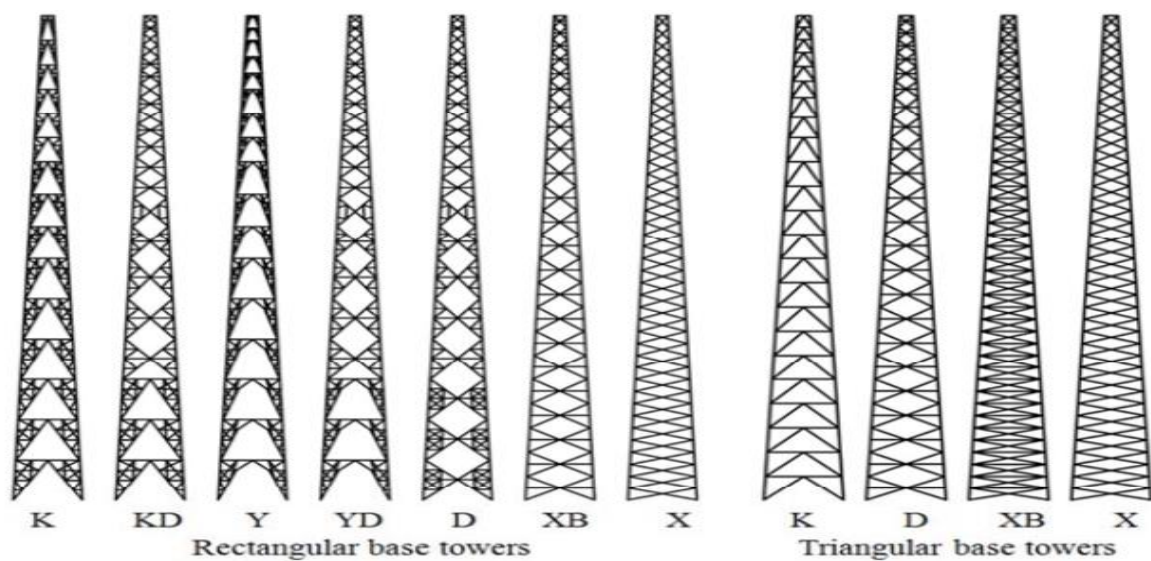


Figure 4: Telecommunication towers with different base and bracing systems

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system for a lateral load on towers using STAAD Pro analysis. The research work will be limited to the analysis of steel towers only [8].

2.4.2 Zero force members

Zero force members are members that have no force, which are provided to increase stability, provide support if the loads changes and to make the truss aesthetically pleasing. These members can be identified by free body diagram and by inspection. The sections usually used as secondary members and they are hot-rolled angles.

Occasionally we encounter truss members that carry no force. There are two conditions that can cause a member to become a so-called zero-force member.

The first condition for a member to be called zero force members is

- When two non-collinear members are connected at a joint,
 - No applied force or support reaction present at that joint,
- Therefore, both members are zero-force members.

The next condition for a member to be called zero force members is

- when three members are connected together at a joint,
- two of the members are collinear,
- No external force or support reaction is acting at that joint in the direction of the non-collinear member,

Therefore, the non-collinear member is a zero-force member.

For instance taking one sub panel sample from any tower geometry as shown in the figure 5, the non collinear members 3, 6, 9, 12 and 13 are zero force members.

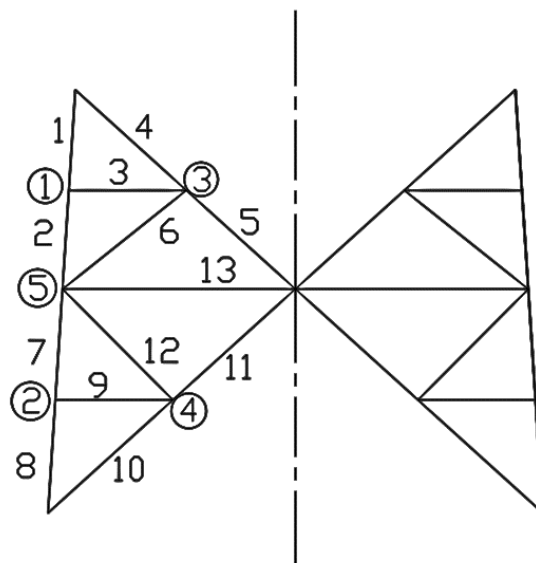


Figure 5: Zero force members

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Zero force members are provided to increase stability of diagonal members and leg members by decreasing the effective length so that members will not fail by buckling. Since, zero force members are short they will never buckle.

2.5 BASIC WIND SPEED

Different locations will have different wind speed distributions. According to weather conditions in Ethiopia it is classified in to five zones such as Frosty Weather, High land, Temperate, Low land, and Desert. EBCS1, 1995, the reference basic wind speed in Ethiopia generally was taken to be 22m/sec is not appropriate for design process. Since the reference basic wind speed should be taken from the wind hazard map which is appropriate for your design location and the manual and commentary of EBCS 1, 1995 should be prepared.

The extreme wind speed analysis at a return period 50 years for Addis Ababa station is 42 m/sec, which is greater than the constant figure, 22 m/sec that is increased by 90.91%, [5].

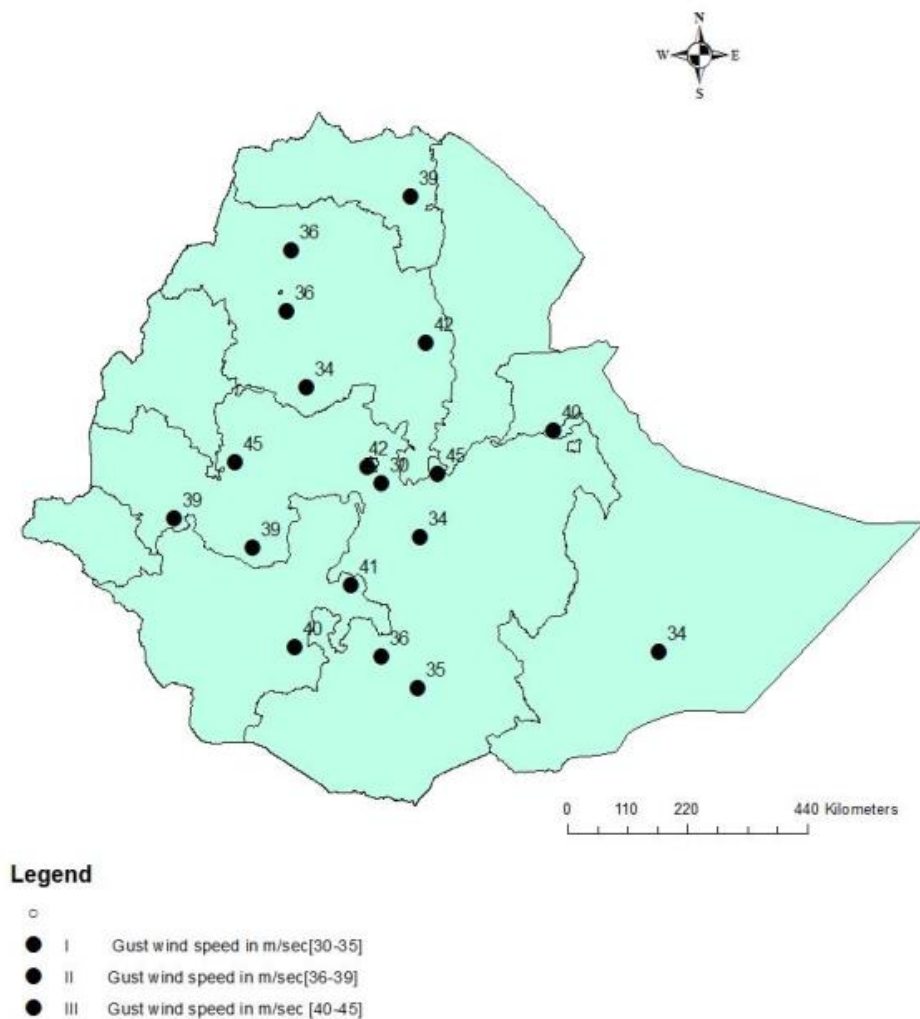


Figure 6: Gust wind speed, at 50 year return period, zonal map for selected stations

2.6 DESIGN PHILOSOPHY

The conventional design of nearly all masonry structures and many wood and steel structures has been accomplished using Allowable Stress Design (ASD) standards.

Allowable Stress Design

Allowable stress design is a method of proportioning structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses contain a factor of safety that may not be exceeded by load effects due to unfactored loads. When using allowable stress design, strengthening is required when the stresses due to nominal loads exceed the specified allowable stresses for the materials of construction, [10].

Compression Member Failures

Compression members fail by two modes of failure: the first one is crushing failure which is caused by yielding of member due to axial compressive force and the second one is buckling failure happens when a very long length compression member is subjected to a critical compressive force which causes elastic instability.



Figure 7: Self-supporting Communication tower failure

So, structural members are proportioned to carry the design loads safely to resist buckling and yielding. Most of communication towers collapse due to the elastic instability of members as a whole and not by failure of material.

The critical stress can be raised by reducing L / r or by using a material with higher modulus of elasticity. Therefore, when members are short it fails by yielding or crushing

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of material hence there are no stability considerations and if member is long it fails by buckling, [13].

Euler's Critical Load

In Euler's curve member with large slenderness ratio buckles at low value of the average compressive stress P_c / A and the curve is valid for long members with higher slenderness ratio otherwise it will suggest an excess of critical load which is actually required for collapse. Failure will only occur by buckling in the elastic range if Euler's critical stress is lower than the yield stress f_y .

The allowable stress used for the comparison can be determined by using the Euler's critical stress for higher slenderness ratio and Rankine's allowable compressive stress for member with smaller slenderness ratio.

$$P_c = \frac{\pi^2 EI}{l^2}$$

Euler's load

$$\sigma_e = \frac{\pi^2 E}{\lambda^2}$$

Critical Stress

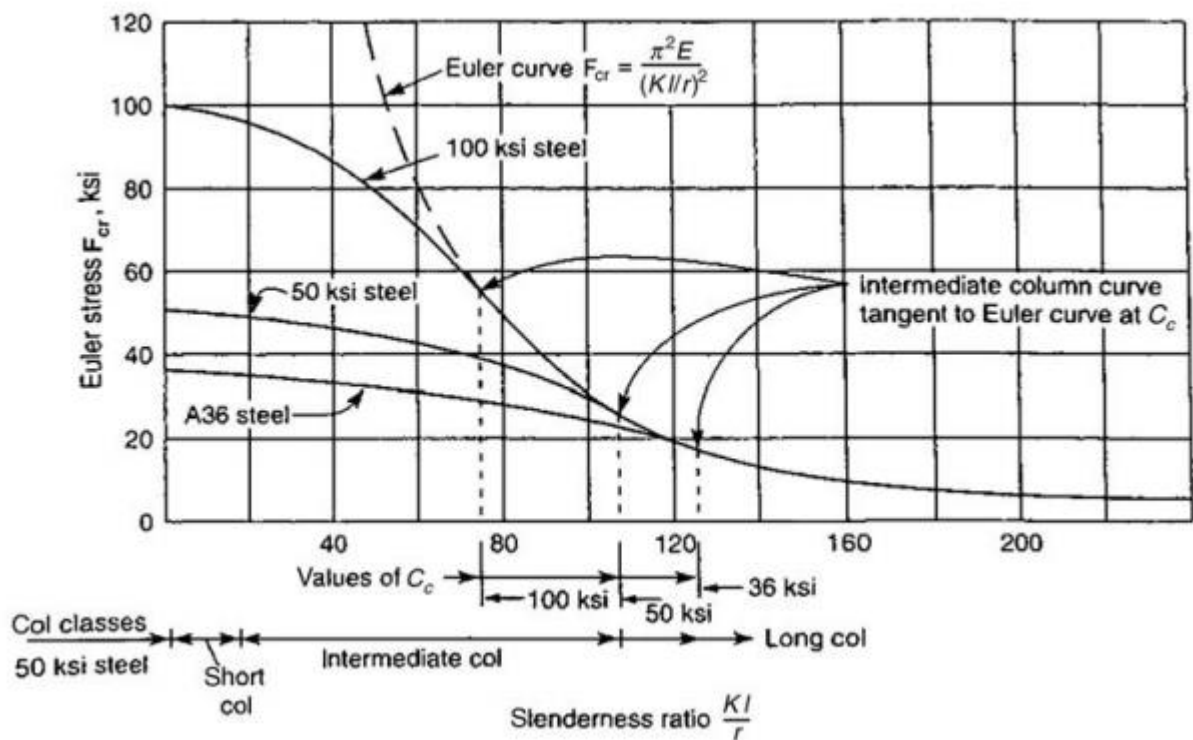


Figure 8: Euler's curve [17],

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From the graph of Figure 8, it is evident that the Euler theory is unsafe for smaller slenderness ratios. It is useful, therefore, to determine the limiting value of slenderness ratio below which the Euler theory should not be applied; this is termed the validity limit.

Merchant Rankines Allowable Compressive Stress (Indian Standard)

The Rankine formula is therefore valid for extreme values of slenderness ratio. It is also found to be fairly accurate for the intermediate values in the range under consideration.

$$\sigma_r = 0.6 \frac{\sigma_y \sigma_e}{(\sigma_y^n + \sigma_e^n)^{\frac{1}{n}}}$$

where $n = 1.4$

$$\sigma_e: \text{Elastic critical stress in compression} = \frac{\pi^2 E}{\lambda^2}$$

$$\lambda: \text{Slenderness ratio} = \frac{L}{r}$$

Therefore, the value of σ_{ac} shall exceed neither of the two values σ_{al}

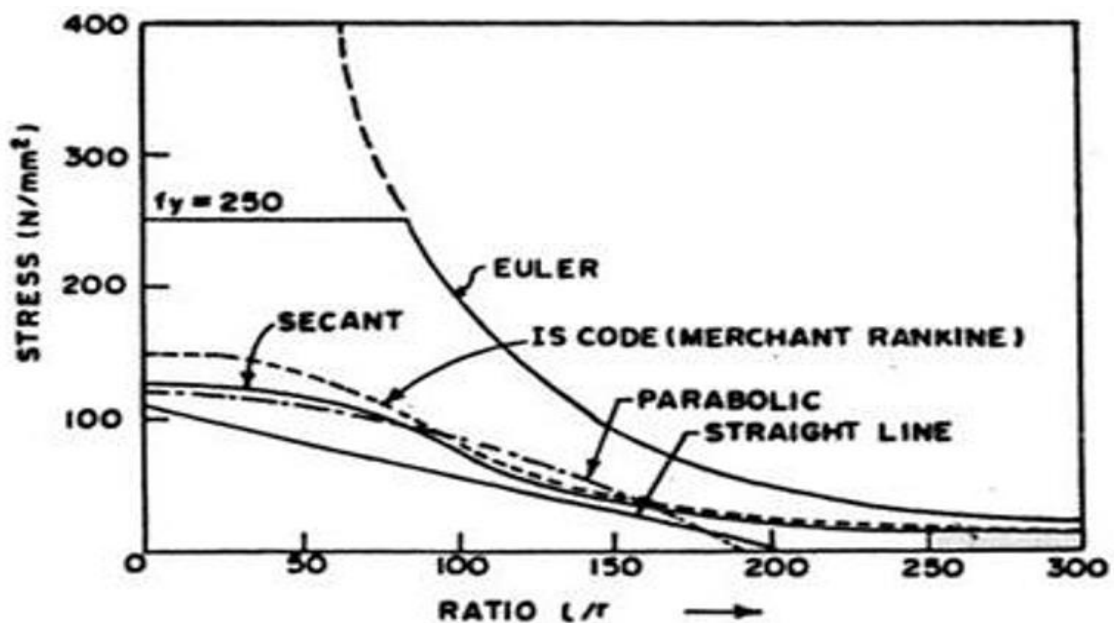


Figure 9: Comparison of various compressive member formula [17]

CHAPTER THREE

SPECIFICATIONS FOR SELF SUPPORTING STEEL LATTICED COMMUNICATION TOWERS

3.1 MATHEMATICAL MODELING

Mathematical modeling of the tower structure that represents height and orientation of members such as leg member, horizontal member, diagonal member and zero force members can be conducted with the help of AutoCAD and MS excel programs.

In designing of heavy duty towers for wind or seismic load, selecting the configurations of the tower in terms of bracing, height and width is the most crucial step.

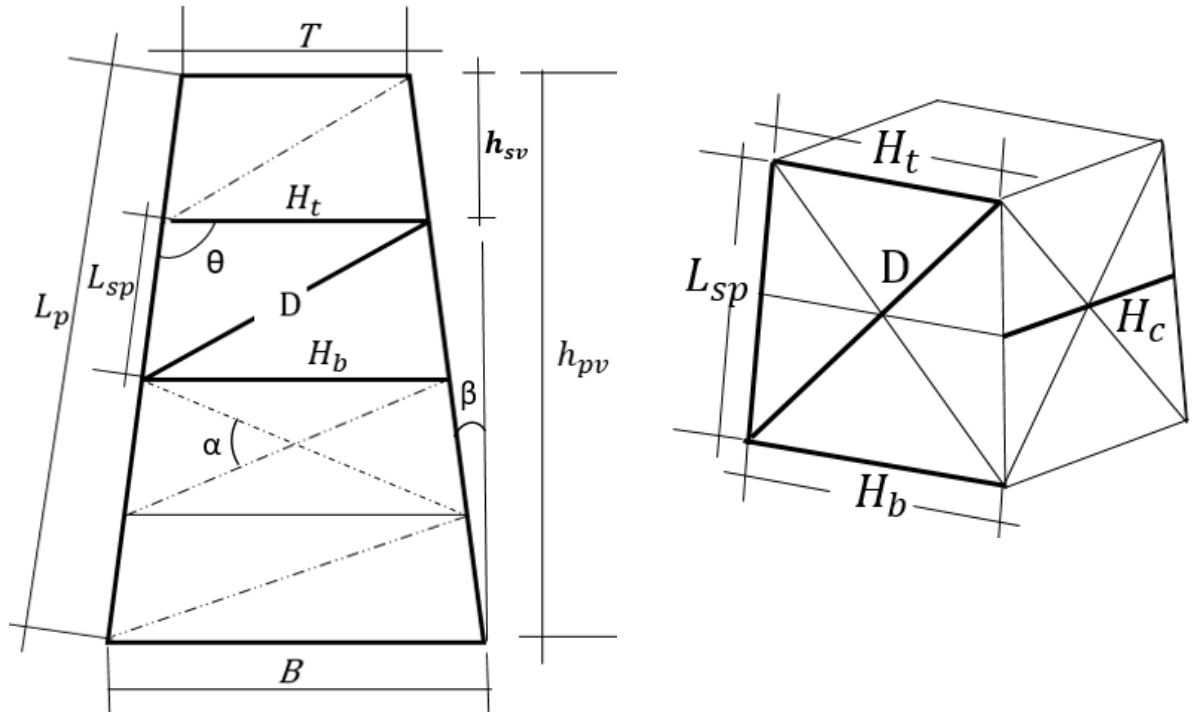


Figure 10: Layout of panel members

The geometrical properties of the tower are to be determined using the following equations.

$$L_p = \sqrt{h_{pv}^2 + 2 \left(\frac{B-T}{2} \right)^2} \quad 3.1$$

$$L_{sp} = \frac{L_p}{n} = \frac{\sqrt{h_{pv}^2 + 2 \left(\frac{B-T}{2} \right)^2}}{n} \quad 3.2$$

SPECIFICATIONS

$$D = \sqrt{H_t^2 + L_{sp}^2 - 2H_tL_{sp}\cos(\theta)} \quad 3.3$$

$$H_T = B - \left(\frac{B-T}{h_{pv}}\right) Z \quad 3.4$$

$$H_c = H_b - \frac{H_b (H_b - H_t)}{H_b + H_t} \quad 3.5$$

$$\theta = \cos^{-1} \left(\frac{\sqrt{h_{pv}^2 + \left(\frac{B-T}{2}\right)^2}}{L_{sp}} \right) + 90^\circ \quad 3.6$$

$$\beta = \theta - 90^\circ \quad 3.7$$

$$3.0^\circ \leq \beta \leq 5.0^\circ$$

$$\alpha = 2 \tan^{-1} \left(\frac{2h_{sv}}{H_t + H_b} \right) \quad 3.8$$

$$50^\circ \leq \alpha \leq 90^\circ \quad \text{for straight part}$$

$$70^\circ \leq \alpha \leq 100^\circ \quad \text{for tapered part}$$

where:

- L_P : Leg Length along corner of panel
- B : Bottom width of panel
- T : Top width of panel
- D : Diagonal length of bracing
- θ : Inclined leg member Slope angle relative to the top horizontal
- β : Inclination angle of tower with respect to vertical
- α : Diagonal bracing cross over angle
- h_{pv} : Vertical height of panel
- h_{sv} : Vertical height of sub panel
- L_{sp} : Leg Length along corner of Subpanel
- H_t : Top width of sub panel
- H_c : Horizontal length at Diagonal Cross over point
- n : Number of similar vertical height sub panels in a panel
- Z : Top joint vertical height of sub panels from the ground under consideration

SPECIFICATIONS

Bracing Type

To investigating the effects of loadings on the communication tower, XX types of bracings with varied arrangement of redundant or zero force members are taken for modeling and design.

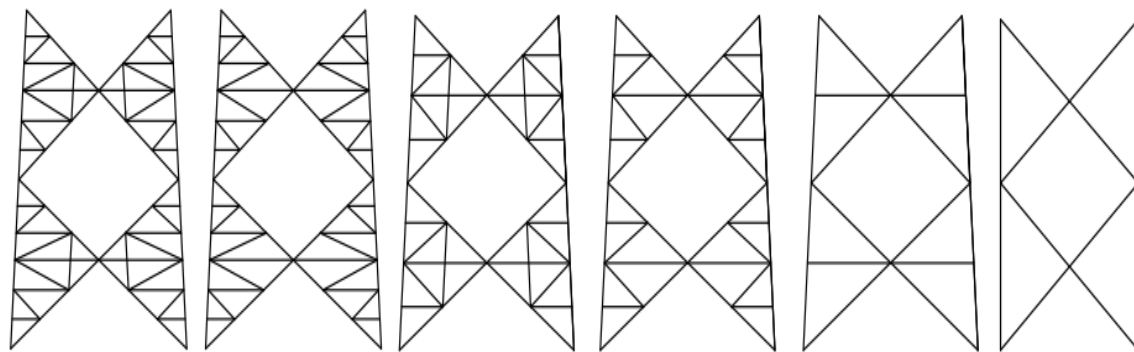


Figure 11: XX bracing types

3.2 WIND LOAD ANALYSIS OF COMMUNICATION TOWER

3.2.1 Basis of wind load analysis

Wind load is the major load on all freestanding lattice towers. Wind load on communication tower has to be calculated and transferred to all panel points to get more realistic effect. As this involves a number of laborious and complex calculations, it is the general practice to consider the equivalent loads.

The projected area is an unknown quantity, but it is required for the wind load calculation. This can be calculated exactly only after the design process is over and actual sections are known. Therefore, it is necessary to assume certain area to arrive at the wind load on the structure. Depending on the spread and size of the structure, 15 to 25% of the gross area is generally taken as the net area. Gross area is the area bounded by the outside perimeter of single face of the tower. The wind load on the tower is assumed to act at selected nodes, first the wind loads on various parts of the tower or the members are calculated. Then the moments about the tower base for all these loads are added and an equivalent load at selected points for that moment is calculated.

An assumed configuration are used in the preliminary design. Based on the preliminary design, approximate member sizes are arrived at for calculating wind load on towers. Recalculation of wind load, if required, is done before arriving at the final designs. A realistic approach is to apply the wind load at each node of the tower and this method is possible in Excel computer programs.

SPECIFICATIONS

The wind load analysis of the self-supporting communication tower is carried out using ASCE 7-05, [10] and ANSI/TIA-222-G, [9] recommendations.

The design wind load shall include the sum of the horizontal design wind forces applied to the structure in the direction of the wind and the design wind forces on appurtenances. All appurtenances, including antennas, mounts and lines, shall be assumed to remain intact and attached to the structure. Loads considered for the wind load design are wind loads on the members of self-supporting latticed telecommunication tower and self-weight of tower including antennas and appurtenance.

3.2.2 Design wind pressure

Determination of the Velocity pressure or design wind pressure Q_z at the middle height of sub panel according to recommendation of TIA 222 G, [9]. The design wind pressure is determined using formula shown below.

$$Q_z = 0.613K_zK_{zt}K_DV^2I \left[\frac{N}{m^2} \right] \quad 3.9$$

where:

K_z : Velocity pressure coefficient

K_{zt} : Topographic factor

K_d : Wind direction probability factor

V : Basic wind speed for the loading condition under investigation, [m/s]

I : Importance factor

Velocity pressure coefficient (K_z), shall be determined based on the exposure category as follows: [9]

$$K_z = 2.01 \left[\frac{z}{z_g} \right]^{\frac{2}{\alpha}} \quad K_{zmin} \leq K_z \leq 2.01 \quad 3.10$$

where:

z = height above ground level at the base of the structure

z_g , α and K_{zmin} are referred from Table below

Table 1: Exposure Category Coefficients

Exposure Category	Z_g [m]	α	K_{zmin}	K_e
B	366	7.0	0.70	0.9
C	274	9.5	0.85	1.0
D	213	11.5	1.03	1.1

SPECIFICATIONS

Since almost all heavy duty towers are constructed in urban and sub urban areas, those tower structures should be designed for exposure category of **B**, Parameters for exposure category used in calculation of the velocity pressure coefficients are given in table above.

$$K_z = 2.01 \times \left[\frac{Z}{366} \right]^{\frac{2}{7}} \quad 0.7 \leq K_z \leq 2.01 \quad \text{rep.3.10}$$

Topographic factor K_{zt} , Wind speed-up effect shall be included for the determination of design wind loads by using the factor called *Topographic factor*.

$$K_{zt} = \left[1 + \frac{K_e K_t}{K_h} \right]^2 \quad 3.11$$

where:

K_h : height reduction factor given by the following equation: $K_h = e^{\left[\frac{f \cdot Z}{H} \right]}$

e : natural logarithmic base = 2.718

K_e : terrain constant

K_t : topographic constant

f : height attenuation factor

Z : height above ground level at the base of the structure

H : height of crest above surrounding terrain

K_{zt} : 1.0 for topographic category 1.

Table 2: Topographic Category Coefficients

Topographic Category	K_t	F
2	0.43	1.25
3	0.53	2.00
4	0.72	1.50

Our tower structures are designed for Topographic category of **1**, $K_{zt} = 1.0$

Wind Direction Probability Factor K_D

Table 3: Wind Direction Probability Factors

Structure Type	Wind direction probability factor, K_d
Latticed structure with triangular, square or rectangular cross section	0.85
Tubular pole structure ,latticed structure with other cross section appurtenances	0.95

Since, the tower structure is Square cross sectional latticed structure **$K_D = 0.85$**

SPECIFICATIONS

Importance Factor I, Since our tower structure is heavy duty communication structure it is classified under site class (Risk category) IV because due to height, use or location represent a high hazard to human life and/or damage to property in the event of failure and/or used primarily for essential communications, [10]. Therefore, its importance factor for wind is **I=1.15**.

Table 4: Importance Factor

Structure class	Wind load without ice	Wind load with ice	Ice thickness	Earthquake
I	0.85	N/A	N/A	N/A
II	1.00	1.00	1.00	1.00
III	1.15	1.00	1.25	1.50
Note ice and earthquake load do not apply to class one structure				

3.2.3 Design wind load

Preliminary design of tower for wind load should be carried out so as to have efficient steel cross section capacity which can be used for the analysis purpose.

The design wind load, F_w , shall be determined in accordance with the following:

$$F_w = F_S + F_A \quad 3.12$$

where:

F_S : Horizontal design wind force on the structure

F_A : design wind force on antennas and appurtenances

The design wind forces, $F_S + F_A$, need not exceed the wind force calculated for a structure using a solidity ratio of 1.0 (solid-faced) plus the wind load on externally mounted appurtenances that are outside the normal projected area of the structure in the direction of the wind.

The design wind force, F_S , applied to each section of a structure shall be determined in accordance with the following:

$$F_S = Q_Z G_h (EPA)_S \quad 3.13$$

$$EPA_S = C_F [D_F \sum A_F + D_r \sum (A_r R_r)] \quad 3.14$$

$$C_F = 4\epsilon^2 - 5.9\epsilon + 4 \text{ (Rectangular cross sections)} \quad 3.15$$

$$C_F = 3.4\epsilon^2 - 4.7\epsilon + 3.4 \text{ (Triangular cross sections)}$$

SPECIFICATIONS

$$\epsilon = \frac{(A_F + A_R)}{A_G} \quad 3.16$$

where:

- ϵ : Solidity ratio
- $(EPA)_S$: Effective projected area of the structure
- Q_Z : Velocity pressure
- G_h : Gust effect factor
- A_F : Projected area of flat structural components in one face of the section
- A_R : Projected area of round structural components in one face of the section including the projected area of ice on flat and round structural components in one face for loading combinations that include ice
- A_G : Gross area of one face as if the face were solid
- D_F : Wind direction factor for flat structural components
- D_r : Wind direction factor for round structural components
- R_R : Reduction factor for a round element

Table 5: Wind direction factors

Tower Cross section	Square		Triangular		
	Wind direction	Normal	45°	Normal	60° ± 90°
	Df	1.00	1+0.75ε (1.2 max)	1.00	0.80 0.85
	Dr	1.00	1+0.75ε (1.2 max)	1.00	1.00 1.00
Wind direction measured from a line normal to the face of the structure					

For wind in diagonal direction, the parallel wind load on structure is multiplied by wind direction factor D_f of 1.2.

SPECIFICATIONS

Wind Directions

To account for the direction of wind loading to the face normal to loading in finite element analysis software, the axis in three dimensional modeling of the trussed tower should be modeled as Parallel and Diagonal as shown below

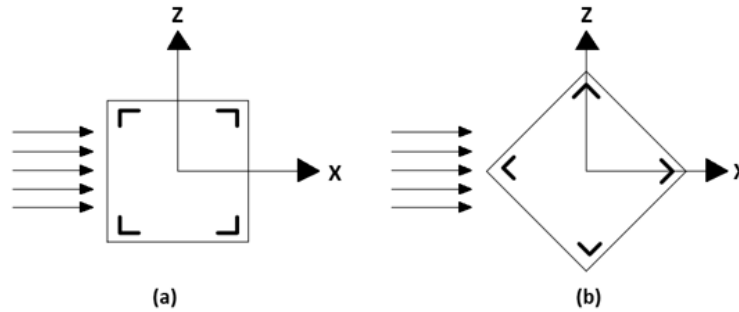


Figure 12: Parallel (a), Diagonal (b) Wind loads direction

The design wind force on appurtenances, F_A shall be determined from the equation:

$$F_A = Q_Z G_h (EPA)_A \quad 3.17$$

where:

q_z : Velocity pressure at the centerline height of the appurtenance

G_h : Gust effect factor

$(EPA)_A$: Effective projected area of the appurtenance

For self-supporting latticed structures, the gust effect factor shall be 1.00 for structures 183 m or greater in height. For structures 137 m or less in height, the gust effect factor shall be 0.85. The gust effect factor shall be linearly interpolated for structure heights between 137 m and 183 m.

$$G_h = 0.85 + 0.15 \left[\frac{h}{45.7} - 3.0 \right] \quad 0.85 \leq G_h \leq 1.00 \quad 3.18$$

where:

h : Height of structure

Therefore, the gust effect factor is $G_h=0.85$

In the absence of more accurate data specifying effective projected area values for each critical wind direction, the effective projected area, $(EPA)_A$, of an appurtenance shall be determined from the equation:

$$(EPA)_A = K_a [(EPA)_N \cos^2 \theta + (EPA)_T \sin^2 \theta] \quad 3.19$$

where:

K_a : 1.0 for round appurtenances

SPECIFICATIONS

- K_a : 1.0 may be conservatively used for any appurtenance, and the value of K_a is constant for all wind directions
- θ : Relative angle between the azimuths associated with the normal face of the appurtenance and the wind direction
- $(EPA)_N$: Effective projected area associated with the windward face normal to the azimuth of the appurtenance.
- $(EPA)_T$: Effective projected area associated with the windward side face of the appurtenance.

The larger value of $(EPA)_N$ or $(EPA)_T$ may be conservatively used for $(EPA)_A$ for all wind directions.

When wind is diagonal to the tower, the relative angle between the azimuth associated with the normal face of the appurtenance and the wind direction so $\theta=00$ and if wind is parallel to the tower, the relative angle between the azimuth associated with the normal face of the appurtenance and the wind direction. So $\theta=45^\circ$ assuming that antenna are arranged diagonally.

