ANALYSIS AND DESIGN OF MICROWAVE ANTENNA LATTICE TOWERS

By

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Major Project

On

ANALYSIS AND DESIGN OF MICROWAVE ANTENNA LATTICE TOWERS

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

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Guide
Prof. U. V. Dave

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CERTIFICATE

This is to certify that the Major Project entitled "Analysis and design of Microwave antenna lattice tower" submitted by Mr. Patel Darshan P. (06MCL012), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Due to the expansion of telecommunications and broadcasting systems, a large number of lattice towers are used to support the microwave antennas. The tower supports the radio, television and telephone antenna to transmit telecommunication signals over the long distances. Therefore damage to them can significantly increases losses due to natural disasters. The main feature in designing the tower is its height; it is usually several times larger than the corresponding horizontal dimension.

The primary objective of work is to understand the effect of wind on microwave towers and to carry out wind analysis of these structures. Lattice tower due to its height is mainly predominant towards the wind, therefore static analysis and dynamic analysis due to the randomly varying wind action is essential as the natural frequency of tower becomes very low due to the wind action. The comparison of wind analysis and seismic analysis is done to evaluate the critical effect out of the two for designing of the tower. The analysis for the lattice tower is based on IS 875 (part-III), 1987. Static analysis is carried out manually, with help of staad-pro software and forces on tower as a whole is calculated.

Parametric study is carried out to find the economical aspect of the tower. The main parameters considered for the study are the base width and vertical profile of the tower. The main objective of parametric study is to see the effects of parameters considered on the weight and deflection of the tower because microwave towers in particular are required to obey very stringent serviceability criteria, usually specified in the terms of tilt and twist limits.

The tower behaves essentially as a cantilever structure fixed at the base. The deflection goes on reducing as the height of the tapering portion increases. At the same time there is reduction in deflection as the base width increases because the stiffness of structure is increased due to its slanting legs which slant more with increase in base width making tower more sensitive for vertical deflections compared to horizontal deflections.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABBREVIATION, NOTATION AND NOMENCLATURE

A_h	Horizontal seismic coefficient
B	Background factor indicating a measure of slowly varying component of fluctuating wind load
C_f	Force coefficient
E	Measure of available energy in wind stream at the natural frequency of structure
F	Wind force on structure
fy	Yield stress of steel
fcc	Elastic critical stress in compression
F_{ij}	Force in the i^{th} member of j^{th} load condition
G	Gust factor
g_f	Peak factor defined as the ratio of expected peak value to the root mean value of fluctuating component
L(h)	A measure of turbulence length scale
P	Axial compressive load
Q_{s1}	Wind force on dish
Q_{t1}	Wind force on tower
Q_i	Design lateral force in panels
r	Roughness factor which is dependent upon size of structure in relation to ground roughness
S	Size reduction factor
T_a	Fundamental natural time period of vibration
U_p	Uplift pressure
V_B	Base shear
V_b	Regional basic wind speed
V_z	Design wind velocity at height z
W_i	Seismic weight of panel
Z	Zone factor

1. INTRODUCTION

1.1 GENERAL

In every country, the development of telecasting and broadcasting networks has continued to rise. The rate of growth is greater in developing countries on account of the comparatively low base of telecasting and broadcasting networks. This in turn, has led to the increase in the construction of steel towers of various configurations and heights as shown in Fig 1.1. These towers are predominately used for-

1. Microwave transmission for communication
2. Radio transmission (short and medium wave wireless)
3. Television transmission
4. Satellite reception
5. Air traffic control
6. Power transmission lines
7. Meteorological measurements
8. Derrick and Crawler Cranes
9. Oil drilling masts
10. Overhead water tanks

The characteristic dimension of a tower is its height. It is usually several times larger than the horizontal dimensions. Frequently the area which may be occupied at ground level is very limited and thus, rather slender structures are commonly used.

According to the size and type of loading, which depends on the purpose of the tower, towers are grouped under two heads.