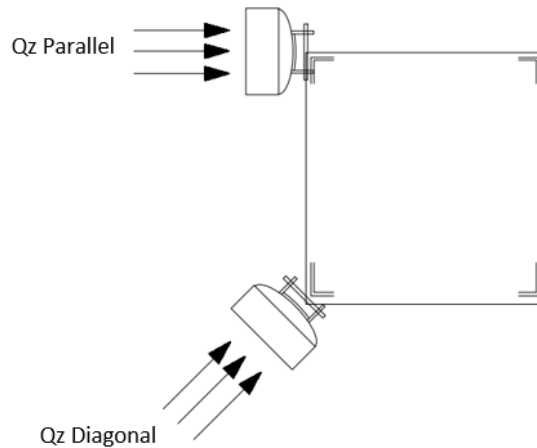


Figure 13: Wind direction top view

SPECIFICATIONS

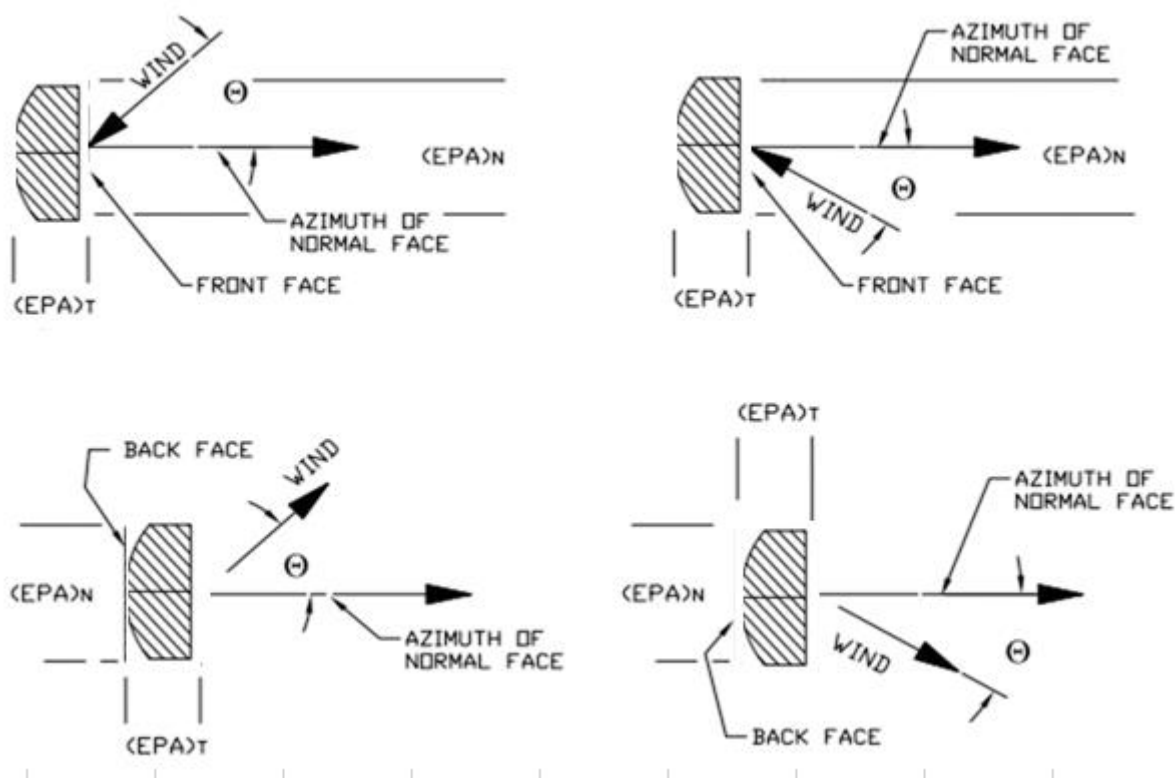


Figure 14: Wind force on appurtenance, [9]

3.2.4 Self-weight of the structure sub panels antennas and appurtenances

To calculate the self-weight of the leg, diagonal, horizontal and zero force members, the unit weight of the member is multiplied by the length of the member for leg, diagonal, horizontal and zero force members respectively.

$$SW [kN] = L [m] \times q [kN/m] \quad 3.20$$

3.2.5 Wind and self weight loads at the joint

The design wind load on sub panel and self-weight of sub panel members and antennas are distributed to each joint adjacent to sub panel accordingly, [2]

- Load at each joint is increased by 15% to account for gussets, bolts and washers
- Wind load on leg, diagonal and zero force members are distributed to number of joint above and below the sub panel.
- Wind load on horizontal member are distributed to joints in which the horizontal member is existed.
- Self-weight of leg, diagonal and zero force members are distributed to number of joint above and below the sub panel.

SPECIFICATIONS

- Self-weight of horizontal member are distributed to joints in which the horizontal member is existed.

Therefore, wind load and self-weight at a joint is calculated as shown below:

$$\text{Wind load at joint} = \frac{F_{LDZA}}{n \times j} + \frac{F_{HJ}}{j} + \frac{F_{LDZB}}{n \times j} \quad 3.21$$

Similarly self weight at joint is computed as follow

$$\text{Self Weight at joint} = \frac{SW_{LDZA}}{n \times j} + \frac{SW_{HJ}}{j} + \frac{SW_{LDZB}}{n \times j} \quad 3.22$$

where:

F_{LDZA}, SW_{LDZA} : Wind load and Self weight on Leg, Diagonal and Zero force member from sub panel, above joint, respectively

F_{HJ}, SW_{HJ} : Wind load and Self weight on horizontal member at the joint, respectively

F_{LDZB}, SW_{LDZB} : Wind load and Self weight on leg, diagonal and zero force member from sub panel above joint, respectively

$n=2, j=4$ constants for square cross section

3.2.6 Axial load due to parallel wind on structure members

Axial load due to horizontal wind loads on joints is calculated by determining the moment at the mid height of sub panels.

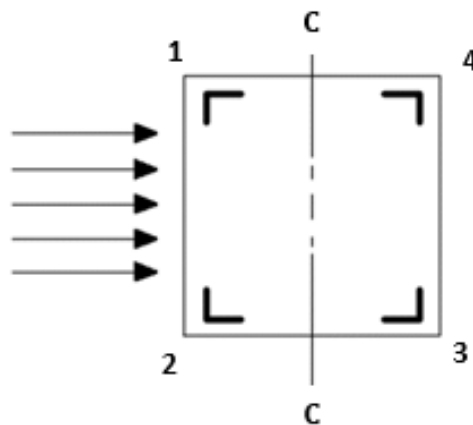


Figure 15: Top view of parallel wind load

SPECIFICATIONS

To determine the moment at mid height of the critical panel two dimensional modeling of horizontal cumulative wind load is calculated at the joints above mid height of the critical panel side face.

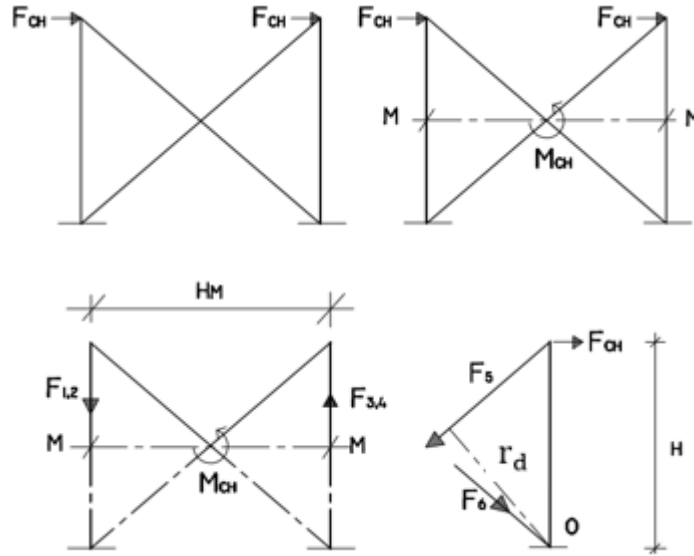


Figure 16: Leg, diagonal axial load and moment at mid height of straight tower part

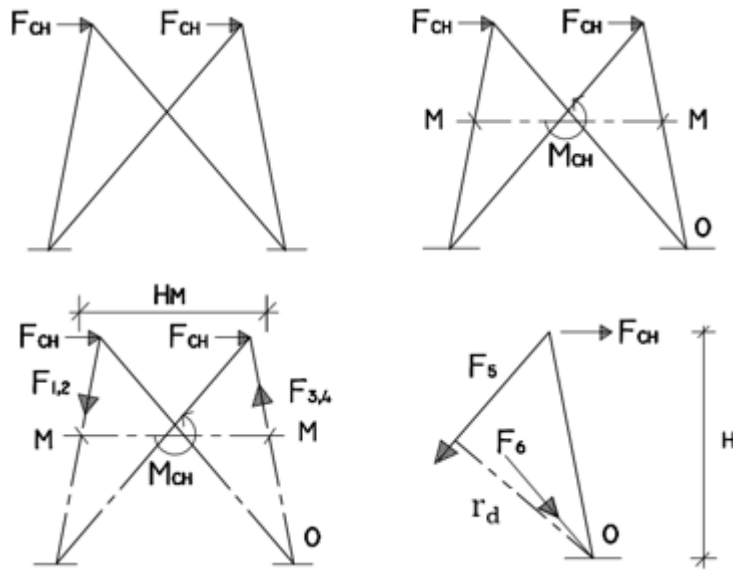


Figure 17: Leg, diagonal axial load and moment at mid height of Tapered tower part

Tensile axial load on wind ward leg 1 and leg 2, compressive axial load on the leeward leg 3 and 4, and compressive or tensile axial load on diagonal bracing member acts due to the parallel horizontal wind load. But the zero force members do not carry axial load they just provided for stability so they don't participate in load distribution coming from joints.

$$\text{Compressive force on wind windward leg, } F_4 = \frac{M_{CH}}{H_m} \quad 3.23$$

SPECIFICATIONS

$$\text{Tensile force on diagonal bracing, } F_5 = \frac{F_{CH} \times H}{r_d} \quad 3.24$$

where:

F_{CH} : Cumulative horizontal wind load at the top of critical sub panel under consideration [N]

M_{CH} : Total moment due to horizontal wind load on joints above at the centroidal height of critical sub panel under consideration, [Nm]

H_m : Lever arm of tensile and compression member at the centroidal height of critical sub panel under consideration, [m]

H : Vertical height of critical sub panel under consideration, [m]

r_d : Lever arm of Diagonal tensile member to the toe of sub panel on the leeward side, [m]

3.2.7 Axial load due to diagonal wind on structure members

For wind in diagonal direction the values of wind load on joints are multiplied by wind direction factor of 1.2 to consider the area of angle member which is in inclined direction to wind load.

The diagonal horizontal wind load on antenna is accounted by using effective projected area of antennas.

$$(EPA)_A = [(EPA)_N \cos^2 \theta + (EPA)_T \sin^2 \theta] \quad 3.25$$

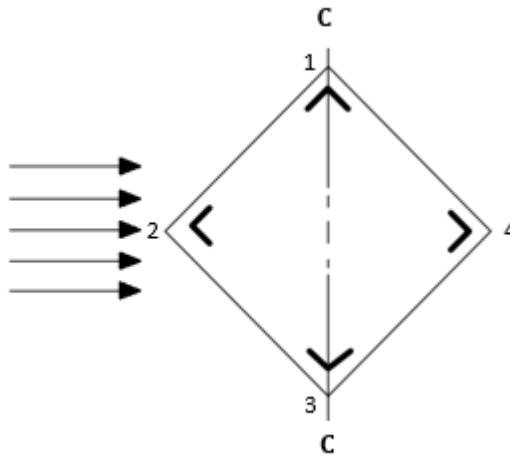


Figure 18: Top view of diagonal wind load

SPECIFICATIONS

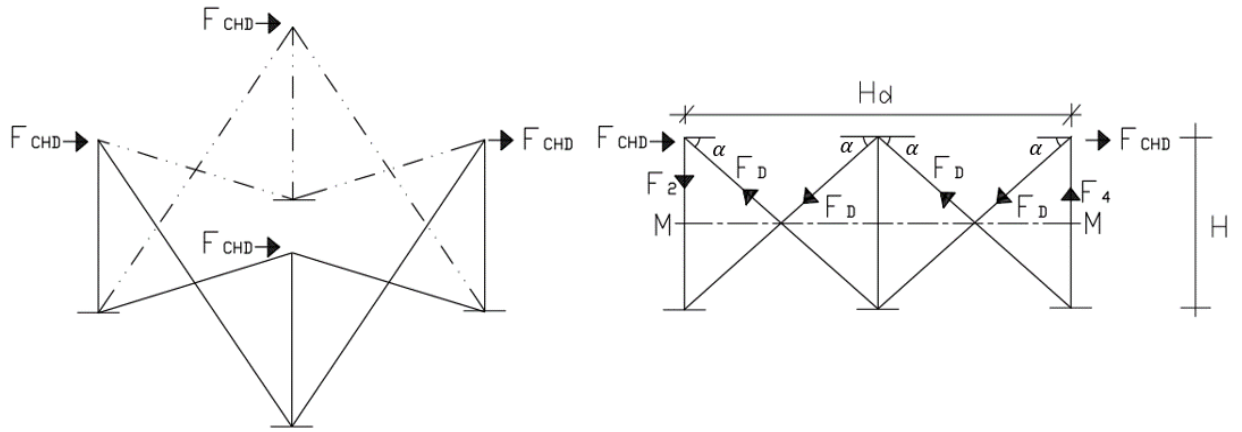


Figure 19: leg, diagonal axial load of straight tower part

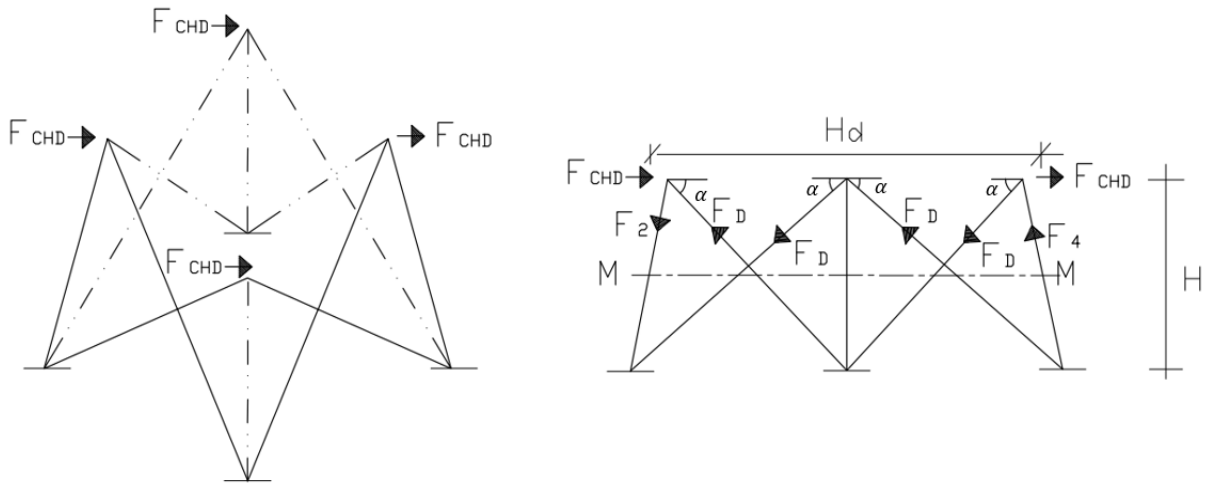


Figure 20: Leg, diagonal axial load of Tapered tower part

Tensile axial load on front leg 2, compressive axial load on the leeward leg 4, and compressive or tensile axial load on diagonal bracing member acts due to the parallel horizontal wind load. But the zero force members do not carry axial load they just provided for stability so they don't participate in load distribution coming from joints.

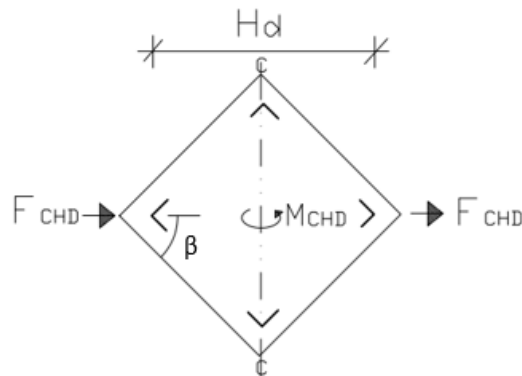


Figure 21: Moment at centroid of tapered tower part due to diagonal wind

SPECIFICATIONS

Compressive force on windward leg, F_4 [N]

$$= \frac{\text{Moment at the mid height of critical Sub Panel}}{\text{Lever arm of tensile and comp Leg}}$$

$$\text{Compressive force on wind windward leg, } F_4 \text{ [N]} = \frac{M_{CHD}}{H_m} \quad 3.26$$

Tensile force on diagonal bracing, F_D [N]

$$= \frac{\text{Total cumulative wind load at the top of sub critical panel}}{8 \sin(\alpha) \times \cos(\beta)}$$

$$\text{Tensile force on diagonal bracing, } F_D \text{ [N]} = \frac{4F_{CHD}}{8 \sin \alpha \times \cos \beta} \quad 3.27$$

where:

F_{CHD} : Cumulative horizontal diagonal wind load at the top of critical sub panel under consideration [Nm]

M_{CHD} : Total moment due to horizontal diagonal wind load on joints above at the centroidal height of critical sub panel under consideration.[Nm]

H_m : Lever arm of tensile and compression member at the centroidal height of critical sub panel under consideration [m]

3.2.8 Maximum compressive axial load on the tower leg

It is assumed that horizontal wind load is resisted by the legs and diagonal bracing of tower, and also the self-weight of tower is resisted by legs of tower and two percent of weight is resisted by the diagonal bracings but the diagonal bracing is not assigned to carry the self-weight of tower.

Since, leg on leeward side of tower is subjected axial compressive load due to the horizontal wind and self-weight of tower, the maximum compressive axial load will be applied on the sub critical panels of the leg. But the wind ward side of the tower leg is subjected to tensile axial load due to the horizontal wind load and axial compressive axial load due to the self-weight of tower.

Basic Allowable stress design (ASCE ASD) load combinations are shown below, [10]:

$$0.6D + W + H \quad 3.28$$

$$0.6D + 0.7E + H \quad 3.29$$

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3.3 SEISMIC LOAD ANALYSIS OF COMMUNICATION TOWER

Antennas supporting structures require special considerations of their response characteristics in regions of high seismicity. The provisions of these Standard specification provide design criteria to insure sufficient strength and stability to resist the effects of seismic ground motions for self-supporting structures.

The earthquake load analysis of the self-supporting communication tower is also carried out using ASCE 7-05, [10] codes and ANSI/TIA-222-G, [9] recommendation's.

Seismic analysis of steel towers can be done by three methods namely Equivalent Lateral analysis, Equivalent Modal analysis, Modal (Response spectrum) analysis and Time history analysis, from the above methods Equivalent modal analysis (Response spectrum) and Modal analysis may be used for seismic analysis.

Modal calculations of structure is carried before the response spectrum so as to determine the natural frequency for each mode shapes or deformation patterns.

Spectral acceleration parameters

Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with ASCE 7-05. Site Classes A and B shall not be assigned to a site if there is more than 10ft (3m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

Assuming the tower is designed for site class **B** the Design Spectral Acceleration Parameters determined using equations shown below.

$$S_{MS} = F_a S_S \quad S_{D S} = \frac{2}{3} S_{MS} \quad S_{DS} = \frac{2}{3} F_a S_S \quad 3.30$$

$$S_{M1} = F_V S_1 \quad S_{D1} = \frac{2}{3} S_{M1} \quad S_{D1} = \frac{2}{3} F_V S_1 \quad 3.31$$

where:

S_S : mapped MCE_R spectral response acceleration parameter at short periods

S_1 : mapped MCE_R spectral response acceleration parameter at a period of 1 second

F_V : long-period site coefficient (at 1.0 s-period)

F_a : short-period site coefficient (at 0.2s-period)

S_{MS} : the MCE_R spectral response acceleration parameter at short periods adjusted for site class effects.

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S_{MI} : the MCE_R spectral response acceleration parameter at period of 1 second adjusted for site class effects.

S_{DS} : design spectral response acceleration parameter at short periods

S_{D1} : design spectral response acceleration parameter at a period of 1 second.

According to the new map, the most seismic area of the country is concentrated near and around the Afar region characterized by a PGA of 0.16g to 0.24g. This alone entails an increase in seismic force demand of 60 to 140 % in this region without including site effect. The capital, Addis Ababa, belongs to the second most seismic zone with PGA in the range of 0.1g to 0.16g. This again implies an increase of 100 to 220 % in seismic hazard level with an average increase of 160 %. Several rapidly growing towns including, Mekele, Dese, Debre Berhan, Ziway, Hawassa, Arbaminch and Dire Dawa belong to this seismic zone, while Semera, the current capital of the Afar Region, is in the heart of the most seismic zone. Considering 0.1g as the lowest-estimate PGA of the region, to which Addis Ababa belongs short-period spectral acceleration, S_s , of 0.45g and the one-second spectral acceleration, S_1 , as 0.18g, [11].

The short period spectral acceleration, S_s and one second spectral acceleration, S_1 for Addis Ababa also tabulated in Unified Facilities Criteria (UFC), UFC 3-310-01, [12] structural load data as 0.553g and, 0.221g respectively which is for Peek ground acceleration PGA of 0.1474g which is much more than the value assigned to Addis Ababa in EBCS EN-1998-1:2014, [14] .

Design Response Spectrum Curve

The design response spectrum curve shall be developed based on the following equation of design spectral response accelerations for different parts of the curve.

The design short period spectral acceleration S_{DS} :

$$S_{DS} = \frac{2}{3} F_a S_s \quad \text{rep.3.30}$$

The design one second spectral acceleration S_{D1} :

$$S_{D1} = \frac{2}{3} F_v S_1 \quad \text{rep.3.31}$$

$$\text{For } T_m < 4.0 \text{ sec} \quad 3.32$$

- For $0 \leq T_m \leq T_0$

$$S_{am} = S_{DS} \left(0.4 + 0.6 \frac{T_m}{T_0} \right)$$

- For $T_0 \leq T_m \leq T_s$

$$S_{am} = S_{DS}$$

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- $T_s \leq T_m \leq T$

$$S_{am} = \frac{S_{D1}}{T_m}$$

For $T_m \geq 4.0$ sec

3.33

- $S_{am} = \frac{4 S_{D1}}{T_m^2}$

where:

S_{am} : Design spectral response acceleration at period T_m for the mode under investigation

f_m : Frequency of structure for the mode under consideration

S_{DS} : Design spectral response acceleration at short period

S_{D1} : Design spectral response acceleration at 1 second

$$T_0 = 0.2 S_{D1}/S_{DS}$$

$$T_s = S_{D1}/S_{DS}$$

$$T_m = 1/f_m$$

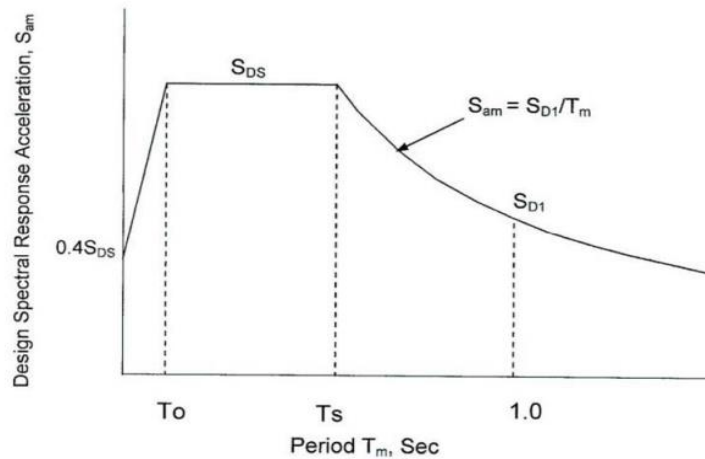


Figure 22: Response spectrum curve

Table 6: Site coefficient F_v

Mapped Risk Targeted Maximum Consider Earthquake (MCE _R) Spectral Response Acceleration Parameter at 1-s period					
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 ASCE 7-05				

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Table 7: Site coefficient F_a

Mapped Risk Targeted Maximum Consider Earthquake (MCE _R) Spectral Response Acceleration Parameter at short period					
Site Class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 ASCE 7-05				

Importance Factor, I_e : a factor that account for the degree of hazard to human life and damage to property according to ASCE7 -05 Table 11.5-1, [10] the importance factor for earthquake load is 1.5.

Response Modification Factor, R : as per the ASCE 7-05 Table 15.4-2, [10] seismic coefficient for non-building structure not similar to building for telecommunication tower steel is 3.

Fundamental Period of the Tower

The fundamental period of the structure for the first mode obtained from finite element analysis should be equal to equation shown below for verification of the software,[9].

$$f_1 = \frac{K_s W_a}{h^2} \sqrt{\frac{W_1}{W_1 + W_2}} \quad 3.34$$

$$W_1 = W \left[\left(\frac{W_a}{W_o} \right)^2 + 0.15 \right] \quad 3.35$$

where:

K_s : 1500 for h and W_a , [m]

W : total weight of the structure including appurtenances

W_2 : weight of structure and appurtenances within top 5% of the structure height

W_a : average face width of structure, [m]

W_o : face width at base of structure, [m]

h : height of structure, [m]

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3.3.1 Equivalent modal analysis

Seismic forces at the level of the structure

$$F_{az} = \frac{S_{az} W_Z I}{R} \quad 3.36$$

$$S_{az} = \frac{a(S_A)^2 + b(S_{DS})^2}{[(S_A)^2 + c(S_{DS})^2]^{\frac{1}{2}}} \quad 3.37$$

where:

$$a = 1.89\left(\frac{z}{h}\right)^2 \quad 3.38$$

$$b = 26.4\left(\frac{z}{h}\right)^5 - 44.5\left(\frac{z}{h}\right)^4 + 27\left(\frac{z}{h}\right)^3 - 8.14\left(\frac{z}{h}\right)^2 + 1.22\left(\frac{z}{h}\right) \quad 3.39$$

$$c = 4.48\left(\frac{z}{h}\right)^5 - 7.16\left(\frac{z}{h}\right)^4 + 6.41\left(\frac{z}{h}\right)^3 - 3.23\left(\frac{z}{h}\right)^2 + 0.64\left(\frac{z}{h}\right) \quad 3.40$$

$$S_A = S_{D1} f_1 \quad f_1 \leq \frac{F_a S_s}{F_v S_1} \quad 3.41$$

$$S_A = S_{DS} \quad f_1 > \frac{F_a S_s}{F_v S_1}$$

F_{az} : Seismic Forces

W_z : portion of total gravity load assigned to the level z under consideration

S_{az} : acceleration coefficient

S_{DS} : Design spectral response acceleration at short period

S_{D1} : Design spectral response acceleration at one second

F_a : Acceleration-Based Site Coefficient

F_v : Velocity-based Site Coefficient

S_s : spectral acceleration short period

S_1 : spectral acceleration at one second

R : response modification factor

I : Importance factor

Z : level under consideration

a, b, c are coefficients function of z/h

3.3.2 Response spectrum (Modal) analysis

Response spectrum analysis is conducted to determine the natural modes of vibration for the structure. Sufficient number of modes have to be included to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model, [9].

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3.3.2.1 Base shear contributed by each mode

The base shear (V_{sm}) contributed by each mode shall be determined from the following equations:

$$V_{sm} = \frac{S_{am} W_m I}{R} \quad 3.42$$

$$W_m = \frac{(\sum_{i=1}^n W_m)^2}{\sum_{i=1}^n W_i \Phi_{im}^2} \quad 3.43$$

where:

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

W_m : Effective modal gravity load

I : Importance factor from

R : Response modification coefficient equal to 3.0 for latticed self-supporting structures,

n : Number designating the uppermost level of the structure with regard to the distribution of gravity loads

i : Number designating the level of the structure starting from the base to the uppermost level

m : Subscript denoting quantities in the m th mode

W_i : Portion of total gravity load (W) assigned to level i

Φ_{im} : Displacement amplitude at the i th level of the structure when vibrating in its m th mode

3.3.2.2 Seismic forces contributed by each mode

The seismic forces (F_{zm}) at each level of the structure (z) for each mode (m) shall be determined in accordance with the following equation, [9]:

$$F_{zm} = C_{zm} V_{sm} \quad 3.44$$

$$C_{zm} = \frac{W_z \Phi_{zm}}{\sum_{i=1}^n W_i \Phi_{im}} \quad 3.45$$

where:

z : Number designating the level under consideration

C_{zm} : Seismic force distribution factor for the m^{th} mode

V_{sm} : Portion of the base shear contributed by the m th mode

W_z : Portion of total gravity load (W) assigned to level z

W_i : Portion of total gravity load (W) assigned to level i

zm : Displacement amplitude at level z when vibrating in the m^{th} mode

im : Displacement amplitude at level i when vibrating in the m^{th} mode

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3.4 DESIGN OF SELF SUPPORTING COMMUNICATION TOWER

3.4.1 Basics of design

Designs shall be made according to the provisions for American Institute Of Steel Construction Allowable Strength Design (AISC ASD) ANSI/AISC 360, specification for structural steel building [19], ASCE/SEI 7, minimum design load for building and other structure [10], ASCE/SEI 10, Design of Latticed Steel Transmission Structures [20] and ANSI/TIA-222-G-2005, Structural Standard for Antenna Supporting Structures and Antennas [9].

Allowable Strength Design (ASD)

Design according to the provisions for allowable strength design (AISC ASD) satisfies the requirements of this Specification when the allowable strength of each structural component equals or exceeds the required strength determined on the basis of the AISC ASD load combinations.

Design shall be performed in accordance with Equation [3.46]:

$$R_a \leq R_n / \Omega \quad 3.46$$

where:

R_a : Required strength using ASD load combinations

R_n : Nominal strength

Ω : Safety factor (AISC ASD)

R_n / Ω : Allowable strength

Required Strength

The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated in Minimum Design Loads for Buildings and Other Structures ASCE/SEI 7-05, [10]. For design purposes, the nominal loads shall be taken as the loads stipulated by the applicable code.

3.4.2 Member and joint properties of equal angle member

3.4.2.1 Classification of sections for local buckling

For compression, sections are classified as nonslender element or slender-element sections. For a non-slender element section, the width-to-thickness ratios of its compression elements shall not exceed λ_r from Table B4.1a of AISC 360-10, [19] if the width-to-thickness ratio of any compression element exceeds λ_r , the section is a slender-element section.

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Unstiffened elements

For unstiffened elements like Equal angle member supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

- (a) For legs of angles and flanges of channels and zees, the width, b , is the full nominal dimension.
- (b) For plates, the width, b , is the distance from the free edge to the first row of fasteners or line of welds.

Gross area and net area

The gross area, A_g , of a member is the total cross-sectional area.

The net area, A_n , of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as 1/16 in. (2mm) greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal, zigzag line or staggered bolt hole, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in this section, of all holes in the chain, and adding, for each gage space in the chain, the quantity $S^2/4g$,

$$A_n = A_g - \sum dt + \sum_{i=1}^n \frac{S^2}{4g} t \quad 3.47$$

where:

S : longitudinal center-to-center spacing (pitch) of any two consecutive holes, [mm]

g : transverse center-to-center spacing (gage) between fastener gage lines, [mm]

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness

Effective net area

The gross area A_g and net area A_n , of tension members shall be determined in accordance with the provision. The effective net area of tension members shall be determined as follows:

$$A_e = A_n U \quad 3.48$$

$$A_n = A_g - \sum_{i=1}^n dt \quad 3.49$$

$$U = 1 - \frac{\bar{x}}{l} \leq 0.9 \quad 3.50$$

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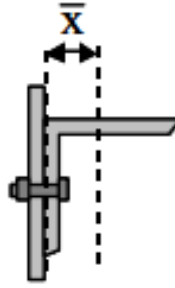


Figure 23: Shear lag parameter

where

U : The shear lag factor

$\bar{x}$: The distance from the centroid of the area connected to the plane of connection

l : The length of the connection

n : Number of bolt in transverse direction

For open cross sections such as single and double angles, the shear lag factor, U , need not be less than the ratio of the gross area of the connected element(s) to the member gross area. For bolted splice plates $A_e = An \leq 0.85A_g$

3.4.2.2 Type of bolted joints

Snug tightened joint

It is firm contact as the condition that exists on a faying surface when the plies are solidly seated against each other. Note that, this does not mean the plies must be in continuous contact. It only means that further tightening of a bolt should not loosen an adjacent bolt in this type of connection Washers are not required.

The snug-tightening is the starting condition for turn-of-nut, calibrated wrench, twist-off type tension-control bolt, and direct tension indicator washer pretensioning,[20].

Pretensioned joint

A pretensioned joint is a joint that transmits shear and/ or tensile loads in which the bolts have been installed to provide a pretension in the installed bolt. Bolts can be pretensioned using turn-of-nut, calibrated wrench, twist off-type tension-control bolt or direct-tension-indicating washer methods. They are preferable in Joints that are subject to significant load reversal, [20].

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Table 8: Minimum Bolt Pretension [kN]

Minimum Bolt Pretension, kN ^(a)	
Bolt Size, mm	A325M Bolt
M16	91
M20	142
M22	176
M24	205
M27	267
M30	326
M36	475

^(a)Equal to 0.70 times the minimum tensile strength of bolts, rounded off to nearest kN, as specified in ASTM F3125/F3125M for Grade A325M

Nut Rotation from Snug –Tight Condition for Turn of Nut Pretension

In both snug-tightened joints and pretensioned joints, the ultimate strength is dependent upon shear transmitted by the bolts and bearing of the bolts against the connected material.

Table 9: Nut rotation from snug –tight condition

Bolt Length	Nut Rotation From Snug –Tight Condition For Turn Of Nut Pretension
Not more than 4de	$\frac{1}{3}$ turn
More than 4de but not more than 8de	$\frac{1}{2}$ turn
More than 8de but not more than 12de	$\frac{2}{3}$ turn
- Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned. For all required nut rotations, the tolerance is plus 60 degrees (6 turn) and minus 30 degrees. - Applicable only to joints in which all material within the grip is steel	

Slip-critical connections

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections. When slip-critical bolts pass through fillers, all surfaces subject to slip shall be prepared to achieve design slip resistance.

For standard hole size the single bolt available slip resistance for the limit state of slip shall be determined as follows,[20]:

$$R_n = \mu D_u h_f T_b n_s \quad 3.51$$

$$\Omega = 1.50 \text{ (ASD)}$$

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where:

D_u : 1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension.

T_b : minimum fastener tension

h_f : Factor for fillers, determined as follows:

For one filler between connected parts $h_f = 1.0$

For two or more fillers between connected parts $h_f = 0.85$

n_s : Number of slip planes required to permit the connection to slip

μ : mean slip coefficient for Class A or B surfaces, as applicable, and determined as follows, or as established by tests:

For Class A surfaces $\mu = 0.30$

For Class B surfaces $\mu = 0.50$

3.4.3 Design of single angle compression members

The nominal compressive strength, P_n , of single angle members shall be determined in accordance with Section 3.7.1 or Section 3.7.3, as appropriate, for axially loaded members.

For single angles with $b/t > 0.71 \sqrt{\frac{E}{F_y}}$, torsional and flexural-torsional buckling shall be used in accordance with section 3.7.2. The effects of eccentricity on single angle members are permitted to be neglected when evaluated as axially loaded compression members:

- (1) Members are loaded at the ends in compression through the same one leg.
- (2) Members are attached by welding or by connections with a minimum of two bolts.
- (3) There are no intermediate transverse loads.
- (4) L_c/r as determined in this section does not exceed 200.

The allowable compressive strength, P_n/Ω_c are determined using the nominal compressive strength, P_n , which is the lowest value obtained based on the applicable limit states of flexural buckling, torsional buckling, and flexural torsional buckling.

$$\Omega_c = 1.67 \text{ (AISC ASD)}$$

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Effective length

The effective length, L_c , for calculation of member slenderness, L_c/r shall be determined in accordance with (ASCE 10-15), [20].

- Other *Compression Members Diagonal and Redundant members*: Member that reduce the unbraced length of primary members by providing intermediate support such that members with a concentric load at both ends of the unsupported panel,

$$L_c/r = L/r; 0 \leq L/r \leq 120 \quad 3.52$$

- For members with *normal framing eccentricities* at both ends of the unsupported panel,

$$L_c/r = 60 + 0.5 L/r; 0 \leq L/r \leq 120 \quad 3.53$$

- For members *partially restrained* against rotation at both ends of the unsupported panel,

$$L_c/r = 46.2 + 0.615 L/r; 120 \leq L/r \leq 250 \quad 3.54$$

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$$L_c/r = L/r; 120 \leq L/r \leq 250 \quad 3.55$$

where:

$$L_c = KL$$

L_c : effective length of member, [mm]

L : laterally unbraced length of the member, [mm]

r : radius of gyration, [mm]

For members designed on the basis of compression, the effective slenderness ratio L_c/r preferably should not exceed 200.

3.4.3.1 Flexural buckling of members without slender elements

This section applies to nonslender element compression members. The nominal compressive strength, P_n , shall be determined based on the limit state of flexural buckling.

$$P_n = F_{cr} A_g \quad 3.56$$

The critical stress, F_{cr} , is determined as follows:

$$(a) \text{ When } \frac{L_c}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} \dots\dots\dots (\text{or } \frac{F_y}{F_e} \leq 2.25)$$

$$F_{Cr} = \left[0.658 \frac{F_y}{F_e} \right] F_y \quad 3.57$$

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(b) When $\frac{L_c}{r} > 4.71 \sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} > 2.25$)

$$F_{Cr} = 0.877 F_e \quad 3.58$$

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \quad 3.59$$

where:

F_e : elastic buckling stress determined according to Equation 3.57 through an elastic buckling analysis as applicable, [Mpa]

A_g : gross cross-sectional area of member, [mm²]

3.4.3.2 Torsional and flexural-torsional buckling of members without slender elements

This section applies to singly symmetric and unsymmetric members and certain doubly symmetric members, such as cruciform or built-up columns without slender elements in uniform compression. These provisions are required for single angles with $b/t > 0.71\sqrt{(E/F_y)}$.

The nominal compressive strength, P_n , shall be determined based on the limit states of torsional and flexural-torsional buckling, as follows:

$$P_n = F_{cr} A_g \quad 3.60$$

For equal angle, singly symmetric members where y is the axis of symmetry the critical stress, F_{cr} , is determined using the torsional or flexural-torsional elastic buckling stress, F_e , as follows

$$F_e = \left(\frac{F_{ez} + F_{ey}}{2H} \right) \left[1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{(F_{ey} + F_{ez})^2}} \right] \quad 3.61$$

where:

A_g : gross cross-sectional area of member, [mm²]

C_w : warping constant, [mm⁶]

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2} \text{ Euler column buckling stress} \quad 3.62$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} \text{ Euler column buckling stress} \quad 3.63$$

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$$F_{ez} = \left(\frac{\pi^2 E C_w}{(k_z L)^2} + GJ \right) \frac{1}{A_g r_o^2} \text{ Torsional buckling stress} \quad 3.64$$

$$H = 1 - \frac{x_o^2 + y_o^2}{r_o^2} \quad 3.65$$

$$r_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \quad 3.66$$

where:

- A_g : gross cross-sectional area of member, [mm²]
- C_w : warping constant, [mm⁶]
- I_x, I_y : moment of inertia about the principal axes, [mm⁴]
- J : torsional constant, [mm⁴]
- G : shear modulus of elasticity of steel, [Mpa]
- K_x : effective length factor for flexural buckling about x-axis
- K_y : effective length factor for flexural buckling about y-axis
- K_z : effective length factor for torsional buckling
- r_o : polar radius of gyration about the shear center, [mm]
- r_x : radius of gyration about x-axis, [mm]
- r_y : radius of gyration about y-axis, [mm]
- x_o, y_o : coordinates of the shear center with respect to the centroid, [mm]

For singly symmetric equal angles the torsional constant J and warping constants C_w are determined as follows

$$J = \frac{1}{3} (b t_1^3 + h t_2^3) \quad 3.67$$

$$C_w = \frac{1}{36} (b^3 t_1^3 + h^3 t_2^3) \quad 3.68$$

3.4.3.3 Members with slender elements

This section applies to slender-element compression members in uniform compression. The nominal compressive strength, P_n , shall be the lowest value based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling.

$$P_n = F_{cr} A_g \quad 3.69$$

The critical stress, F_{cr} , shall be determined as follows:

(a) When $\frac{L_c}{r} \leq 4.71 \sqrt{\frac{E}{Q F_y}} \dots\dots\dots (\text{or } \frac{Q F_y}{F_e} \leq 2.25)$

$$F_{Cr} = \left[0.658 \frac{Q F_y}{F_e} \right] F_y \quad 3.70$$

(b) When $\frac{L_c}{r} > 4.71 \sqrt{\frac{E}{Q F_y}} \dots\dots\dots (\text{or } \frac{Q F_y}{F_e} > 2.25)$

$$F_{Cr} = 0.877 F_e \quad 3.71$$

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where:

Fe: Elastic buckling stress, calculated using Equations 3.59 and Equation 3.61 for singly symmetric members [Mpa]

Q: Net reduction factor accounting for all slender compression elements;

1.0 For members without slender elements in uniform compression.

QsQa For members with slender-element sections in uniform compression

For cross sections composed of only unstiffened slender elements, $Q = Q_s$ ($Q_a = 1.0$). For cross sections composed of only stiffened slender elements, $Q = Q_a$ ($Q_s = 1.0$). For cross sections composed of both stiffened and unstiffened slender elements, $Q = Q_s Q_a$. For cross sections composed of multiple unstiffened slender elements, it is conservative to use the smaller Q_s from the more slender element in determining the member strength for pure compression.

Reduction factor for slender unstiffened elements, Q_s

The reduction factor, Q_s , for slender unstiffened elements is defined as follows:

(c) For single angles compression members:

$$Q_s = 1 \quad \frac{b}{t} \leq 0.45 \sqrt{\frac{E}{F_y}} \quad 3.72$$

$$Q_s = 1.34 - 0.76 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \quad 0.45 \sqrt{\frac{E}{F_y}} < \frac{b}{t} < 0.91 \sqrt{\frac{E}{F_y}} \quad 3.73$$

$$Q_s = \frac{0.53E}{F_y \left(\frac{b}{t} \right)^2} \quad \frac{b}{t} \geq 0.91 \sqrt{\frac{E}{F_y}} \quad 3.74$$

where:

b: full width of longest leg, [mm]

t: thickness of the leg, [mm]

3.4.3.4 Bracing and secondary compression member

Bracing and secondary members are provided where the slenderness ratio values of leg and diagonals structural member values are beyond the limit. In order to consider a reduction in the unbraced length of a supported member at a node or Panel point, bracing and secondary members (zero force members) shall provide a minimum resistance, P_s ,

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normal to the supported member (in both directions) in the plane of buckling under consideration. P_s shall be determined in accordance with the following, [9].

$$P_s = \left[1.5 + \frac{\frac{KL}{r} - 60}{60} \right] \frac{F_s}{100} \quad 3.75$$

$$1.5 \frac{F_s}{100} \leq P_s \leq 2.5 \frac{F_s}{100}$$

where:

F_s : axial design compressive force in the supported member

$\frac{KL}{r}$: effective slenderness ratio of the supported member in the plane of buckling under consideration

3.4.4 Affected elements of members and connecting elements

This section applies to elements of members at connections and connecting elements, such as plates, gussets, angles and brackets.

- To prevent member failure in tension, because of reduced net area due to bolt holes for connection, capacity of members has to be checked for fracture failure criteria over the net area through bolt holes.
- To prevent bolt failure in shearing, bolted joint shall be checked for shearing of bolt with a shear plane.
- To prevent bearing stress failure due to crushing of connected members when the shank of bolt bears on connected members and deformations of bolt by other member acting on it, the bearing pressure or contact pressure acting over the projected area of the bolt shank on the connected member due to the joint-load shall be less than or equal to the allowable bearing stress specified by the code.

3.4.4.1 Spacing and edge distance of bolts

Minimum spacing

The distance between centers of standard holes shall not be less than $2\frac{2}{3}$ times the nominal diameter d , of the fastener or the clear distance between bolts holes shall not be less than nominal diameter d , as shown in the figure below.

$$S_c \geq 2\frac{2}{3}d_b \quad 3.76$$

$$S \geq d_b$$

However, a distance of three time the nominal diameter of bolt is preferred for standard holes between centers of holes, $S=3d_b$.

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where:

S_c : Clear distance between bolts holes

S : Distance between centers of standard holes

d_b : Nominal Diameter of bolt

Minimum edge distance

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table 9, or as required in Section 3.8.5.

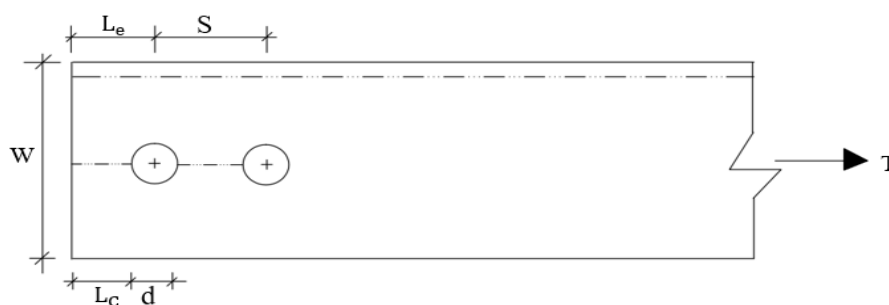


Figure 24: Minimum edge distance, L_e

Table 10: Minimum Edge Distance, [mm]

Minimum Edge Distance From Center of Standard Hole To Edge of Connected Part, [mm]	
Bolt Diameter, mm	Minimum Edge Distance L_e , [mm]
16	22
20	26
22	28
24	30
27	34
30	38
36	46
Over 36	1.25d

Equivalent workable gage length

Table 11: Equivalent workable gages length for holes in single angle, [mm]

Leg	250	200	175	150	125	100	90	75	60	50	45
g₁	125	115	100	90	75	60	50	45	35	30	25
g₂	75	75	60	55	50	-	-	-	-	-	-
g₃	60	75	75	60	45	-	-	-	-	-	-
g₄	60	-	-	-	-	-	-	-	-	-	-

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Standard holes dimensions

Standard holes are used in the absence of Engineers record approval for the use of other hole types, standard holes shall be used in all plies of bolted joints

Table 12: Standard hole dimensions [mm]

Bolt Diameter, mm	Standard Diameter, mm
M16	18
M20	22
M22	24
M24	27
M27	30
M30	33
≥M36	d+3

3.4.4.2 Strength of elements in tension

The allowable strength, R_n/Ω , of affected and connecting elements loaded in tension shall be the lower value obtained according to the limit states of tensile yielding and tensile rupture.

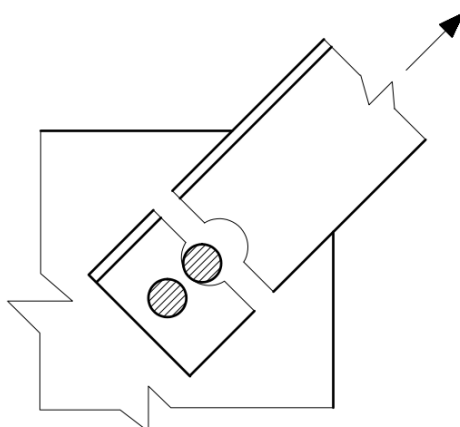


Figure 25: Tension Failure

- For Tensile yielding of connecting elements

$$R_n = F_y A_g \quad 3.77$$

$$\Omega = 1.67 \text{ (AISC ASD)}$$

- For Tensile rupture of connecting elements

$$R_n = F_u A_e \quad 3.78$$

$$\Omega = 2.00 \text{ (AISC ASD)}$$

where:

A_e : effective net area, [mm²]

A_g : gross area of member, [mm²]

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F_y : specified minimum yield stress, [Mpa]

F_u : specified minimum tensile strength, [Mpa]

3.4.4.3 Strength of elements in shear

The available shear strength of affected and connecting elements in shear shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

- For *shear yielding* of the element:

$$R_n = 0.6F_y A_{gv} \quad 3.79$$

$$\Omega = 1.50 \text{ (AISC ASD)}$$

where:

A_{gv} = Gross area subject to shear, [mm²]

- For *shear rupture* of the element:

$$R_n = 0.6F_u A_{nv} \quad 3.80$$

$$\Omega = 2.00 \text{ (AISC ASD)}$$

where:

A_{nv} = net area subject to shear, [mm²]

3.4.4.4 Block shear strength

The available strength for the limit state of block shear rupture (*Tearing failure*) along a shear failure path or paths and a perpendicular tension failure path shall be taken as:

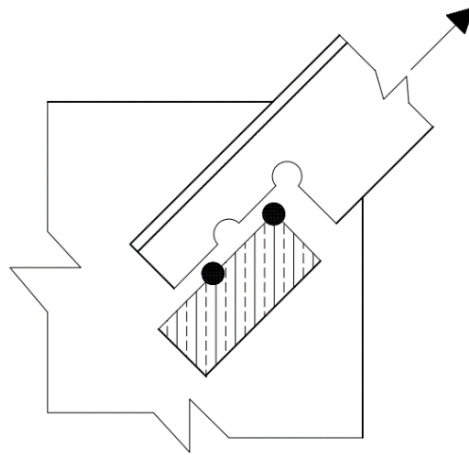


Figure 26: Block Shear failure

$$R_n = 0.60F_u A_{nv} + F_u A_{nt} \leq 0.60F_y A_{gv} + F_u A_{nt} \quad 3.81$$

$$\Omega = 2.00 \text{ (AISC ASD)}$$

where:

A_{nt} = net area subject to tension, [mm²]

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3.4.4.5 Bearing and tearout strength at bolt holes

The available strength R_n / Ω at bolt holes shall be determined for the limit states of *bearing and tearout*, as follows:

For a bolt in a connection with standard hole, independent of the direction of loading the nominal strength of the connected material, R_n , is determined as follows:

(1) Bearing

When deformation at the bolt hole at service load is a design consideration

$$R_n = 2.4 d t F_u \quad 3.82$$

(2) Tearout

When deformation at the bolt hole at service load is a design consideration

$$R_n = 1.2 l_c t F_u \quad 3.83$$

$$\Omega = 2.00 \text{ (ASD)}$$

where:

F_u : Specified minimum tensile strength of the connected material, MPa

d : Nominal fastener diameter, mm

l_c : clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole, S- d_h or edge of the material, $L_e - 0.5d_h$, mm

t : thickness of connected material, mm

3.4.4.6 Tensile and shear strength of bolts and threaded parts

The allowable tensile or shear strength, R_n / Ω , of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the limit states of tension rupture and shear rupture as:

$$R_n = F_n A_b \quad 3.84$$

$$\Omega = 2.00 \text{ (ASD)}$$

where:

A_b : Nominal unthreaded body area of bolt or threaded part, mm²

F_n : Nominal tensile stress, F_{nt} , or shear stress, F_{nv} ,

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Table 13: Nominal Strength of Fasteners and Threaded Parts, MPa

Description Of Fasteners	Nominal Tensile Strength F_{nt} ,[Mpa]	Nominal Shear Strength In Bearing Type Connections F_{nv} ,[Mpa]
A325 bolts, when threads are included in the shear planes	620	372
A325 bolts, when threads are excluded from shear planes	620	469

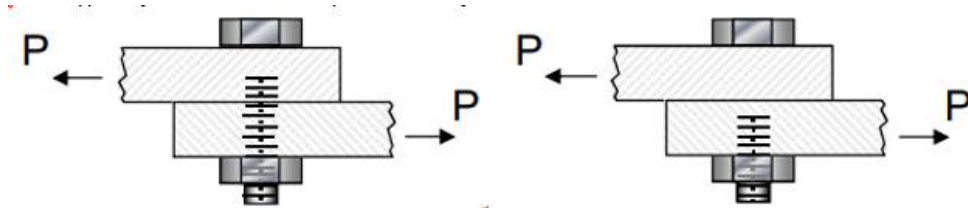


Figure 27: Shear plane and threaded part

CHAPTER FOUR

SEISMIC AND WIND LOAD BEHAVIOUR OF COMMUNICATION TOWER

To study the seismic and wind behavior of the tower, analysis of self-supporting latticed steel communication towers under various heights, wind Loads and seismic zones are done. For the analysis and design purpose, a finite element software STAAD Pro V8i is used.

For wind load analysis, the wind intensity versus height graph value from the preliminary analysis is used as an input.

Exposure factor is percent of the face of the structure exposed to wind, and it is always less than one, from the excel exposure factors of various height of tower are calculated as per the definition.

Effect of wind load on the proposed tower structure for various wind cases have been studied using gust factor method

Using the finite element software, analysis of tower for wind load, the horizontal joint displacements at the top of the structure and member stresses at the base of panel or critical members are determined for wind cases proposed based on the 50-year recurrence interval basic wind speed for the site.

4.1 FINITE ELEMENT THREE DIMENSIONAL MODELLING

Heavy duty tower models having a height of 45m, 60m, 80m, and 100m with XX type of bracings were taken for the analysis. So as to make easy for the analysis, the SSCT100 tower modular panel and sub panel member modelling is used for the other smaller in height towers by taking the base panels at mid height of, D, E and F for SSCT45, SSCT60 and SSCT80 towers respectively.

Table 14: Specifications of self-supporting communication towers

Tower Name	Tower height [m]	Base Width [m]	Top width [m]	Height of Straight [m]	Height of Tapered [m]	Bracing Type
SSCT45	45	7.17	2	5	40	XX
SSCT60	60	9.11	2	5	55	XX
SSCT80	80	11.70	2	5	75	XX
SSCT100	100	14.23	2	5	95	XX

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Table 15 : Panel and sub panel dimension for SSCT100

Panel Name	Panel height h_{pv} [m]	Number of Sub panel n	Bottom of panel from tower base [m]	Top width T [m]	Bottom Width B [m]	Panel Leg length Lp [m]
A	5.00	5	95.00	2.00	2.00	5.00
B	8.00	4	87.00	2.00	3.03	8.03
C	12.00	3	75.00	3.03	4.59	12.05
D	20.00	4	55.00	4.59	7.17	20.08
E	15.00	2	40.00	7.17	9.12	15.06
F	20.00	2	20.00	9.12	11.70	20.08
G	20.00	2	0.00	11.70	14.29	20.08
Straight part	5.00	5	-	2.00	2.00	5.00
Tapered part	95.00	17	-	2.00	14.29	95.40
Total	100.00	22	-	2.00	14.29	100.40

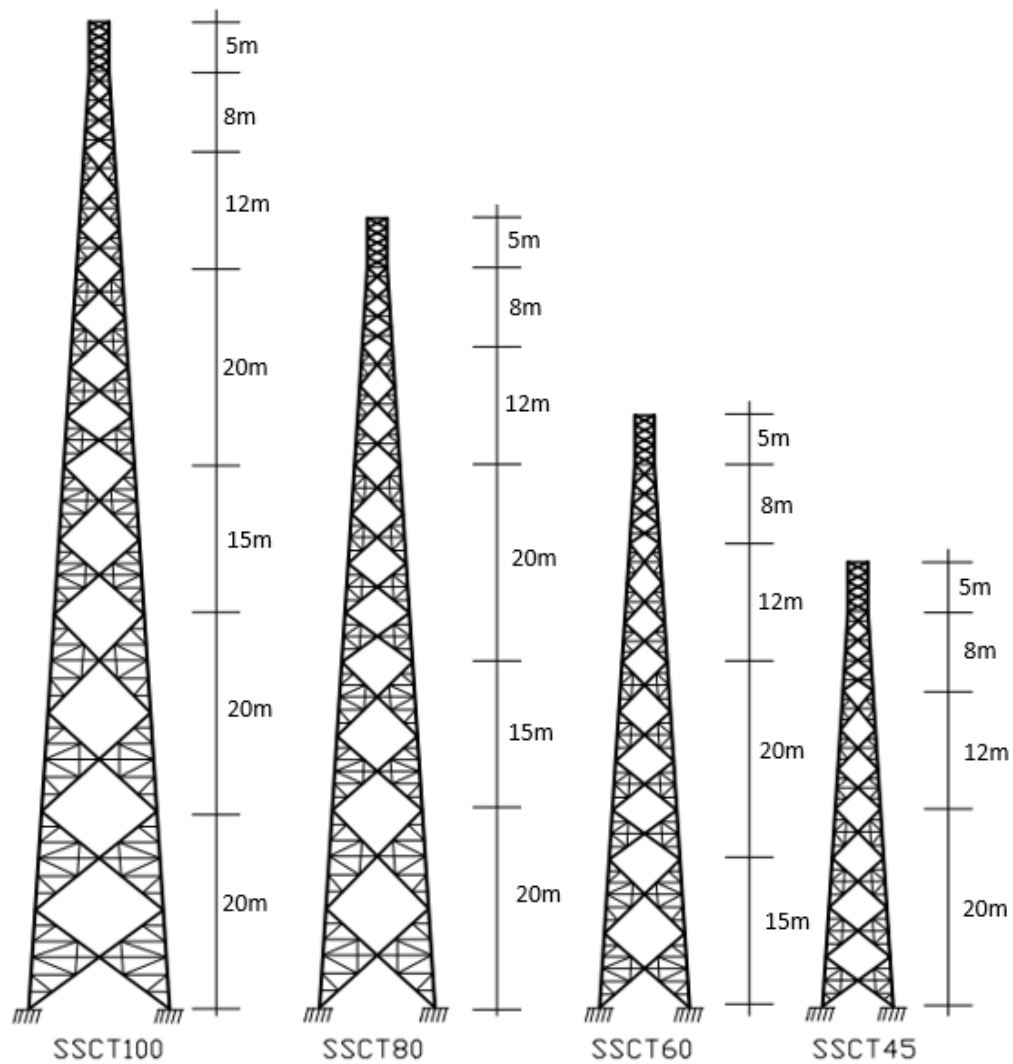


Figure 28: Self supporting communication towers

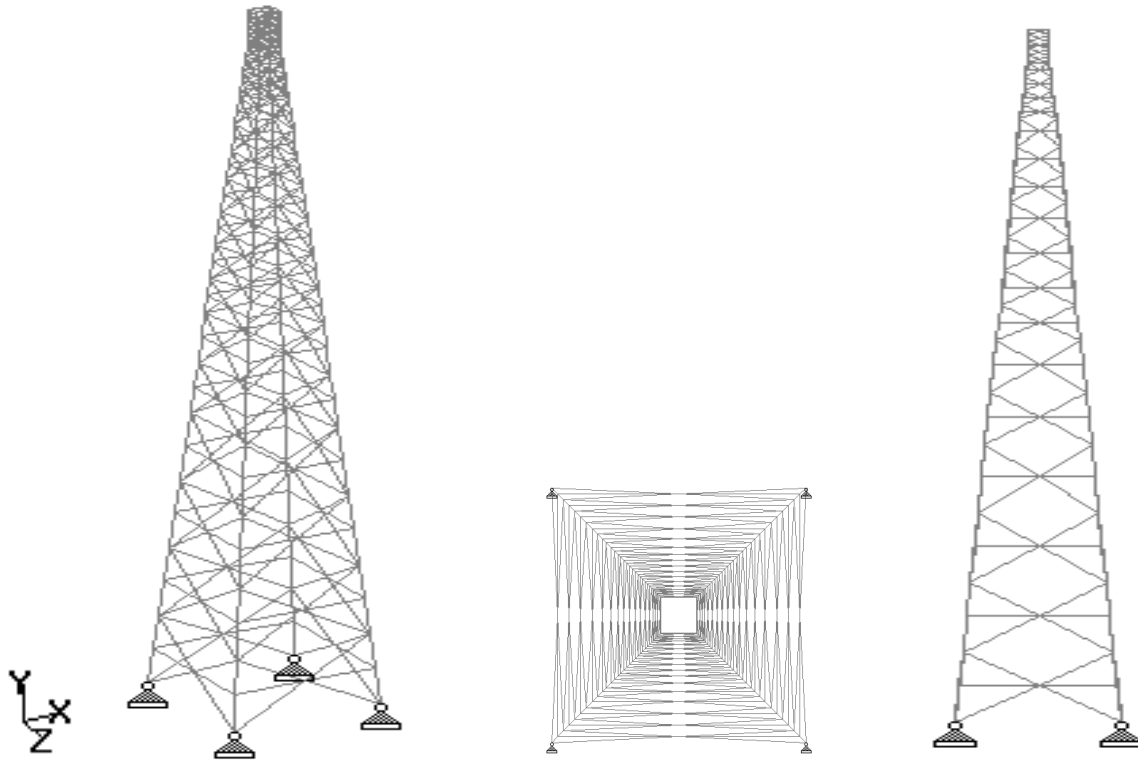


Figure 29: Three Dimensional, Top and Front view of SSCT Structural model

The following equal angle steel sections are used for all tower height for seismic and wind behavior study

Panel members Steel Section profile for Seismic and wind behavior study				
Panel	Leg members [mm×mm×mm]	Diagonal members [mm×mm×mm]	Horizontal Members [mm×mm×mm]	Zero force member [mm×mm×mm]
A	75×75×6	75×75×6	75×75×6	75×75×6
B	100×100×13	75×75×8	75×75×6	75×75×6
C	100×100×13	75×75×8	75×75×6	75×75×6
D	150×150×15	75×75×8	75×75×6	75×75×6
E	200×200×20	100×100×8	100×100×8	75×75×8
F	250×250×25	100×100×13	100×100×8	75×75×8
G	250×250×35	100×100×13	100×100×8	75×75×8

4.2 WIND ANALYSIS

4.2.1 Wind Intensity

Four wind velocity cases have been selected for the studies of the behavior of tower when it is subjected to wind load.

- Case I 35m/s (80mph)
- Case II 40m/s (90mph)
- Case III 45m/s (100mph)
- Case IV 50m/s (110mph)

SEISMIC AND WIND LOAD BEHAVIOUR

Wind intensity for each type towers is determined at the centroid of the sub panels. To consider the diagonal wind load intensity the parallel wind load intensity is multiplied by a factor of 1.2 as shown in Table 5. The Graph of wind intensity versus height is needed for thirty two (32) STAAD Pro finite element analysis considering parallel and diagonal wind loads.

Table 16: Wind intensity for SSCT100

Height from Ground [m]	Wind Velocity pressure (Parallel) q_{zp} [kN/m ²]				Wind Velocity pressure(Diagonal) q_{zd} [kN/m ²]			
	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.921	0.431	0.563	0.712	0.879	0.517	0.675	0.854	1.055
14.913	0.591	0.772	0.977	1.207	0.710	0.927	1.173	1.448
24.902	0.685	0.894	1.132	1.397	0.821	1.073	1.358	1.676
34.890	0.754	0.985	1.246	1.538	0.905	1.181	1.495	1.846
43.680	0.804	1.050	1.329	1.640	0.965	1.260	1.594	1.968
51.171	0.841	1.098	1.390	1.716	1.009	1.318	1.668	2.060
57.461	0.869	1.135	1.437	1.774	1.043	1.362	1.724	2.129
62.457	0.890	1.163	1.472	1.817	1.068	1.395	1.766	2.180
67.452	0.910	1.189	1.504	1.857	1.092	1.426	1.805	2.229
72.445	0.929	1.213	1.535	1.895	1.115	1.456	1.842	2.275
76.960	0.945	1.234	1.562	1.929	1.134	1.481	1.875	2.314
80.955	0.959	1.252	1.585	1.957	1.150	1.503	1.902	2.348
84.948	0.972	1.270	1.607	1.984	1.166	1.523	1.928	2.380
87.985	0.982	1.282	1.623	2.004	1.178	1.539	1.948	2.404
89.984	0.988	1.291	1.633	2.017	1.186	1.549	1.960	2.420
91.982	0.994	1.299	1.644	2.029	1.193	1.559	1.972	2.435
93.980	1.000	1.307	1.654	2.042	1.201	1.568	1.985	2.450
95.500	1.005	1.313	1.661	2.051	1.206	1.575	1.994	2.461
96.500	1.008	1.317	1.666	2.057	1.210	1.580	2.000	2.469
97.500	1.011	1.321	1.671	2.063	1.213	1.585	2.006	2.476
98.500	1.014	1.324	1.676	2.069	1.217	1.589	2.011	2.483
99.500	1.017	1.328	1.681	2.075	1.220	1.594	2.017	2.490

The remaining wind intensity tables for SSCT80, SSCT60, and SSCT45 towers are shown in appendix C. The following sixteen (16) graphs shown bellows are wind intensity graph for parallel loadings based on Table B78, Table B79, Table B80 and Table B81.

SEISMIC AND WIND LOAD BEHAVIOUR

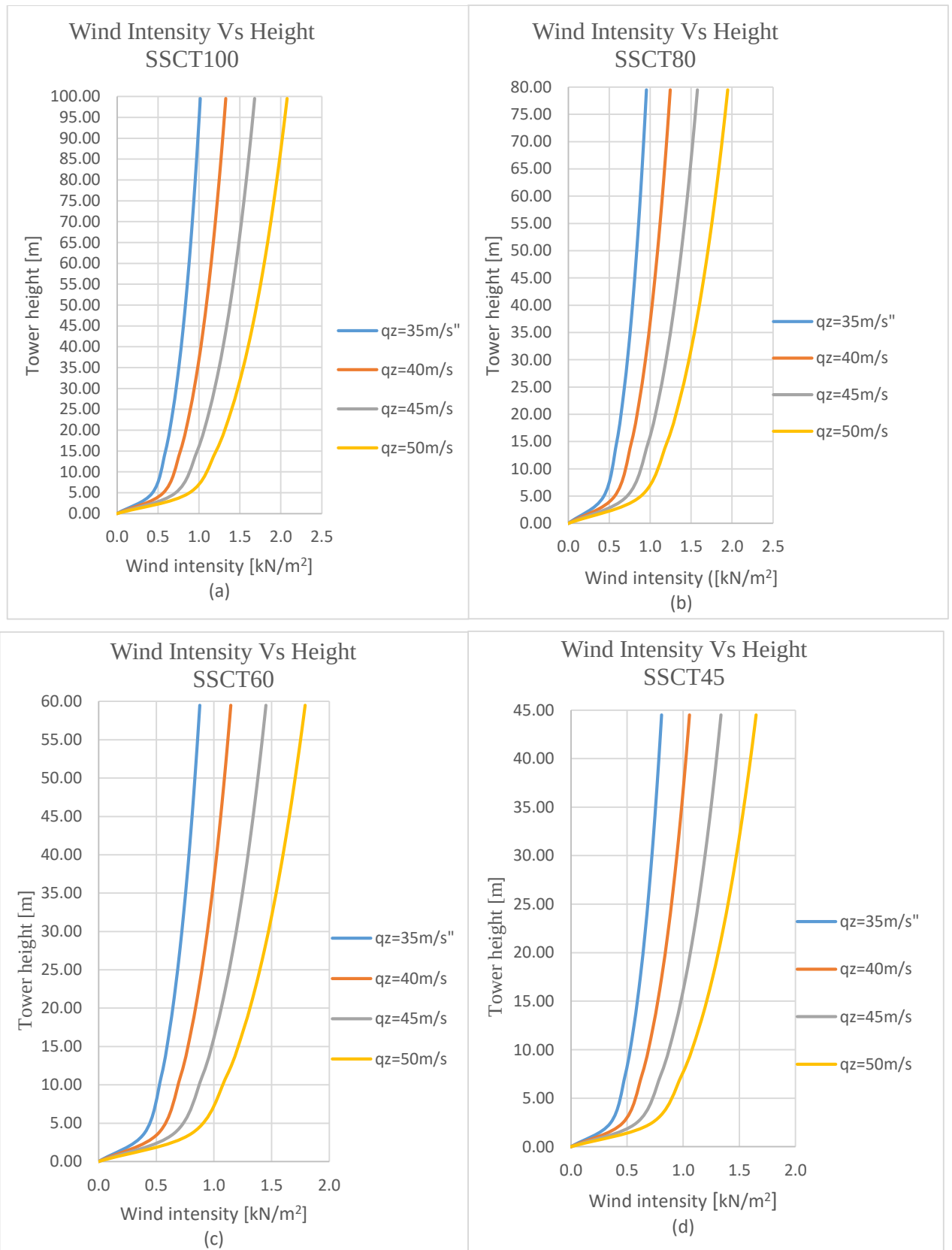


Figure 30: Wind intensity versus height graph

4.3 RESPONSE SPECTRUM ANALYSIS

For a rigid structure on the reference ground type, Class B, F_a takes the value of unity and the design spectral ordinate S_{ao} should be equal or the same as the peak ground acceleration (PGA) of the site for the design earthquake. This enables us to estimate the value of S_s for a known PGA of a site, [11].

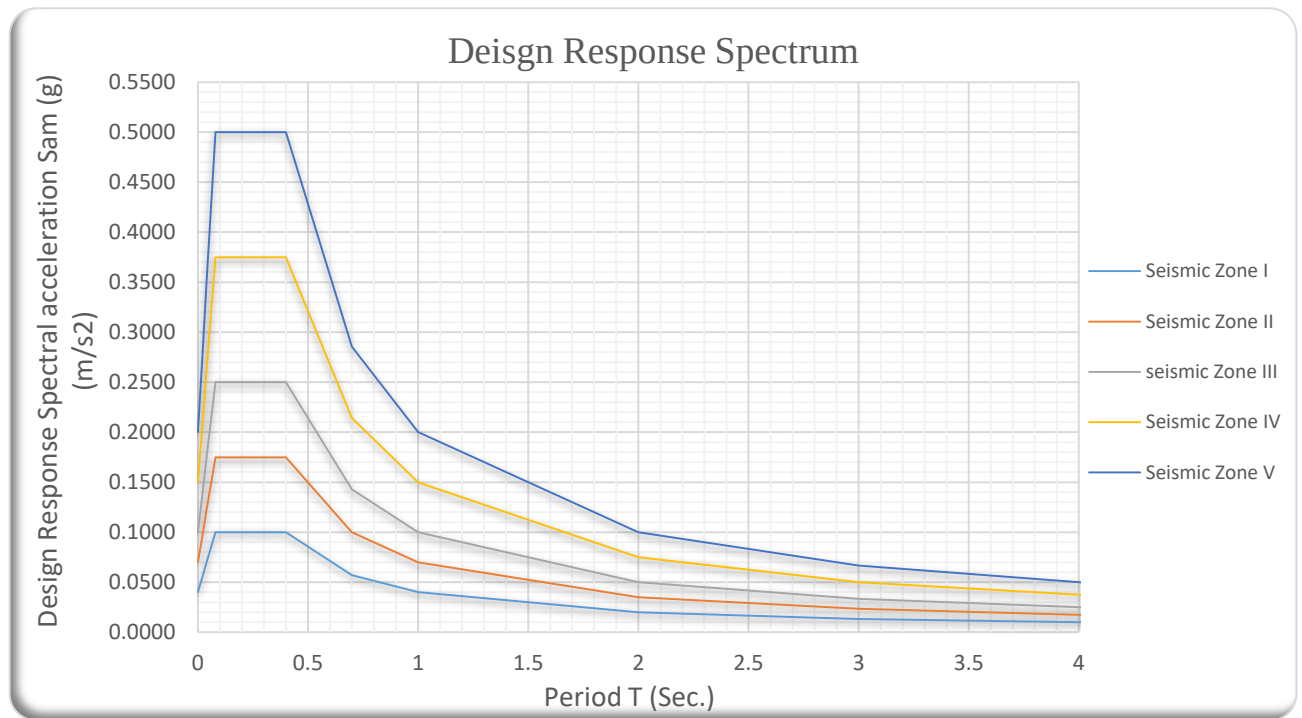


Figure 31: Design response spectrum curve

Table 17: Spectral acceleration parameters

Seismic Zone	PGA [g]	S_s [g]	S_1 [g]	S_{Ds} [g]	S_{D1} [g]	T_0 [Sec.]	T_s [Sec.]	T [sec.]
I	0.040	0.150	0.060	0.100	0.040	0.080	0.400	4.000
II	0.070	0.263	0.105	0.175	0.070	0.080	0.400	4.000
III	0.100	0.375	0.150	0.250	0.100	0.080	0.400	4.000
IV	0.150	0.563	0.225	0.375	0.150	0.080	0.400	4.000
V	0.200	0.750	0.300	0.500	0.200	0.080	0.400	4.000

4.4 NODAL DISPLACEMENT AND BASE SHEAR RESULTS

4.4.1 Nodal Displacement Results

The Horizontal Nodal displacement for all wind cases and seismic zones acting in parallel and diagonal direction was done by performing 320 (8×40) finite element analysis using STAAD Pro V8i Finite element software.

Table 18: Horizontal Nodal displacement due to parallel wind load in [mm]

Wind Speed	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35m/s	4.72	7.87	14.50	23.69
Case (II) 40m/s	6.16	10.28	18.94	30.94
Case (III) 45m/s	7.80	13.01	23.98	39.16
Case (IV) 50m/s	9.63	16.06	29.60	48.35

Table 19: Horizontal Nodal Displacement due to Diagonal wind load in [mm]

Wind Speed	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35m/s	6.91	13.30	23.35	39.08
Case (II) 40m/s	9.03	17.37	30.49	51.05
Case (III) 45m/s	11.43	21.99	38.60	64.60
Case (IV) 50m/s	14.11	27.15	47.64	79.76

Table 20: Horizontal Nodal displacement due to Parallel seismic load in [mm]

Seismic Zone	SSCT45	SSCT60	SSCT80	SSCT100
Seismic Zone I	1.52	2.93	4.90	6.26
Seismic Zone II	2.67	5.14	8.58	10.96
Seismic Zone III	3.81	7.33	12.26	15.65
Seismic Zone IV	5.72	11.00	18.39	23.48
Seismic Zone V	7.62	14.65	24.52	31.30

Table 21: Horizontal Nodal displacement due to Diagonal seismic load in [mm]

Seismic Zone	SSCT45	SSCT60	SSCT80	SSCT100
Seismic Zone I	1.52	2.93	4.91	6.26
Seismic Zone II	2.67	5.14	8.59	10.96
Seismic Zone III	3.81	7.32	12.26	15.66
Seismic Zone IV	5.72	11.00	18.40	23.49
Seismic Zone V	7.62	14.65	24.53	31.32

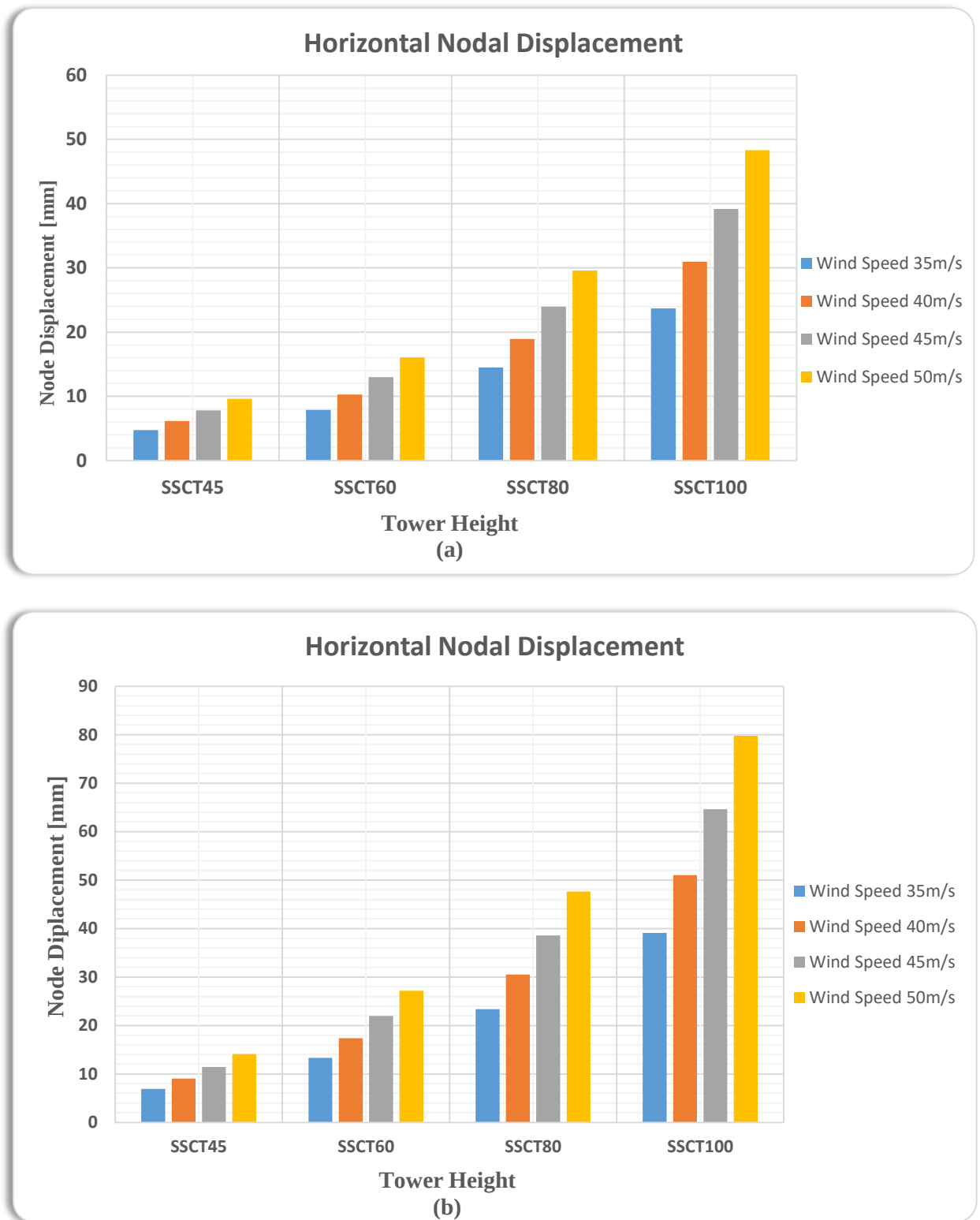


Figure 32: Horizontal Nodal Displacement for (a) Parallel Wind (b) Diagonal Wind

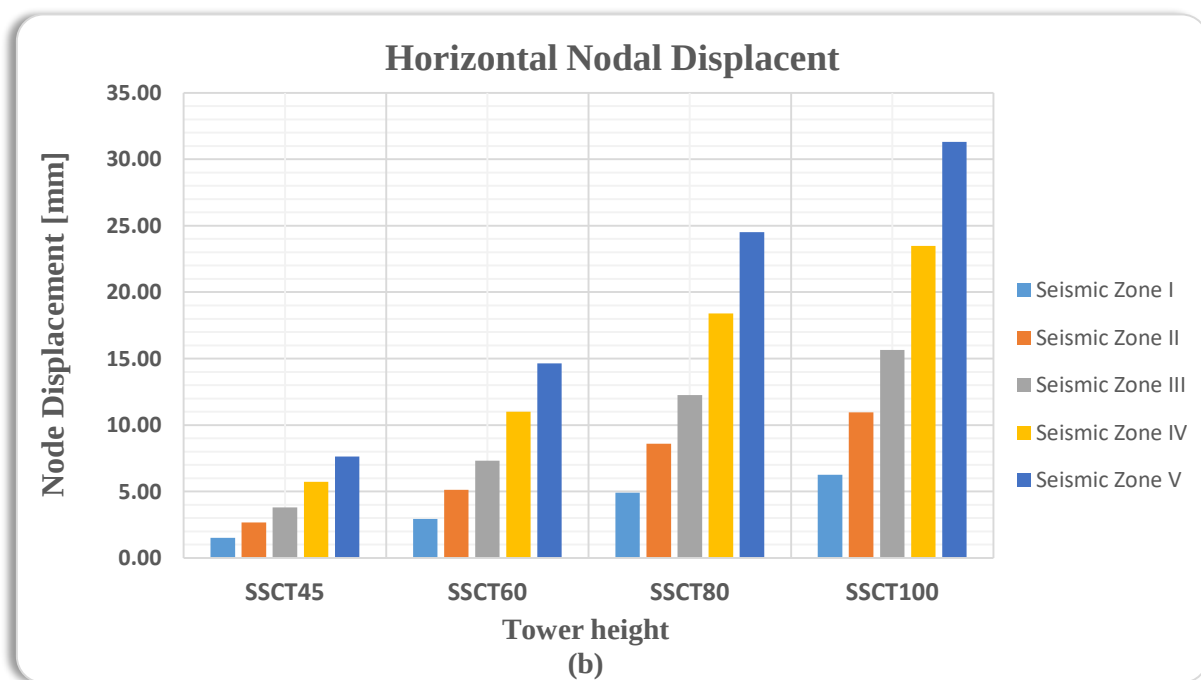
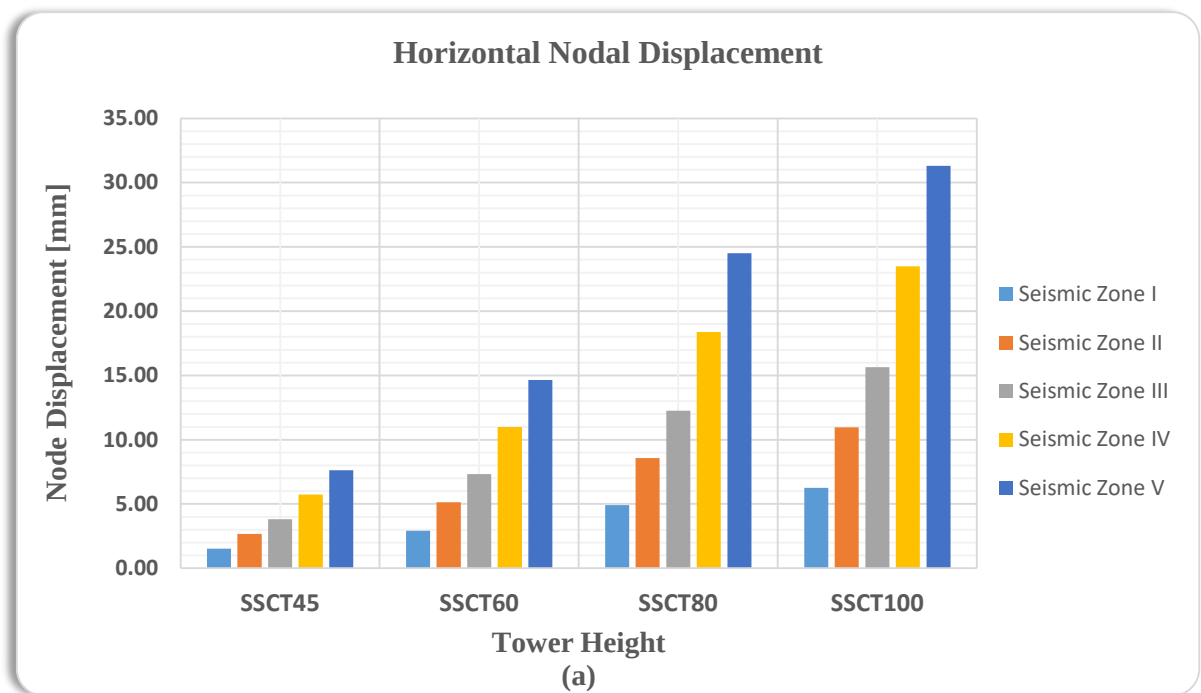


Figure 33: Horizontal Nodal Displacement for Seismic Zones, (a) Parallel seismic load
(b) Diagonal Seismic load

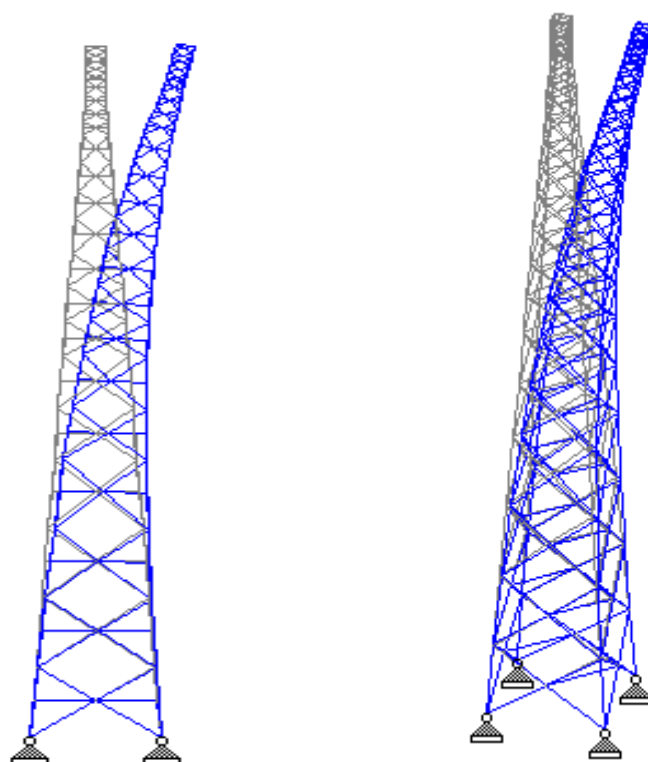


Figure 34: Horizontal Node Displacement Views

Horizontal Node displacement difference between wind and seismic load in percent

Table 22: Horizontal Node Displacement Difference between parallel wind and seismic load in Percent for Seismic Zone I [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	68	63	66	74
Case (II) 40	75	71	74	80
Case (III) 45	80	77	80	84
Case (IV) 50	84	82	83	87

Table 23: Horizontal Node Displacement Difference between Diagonal wind and seismic load in Percent for Seismic Zone I [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	78	78	79	84
Case (II) 40	83	83	84	88
Case (III) 45	87	87	87	90
Case (IV) 50	89	89	90	92

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Table 24: Horizontal Node Displacement Difference between Parallel wind and seismic load in Percent for Seismic Zone II. [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	43	35	41	54
Case (II) 40	57	50	55	65
Case (III) 45	66	60	64	72
Case (IV) 50	72	68	71	77

Table 25: Horizontal Node Displacement Difference between Diagonal wind and seismic load in Percent for Seismic Zone II.[%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	61	61	63	72
Case (II) 40	70	70	72	79
Case (III) 45	77	77	78	83
Case (IV) 50	81	81	82	86

Table 26: Horizontal Node Displacement Difference between Parallel wind and seismic load in Percent for Seismic Zone III. [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	19	7	15	34
Case (II) 40	38	29	35	49
Case (III) 45	51	44	49	60
Case (IV) 50	60	54	59	68

Table 27: Horizontal Node Displacement Difference between Diagonal wind and seismic load in Percent for Seismic Zone III. [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	45	45	47	60
Case (II) 40	58	58	60	69
Case (III) 45	67	67	68	76
Case (IV) 50	73	73	74	80

SEISMIC AND WIND LOAD BEHAVIOUR

Table 28: Horizontal Node Displacement Difference between Parallel wind and seismic load in Percent for Seismic Zone IV. [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	-21	-40	-27	1
Case (II) 40	7	-7	3	24
Case (III) 45	27	15	23	40
Case (IV) 50	41	32	38	51

Table 29: Horizontal Node Displacement Difference between Diagonal wind and seismic load in Percent for Seismic Zone IV. [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	17	17	21	40
Case (II) 40	37	37	40	54
Case (III) 45	49	50	52	64
Case (IV) 50	59	59	61	71

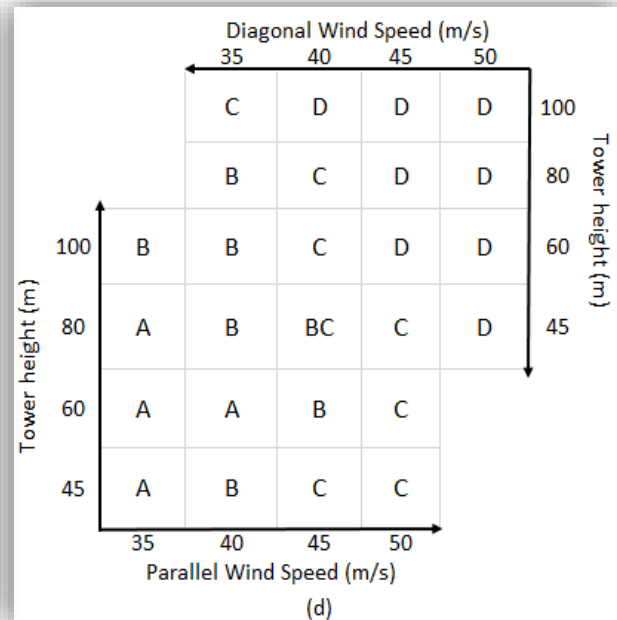
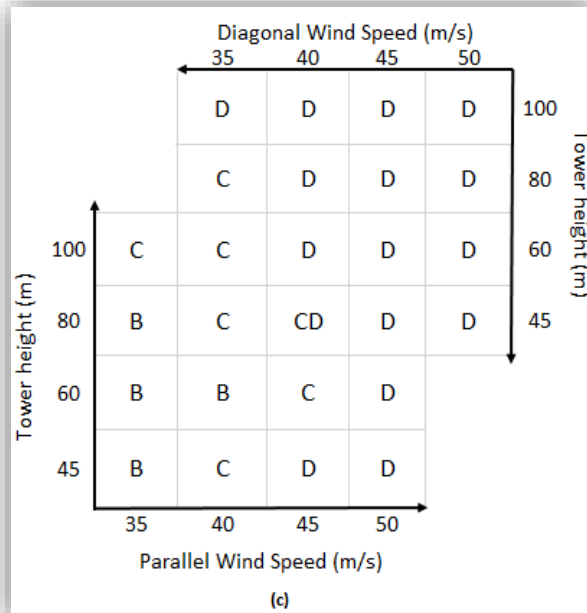
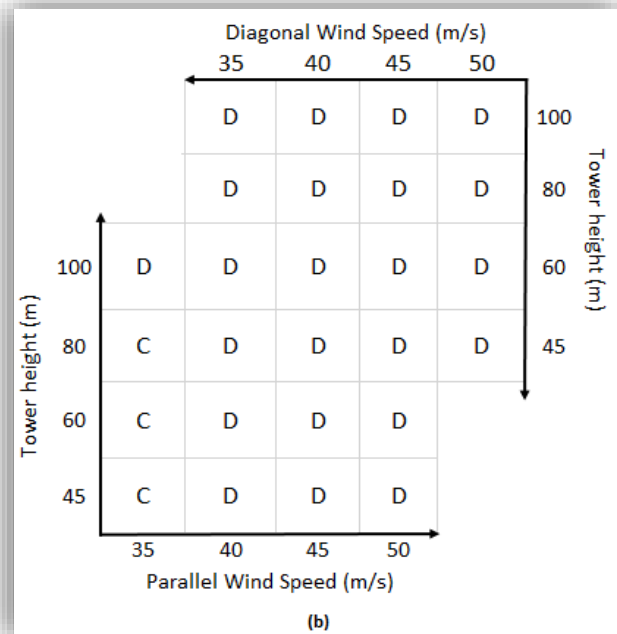
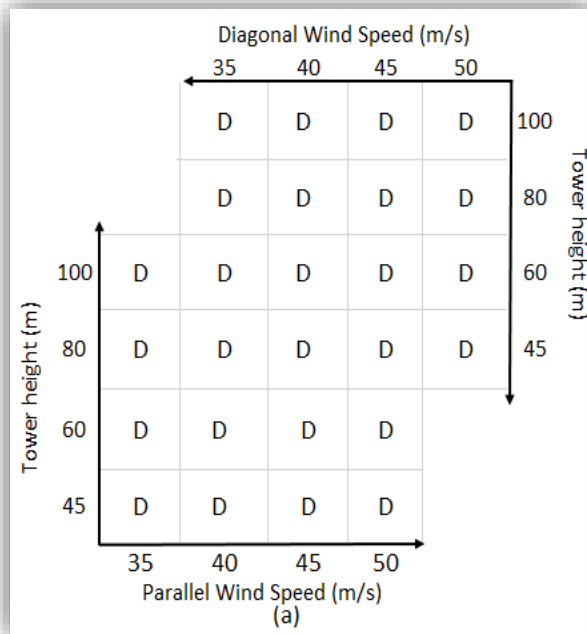
Table 30: Horizontal Node Displacement Difference between Parallel wind and seismic load in Percent for Seismic Zone V. [%]

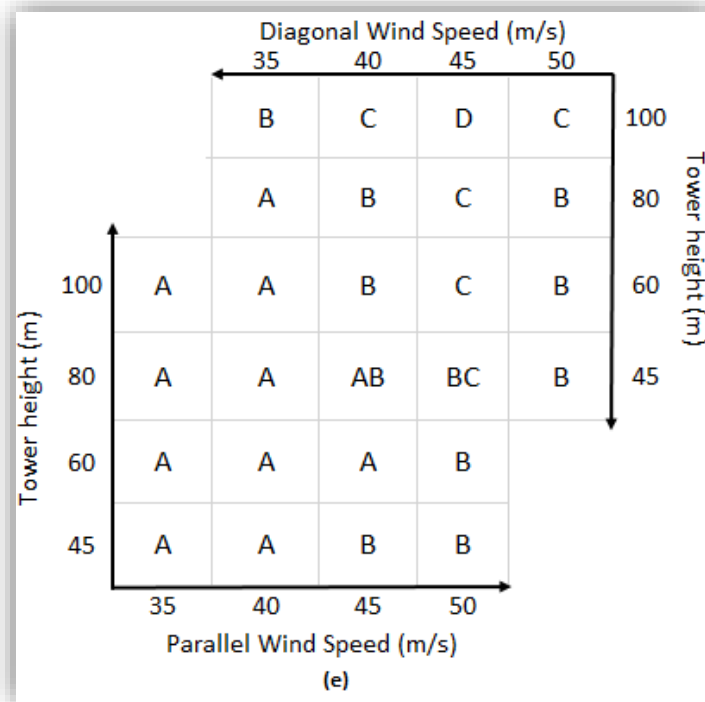
Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	-62	-86	-69	-32
Case (II) 40	-24	-43	-29	-1
Case (III) 45	2	-13	-2	20
Case (IV) 50	21	9	17	35

Table 31: Horizontal Node Displacement Difference between Diagonal wind and seismic load in Percent for Seismic Zone V. [%]

Wind Speed [m/s]	SSCT45	SSCT60	SSCT80	SSCT100
Case (I) 35	-10	-10	-5	20
Case (II) 40	16	16	20	39
Case (III) 45	33	33	36	52
Case (IV) 50	46	46	49	61

SEISMIC AND WIND LOAD BEHAVIOUR





Labels	A	B	C	D
Value [%]	≤ 0	≤ 30	≤ 50	≥ 50

Figure 35: Chart for Percentage Node Displacement Difference between Wind load and Seismic load in (a) seismic Zone I (b) seismic Zone II (c) seismic Zone III (d) seismic Zone IV (e) seismic Zone V.

Most of the time it is argued that, any tower structure designed for wind load can withstand seismic load. Even if it is true it is wrong to perceive the statement works for all seismic zones because for higher seismic zones some tower exhibited responses more than wind load.

Note that deformation of the steel section is not included in total node displacement of tower.

4.4.2 Base shear Results

Table 32: Base Shear for Seismic Zone [kN]

Seismic Zone	SSCT45	SSCT60	SSCT80	SSCT100
Seismic Zone I	3.66	6.02	9.23	11.54
Seismic Zone II	6.42	10.56	16.15	20.21
Seismic Zone III	9.16	15.06	23.06	28.86
Seismic Zone IV	13.75	22.61	34.60	43.30
Seismic Zone V	18.32	30.11	46.12	57.71

SEISMIC AND WIND LOAD BEHAVIOUR

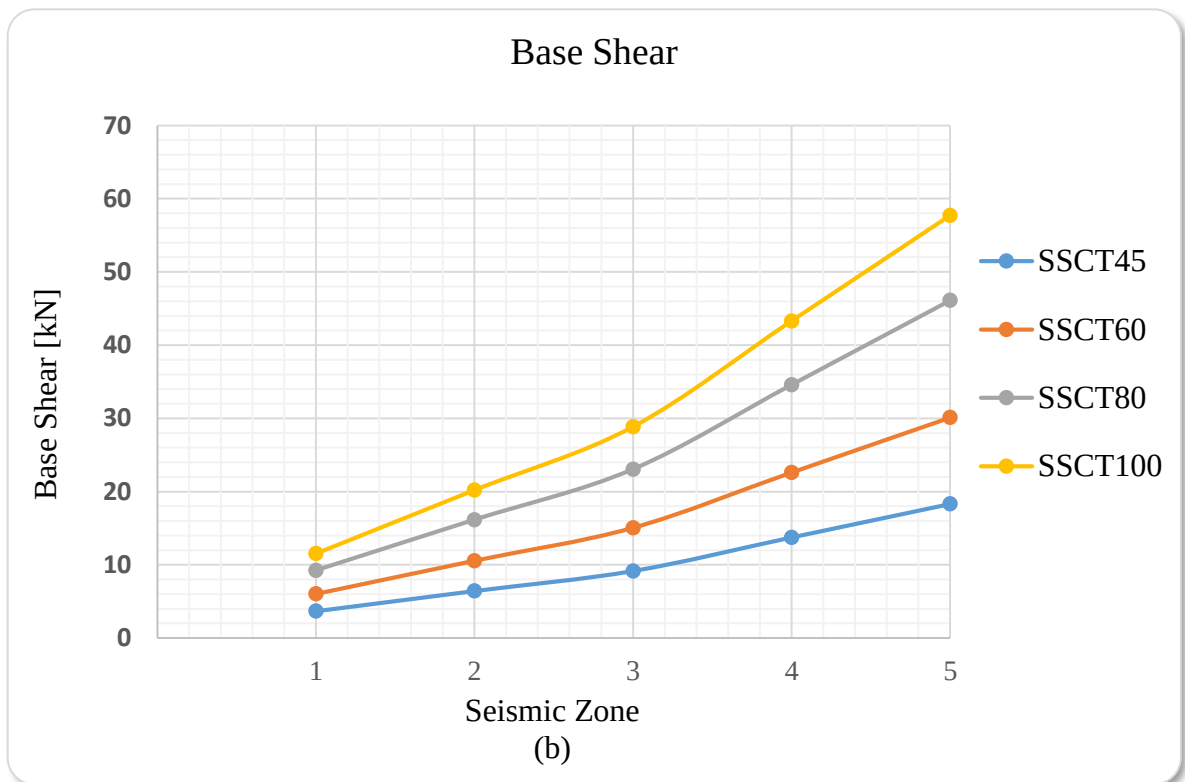
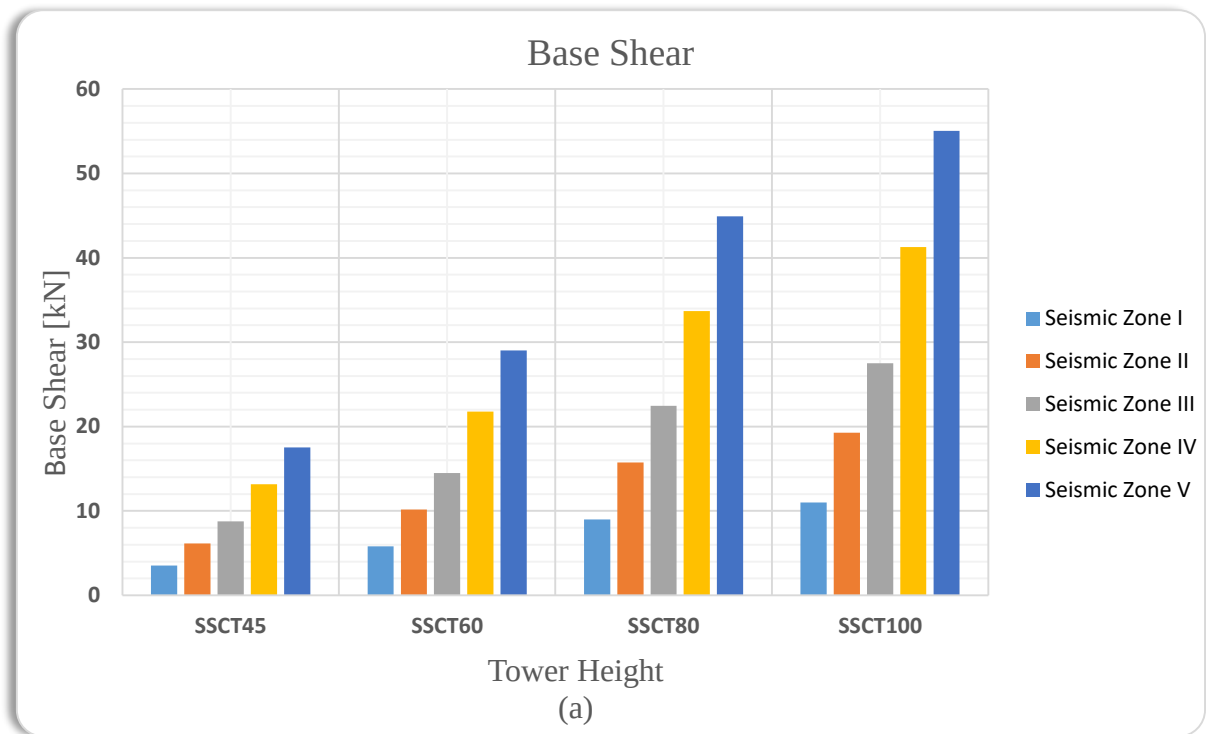


Figure 36: Base shear versus tower height (a) Base Shear versus Seismic Zones, (b)

Base shear value of communication tower's is directly proportional to height of tower and seismic zone.

SEISMIC AND WIND LOAD BEHAVIOUR

Table 33: Seismic Zones Base shear Difference in Percent [%]

Seismic Zone	SSCT45	SSCT60	SSCT80	SSCT100	Average Increment %
I-II	76	75	75	75	75
II-III	43	36	43	43	41
III-IV	50	57	50	50	52
IV-V	33	33	36	33	34

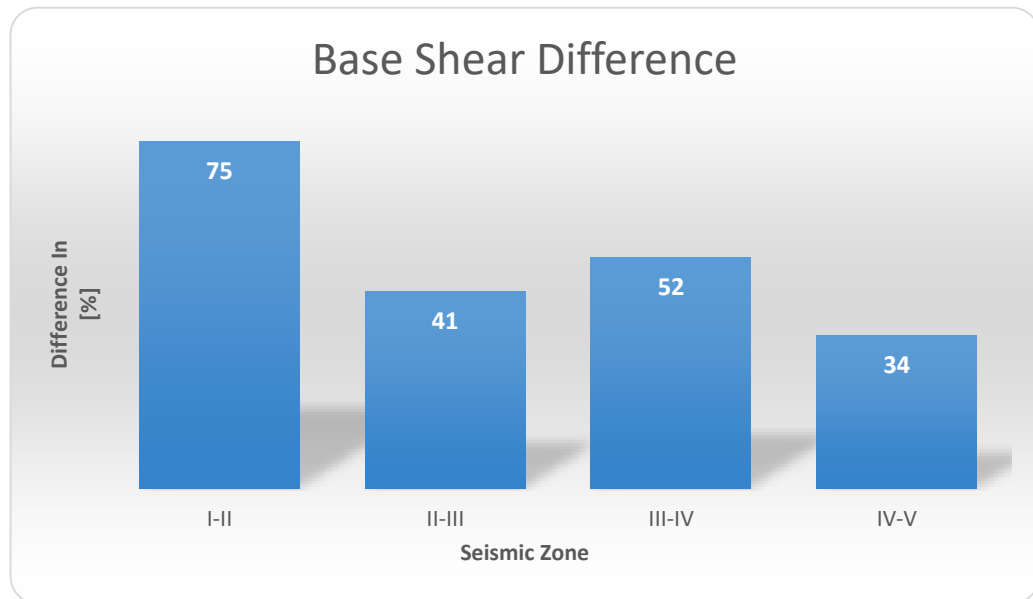


Figure 37: Base Shear Difference [%]

- Seventy five percent [75%] increase in base shear of communication tower, occur from seismic zone one to seismic zone two.
- Forty one percent [41%] increase in base shear of communication tower, occur from seismic zone two to seismic zone three.
- Fifty two percent [52%] increase in base shear of communication tower, occur from seismic zone three to seismic zone four.
- Thirty four percent [34%] increase in base shear of communication tower, occur from seismic zone four to seismic zone five.

CHAPTER FIVE

ALGORITHM AND FLOWCHART FOR COMMUNICATION TOWER DESIGN

5.1 ALGORITHM FOR COMMUNICATION TOWER DESIGN

5.1.1 Input Data

Input data consists of tower dimension wind speed and material property.

Tower dimensions

- Base width (B_w) and top width (T_w) of the four leg tower
- Tower height (H), Straight part height length (H_{str}) and tapered part height length (H_{tpr})
- Panel (modular) heights where $A, B, C, \dots$ are panel designation
- Sub-panel height ($A_1, A_2, \dots, B_4, B_5, \dots, C_7, C_8, \dots$) where, $A, B, C, \dots$ are panel designation under consideration and $1, 2, 3, \dots$ are sub panel number from top to bottom

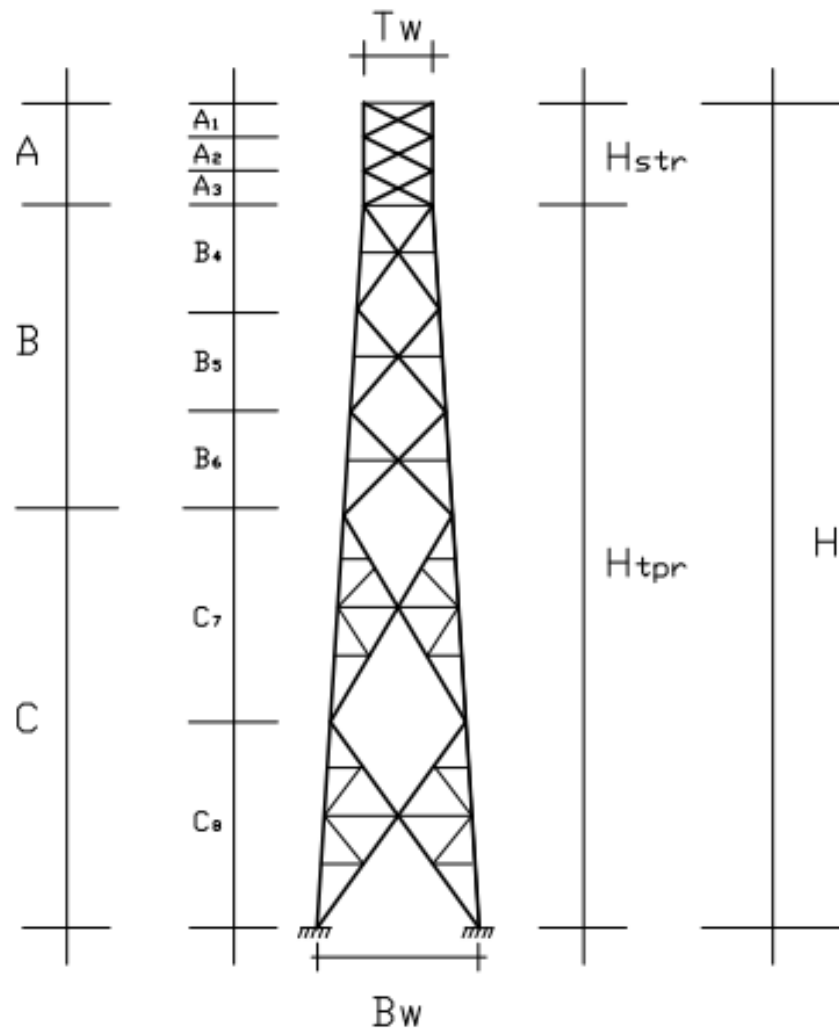


Figure 38: Tower Dimension

ALGORITHM AND FLOWCHART

The above tower dimensions are allowable if:

- Leg member slope angle satisfies equation [3.7]
- The diagonal bracing cross over angle satisfies equation [3.8]

Note that, Panel is set of sub panels having similar vertical height also, Sub Panel is a span between floors of the tower formed by two over crossed diagonal members.

Wind speed

- Basic wind speed (3 second gust, 50 year return period)

Material property

- Equal angle steel grade
- Cross sectional property
- Modulus of elasticity
- Number ,type and elevation of antenna

5.1.2 Analysis

Given the above input data of dimension and material property of self supporting trussed latticed communication tower the analysis will be conducted as follow:

The length of leg and diagonal member are determined from equation [3.2] and [3.3], respectively

Identify the site in which the communication tower is going to be installed.

Having the seismic zone, wind zone and height select the governing load from Figure 35

The structural analysis of self supporting tower for wind and self weight will be conducted simultaneously as follow:

- Wind velocity pressure on tower is determined using equation [3.9]
- The design wind load on structural member, and antenna for each sub panel will be determined using equation [3.13] and [3.17], respectively
- From equation [3.20], the self weight of structural member determined
- Self weight of the antenna is taken from antenna specification
- Total wind load and self weight loads are distributed to each joint using equation [3.21] and [3.22]
- The compressive and tensile axial load on structural member will be determined from equation [3.23] and [3.24] respectively

ALGORITHM AND FLOWCHART

- Appropriate load factor and load combination are applied for allowable strength design from Equation 28 or Equation 29

5.1.3 Designing

Having the axial compressive and axial tensile load strength from the analysis for comparison, the compression member and tension members are designed as follow:

Compression member design

- Flexural buckling of Leg and Diagonal structural member from equation [3.56]
- Flexural and torsional buckling of Leg and Diagonal structural member from equation [3.60]
- If compression members are slender members are designed from equation ,[3.69]
- Bracing and secondary compression member from equation [3.75]

Tension member design

Tension members which are affected due to hole provided for bolt connections are designed for:

- Tension from equation [3.77] and [3.78]
- Shear from equation [3.79], [3.80] and [3.81]
- Bearing and tearout from equation [3.82] and [3.83]
- Bolt tensile and shear strength from equation [3.84]

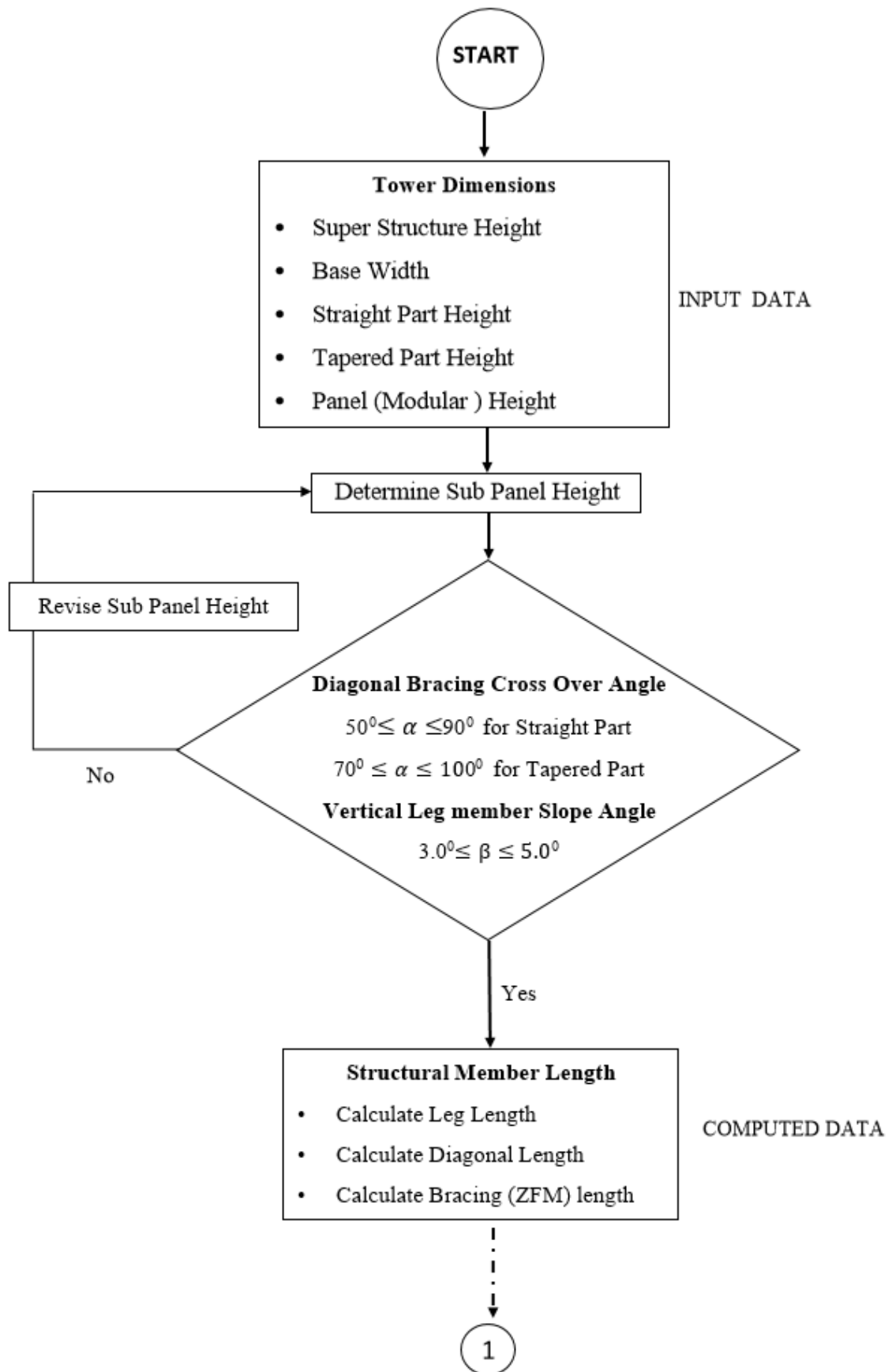
The minimum edge distance, workable gage distance, and minimum spacing are determined from Table 10,

Table 11, and Equation [3.76].

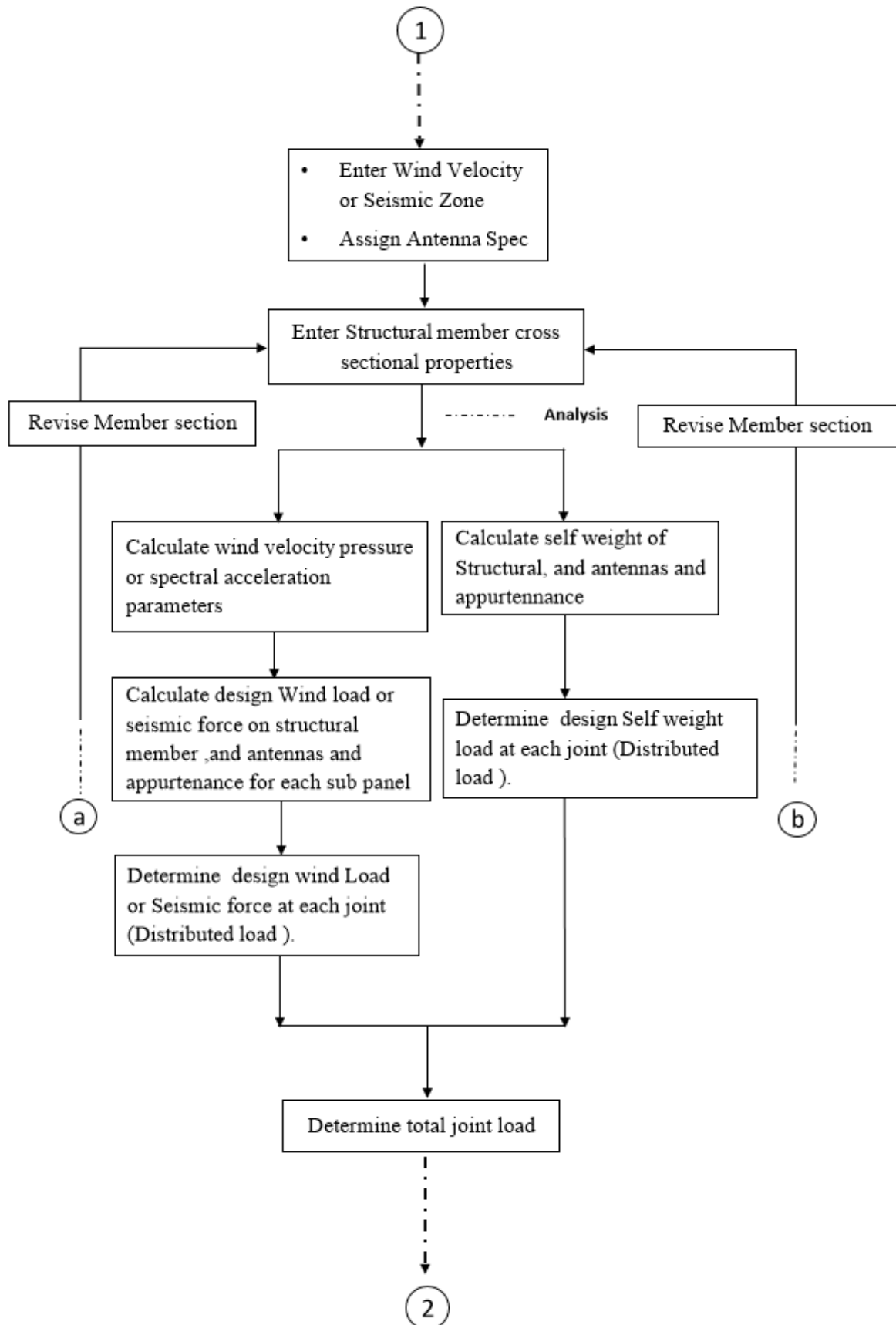
Note that, the gusset plates installed between structural members at bolted connection may be provided with thickness more than the thickness of diagonal members at cross bracings and less than thickness of leg member under connection considered

The above algorithm can be summarized in a flow chart as shown in Figure 39.

ALGORITHM AND FLOWCHART



ALGORITHM AND FLOWCHART



ALGORITHM AND FLOWCHART

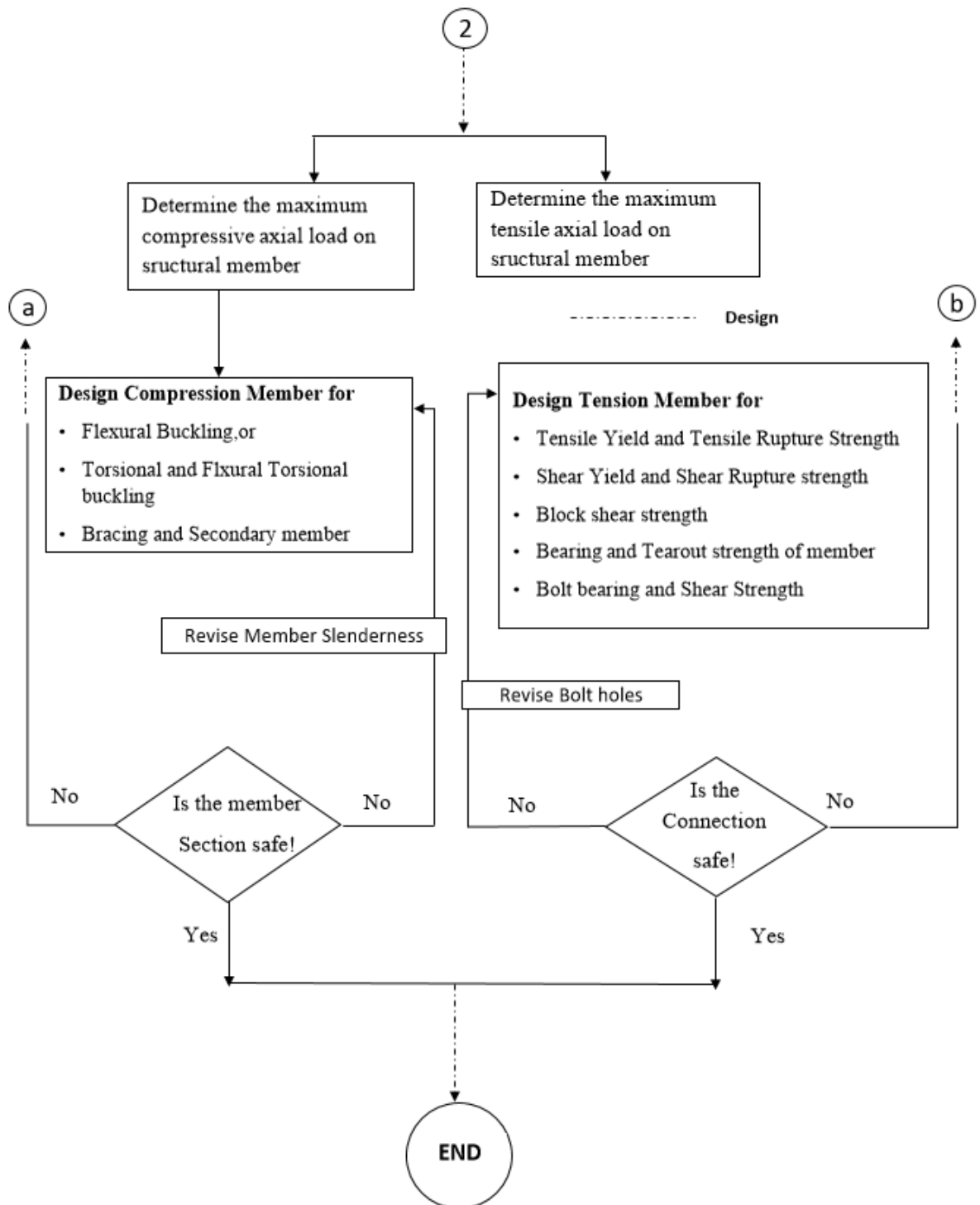


Figure 39: Flowchart for Tower Design

CHAPTER SIX

DESIGN SAMPLE FOR ARTICULATED MODULAR TOWER

Articulated modular towers are trussed towers consisting of segmented sub panels held together by joints to form modular panel. Therefore, design for the articulated modular tower which can represent the design of various height of tower with number of antennas combinations is presented. Tower modular heights are configured in a manner that can be used as other tower height listed in Table 34.

The design of an articulated modular tower with height having 100m is conducted using step by step design procedure as shown below.

Steps for Designing Self-Supporting Communication Tower

- Step 1: Mathematical modeling
- Step 2: Determination of design wind pressure
- Step 3: Determination of design wind load on structural member and antennas
- Step 4: Determination self weight of structural member and antennas
- Step 5: Cumulative wind load and self weight loads at joint
- Step 6: Determination of joint load
- Step 7: Determination of axial compressive and tensile load
- Step 8: Design of compression member for buckling
- Step 9: Design of tension member affected by connection

Note that design for deflection should be conducted based on the specifications of the antennas, that are mounted on the communication towers.

Step1: Mathematical Modeling

- Base Width=14.29m
- Top Width=2m
- Total Modular Height=100m
- Number of Modular Panel=7
- Number of Sub Panel=22
- Number of Dish =3 parabolic antenna with a mass of 126.0 Kg and 4 parabolic antenna with a mass of 170.0Kg.

STEP BY STEP DESIGN SAMPLE

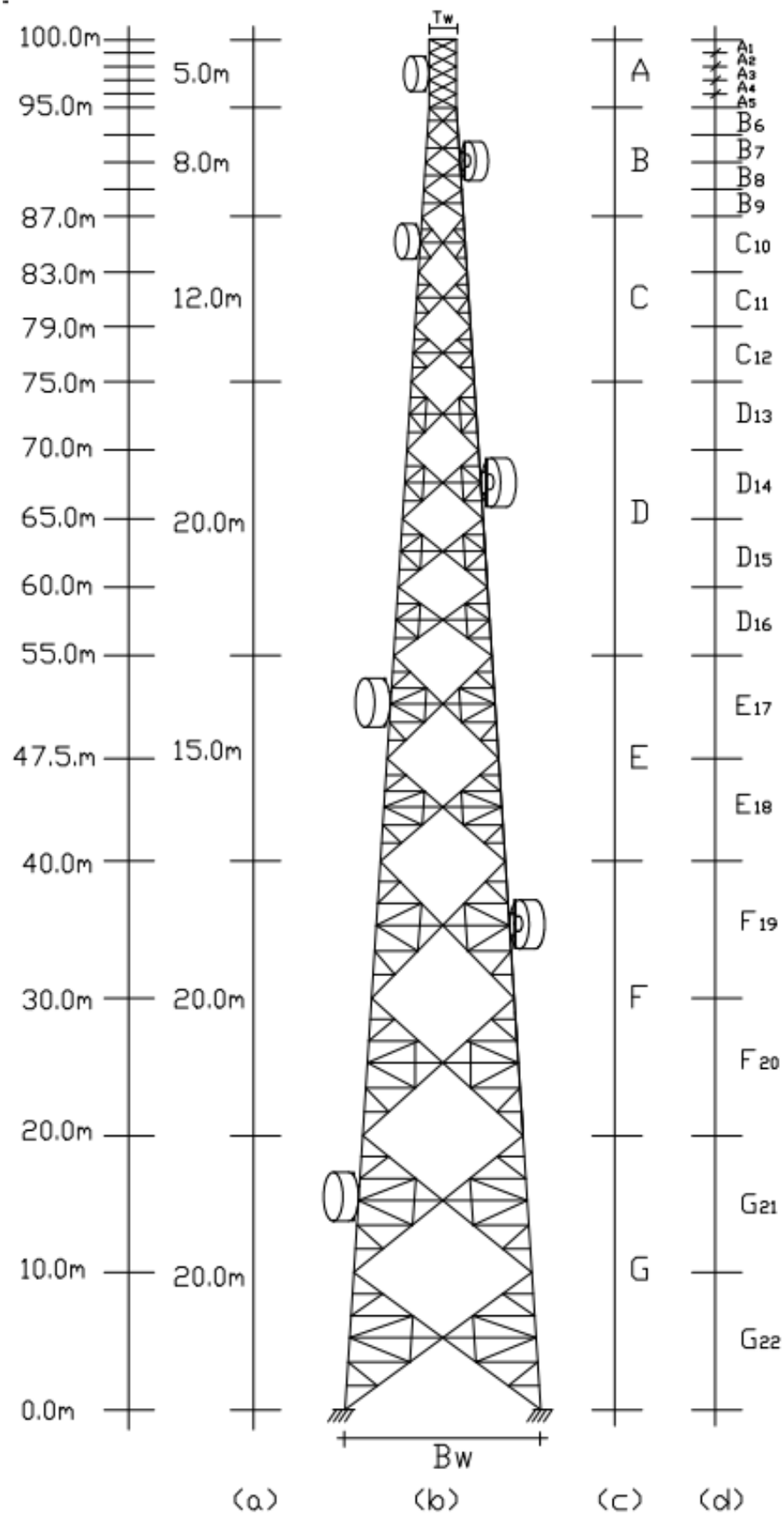


Figure 40:100 meter Self-Supporting Communication Tower, (a) Panel dimension (b) 2D View (c) Panel mark (d) Sub panel mark

STEP BY STEP DESIGN SAMPLE

Table 34: Articulated Modular Towers Geometry and Antenna Specifications

Tower Name	Tower height [m]	Modular Base Width [m]	Top width [m]	Height of Straight [m]	Height of Tapered [m]	Number of antennas considered
SSCT45	45	7.17	2	5	40	4
SSCT60	60	9.11	2	5	55	5
SSCT80	80	11.70	2	5	75	6
SSCT100	100	14.23	2	5	95	7

Equal Angle Steel Section Assigned for leg ,diagonal, Bracing and secondary members. Hot-rolled Equal angle Steel sections used for the design of towers are dip galvanized Low carbon with yield stress of 235 MPa and modulus of elasticity of 210GPa.

Table 35: Tower Steel Section

Panel	Sub panel	Leg Members D×B×t [mm×mm×mm]	Diagonal Members D×B×t [mm×mm×mm]	Horizontal Members D×B×t [mm×mm×mm]	Zero force Member D×B×t [mm×mm×mm]
A	A1	75×75×6	60×60×5	60×60×5	60×60×5
	A2	<<	<<	<<	<<
	A3	<<	<<	<<	<<
	A4	<<	<<	<<	<<
	A5	<<	<<	<<	<<
B	B6	100×100×13	75×75×6	75×75×6	75×75×6
	B7	<<	<<	<<	<<
	B8	<<	<<	<<	<<
	B9	<<	<<	<<	<<
C	C10	150×150×15	75×75×8	75×75×6	75×75×6
	C11	<<	<<	<<	<<
	C12	<<	<<	<<	<<
D	D13	200×200×20	100×100×13	100×100×8	75×75×6
	D14	<<	<<	<<	<<
	D15	<<	<<	<<	<<
	D16	<<	<<	<<	<<
E	E17	200×200×25	100×100×13	100×100×8	75×75×8
	E18	<<	<<	<<	<<
F	F19	250×250×28	100×100×15	100×100×8	75×75×8
	F20	<<	<<	<<	<<
G	G21	250×250×35	120×120×15	100×100×8	75×75×8
	G22	<<	<<	<<	<<

STEP BY STEP DESIGN SAMPLE

Step 2: Determination of Design Wind Pressure

Design wind pressure Q_z at the centroid of sub panel determined according to recommendation of TIA 222 G, [9] and [10]. Wind Speed of 44m/s is used to calculate the wind pressure from equation [3.9] [3.10] [3.11] and Table 3.

Table 36: Velocity Pressure

Part	Panel	Sub Panel	Panel centroid height from Tower base Z [m]	Velocity pressure coefficient K_z	Velocity Pressure Q_z [N/m ²]
Straight part	A	A1	99.50	1.39	1607.17
		A2	98.50	1.38	1602.54
		A3	97.50	1.38	1597.87
		A4	96.50	1.37	1593.17
		A5	95.50	1.37	1588.44
Tapered Part	B	B6	93.98	1.36	1581.17
		B7	91.98	1.35	1571.50
		B8	89.98	1.35	1561.67
		B9	87.99	1.34	1551.68
	C	C10	84.95	1.32	1536.18
		C11	80.95	1.31	1515.19
		C12	76.96	1.29	1493.44
	D	D13	72.45	1.27	1467.87
		D14	67.45	1.24	1438.22
		D15	62.46	1.21	1406.95
		D16	57.46	1.18	1373.83
	E	E17	51.17	1.15	1329.07
		E18	43.68	1.10	1270.30
	F	F19	34.89	1.03	1191.31
		F20	24.90	0.93	1081.89
	G	G21	14.91	0.81	934.45
		G22	4.92	0.59	680.75

STEP BY STEP DESIGN SAMPLE

Step 3: Determination of Design Wind Load on Structural member and antenna

- The design wind load F_S on structural member, shall be determined in accordance with the equation [3.13].
- The effective projected area of members EPA_S determined using equation [3.14]
- The design wind load F_A on antenna, shall be determined in accordance with the equation [3.17].
- The effective projected area of antenna EPA_A determined using equation [3.19]

Table 37: Projected areas of Structural members

Sub Panel	No Leg Exposed to wind	Leg Length V [m]	Leg Wind obstruction area A_L [m ²]	No Diagonal Exposed to wind	Diagonal Length D [m]	Diagonal Wind obstruction area A_D [m ²]
A1	2	1.00	0.15	2	2.24	0.27
A2	2	1.00	0.15	2	2.24	0.27
A3	2	1.00	0.15	2	2.24	0.27
A4	2	1.00	0.15	2	2.24	0.27
A5	2	1.00	0.15	2	2.24	0.27
B6	2	2.00	0.40	2	2.92	0.44
B7	2	2.00	0.40	2	3.12	0.47
B8	2	2.00	0.40	2	3.32	0.50
B9	2	2.00	0.40	2	3.53	0.53
C10	2	4.01	1.20	2	5.19	0.78
C11	2	4.01	1.20	2	5.53	0.83
C12	2	4.01	1.20	2	5.90	0.88
D13	2	5.01	2.00	2	7.02	1.40
D14	2	5.01	2.00	2	7.48	1.50
D15	2	5.01	2.00	2	7.98	1.60
D16	2	5.01	2.00	2	8.49	1.70
E17	2	7.53	3.01	2	10.73	2.15
E18	2	7.53	3.01	2	11.44	2.29
F19	2	10.02	5.01	2	13.99	2.80
F20	2	10.02	5.01	2	14.92	2.98
G21	2	10.02	5.01	2	15.90	3.82
G22	2	10.02	5.01	2	16.93	4.06

STEP BY STEP DESIGN SAMPLE

Table 38: Projected areas of Structural members

Sub Panel	Number of Horizontal Exposed to wind	Horizontal Length at a joint H [m]	Horizontal Wind obstruction area A_H [m ²]	No Horizontal at cross over Exposed to wind	Horizontal length at cross over H_C [m]	Horizontal length at cross over Wind obstruction area A_{HC} [m ²]	Zero Force Member Area ZFM_H [m ²]
A1	1	2.00	0.12	-	2.00	0.00	0.00
A2	-	2.00	-	-	2.00	0.00	0.00
A3	-	2.00	-	1	2.00	0.12	0.00
A4	-	2.00	-	-	2.00	0.00	0.00
A5	1	2.00	0.12	-	2.00	0.00	0.00
B6	-	2.00	-	1	2.12	0.16	0.00
B7	-	2.26	-	1	2.38	0.18	0.00
B8	-	2.52	-	1	2.64	0.20	0.00
B9	-	2.78	-	1	2.90	0.22	0.00
C10	-	3.03	-	1	3.27	0.25	0.94
C11	-	3.55	-	1	3.79	0.28	1.00
C12	-	4.07	-	1	4.31	0.32	1.07
D13	-	4.59	-	1	4.89	0.49	2.04
D14	-	5.23	-	1	5.54	0.55	2.20
D15	-	5.88	-	1	6.19	0.62	2.38
D16	-	6.53	-	1	6.84	0.68	2.55
E17	-	7.17	-	1	7.63	0.76	3.13
E18	-	8.15	-	1	8.60	0.86	3.38
F19	-	9.12	-	1	9.72	0.97	4.06
F20	-	10.41	-	1	11.02	1.10	4.39
G21	-	11.70	-	1	12.32	1.23	4.74
G22	-	13.00	-	1	13.61	1.36	5.08

STEP BY STEP DESIGN SAMPLE

Table 39: Design wind Load on Structural members

Sub Panel	Wind Load On leg F_L [N]	Wind Load On Diagonal F_D [N]	Wind Load On Horizontal at cross over point F_{HC} [N]	Wind Load On Zero Force Members F_Z [N]	Wind Load On Horizontal at joints F_H [N]	Wind Load on Structure F_S [N]
A1	553.62	990.35	-	-	442.90	1986.87
A2	600.90	1074.93	-	-	-	1675.83
A3	550.42	984.62	440.34	-	-	1975.38
A4	597.39	1068.64	-	-	-	1666.03
A5	547.17	978.81	-	-	437.74	1963.72
B6	1528.07	1672.26	606.57	-	-	3806.90
B7	1552.08	1810.90	691.49	-	-	4054.47
B8	1569.20	1949.71	775.29	-	-	4294.19
B9	1581.10	2088.59	857.87	-	-	4527.56
C10	4419.01	2859.92	902.18	3439.19	-	11620.30
C11	4499.86	3104.79	1064.69	3744.39	-	12413.74
C12	4543.16	3343.51	1222.16	4032.07	-	13140.90
D13	7021.35	4916.02	1712.96	7135.95	-	20786.27
D14	7041.55	5258.37	1946.02	7744.84	-	21990.77
D15	7014.74	5582.91	2165.70	8314.40	-	23077.76
D16	6949.78	5886.98	2370.50	8841.52	-	24048.78
E17	10782.53	7685.92	2731.91	11210.74	-	32411.09
E18	10459.40	7950.79	2988.27	11753.52	-	33151.97
F19	16708.69	9331.19	3241.14	13534.45	-	42815.47
F20	15387.90	9165.79	3383.79	13492.02	-	41429.50
G21	13334.48	10158.64	3277.59	12601.76	-	39372.48
G22	9801.29	7947.81	2662.83	9943.80	-	30355.74

Table 40: Design wind Load on Antennas

Sub panel	Antenna diameter [m]	Width of antenna [m]	Area A_N [m ²]	Area A_T [m ²]	Area of antenna (EPA) A_A [m ²]	Wind pressure on antenna [N/m ²]	Wind load on antenna F_A [N]
A3	2.00	1.10	3.77	2.20	2.64	1597.87	3580.38
B7	2.00	1.10	3.77	2.20	2.64	1571.50	3521.27
C10	2.00	1.10	3.77	2.20	2.64	1536.18	3442.13
D14	2.40	1.38	5.43	3.30	3.89	1438.22	4752.33
E17	2.40	1.38	5.43	3.30	3.89	1329.07	4391.67
F19	2.40	1.38	5.43	3.30	3.89	1191.31	3936.48
G21	2.40	1.38	5.43	3.30	3.89	934.45	3087.73

STEP BY STEP DESIGN SAMPLE

Step 4: Self weight of Structural member and antennas

i. Self Weight of Structure

Self weight of the structural members is the product of length and mass per length of the section calculated using equation [3.20].

Table 41: Self-Weight of Structural members

Sub Panel	Self-weight of Leg Member SW_L [N]	Self-weight of Diagonal Member SW_D [N]	Self-weight of Horizontal at cross over point SW_{HC} [N]	Self-weight of Horizontal Member SW_H [N]	Self-weight of Zero Force Member SW_Z [N]	Total Self-weight S_W [N]
A1	269.58	798.46	-	357.08	-	1425.13
A2	269.58	798.46	-	-	-	1068.04
A3	269.58	798.46	357.08	-	-	1425.13
A4	269.58	798.46	-	-	-	1068.04
A5	269.58	798.46	-	357.08	-	1425.13
B6	1502.10	1576.70	571.91	-	-	3650.72
B7	1502.10	1681.01	641.89	-	-	3825.01
B8	1502.10	1790.12	711.83	-	-	4004.05
B9	1502.10	1903.20	781.73	-	-	4187.03
C10	5316.34	3676.82	882.43	-	840.98	10716.56
C11	5316.34	3919.91	1022.67	-	899.16	11158.08
C12	5316.34	4181.08	1162.74	-	959.01	11619.16
D13	11776.94	10517.01	2340.74	-	1830.34	26465.03
D14	11776.94	11217.13	2651.58	-	1980.82	27626.48
D15	11776.94	11954.96	2962.18	-	2134.62	28828.70
D16	11776.94	12723.92	3272.61	-	2291.17	30064.63
E17	21740.89	16086.75	3652.28	-	3697.77	45177.69
E18	21740.89	17155.22	4118.43	-	3996.57	47011.10
F19	40894.89	24046.09	4652.87	-	4793.70	74387.55
F20	40894.89	25647.23	5274.60	-	5188.83	77005.55
G21	46399.97	33202.12	5895.82	-	5592.78	91090.68
G22	46399.97	35340.30	6516.68	-	6004.01	94260.96

STEP BY STEP DESIGN SAMPLE

ii. Self Weight of Antenna

The self weight of antennas and appurtenance which can be fixed along tower height taken from antenna manual. Two types of antenna with net weight of 126 kg and 170kg are used at different panels [15].

Table 42: Self weight of antennas

Sub Panel	Number of Antenna	Mass of Antenna [Kg]	Weight of Antenna at the panel [N]
A3	1	126.00	1236.06
B7	1	126.00	1236.06
C10	1	126.00	1236.06
D14	1	170.00	1667.70
E17	1	170.00	1667.70
F19	1	170.00	1667.70
G21	1	170.00	1667.70

Step 5: Determination of Joint load

i. Wind load at a joint

Based on the assumptions drawn, for the distributing wind load to joints

Wind load at joint

$$= \frac{(F_L + F_D + F_Z) \text{ above joint}}{8} + \frac{(F_H) \text{ at joint}}{4} + \frac{(F_L + F_D + F_Z) \text{ below joint}}{8}, \text{ rep.3.21}$$

Table 43: Joint wind load due to Structural members

Sub Panel	Joint above sub panel	Wind load $F_L + F_D + F_Z$ on joint from panel above [N]	Wind load from Horizontal at the joint [N]	Wind load $F_L + F_D + F_Z$ on joint from panel below [N]	Wind load at each joints without antenna [N]
A1	1,2,3,4	0.00	125.85	221.95	347.80
A2	5,6,7,8	221.95	-	240.90	462.85
A3	9,10,11,12	240.90	-	283.96	524.86
A4	13,14,15,16	283.96	-	239.49	523.45
A5	17,18,19,20	239.49	-	219.36	458.85
B6	21,22,23,24	219.36	125.85	547.24	892.45
B7	25,26,27,28	547.24	-	582.83	1130.07
B8	29,30,31,32	582.83	-	617.29	1200.12
B9	33,34,35,36	617.29	-	650.84	1268.13
C10	37,38,39,40	650.84	-	1670.42	2321.25
C11	41,42,43,44	1670.42	-	1784.48	3454.89
C12	45,46,47,48	1784.48	-	1889.00	3673.48

STEP BY STEP DESIGN SAMPLE

Table 43: Joint wind load due to structural membercont'd

Sub Panel	Joint above sub panel	Wind load $F_L + F_D + F_Z$ on joint from panel above [N]	Wind load from Horizontal at the joint [N]	Wind load $F_L + F_D + F_Z$ on joint from panel below [N]	Wind load at each joints without antenna [N]
D13	49,50,51,52	1889.00	-	2988.03	4877.03
D14	53,54,55,56	2988.03	-	3161.17	6149.20
D15	57,58,59,60	3161.17	-	3317.43	6478.60
D16	61,62,63,64	3317.43	-	3457.01	6774.44
E17	65,66,67,68	3457.01	-	4659.09	8116.11
E18	69,70,71,72	4659.09	-	4765.60	9424.69
F19	73,74,75,76	4765.60	-	6154.72	10920.32
F20	77,78,79,80	6154.72	-	5955.49	12110.21
G21	81,82,83,84	5955.49	-	5659.79	11615.28
G22	85,86,87,88	5659.79	-	4363.64	10023.43
Ground	89,90,91,92	4363.64	-	-	4363.64

Table 44: Antenna wind load at Joint

Sub Panel	Joint above sub panel	Wind Load on Antenna F_A [N]	Antenna wind load at each joints F_{AJ} [N]
A3	9,10,11,12	3580.38	895.09
B7	25,26,27,28	3521.27	880.32
C10	37,38,39,40	3442.13	860.53
D14	53,54,55,56	4752.33	1188.08
E17	65,66,67,68	4391.67	1097.92
F19	73,74,75,76	3936.48	984.12
G21	81,82,83,84	3087.73	771.93

ii. Self weight Load to Joints

Based on the assumptions drawn, for the distributing self weight load to joints

Self Weight Load at joint

$$= \frac{(SW_L + SW_D + SW_Z) \text{ above joint}}{8} + \frac{(SW_H) \text{ at joint}}{4} + \frac{(SW_L + SW_D + SW_Z) \text{ below joint}}{8},$$

rep.[3.22]

STEP BY STEP DESIGN SAMPLE

Table 45: Structural members self weight load at joint

Sub Panel	Joint above Sub panel	Self-weight load $SW_L + SW_D + SW_Z$ on joint from panel above [N]	Self-weight Horizontal at the joint [N]	Self-weight load $SW_L + SW_D + SW_Z$ on joint from panel below [N]	Self-Weight on each four joints [N]
A1	1,2,3,4	0.00	102.66	153.53	256.19
A2	5,6,7,8	153.53	-	153.53	307.06
A3	9,10,11,12	153.53	-	204.86	358.39
A4	13,14,15,16	204.86	-	153.53	358.39
A5	17,18,19,20	153.53	102.66	153.53	409.72
B6	21,22,23,24	153.53	-	524.79	678.32
B7	25,26,27,28	524.79	-	549.84	1074.64
B8	29,30,31,32	549.84	-	575.58	1125.43
B9	33,34,35,36	575.58	-	601.89	1177.47
C10	37,38,39,40	601.89	-	1540.51	2142.39
C11	41,42,43,44	1540.51	-	1603.97	3144.48
C12	45,46,47,48	1603.97	-	1670.25	3274.23
D13	49,50,51,52	1670.25	-	3804.35	5474.60
D14	53,54,55,56	3804.35	-	3971.31	7775.65
D15	57,58,59,60	3971.31	-	4144.13	8115.43
D16	61,62,63,64	4144.13	-	4321.79	8465.92
E17	65,66,67,68	4321.79	-	6494.29	10816.08
E18	69,70,71,72	6494.29	-	6757.85	13252.14
F19	73,74,75,76	6757.85	-	10693.21	17451.06
F20	77,78,79,80	10693.21	-	11069.55	21762.76
G21	81,82,83,84	11069.55	-	13094.29	24163.83
G22	85,86,87,88	13094.29	-	13550.01	26644.30
Ground	89,90,91,92	13550.01	-	0.00	13550.01

Table 46: Antenna self weight load at joint

Sub Panel	Joint above Sub Panel	Weight of Antenna at the panel [N]	Weight of Antenna at the joints [N]
A3	9,10,11,12	1236.06	309.02
B7	25,26,27,28	1236.06	309.02
C10	37,38,39,40	1236.06	309.02
D14	53,54,55,56	1667.70	416.93
E17	65,66,67,68	1667.70	416.93
F19	73,74,75,76	1667.70	416.93
G21	81,82,83,84	1667.70	416.93

STEP BY STEP DESIGN SAMPLE

The total wind load due to wind on structural member at a joint, and wind on antenna and appurtenance are determined and tabulated in Table 47.

Similarly, the total self weight due to the self weight of structural members at a joint, and self weight of antennas at a joints are determined and tabulated in Table 47 below.

Table 47: Wind load and Self-weight load of Truss and Antenna at each joints

Sub Panel	Joint above Sub panel	Antenna wind load on joints [N]	Structure Wind load at each joints [N]	Wind load on joints from Truss and antenna [N]	Weight of Antenna at joints [N]	Self-Weight of structure at each joints [N]	Self-weight of Truss and Antenna at each joints [N]
A1	1,2,3,4	-	347.80	347.80	-	256.19	256.19
A2	5,6,7,8	-	462.85	462.85	-	307.06	307.06
A3	9,10,11,12	895.09	524.86	1419.96	309.02	358.39	667.41
A4	13,14,15,16	-	523.45	523.45	-	358.39	358.39
A5	17,18,19,20	-	458.85	458.85	-	409.72	409.72
B6	21,22,23,24	-	892.45	892.45	-	678.32	678.32
B7	25,26,27,28	880.32	1130.07	2010.39	309.02	1074.64	1383.65
B8	29,30,31,32	-	1200.12	1200.12	-	1125.43	1125.43
B9	33,34,35,36	-	1268.13	1268.13	-	1177.47	1177.47
C10	37,38,39,40	860.53	2321.25	3181.79	309.02	2142.39	2451.41
C11	41,42,43,44	-	3454.89	3454.89	-	3144.48	3144.48
C12	45,46,47,48	-	3673.48	3673.48	-	3274.23	3274.23
D13	49,50,51,52	-	4877.03	4877.03	-	5474.60	5474.60
D14	53,54,55,56	1188.08	6149.20	7337.28	416.93	7775.65	8192.58
D15	57,58,59,60	-	6478.60	6478.60	-	8115.43	8115.43
D16	61,62,63,64	-	6774.44	6774.44	-	8465.92	8465.92
E17	65,66,67,68	1097.92	8116.11	9214.02	416.93	10816.08	11233.01
E18	69,70,71,72	-	9424.69	9424.69	-	13252.14	13252.14
F19	73,74,75,76	984.12	10920.32	11904.44	416.93	17451.06	17867.98
F20	77,78,79,80	-	12110.21	12110.21	-	21762.76	21762.76
G21	81,82,83,84	771.93	11615.28	12387.22	416.93	24163.83	24580.76
G22	85,86,87,88	-	10023.43	10023.43	-	26644.30	26644.30
Ground	89,90,91,92	-	4363.64	4363.64	-	13550.01	13550.01

STEP BY STEP DESIGN SAMPLE

Step 6: Cumulative Wind Load and Self Weight Loads at Joint

For two dimensional parallel wind load analysis the side face of the critical panel parallel to wind direction is selected because diagonal members along the face normal to wind are not affected by parallel wind load.

The cumulative self weight of the structure up to the top joint of the critical sub panel leg and cumulative wind load up to top joint of critical sub panel are determined to calculate the total axial compressive and tensile load on structural member. The compressive axial load is determined only for the critical sub panel.

Table 48: parallel wind and self-weight load at a critical panel

Sub Panel	Joints along Side face above sub panel	Self-weight $\sum SW_{VC}^*$ [N]	Wind Load F_{HC}^{**} [N]	Wind load $2F_{HC}^{***}$ [N]	Remark
A1	2,3	256.19	347.80	695.59	-
A2	6,7	563.26	462.85	1621.28	-
A3	10,11	1230.66	1419.96	4461.19	-
A4	14,15	1589.06	523.45	5508.10	-
A5	18,19	1998.78	458.85	6425.80	Critical panel
B6	22,23	2677.10	892.45	8210.71	-
B7	26,27	4060.75	2010.39	12231.49	-
B8	30,31	5186.18	1200.12	14631.73	-
B9	34,35	6363.65	1268.13	17167.98	Critical panel
C10	38,39	8815.05	3181.79	23531.56	-
C11	42,43	11959.53	3454.89	30441.35	-
C12	46,47	15233.76	3673.48	37788.30	Critical panel
D13	50,51	20708.36	4877.03	47542.37	-
D14	54,55	28900.94	7337.28	62216.93	-
D15	58,59	37016.37	6478.60	75174.14	-
D16	62,63	45482.29	6774.44	88723.01	Critical panel
E17	66,67	56715.30	9214.02	107151.06	-
E18	70,71	69967.44	9424.69	126000.44	Critical panel
F19	74,75	87835.42	11904.44	149809.32	-
F20	78,79	109598.18	12110.21	174029.74	Critical panel
G21	82,83	134178.93	12387.22	198804.18	-
G22	86,87	160823.23	10023.43	218851.04	Critical panel
Ground	90,91	174373.24	4363.64	227578.32	-

*Cumulative Self-weight of tower at each joint at side face of towers

** Horizontal wind load at each joint on the side face of tower

*** Cumulative Horizontal wind load on the side face of critical panel

STEP BY STEP DESIGN SAMPLE

Step 7: Axial Compressive and Tensile Load

The axial compressive force on the leg and diagonal members are calculated for the critical sub panels of modularized panels due to wind and self-weight using the diagonal and parallel wind load analysis methods.

For parallel wind load, the critical member's cumulative horizontal wind load at the top of critical sub panel and moment at the centroid of critical sub panel face along the direction of wind load are determined to get the compressive axial load

When the wind load act parallel to the normal face of the tower the diagonals are subjected to higher axial load rather than wind acts in the diagonal direction because only four diagonals parallel to wind directions are exposed to wind.

Similarly, when the wind load act in parallel or normal to face of tower the legs are subjected to a lower axial load because all of four legs are exposed to wind.

Table 49: Leg and Diagonal Member's Compressive Axial Load for Parallel Wind

Sub Panel	Wind Load $2F_{CH}$ [kN]	Moment M_{CH}^* [kNm]	Lever arm r_d^{**} [m]	Lever arm r_l^{***} [m]	lever arm H^{****} [m]	Compressive force on leeward leg F_4 [kN]	Compressive force on diagonal bracing F_6 [kN]
A5	6.425	15.499	0.89	2.00	1.00	-7.749	-3.592
B9	17.167	106.282	1.72	2.90	2.00	-36.651	-9.976
C12	37.788	416.169	3.11	4.31	4.00	-96.488	-24.275
D16	88.723	1640.569	4.23	6.84	5.00	-239.987	-52.442
E18	126.000	3297.114	5.98	8.60	7.50	-383.257	-79.024
F20	174.029	5992.370	7.85	11.02	10.00	-543.873	-110.843
G22	218.851	9945.140	8.45	13.61	10.00	-730.589	-129.530

* Moment at the Centroid of critical Panel

** Lever arm Diagonal tensile member to the toe face

*** Lever arm of tensile Leg and comp Leg force member

**** Lever arm of horizontal load to the toe face

STEP BY STEP DESIGN SAMPLE

When the wind load act diagonally to the normal face of the tower the diagonals are subjected to lower axial load rather than wind acts in the parallel direction because all of eight diagonals members are exposed in a diagonal direction to wind load

Similarly, when the wind load act diagonally at forty five degree to the face of the tower the legs are subjected to a higher axial load because only two legs are exposed to wind.

Table 50: Leg and Diagonal Member's Compressive Axial Load for Diagonal Wind

Sub Panel	Effective Antenna Area EPA [m ²]	Diagonal wind load on antenna [kN]	Cumulative wind load at the top of critical panel face $2F_{CH}$ [kN]	Moment at the Centroid of critical Panel face M_{CHD} [kNm]	Horizontal Diagonal length of tower W_d [m]	Compressive force on leeward leg F_4 [kN]	compressive force on diagonal bracing F_6 [kN]
A3	3.77	7.228	-	-	-	-	-
A5	-	-	18.354	156.077	2.83	-55.181	-3.627
B7	3.77	7.109	-	-	-	-	-
B9	-	-	47.019	476.456	4.11	-115.874	-9.660
C10	3.77	6.949	-	-	-	-	-
C12	-	-	96.443	1303.466	6.13	-212.675	-21.904
D14	5.43	9.369	-	-	-	-	-
D16	-	-	225.236	4288.014	9.70	-442.226	-47.070
E17	5.43	8.658	-	-	-	-	-
E18	-	-	318.090	8263.722	12.22	-676.372	-70.533
F19	5.43	7.760	-	-	-	-	-
F20	-	-	436.397	14732.337	15.65	-941.177	-98.271
G21	5.43	6.087	-	-	-	-	-
G22	-	-	546.350	24218.983	19.31	-1254.297	-114.328

STEP BY STEP DESIGN SAMPLE

Step 8: Design of Compression Member for Buckling

For modular tower the critical sub panel's leg and diagonal members are checked for Flexural, Torsional, or Flexural Torsional buckling due to axial load from wind and self weight of the structure using Allowable stress design method.

The Maximum compressive force on the leg member is the sum of the cumulative self weight and compressive axial load because of horizontal load on the sub panel. Also, the Maximum Tensile or compressive axial on the Diagonal member is equal to the axial load due to horizontal load.

Table 51: Leg Axial Compressive Strength AISC 360-16, ASCE 10-15 Based on Flexural Buckling

Sub Panel	Required Compressive Load P_a [kN]	Slenderness ratio l/r	Effective Slenderness ratio L_c/r	Critical stress F_{cr} [MPa]	Gross cross Sectional area A_g [mm ²]	Strength P_n [kN]	Allowable Strength P_n/Ω_c [kN]	Remark $P_n/\Omega_c > P_a$
A5	-56	34	77	198	876	-173	-104	Ok!
B9	-120	51	86	183	2450	-448	-269	Ok!
C12	-222	68	94	168	4320	-727	-436	Ok!
D16	-470	63	92	172	7660	-1317	-790	Ok!
E18	-718	96	108	143	9430	-1354	-812	Ok!
F20	-1007	102	111	138	13300	-1839	-1103	Ok!
G22	-1351	102	111	138	16400	-2262	-1357	Ok!

Table 52: Diagonal Axial Compressive Strength; AISC 360-16, ASCE 10-15 Based on Flexural Buckling

Sub Panel	Design Compressive or tensile load [kN]	Slenderness ratio l/r	Effective Slenderness ratio L_c/r	Critical stress F_{cr} [MPa]	Gross cross Sectional area A_g [mm ²]	Strength P_n [kN]	Allowable Strength P_n/Ω_c [kN]	Remark $P_n/\Omega_c > P_a$
A5	-4	94	107	146	586	-85	-51	Ok!
B9	-10	120	120	123	876	-108	-65	Ok!
C12	-24	101	111	140	1150	-160	-96	Ok!
D16	-52	109	114	133	2450	-326	-195	Ok!
E18	-79	147	147	84	2450	-207	-124	Ok!
F20	-111	128	128	110	2800	-309	-185	Ok!
G22	-130	121	121	123	3400	-417	-250	Ok!

STEP BY STEP DESIGN SAMPLE

Step 9: Design of Tension Member Affected by Connection

a) Leg Splice Connection Design

- Tensile strength of leg splice connection is conducted according to equation [3.77] for *yielding* and [3.78] for *rupture*
Net area subjected to tension is calculated from equation [3.47]
- Shear strength of leg splice connection is conducted according to equation [3.81] for *block shear*.
- For block shear strength, net area subjected to shear (A_{nv}), net area subjected to tension (A_{nt}) and gross area subjected to shear (A_{gv}) are calculated using equation shown below

$$A_{gv} = [(n_{bv} - 1)S + L_e]t$$

$$A_{nv} = [(n_{bv} - 1)S + L_e - (n_{bv} - 0.5)d_h]t$$

$$A_{nt} = \min[(A_{nt1}, A_{nt2})]$$

$$A_{nt1} = [(\sum_{i=1}^4 g_i + e) - (\frac{n_l}{2} - 0.5)d_h + (\sum_{i=1}^4 \frac{p_i^2}{4g_i})]t$$

$$A_{nt2} = [(\sum_{i=1}^4 g_i + e) - 0.5d_h]t$$

$$e = w - \sum_{i=1}^4 g_i$$

n_{bv} : Number of bolt in longitudinal direction in the line nearest to shear center

Equivalent workable *gages* length g_1, g_2, g_3 , and g_4 of the equal angle steel for the provision of bolt holes are referred from table [10] based on their width.

Pitch distance (p) of bolt in transverse loading is taken as half the spacing (S) between bolts.

Width of equal angle steel section is taken from panel input data

Bolts in leg and splice connections are subjected to shear in double plane and should be arranged in staggered manner.

- Bearing and tearout strength of leg splice connection is conducted according to equation [3.82] for *tearout* and [3.83] for *bearing*
- Bolt tensile and shear strength of leg splice connection is conducted according to equation [3.84]. Bolts provided in two leg connected by splice should be arranged in symmetrical arrangement

STEP BY STEP DESIGN SAMPLE

i. Panel G (Bottom Panel)

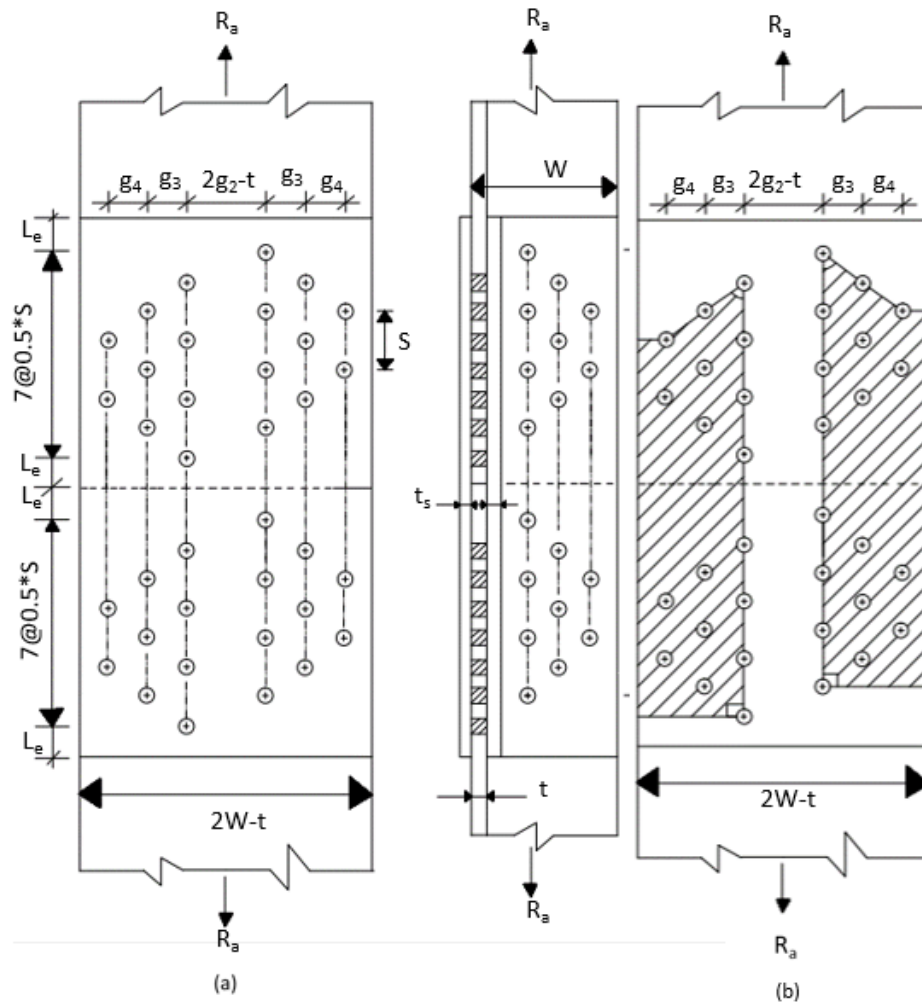


Figure 41: Panel G (a) Splice Connection View (b) Block Shear Path

Table 53: Panel G splice connection Design

Panel G (Critical Sub Panel)	Values	Comment $R_n/\Omega > R_a$
Equal angle steel, Leg width W & thickness t [mm]	250/35	-
Thickness of splice t_s [mm]	37	-
Bolt Grade A325N f_{nv} [Mpa]	372.00	-
Diameter of bolt d_b [mm]	20.00	-
Required Strength R_a [kN]	1350.79	-
Available Tensile Strength R_n/Ω [kN]	2382.05	Ok!
Available Block Shear strength R_n/Ω [kN]*	1505.23	Ok!
Available Bearing & Tearout Strength R_n/Ω [kN]	6048.00	Ok!
Available Bolt Strength (Shear bearing) R_n/Ω [kN]	2103.61	Ok!

STEP BY STEP DESIGN SAMPLE

ii. Panel F

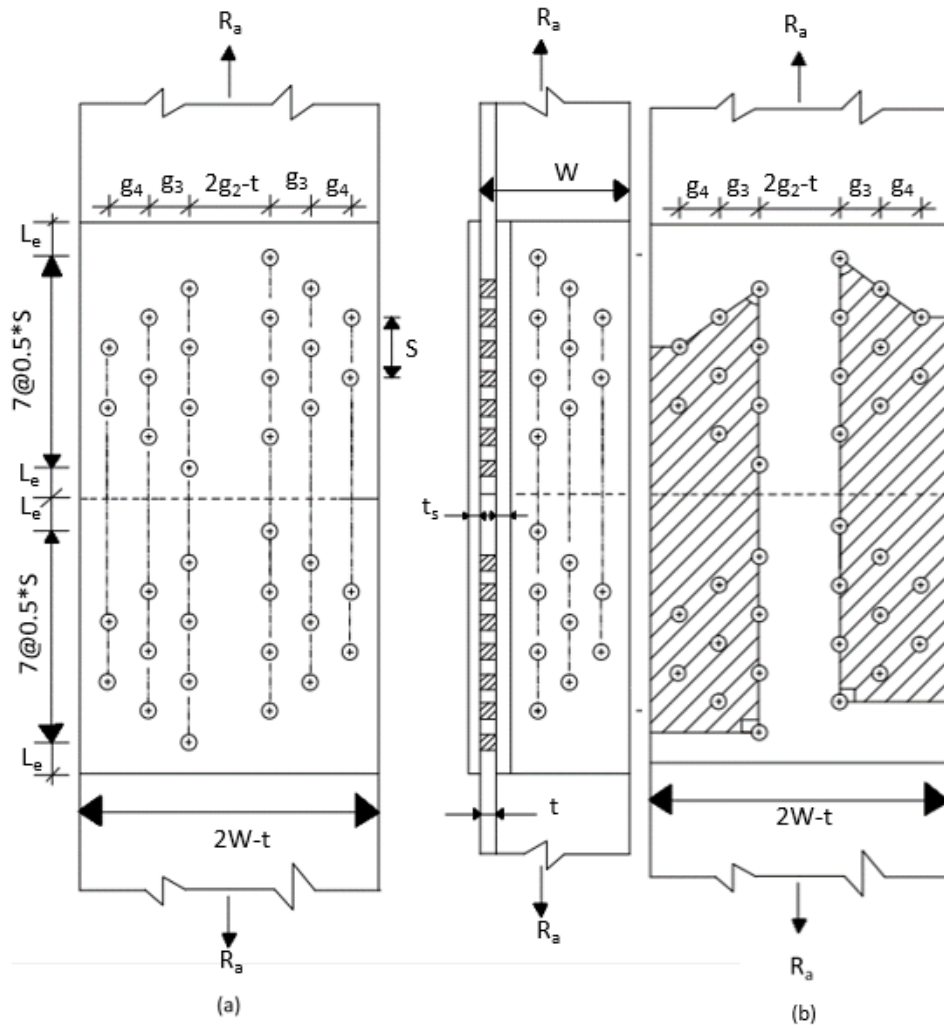


Figure 42: Panel F (a) Splice Connection View (b) Block Shear Path

Table 54: Panel F splice connection Design

Panel F (Critical Sub Panel)	Values	Comment $R_n/\Omega > R_a$
Equal angle steel, Leg width W & thickness t [mm]	250/28	-
Thickness of splice t_s [mm]	30	-
Bolt Grade A325N f_{nv} [Mpa]	372.00	-
Diameter of bolt d_b [mm]	16.00	-
Required Strength R_a [kN]	1006.94	-
Available Tensile Strength R_n/Ω [kN]	1978.44	Ok!
Available Block Shear strength R_n/Ω [kN]*	1147.35	Ok!
Available Bearing & Tearout Strength R_n/Ω [kN]	3870.72	Ok!
Available Bolt Strength (shear bearing) R_n/Ω [kN]	1346.31	Ok!

STEP BY STEP DESIGN SAMPLE

iii. Panel E

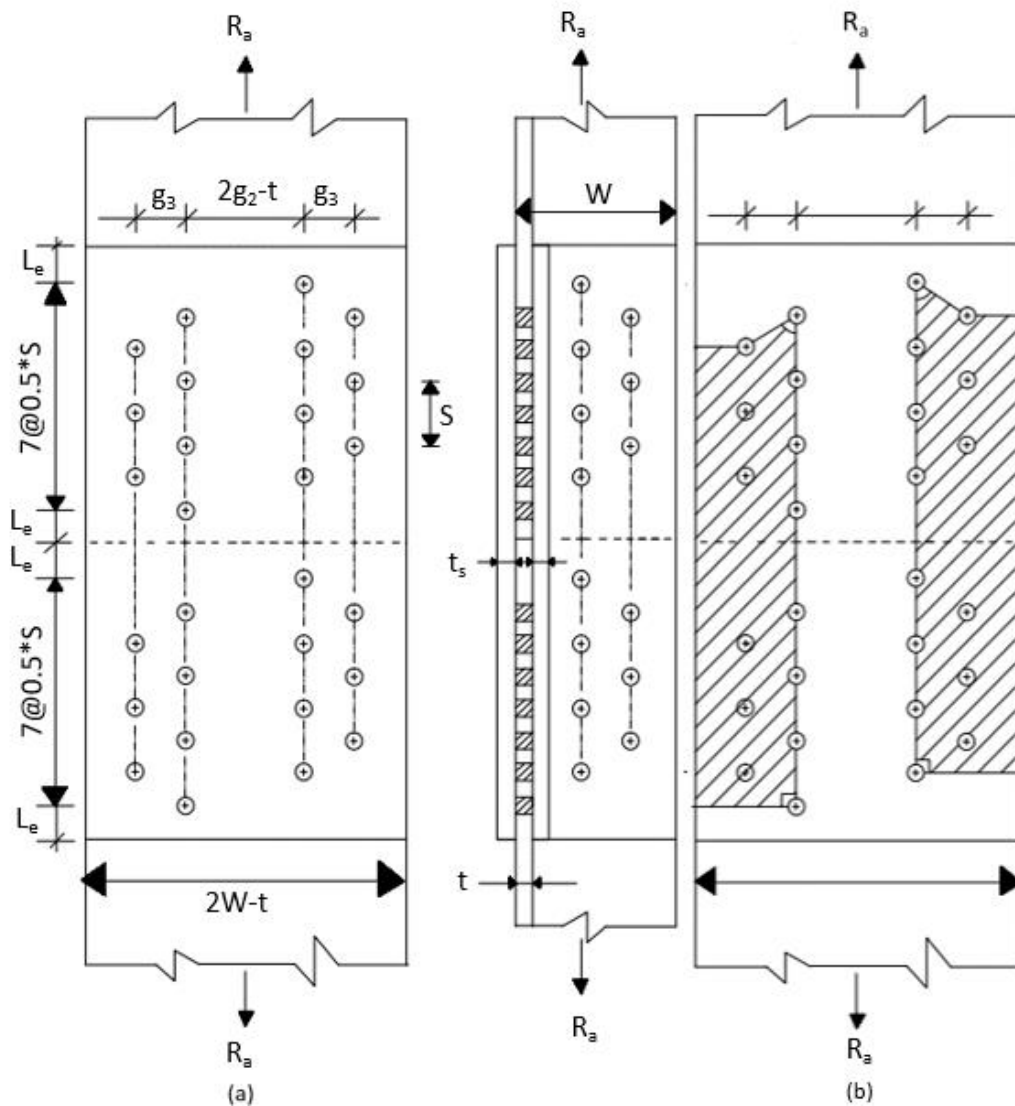


Figure 43: Panel E (a) Splice Connection View (b) Block Shear Path

Table 55: Panel E splice connection Design

Panel E (Critical Sub Panel)	Values	Comment $R_n/\Omega > R_a$
Equal angle steel, Leg width W & thickness t [mm]	200/25	-
Thickness of splice t_s [mm]	27	-
Bolt Grade A325N f_{nv} [Mpa]	372.00	-
Diameter of bolt d_b [mm]	16.00	-
Required Strength R_a [kN]	718.35	-
Available Tensile Strength R_n/Ω [kN]	1403.44	Ok!
Available Block Shear strength R_n/Ω [kN]*	857.52	Ok!
Available Bearing & Tearout Strength R_n/Ω [kN]	2688.00	Ok!
Available Bolt Strength (shear bearing) R_n/Ω [kN]	1047.13	Ok!

STEP BY STEP DESIGN SAMPLE

iv. Panel D

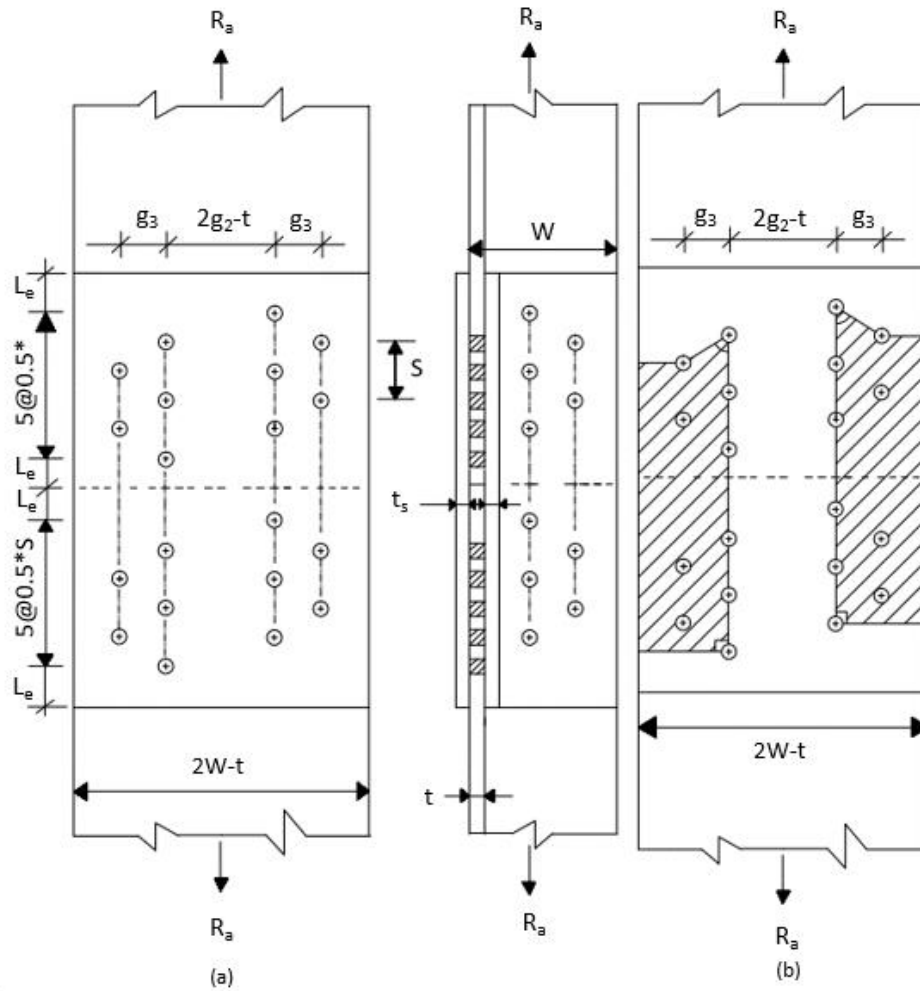


Figure 44: Panel D (a) Splice Connection View (b) Block Shear Path

Table 56: Panel D splice connection Design

Panel D (Critical Sub Panel)		Values	Comment $R_n/\Omega > R_a$
Equal angle steel, Leg width W & thickness t [mm]		200/20	-
Thickness of splice	t_s [mm]	22	-
Bolt Grade A325N	f_{nv} [Mpa]	372.00	-
Diameter of bolt	d_b [mm]	16.00	-
Required Strength	R_a [kN]	469.52	-
Available Tensile Strength	R_n/Ω [kN]	1137.72	Ok!
Available Block Shear strength	R_n/Ω [kN]*	608.01	Ok!
Available Bearing & Tearout Strength	R_n/Ω [kN]	1536.00	Ok!
Available Bolt Strength (shear bearing)	R_n/Ω [kN]	747.95	Ok!

STEP BY STEP DESIGN SAMPLE

v. Panel C

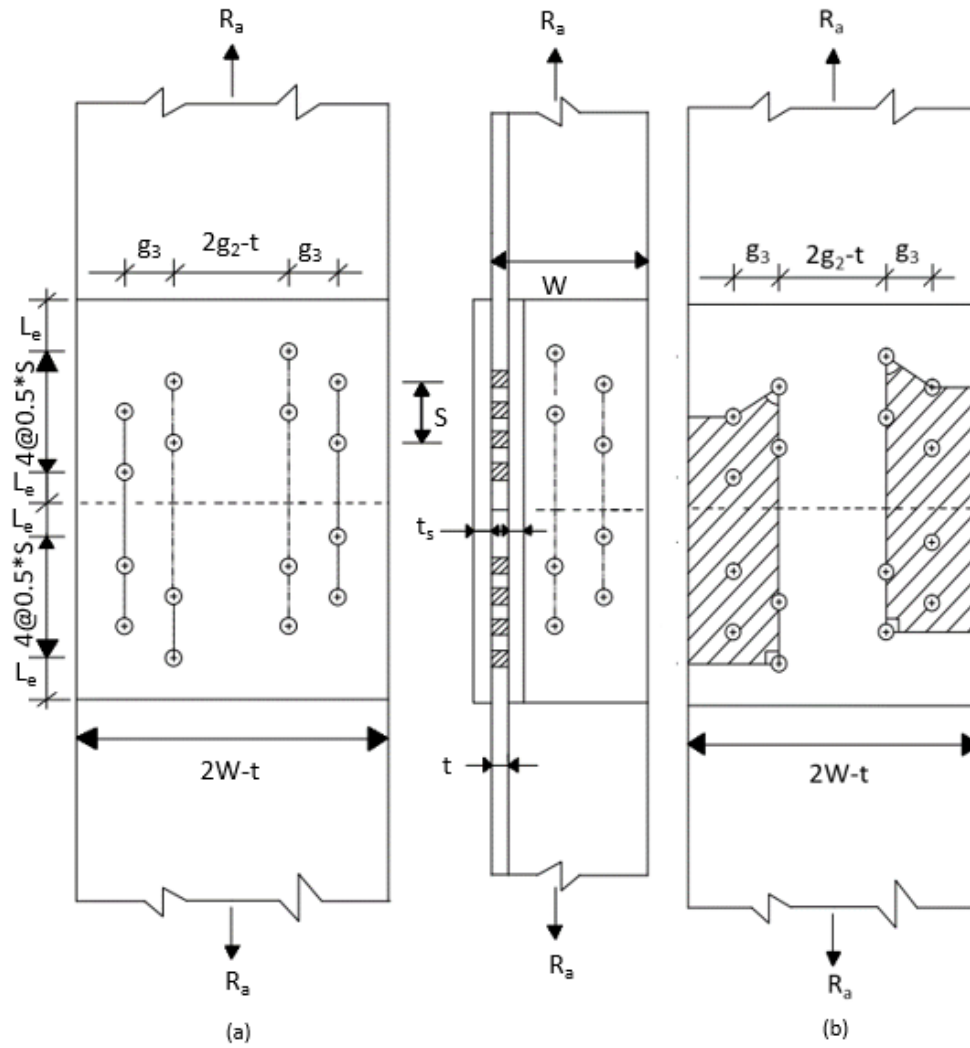


Figure 45: Panel C (a) Splice Connection View (b) Block Shear Path

Table 57: Panel C splice connection Design

Panel C (Critical Sub Panel)		Values	Comment $R_n/\Omega > R_a$
Equal angle steel, Leg width W & thickness t [mm]		150/15	-
Thickness of splice	t_s [mm]	17	-
Bolt Grade A325N	f_{nv} [Mpa]	372.00	-
Diameter of bolt	d_b [mm]	16.00	-
Required Strength	R_a [kN]	221.82	-
Available Tensile Strength	R_n/Ω [kN]	637.24	Ok!
Available Block Shear strength	R_n/Ω [kN]*	309.20	Ok!
Available Bearing & Tearout Strength	R_n/Ω [kN]	921.60	Ok!
Available Bolt Strength (shear bearing)	R_n/Ω [kN]	598.36	Ok!

STEP BY STEP DESIGN SAMPLE

vi. Panel B

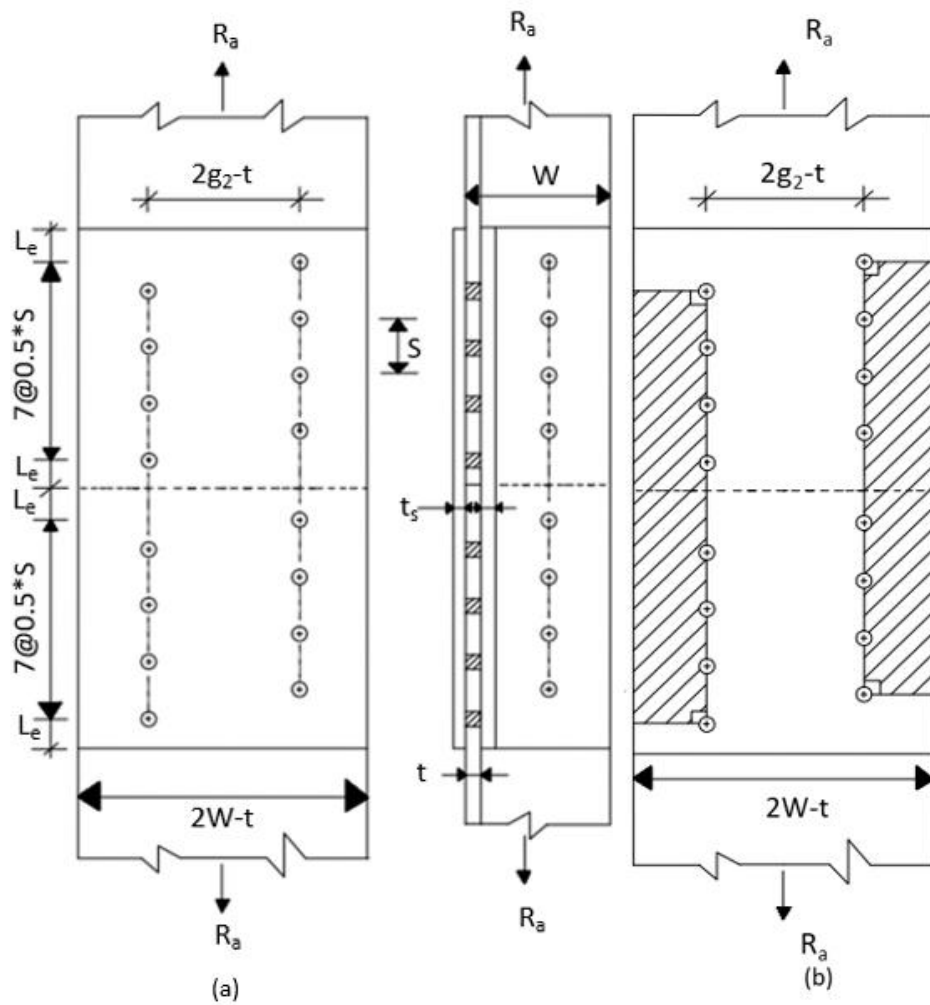


Figure 46: Panel B (a) Splice Connection View (b) Block Shear Path

Table 58: Panel B splice connection Design

Panel B (Critical Sub Panel)	Values	Comment $R_n/\Omega > R_a$
Equal angle steel, Leg width W & thickness t [mm]	100/13	-
Thickness of splice t_s [mm]	15	-
Bolt Grade A325N f_{nv} [Mpa]	372.00	-
Diameter of bolt d_b [mm]	16.00	-
Required Strength R_a [kN]	119.69	-
Available Tensile Strength R_n/Ω [kN]	363.92	Ok!
Available Block Shear strength R_n/Ω [kN]*	271.05	Ok!
Available Bearing & Tearout Strength R_n/Ω [kN]	798.72	Ok!
Available Bolt Strength (shear bearing) R_n/Ω [kN]	598.36	Ok!

STEP BY STEP DESIGN SAMPLE

vii. Panel A

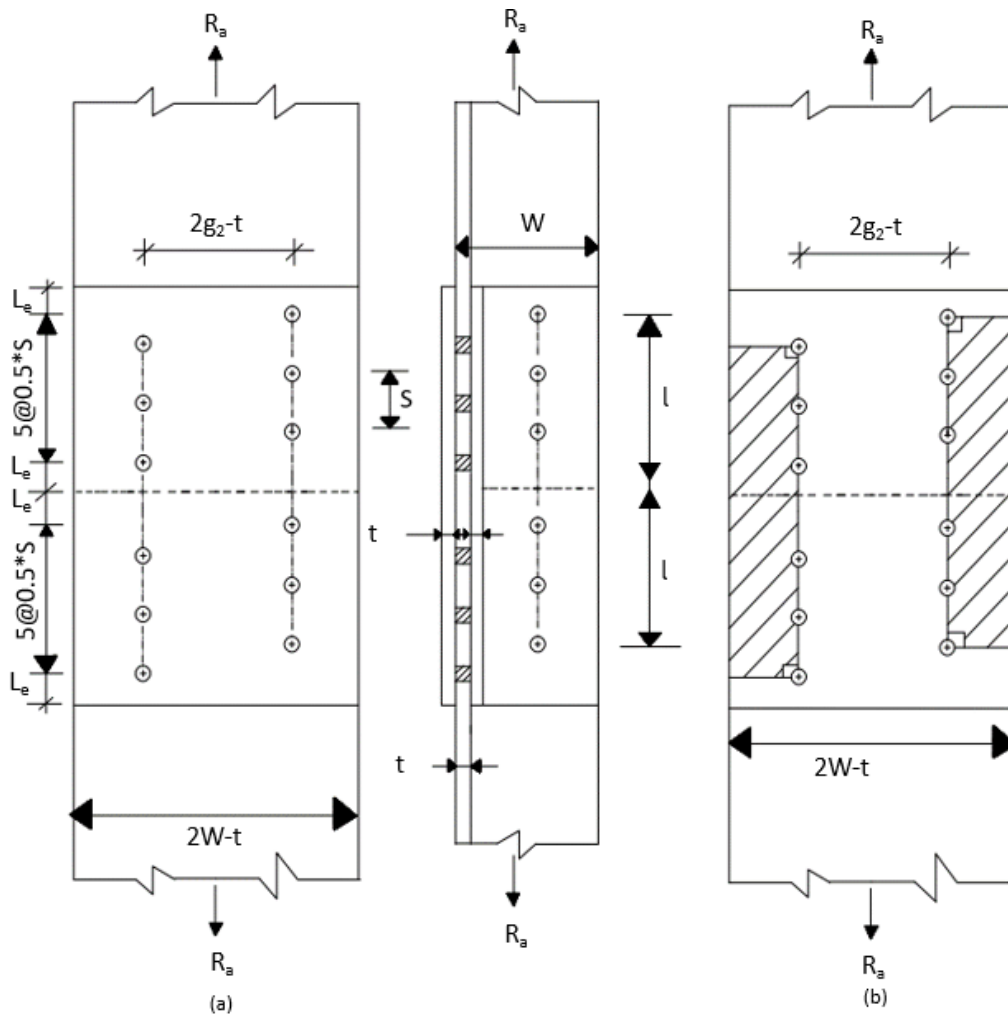


Figure 47: Panel A (a) Splice Connection View (b) Block Shear Path

Table 59: Panel A splice connection Design

Panel A (Critical Sub Panel)		Values	Comment $R_n/\Omega > R_a$
Equal angle steel, Leg width W & thickness t [mm]		75/6	-
Thickness of splice t_s [mm]		8	-
Bolt Grade A325N f_{nv} [Mpa]		372.00	-
Diameter of bolt d_b [mm]		16.00	-
Required Strength R_a [kN]		56.38	-
Available Tensile Strength R_n/Ω [kN]		127.21	Ok!
Available Block Shear strength R_n/Ω [kN]*		113.10	Ok!
Available Bearing & Tearout Strength R_n/Ω [kN]		276.48	Ok!
Available Bolt Strength (shear bearing) R_n/Ω [kN]		276.48	Ok!

STEP BY STEP DESIGN SAMPLE

b) Diagonal and Gusset Plate Connection Design

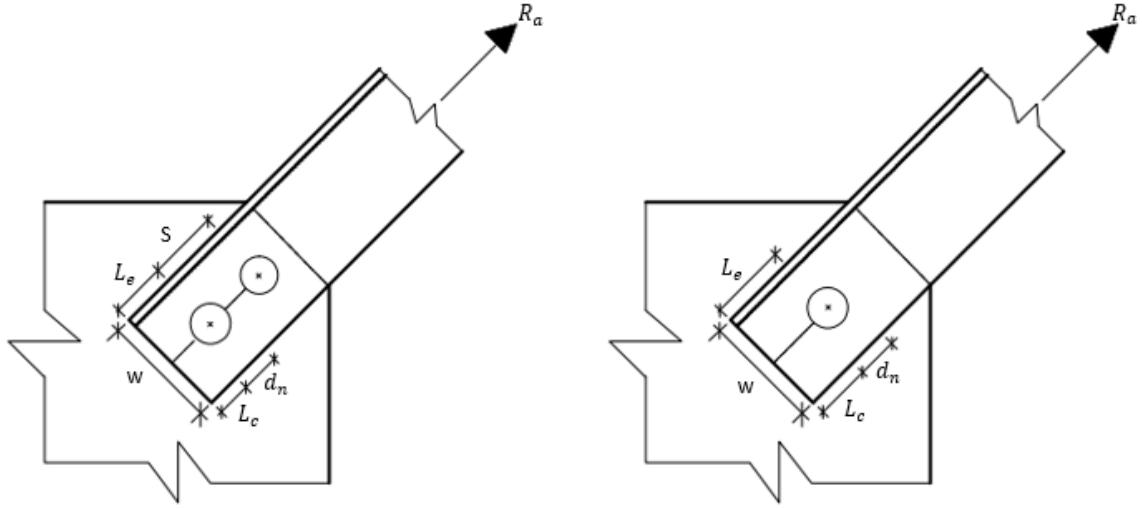


Figure 48: Diagonal-Gusset Plate Connection, (Left) double bolt (right) Single bolt

- Tensile strength of Diagonal and Gusset connection is conducted according to equation [3.77] for *yielding* and [3.78] for *rupture*.

Net area subjected to tension is calculated from equation [3.47]

- Shear strength of Diagonal and Gusset connection is conducted according to equation [3.81] for *block shear*. For block shear strength, net area subjected to shear (A_{nv}), net area subjected to tension (A_{nt}) and gross area subjected to shear (A_{gv}) are calculated using equation shown below

$$A_{gv} = [(n_{bv} - 1)S + L_e]t$$

$$A_{nv} = [(n_{bv} - 1)S + L_e - (n_{bv} - 0.5)d_h]t$$

$$A_{nt} = \min[(A_{nt1}, A_{nt2})]$$

$$A_{nt1} = [(\sum_{i=1}^4 g_i + e) - (\frac{n_1}{2} - 0.5)d_h + (\sum_{i=1}^4 \frac{p^2}{4g_i})]t$$

$$A_{nt2} = [(\sum_{i=1}^4 g_i + e) - 0.5d_h]t$$

$$e = w - \sum_{i=1}^4 g_i$$

n_{bv} : Number of bolt in longitudinal direction in the line nearest to shear center.

Equivalent workable *gages* length g_1 , g_2 , g_3 , and g_4 of the equal angle steel for the provision of bolt holes are referred from table [10] based on their width.

Pitch distance (p) of bolt in transverse loading is taken as half the spacing (S) between bolts. Width and thickness of diagonal section is taken from panel input data

STEP BY STEP DESIGN SAMPLE

- Bearing and tearout strength of Diagonal and Gusset connection is conducted according to equation, [3.82] for *tearout* and, [3.83] for *bearing*
- Bolt tensile and shear strength of Diagonal and Gusset connection is conducted according to equation, [3.84]. Bolts in Diagonal and Gusset connection are subjected to shear in single plane

Diagonal and Gusset Plate Connection Design Output of Critical Sub Panel

Table 60: Panel G Diagonal and gusset plate connection

Panel G specifications		Values	Comment $R_n/\Omega > R_a$
Diagonal steel width & thickness	W/t [mm]	120/15	-
Thickness of Gusset	t_G [mm]	17	-
Bolt Grade A325N	f_{nv} [MPa]	372	-
Diameter of Bolt	d_b [mm]	22.00	-
Number of bolt	n_{bv}	2	-
Required Strength	R_n [kN]	129.53	-
Available Tensile Strength of Diagonal	R_n/Ω [kN]	155.95	Ok!
Available Shear Strength of Diagonal	R_n/Ω [kN]	243.60	Ok!
Available Bearing & Tearout Strength Diagonal	R_n/Ω [kN]	205.20	Ok!
Available Total connection Strength	R_n/Ω [kN]*	131.90	Ok!

Table 61: Panel F Diagonal and gusset plate connection Design

Panel F specifications		Values	Comment $R_n/\Omega > R_a$
Diagonal steel width & thickness	W/t [mm]	100/15	-
Thickness of Gusset	t_G [mm]	17	-
Bolt Grade A325N	f_{nv} [MPa]	372	-
Diameter of Bolt	d_b [mm]	20.00	-
Number of bolt	n_{bv}	2	-
Required Strength	R_n [kN]	110.84	-
Available Tensile Strength of Diagonal	R_n/Ω [kN]*	115.90	Ok!
Available Shear Strength of Diagonal	R_n/Ω [kN]	177.60	Ok!
Available Bearing & Tearout Strength Diagonal	R_n/Ω [kN]	187.20	Ok!
Available Total connection strength	R_n/Ω [kN]	116.03	Ok!

STEP BY STEP DESIGN SAMPLE

Table 62: Panel E Diagonal and gusset plate connection Design

Panel E specifications		Values	Comment $R_n/\Omega > R_a$
Diagonal steel width & thickness	W/t [mm]	100/13	-
Thickness of Gusset	t_G [mm]	15	-
Bolt Grade A325N	f_{nv} [MPa]	372	-
Diameter of Bolt	d_b [mm]	20.00	-
Number of bolt	n_{bv}	2	-
Required Strength	R_n [kN]	79.02	-
Available Tensile Strength of Diagonal	R_n/Ω [kN]*	100.45	Ok!
Available Shear Strength of Diagonal	R_n/Ω [kN]	150.80	Ok!
Available Bearing & Tearout Strength Diagonal	R_n/Ω [kN]	156.00	Ok!
Available Total connection strength	R_n/Ω [kN]	102.11	Ok!

Table 63: Panel D Diagonal and gusset plate connection Design

Panel D specifications		Values	Comment $R_n/\Omega > R_a$
Diagonal steel width & thickness	W/t [mm]	100/13	-
Thickness of Gusset	t_G [mm]	15	-
Bolt Grade A325N	f_{nv} [MPa]	372	-
Diameter of Bolt	d_b [mm]	20.00	-
Number of bolt	n_{bv}	2	-
Required Strength	R_n [kN]	52.44	-
Available Tensile Strength of Diagonal	R_n/Ω [kN]*	100.45	Ok!
Available Shear Strength of Diagonal	R_n/Ω [kN]	150.80	Ok!
Available Bearing & Tearout Strength Diagonal	R_n/Ω [kN]	156.00	Ok!
Available Total connection strength	R_n/Ω [kN]	102.11	Ok!

Table 64: Panel C Diagonal and gusset plate connection Design

Panel C specifications		Values	Comment $R_n/\Omega > R_a$
Diagonal steel width & thickness	W/t [mm]	75/8	-
Thickness of Gusset	t_G [mm]	10	-
Bolt Grade A325N	f_{nv} [MPa]	372	-
Diameter of Bolt	d_b [mm]	16.00	-
Number of bolt	n_{bv}	1	-
Required Strength	R_n [kN]	24.28	-
Available Tensile Strength of Diagonal	R_n/Ω [kN]	88.00	Ok!
Available Shear Strength of Diagonal	R_n/Ω [kN]*	26.88	Ok!
Available Bearing & Tearout Strength Diagonal	R_n/Ω [kN]	80.64	Ok!
Available Total connection strength	R_n/Ω [kN]	64.28	Ok!

STEP BY STEP DESIGN SAMPLE

Table 65: Panel B Diagonal and gusset plate connection Design

Panel B specifications		Values	Comment $R_n/\Omega > R_a$
Diagonal steel width & thickness	W/t [mm]	75/6	-
Thickness of Gusset	t_G [mm]	8	-
Bolt Grade A325N	f_{nv} [MPa]	372	-
Diameter of Bolt	d_b [mm]	20.00	-
Number of bolt	n_{bv}	1	-
Required Strength	R_n [kN]	10.00	-
Available Tensile Strength of Diagonal	R_n/Ω [kN]	66.00	Ok!
Available Shear Strength of Diagonal	R_n/Ω [kN]*	17.28	Ok!
Available Bearing & Tearout Strength Diagonal	R_n/Ω [kN]	57.60	Ok!
Available Total connection strength	R_n/Ω [kN]	54.68	Ok!

Table 66: Panel A Diagonal and gusset plate connection Design

Panel A specifications		Values	Comment $R_n/\Omega > R_a$
Diagonal steel width & thickness	W/t [mm]	60/5	-
Thickness of Gusset	t_G [mm]	7	-
Bolt Grade A325N	f_{nv} [MPa]	372	-
Diameter of Bolt	d_b [mm]	16.00	-
Number of bolt	n_{bv}	1	-
Required Strength	R_n [kN]	3.59	-
Available Tensile Strength of Diagonal	R_n/Ω [kN]	40.00	Ok!
Available Shear Strength of Diagonal	R_n/Ω [kN]*	14.40	Ok!
Available Bearing & Tearout Strength Diagonal	R_n/Ω [kN]	48.00	Ok!
Available Total connection strength	R_n/Ω [kN]	48.00	Ok!

STEP BY STEP DESIGN SAMPLE

c) Leg and Gusset Plate Connection Design

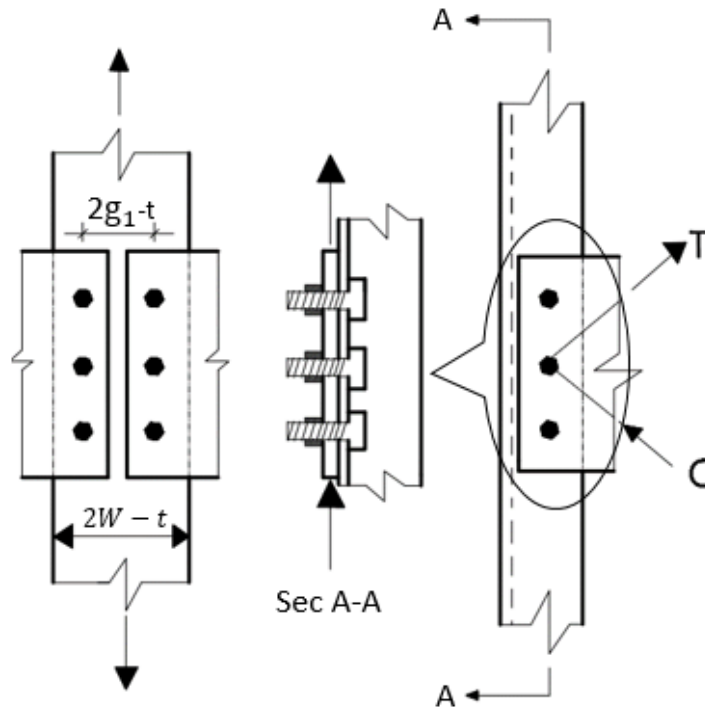


Figure 49: Leg-Gusset Plate Connection

- Bearing and tearout strength at bolt holes is determined independent of loading direction.
- Shear strength of gusset plate is conducted according to equation [3.79] for *yielding* and [3.80] *rupture*. Net area subjected to shear (A_{nv}), and gross area subjected to shear (A_{gv}) are calculated for leg and gusset plate connection using equation shown below

$$A_{gv} = 2[(n_{bv} - 1)S + L_e]t_G$$

$$A_{nv} = 2[(n_{bv} - 1)S + L_e - (n_{bv} - 0.5)d_h]t_G$$

n_{bv} : Number of bolt in longitudinal direction in the line nearest to shear center

Equivalent workable *gage* length g_1 , g_2 , g_3 , and g_4 of the equal angle steel for the provision of bolt holes are referred from table [10] based on their width. Pitch distance (p) of bolt in transverse loading is taken as half the spacing (S) between bolts. Thickness of gusset is taken from diagonal and gusset connection for each panel.

- Bearing and tearout strength gusset plate is conducted according to equation [3.82] for *tearout* and [3.83] for *bearing*.
- Bolt shear and bearing strength in leg gusset connection is conducted according to equation [3.84]. Bolts in leg gusset plate connections are subjected to shear in single plane and are not arranged in staggered manner.

STEP BY STEP DESIGN SAMPLE

Leg and Gusset Plate Connection Design Output of Critical Sub Panel

Table 67: Panel G Leg and gusset plate connection Design

Strength [kN]	Value	Comment $R_n/\Omega > R_a$
Diameter of Bolt d_b [mm]	20	-
Required Design Strength R_a	129.53	-
Allowable Bearing & Tearout strength of gusset R_n/Ω	489.60	Ok!
Allowable Shear strength of gusset R_n/Ω	612.00	Ok!
Allowable Bolt Shear & Bearing Strength R_n/Ω^*	175.30	Ok!

Table 68: Panel F Leg and gusset plate connection Design

Strength [kN]	Value	Comment $R_n/\Omega > R_a$
Diameter of Bolt d_b [mm]	20	-
Required Design Strength R_a	110.84	-
Allowable Bearing & Tearout strength of gusset R_n/Ω	806.40	Ok!
Allowable Shear strength of gusset R_n/Ω	612.00	Ok!
Allowable Bolt Shear & Bearing Strength R_n/Ω^*	175.30	Ok!

Table 69: Panel E Leg and gusset plate connection Design

Strength [kN]	Value	Comment $R_n/\Omega > R_a$
Diameter of Bolt d_b [mm]	16	-
Required Design Strength R_a	79.02	-
Allowable Bearing & Tearout strength of gusset R_n/Ω	576.00	Ok!
Allowable Shear strength of gusset R_n/Ω	435.60	Ok!
Allowable Bolt Shear & Bearing Strength R_n/Ω^*	112.19	Ok!

Table 70: Panel D Leg and gusset plate connection Design

Strength [kN]	Value	Comment $R_n/\Omega > R_a$
Diameter of Bolt d_b [mm]	16	-
Required Design Strength R_a	52.44	-
Allowable Bearing & Tearout strength of gusset R_n/Ω	460.80	Ok!
Allowable Shear strength of gusset R_n/Ω	435.00	Ok!
Allowable Bolt Shear & Bearing Strength R_n/Ω^*	112.19	Ok!

Table 71: Panel C Leg and gusset plate connection Design

Strength [kN]	Value	Comment $R_n/\Omega > R_a$
Diameter of Bolt d_b [mm]	16	-
Required Design Strength R_a	24.28	-
Allowable Bearing & Tearout strength of gusset R_n/Ω	345.60	Ok!
Allowable Shear strength of gusset R_n/Ω	290.40	Ok!
Allowable Bolt Shear & Bearing Strength R_n/Ω^*	112.19	Ok!

STEP BY STEP DESIGN SAMPLE

Table 72: Panel B Leg and gusset plate connection Design

Strength [kN]		Value	Comment $R_n/\Omega > R_a$
Diameter of Bolt	d_b [mm]	16	-
Required Strength	R_a	10.00	-
Allowable Bearing & Tearout strength of gusset	R_n/Ω	299.52	Ok!
Allowable Shear strength of gusset	R_n/Ω	232.32	Ok!
Allowable Bolt Shear & Bearing Strength	R_n/Ω^*	112.19	Ok!

Table 73: Panel A Leg and gusset plate connection Design

Strength [kN]		Value	Comment $R_n/\Omega > R_a$
Diameter of Bolt	d_b [mm]	16	-
Required Strength	R_a	3.59	-
Allowable Bearing & Tearout strength of gusset	R_n/Ω	138.24	Ok!
Allowable Shear strength of gusset	R_n/Ω	203.30	Ok!
Allowable Bolt Shear & Bearing Strength	R_n/Ω^*	112.19	Ok!

CHAPTER SEVEN

CONCLUSION AND RECOMENDATION

7.1 CONCLUSIONS

The following conclusions are made, from the wind and seismic behavior study of self-supporting communication tower and design of articulated modular tower.

- The Tower's top nodal displacement, arising from the wind load in diagonal direction, is larger than that of the top nodal displacement that arises due to the wind load in parallel direction. Therefore, the diagonal wind direction must be used as governing wind direction for analyzing communication towers.
- The Tower's top nodal displacement, arising from the seismic load in parallel and diagonal direction to the side face of the tower are equal. Therefore, top nodal displacement due to seismic load does not affected by orientation of tower with respect to seismic load direction.
- Tower's in seismic zone, I, II, III, and IV, having height of 45m, 60m, 80m and 100m, exposed to wind speed of 35m/s, 40m/s, 45m/s, and 50m/s, exhibited more top nodal displacement due to wind load than the seismic load. So for those seismic zones, wind load is governing load.
- Tower's in seismic zone, V, having height of 45m, 60m, and 80m, exposed to speed of 35 m/s, exhibited less top nodal displacement due to wind load than seismic load produced. So for those seismic zones, seismic load is governing load.
- Tower in seismic zone, V, having height of 100m exposed to speed of 35 m/s the top nodal displacement, exhibited more top nodal displacement due to wind load than the seismic load produced. So for those seismic zones, wind load is governing load.
- There is an increase of 75% to 200% seismic force due to site effect from seismic zone I to Seismic Zone V.
- Leg members of the tower are more stressed due to wind in a diagonal direction, while the diagonal members of the tower are more stressed due to the wind in parallel direction. Thus, parallel wind load analysis and diagonal wind load analysis should be done for diagonal member and leg member design respectively.
- The failure of the leg structural members at the splice connection is governed by block shear failure.

CONCLUSION AND RECOMENDATION

- The failure of the diagonal structural members at the singly bolted diagonal-gusset connection is governed by shear failure
- For Communication Towers, Leg and Diagonal structural member's failure governed by Block shear failure and Shear failure respectively, the leg-gusset connections failure may governed by Bolt shear-bearing failure.

7.2 RECOMMENDATIONS

From this study the following recommendations are made:-

- For further understanding of self-supporting communication tower behavior and performance under various conditions, the effect of accidental explosive load under various heights and bracing systems should be studied.
- Since, this design module is prepared for rectangular base communication tower, study on triangular base cross section is recommended.
- Study on wind and seismic performance of existing self supporting communication towers are recommended.

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APPENDIX A

SECTION DETAILING OF STRUCTURAL MEMBERS

Panel G (Bottom Panel) Section Detailing

Mark	Section	Grade	Length [m]	Weight [kg]	Description	Quantity
GL	250mm×250mm×35mm	S235	10.04	1285.34	Leg	8
GD1	120mm×120mm×15mm	S235	15.90	537.58	Diagonal	8
GD2	120mm×120mm×15mm	S235	16.93	572.20	Diagonal	8
GH1	100mm×100mm×8mm	S235	12.32	150.25	Horizontal	4
GH2	100mm×100mm×8mm	S235	13.61	166.07	Horizontal	4

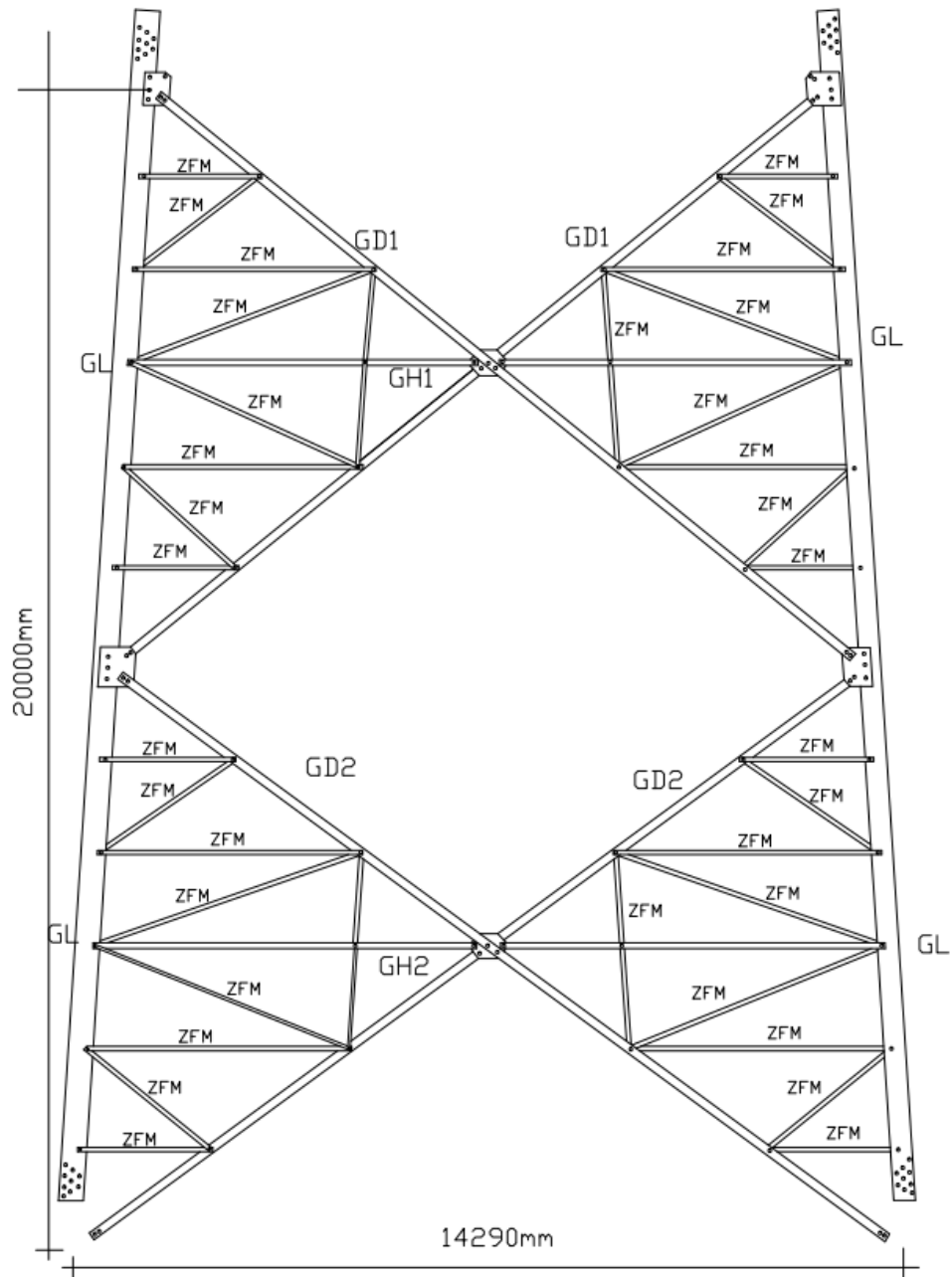


Figure 50: Panel G section detailing

DETAILING

Panel F

Mark	Section	Grade	Length[m]	Weight [kg]	Description	Quantity
FL	250mm×250mm×28mm	S235	10.04	940.9	Leg	8
FD1	150mm×150mm×15mm	S235	13.99	472.9	Diagonal	8
FD2	100mm×100mm×15mm	S235	14.92	504.4	Diagonal	8
FH1	100mm×100mm×8mm	S235	9.72	118.6	Horizontal	4
FH2	100mm×100mm×8mm	S235	11.02	134.4	Horizontal	4

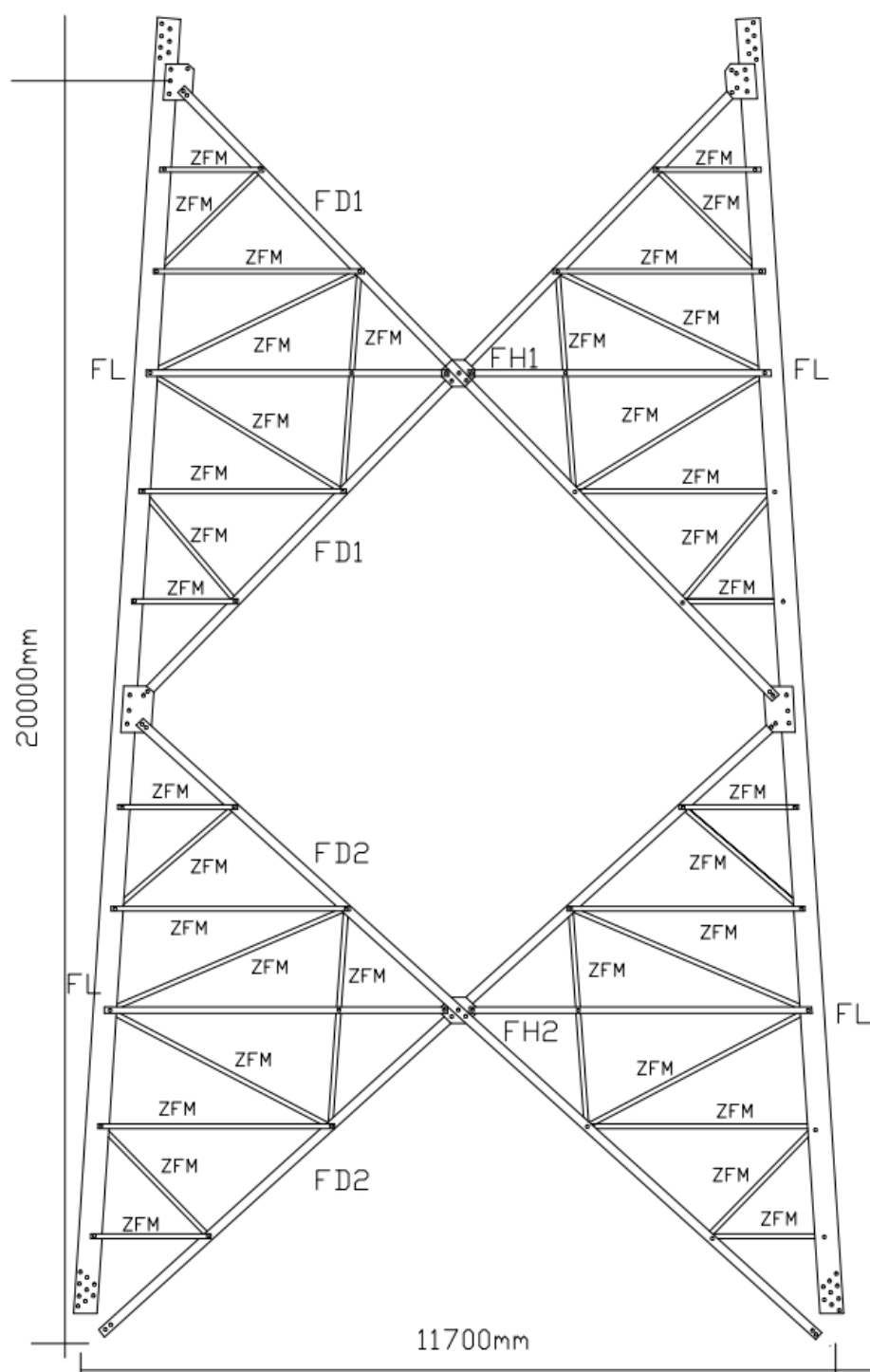


Figure 51: Panel F section detailing

DETAILING

Panel E

Mark	Section	Grade	Length[m]	Weight [kg]	Description	Quantity
EL	200mm×200mm×25mm	S235	7.53	554.3	Leg	8.00
ED1	100mm×100mm×13mm	S235	10.73	362.7	Diagonal	8.00
ED2	100mm×100mm×13mm	S235	11.44	386.8	Diagonal	8.00
EH1	100mm×100mm×8mm	S235	7.63	93.1	Horizontal	4.00
EH2	100mm×100mm×8mm	S235	8.60	105.0	Horizontal	4.00

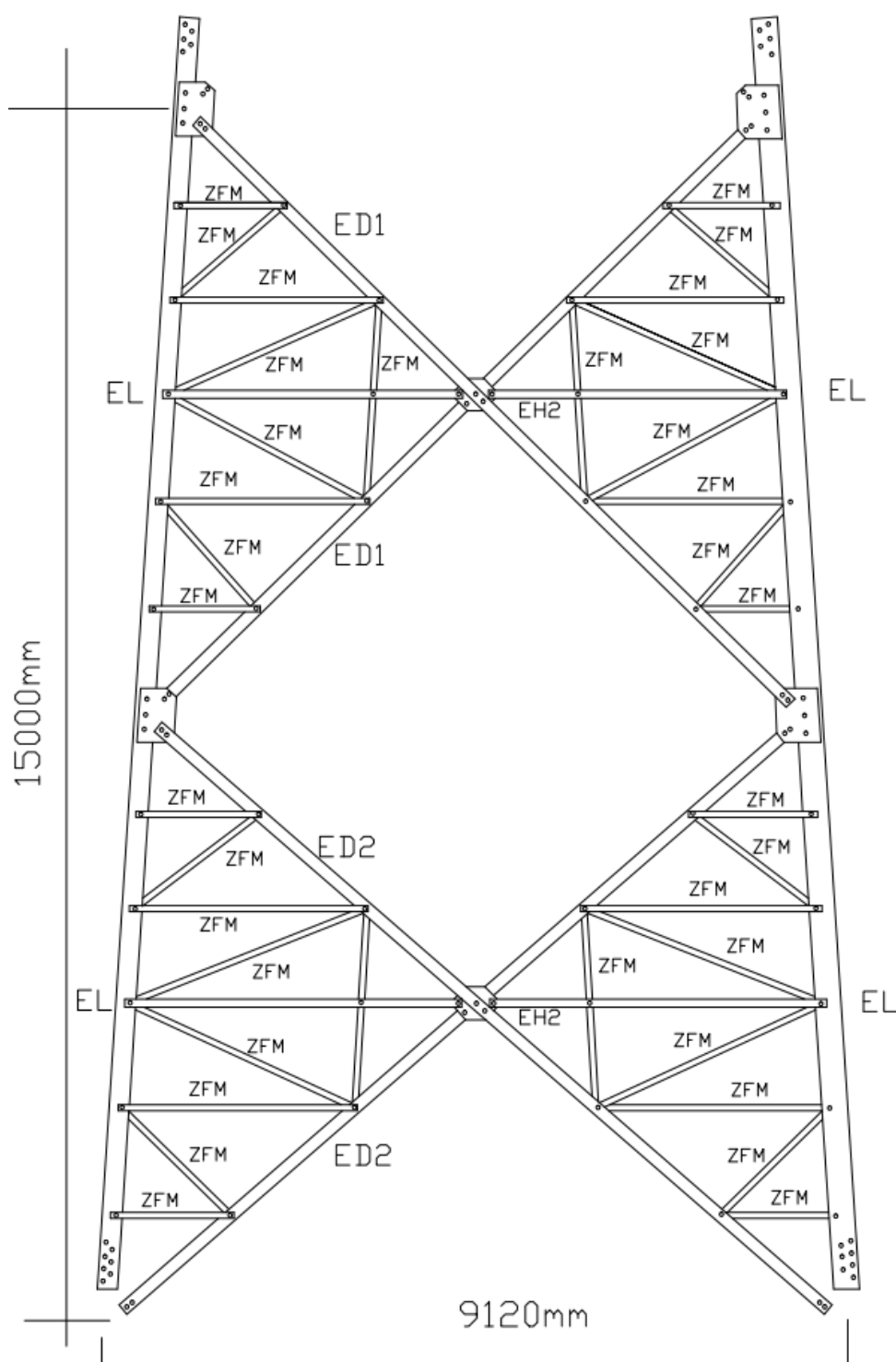


Figure 52: Panel E section detailing

DETAILING

Panel D

Mark	Section	Grade	Length[m]	Weight [kg]	Description	Quantity
DL	200mm×200mm×20mm	S235	5.02	300.8	Leg	16
DD1	100mm×100mm×15mm	S235	7.02	153.7	Diagonal	8
DD2	100mm×100mm×13mm	S235	7.48	163.9	Diagonal	8
DD3	100mm×100mm×13mm	S235	7.98	174.7	Diagonal	8
DD4	100mm×100mm×13mm	S235	8.49	185.9	Diagonal	8
DH1	100mm×100mm×8mm	S235	4.89	33.6	Horizontal	4
DH2	100mm×100mm×8mm	S235	5.54	38.1	Horizontal	4
DH3	100mm×100mm×8mm	S235	6.19	42.5	Horizontal	4
DH4	100mm×100mm×8 mm	S235	6.84	47.0	Horizontal	4

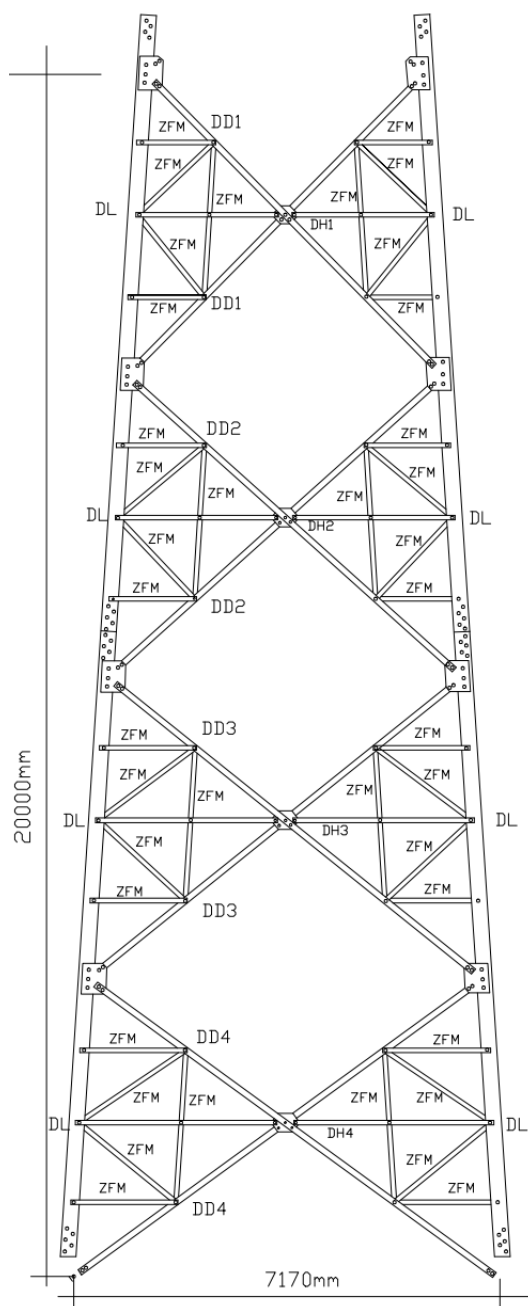


Figure 53: Panel D section detailing

DETAILING

Panel C

Mark	Section	Grade	Length[m]	Weight [kg]	Description	Quantity
CL	150mm×150mm×15mm	S235	4.02	21.9	Leg	12
CD1	75mm×75mm×8mm	S235	5.19	46.9	Diagonal	8
CD2	75mm×75mm×8mm	S235	5.53	49.9	Diagonal	8
CD3	75mm×75mm×8mm	S235	5.90	53.3	Diagonal	8
CH1	75mm×75mm×6mm	S235	3.27	22.5	Horizontal	4
CH2	75mm×75mm×6mm	S235	3.79	26.1	Horizontal	4
CH3	75mm×75mm×6mm	S235	4.31	29.6	Horizontal	4

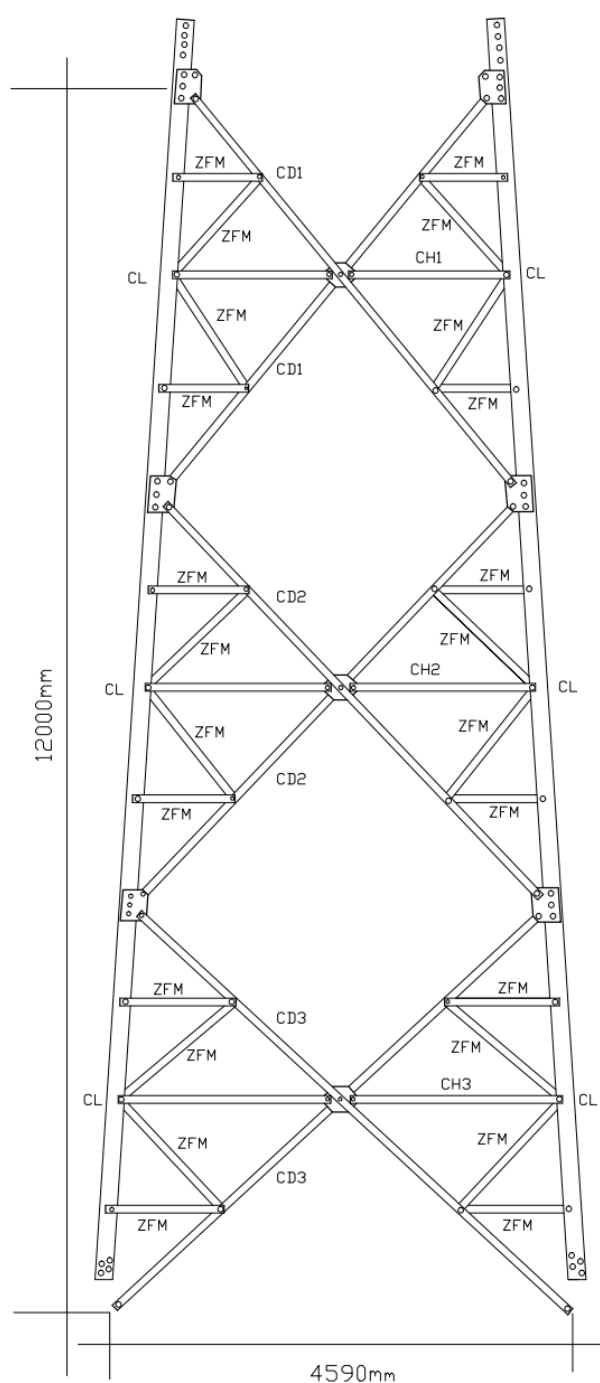


Figure 54: Panel C section detailing

DETAILING

Panel B

Mark	Section	Grade	Length[m]	Weight [kg]	Description	Quantity
BL	100mm×100mm×13mm	S235	2.01	38.4	Leg	16
BD1	75mm×75mm×6mm	S235	2.92	20.1	Diagonal	8
BD2	75mm×75mm×6mm	S235	3.12	21.4	Diagonal	8
BD3	75mm×75mm×6mm	S235	3.32	22.8	Diagonal	8
BD4	75mm×75mm×6mm	S235	3.53	24.3	Diagonal	8
BH1	75mm×75mm×6mm	S235	2.12	14.6	Horizontal	4
BH2	75mm×75mm×6mm	S235	2.38	16.4	Horizontal	4
BH3	75mm×75mm×6mm	S235	2.64	18.1	Horizontal	4
BH4	75mm×75mm×6mm	S235	2.90	19.9	Horizontal	4

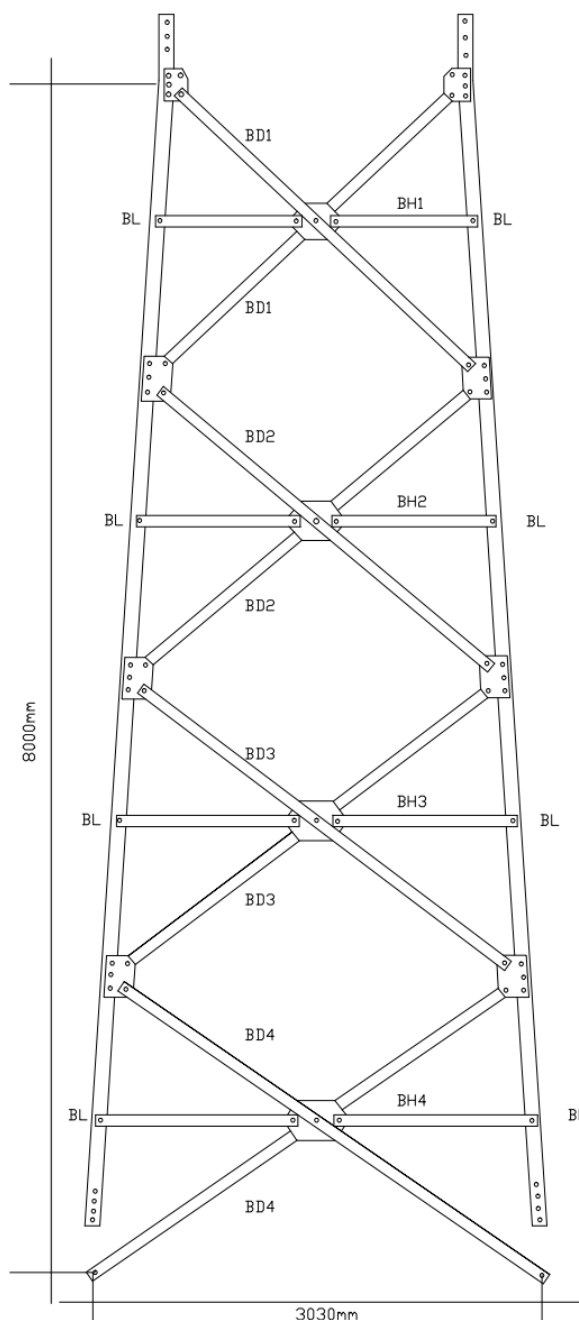


Figure 55: Panel B section detailing

DETAILING

Panel A (top panel)

Mark	Section	Grade	Length[m]	Weight [kg]	Description	Quantity
AL	75mm×75mm×6mm	S235	1.00	7	Leg	20
AD	60mm×60mm×5mm	S235	2.24	15	Diagonal	40
AH	60mm×60mm×5mm	S235	2.00	14	Horizontal	12

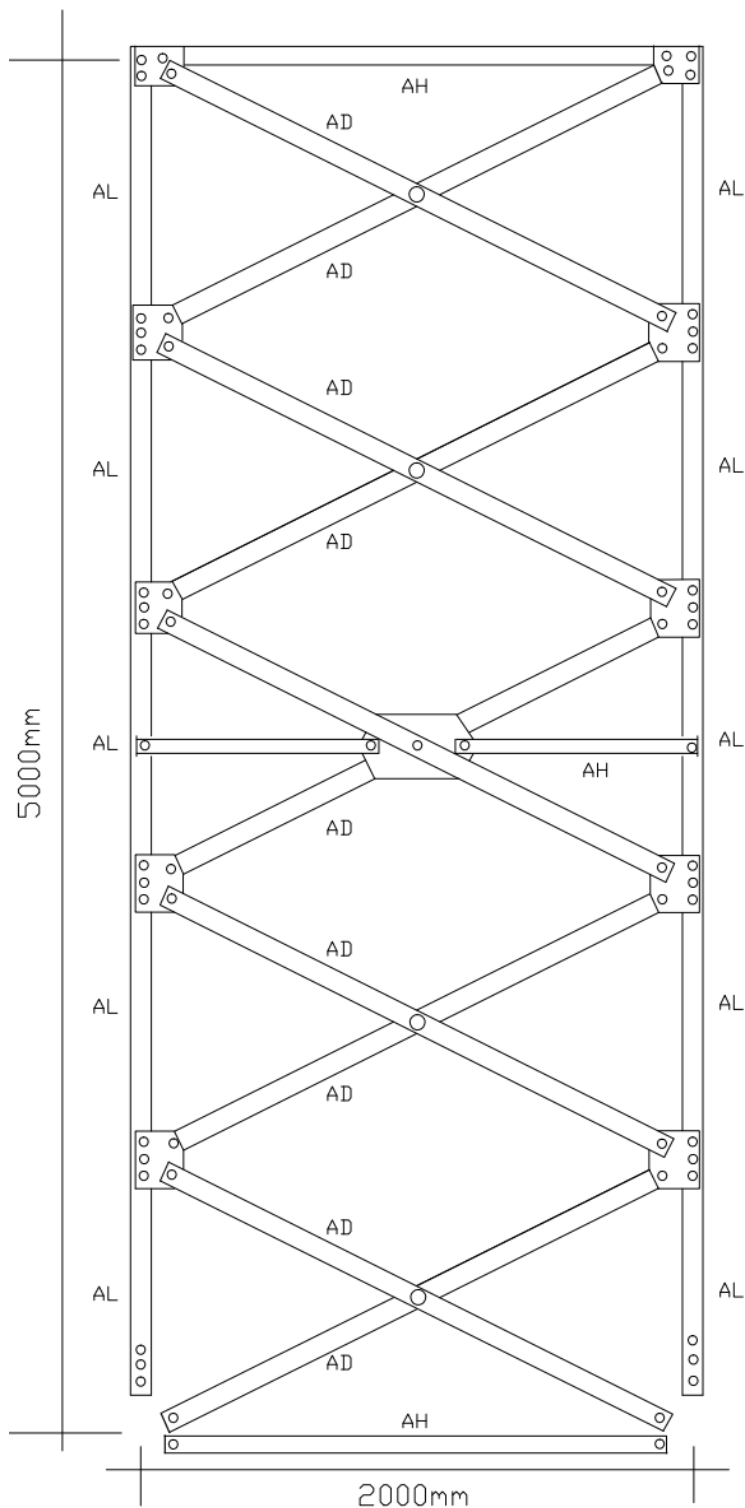


Figure 56: Panel A section detailing

APPENDIX B

GEOMETRIC COORDINATES OF BASIC NODES

Basic Nodes for Three Dimensional Modeling

Basic nodes are corner coordinates points of the tower at base, top of tapered and top of straight part of the tower. The centroid of the base is used as the origin for geometric coordinate in the three dimensional modeling for parallel and diagonal directions of loading as shown below.

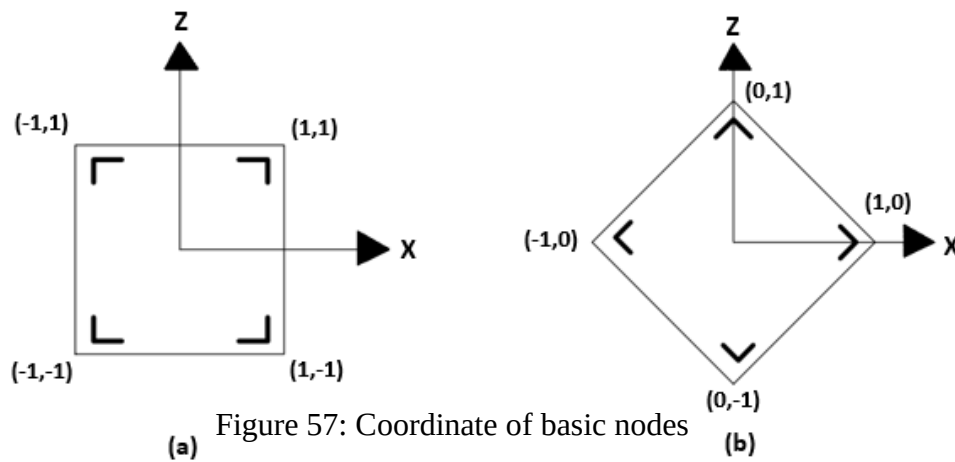


Figure 57: Coordinate of basic nodes

Table 74: Geometric coordinates of Basic nodes for parallel loading

Basic Nodes	Basic node Unit Vectors		SSCT100			SSCT80		
	X_i	Z_i	$X[m]$	$Z[m]$	$Y[m]$	$X[m]$	$Z[m]$	$Y[m]$
1	-1.00	-1.00	-7.15	-7.15	0.00	-5.85	-5.85	0.00
2	-1.00	1.00	-7.15	7.15	0.00	-5.85	5.85	0.00
3	1.00	-1.00	7.15	-7.15	0.00	5.85	-5.85	0.00
4	1.00	1.00	7.15	7.15	0.00	5.85	5.85	0.00
5	-1.00	-1.00	-1.00	-1.00	95.00	-1.00	-1.00	75.00
6	-1.00	1.00	-1.00	1.00	95.00	-1.00	1.00	75.00
7	1.00	-1.00	1.00	-1.00	95.00	1.00	-1.00	75.00
8	1.00	1.00	1.00	1.00	95.00	1.00	1.00	75.00
9	-1.00	-1.00	-1.00	-1.00	100.00	-1.00	-1.00	80.00
10	-1.00	1.00	-1.00	1.00	100.00	-1.00	1.00	80.00
11	1.00	-1.00	1.00	-1.00	100.00	1.00	-1.00	80.00
12	1.00	1.00	1.00	1.00	100.00	1.00	1.00	80.00

BASIC NODES

Table 75: Geometric coordinates of Basic nodes for parallel loading

Basic Nodes	Basic node Unit Vectors		SSCT60			SSCT45		
	Xi	Zi	X[m]	Z[m]	Y[m]	X[m]	Z[m]	Y[m]
1	-1.00	-1.00	0.00	-6.44	0.00	0.00	-5.07	0.00
2	-1.00	1.00	-6.44	0.00	0.00	-5.07	0.00	0.00
3	1.00	-1.00	0.00	6.44	0.00	0.00	5.07	0.00
4	1.00	1.00	6.44	0.00	0.00	5.07	0.00	0.00
5	-1.00	-1.00	0.00	-1.41	55.00	0.00	-1.41	40.00
6	-1.00	1.00	-1.41	0.00	55.00	-1.41	0.00	40.00
7	1.00	-1.00	0.00	1.41	55.00	0.00	1.41	40.00
8	1.00	1.00	1.41	0.00	55.00	1.41	0.00	40.00
9	-1.00	-1.00	0.00	-1.41	60.00	0.00	-1.41	45.00
10	-1.00	1.00	-1.41	0.00	60.00	-1.41	0.00	45.00
11	1.00	-1.00	0.00	1.41	60.00	0.00	1.41	45.00
12	1.00	1.00	1.41	0.00	60.00	1.41	0.00	45.00

Table 76: Geometric Coordinates Of Basic Nodes For Diagonal Loading

Basic Nodes	Basic node Unit Vectors		SSCT100			SSCT80		
	Xi	Zi	X[m]	Z[m]	Y[m]	X[m]	Z[m]	Y[m]
1	0.00	-1.00	0.00	-10.11	0.00	0.00	-8.27	0.00
2	-1.00	0.00	-10.11	0.00	0.00	-8.27	0.00	0.00
3	0.00	1.00	0.00	10.11	0.00	0.00	8.27	0.00
4	1.00	0.00	10.11	0.00	0.00	8.27	0.00	0.00
5	0.00	-1.00	0.00	-1.41	95.00	0.00	-1.41	75.00
6	-1.00	0.00	-1.41	0.00	95.00	-1.41	0.00	75.00
7	0.00	1.00	0.00	1.41	95.00	0.00	1.41	75.00
8	1.00	0.00	1.41	0.00	95.00	1.41	0.00	75.00
9	0.00	-1.00	0.00	-1.41	100.00	0.00	-1.41	80.00
10	-1.00	0.00	-1.41	0.00	100.00	-1.41	0.00	80.00
11	0.00	1.00	0.00	1.41	100.00	0.00	1.41	80.00
12	1.00	0.00	1.41	0.00	100.00	1.41	0.00	80.00

BASIC NODES

Table 77: Geometric Coordinates of Basic Nodes For Diagonal Loading

Basic Nodes	Unit Vectors		SSCT60			SSCT45		
	Xi	Zi	X[m]	Z[m]	Y[m]	X[m]	Z[m]	Y[m]
1	0.00	-1.00	0.00	-6.44	0.00	0.00	-5.07	0.00
2	-1.00	0.00	-6.44	0.00	0.00	-5.07	0.00	0.00
3	0.00	1.00	0.00	6.44	0.00	0.00	5.07	0.00
4	1.00	0.00	6.44	0.00	0.00	5.07	0.00	0.00
5	0.00	-1.00	0.00	-1.41	55.00	0.00	-1.41	40.00
6	-1.00	0.00	-1.41	0.00	55.00	-1.41	0.00	40.00
7	0.00	1.00	0.00	1.41	55.00	0.00	1.41	40.00
8	1.00	0.00	1.41	0.00	55.00	1.41	0.00	40.00
9	0.00	-1.00	0.00	-1.41	60.00	0.00	-1.41	45.00
10	-1.00	0.00	-1.41	0.00	60.00	-1.41	0.00	45.00
11	0.00	1.00	0.00	1.41	60.00	0.00	1.41	45.00
12	1.00	0.00	1.41	0.00	60.00	1.41	0.00	45.00

APPENDIX C

WIND INTENSITY

Table B78: Wind intensity for SSCT100

Sub Panel centroid from Ground [m]	Parallel Wind Velocity pressure q_{zp} [kN/m ²]				Diagonal Wind Velocity pressure q_{zd} [kN/m ²]			
	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.921	0.431	0.563	0.712	0.879	0.517	0.675	0.854	1.055
14.913	0.591	0.772	0.977	1.207	0.710	0.927	1.173	1.448
24.902	0.685	0.894	1.132	1.397	0.821	1.073	1.358	1.676
34.890	0.754	0.985	1.246	1.538	0.905	1.181	1.495	1.846
43.680	0.804	1.050	1.329	1.640	0.965	1.260	1.594	1.968
51.171	0.841	1.098	1.390	1.716	1.009	1.318	1.668	2.060
57.461	0.869	1.135	1.437	1.774	1.043	1.362	1.724	2.129
62.457	0.890	1.163	1.472	1.817	1.068	1.395	1.766	2.180
67.452	0.910	1.189	1.504	1.857	1.092	1.426	1.805	2.229
72.445	0.929	1.213	1.535	1.895	1.115	1.456	1.842	2.275
76.960	0.945	1.234	1.562	1.929	1.134	1.481	1.875	2.314
80.955	0.959	1.252	1.585	1.957	1.150	1.503	1.902	2.348
84.948	0.972	1.270	1.607	1.984	1.166	1.523	1.928	2.380
87.985	0.982	1.282	1.623	2.004	1.178	1.539	1.948	2.404
89.984	0.988	1.291	1.633	2.017	1.186	1.549	1.960	2.420
91.982	0.994	1.299	1.644	2.029	1.193	1.559	1.972	2.435
93.980	1.000	1.307	1.654	2.042	1.201	1.568	1.985	2.450
95.500	1.005	1.313	1.661	2.051	1.206	1.575	1.994	2.461
96.500	1.008	1.317	1.666	2.057	1.210	1.580	2.000	2.469
97.500	1.011	1.321	1.671	2.063	1.213	1.585	2.006	2.476
98.500	1.014	1.324	1.676	2.069	1.217	1.589	2.011	2.483
99.500	1.017	1.328	1.681	2.075	1.220	1.594	2.017	2.490

WIND INTENSITY

Table B79: Wind intensity for SSCT80

Sub Panel centroid from Ground [m]	Parallel Wind Velocity pressure q_{zp} [kN/m ²]				Diagonal Wind Velocity pressure q_{zd} [kN/m ²]			
	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.902	0.430	0.562	0.711	0.878	0.516	0.674	0.854	1.054
14.890	0.591	0.772	0.977	1.206	0.709	0.926	1.172	1.447
23.680	0.675	0.881	1.115	1.377	0.810	1.058	1.339	1.653
31.171	0.730	0.953	1.207	1.490	0.876	1.144	1.448	1.788
37.461	0.769	1.005	1.272	1.570	0.923	1.206	1.526	1.884
42.457	0.797	1.041	1.318	1.627	0.957	1.250	1.582	1.953
47.452	0.823	1.075	1.361	1.680	0.988	1.290	1.633	2.016
52.445	0.847	1.106	1.400	1.728	1.016	1.327	1.680	2.074
56.960	0.867	1.133	1.433	1.770	1.041	1.359	1.720	2.124
60.955	0.884	1.155	1.461	1.804	1.061	1.386	1.754	2.165
64.948	0.900	1.176	1.488	1.837	1.080	1.411	1.786	2.205
67.985	0.912	1.191	1.508	1.861	1.094	1.430	1.809	2.234
69.984	0.920	1.201	1.520	1.877	1.104	1.441	1.824	2.252
71.982	0.927	1.211	1.533	1.892	1.113	1.453	1.839	2.270
73.980	0.934	1.220	1.545	1.907	1.121	1.464	1.853	2.288
75.500	0.940	1.228	1.554	1.918	1.128	1.473	1.864	2.302
76.500	0.943	1.232	1.559	1.925	1.132	1.479	1.871	2.310
77.500	0.947	1.237	1.565	1.932	1.136	1.484	1.878	2.319
78.500	0.950	1.241	1.571	1.939	1.140	1.490	1.885	2.327
79.500	0.954	1.246	1.577	1.946	1.145	1.495	1.892	2.336

WIND INTENSITY

Table B80: Wind intensity for SSCT60

Sub Panel centroid from Ground [m]	Parallel Wind Velocity pressure q_{zp} [kN/m ²]				Diagonal Wind Velocity pressure q_{zd} [kN/m ²]			
	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3.680	0.396	0.518	0.655	0.809	0.476	0.621	0.786	0.971
11.171	0.544	0.711	0.900	1.111	0.653	0.853	1.080	1.333
17.461	0.619	0.808	1.022	1.262	0.742	0.969	1.227	1.515
22.457	0.665	0.868	1.099	1.356	0.798	1.042	1.318	1.628
27.452	0.704	0.919	1.164	1.437	0.845	1.103	1.396	1.724
32.445	0.738	0.964	1.220	1.507	0.886	1.157	1.465	1.808
36.960	0.766	1.001	1.267	1.564	0.920	1.201	1.520	1.877
40.955	0.789	1.031	1.304	1.610	0.947	1.237	1.565	1.933
44.948	0.810	1.058	1.340	1.654	0.972	1.270	1.608	1.985
47.985	0.826	1.078	1.365	1.685	0.991	1.294	1.638	2.022
49.984	0.835	1.091	1.381	1.705	1.002	1.309	1.657	2.046
51.982	0.845	1.103	1.396	1.724	1.014	1.324	1.676	2.069
53.980	0.854	1.115	1.412	1.743	1.025	1.338	1.694	2.091
55.500	0.861	1.124	1.423	1.757	1.033	1.349	1.707	2.108
56.500	0.865	1.130	1.430	1.766	1.038	1.356	1.716	2.119
57.500	0.869	1.136	1.437	1.774	1.043	1.363	1.725	2.129
58.500	0.874	1.141	1.444	1.783	1.049	1.369	1.733	2.140
59.500	0.878	1.147	1.451	1.792	1.054	1.376	1.742	2.150

WIND INTENSITY

Table B81: Wind intensity for SSCT45

Sub Panel centroid from Ground [m]	Parallel Wind Velocity pressure q_{zp} [kN/m ²]				Diagonal Wind Velocity pressure q_{zd} [kN/m ²]			
	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s	Case I 35m/s	Case II 40m/s	Case III 45m/s	Case IV 50m/s
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2.461	0.353	0.462	0.584	0.721	0.424	0.554	0.701	0.865
7.457	0.485	0.634	0.802	0.990	0.582	0.760	0.962	1.188
12.452	0.562	0.733	0.928	1.146	0.674	0.880	1.114	1.375
17.445	0.618	0.808	1.022	1.262	0.742	0.969	1.227	1.514
21.960	0.660	0.863	1.092	1.348	0.792	1.035	1.310	1.617
25.955	0.693	0.905	1.145	1.414	0.831	1.086	1.374	1.696
29.948	0.722	0.943	1.193	1.473	0.866	1.131	1.431	1.767
32.985	0.742	0.969	1.226	1.514	0.890	1.163	1.472	1.817
34.984	0.754	0.985	1.247	1.540	0.905	1.182	1.496	1.847
36.982	0.766	1.001	1.267	1.564	0.920	1.201	1.520	1.877
38.980	0.778	1.016	1.286	1.588	0.934	1.219	1.543	1.905
40.500	0.787	1.027	1.300	1.605	0.944	1.233	1.560	1.926
41.500	0.792	1.035	1.309	1.617	0.951	1.242	1.571	1.940
42.500	0.798	1.042	1.318	1.628	0.957	1.250	1.582	1.953
43.500	0.803	1.049	1.327	1.638	0.963	1.258	1.593	1.966
44.500	0.808	1.055	1.336	1.649	0.970	1.267	1.603	1.979

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