According to the type of loading

- Towers with Large Vertical loads
- Towers with mainly horizontal wind loads

According to the size of tower

- Light weight tower

➢ Heavy weight tower

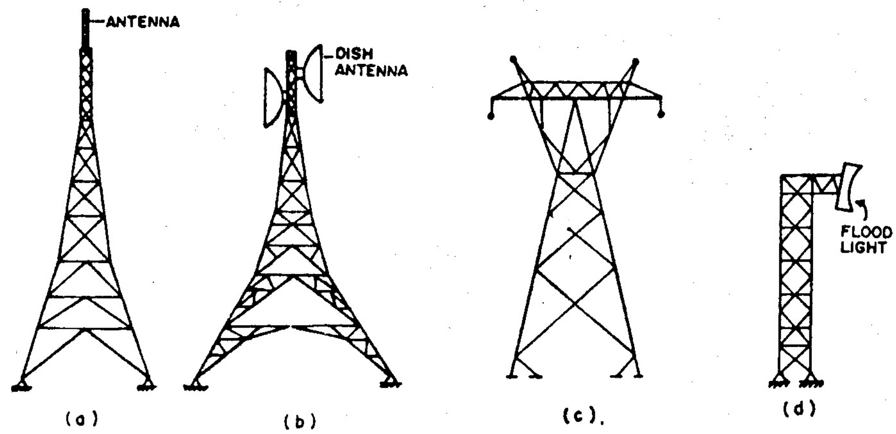


Fig 1.1 various types of towers

The types of tower to be used depend upon number and types of antennas to be mounted on tower and wind load of the place. If the wind pressure is very high and numbers of antennas are less than four, the requirement of tower is of light weight tower and if numbers of antennas are more than four, then heavy weight towers are required.

The cross-section of the towers in the plan are either a triangle or square as shown in Fig1.2.These towers are called lattice towers.

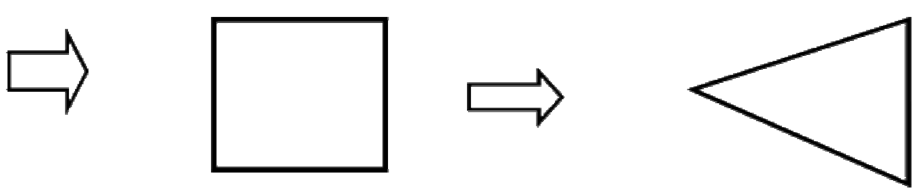


Fig 1.2 Plan shapes of tower

1.2 TYPES OF TOWER

The towers, subjected predominately to wind loads, are called lattice towers. The height of towers may vary from 20 m to 500 m, in which commonly used heights are:

- 100 to 400 m for television,
- 50 to 200m for radio transmission and communication networks,
- 15 to 50 m for flood lights,

- 10 to 45 m for power transmission

Steel towers are constructed in a number of ways but the most efficient use of material is achieved by using an open steel lattice. The use of an open Lattice avoids presenting the full width of structure to the wind but enables the construction of extremely lightweight and stiff structures. Lattice towers are typically square or triangular and have low redundancy. The legs are braced by the main bracings: both of these are often propped by additional secondary bracing to reduce the effective buckling lengths.

Towers are classified into two major groups based on their structural reason. They are:

(1) Self supporting towers

(2) Conventional guyed towers

1.2.1 SELF SUPPORTING TOWER

The use of free standing latticed steel towers to support cellular and microwave antennas has been intensive in last few years with the expansion in telecommunicating systems. Due to light weight of this structure, wind forces are the primary concern in the design. Free-standing towers are normally square in plan and are supported on ground or on tall buildings usually by four legs, and they act as a cantilever trusses in carrying the wind and seismic loads.These towers demand more steel but less base area and are suitable in many situations.

Fig 1.3 Typical view of a microwave tower

Telecommunication towers, such as the ones used for emergency response systems, require elevated antennas to effectively transmit and receive microwave communication as shown Fig 1.3. The stress calculation in self-supported steel lattice tower is usually based on conventional linear elastic analysis and this is generally found to be adequate, as the deflections are small in most cases.

Self supporting towers used for communication purposes must be designed to meet stringent deflection requirements. This is necessary since a minor misalignment of satellite dishes mounted on the tower may result in loss of communication signals, which could lead to disruptions or poor quality service to thousands of customers. Wind induced vibrations are the primary source for excessive tower deflections.

1.2.2 CONVENTIONAL GUYED TOWER

Guyed towers are exclusively used for communication purposes and structural reliability of guyed communication tower is becoming an important factor in the ever-increasing demand for wireless communication technologies. Guyed towers are frequently designed to heights of 300 meters and are used to transmit and receive high frequency signals for various electronic communication systems including those associated with electric power distribution.

Guyed towers are hinged to base, and are supported by guyed wires attached to it at various levels, to transmit the wind forces to the ground as shown in Fig 1.4. Due to this reason, guyed towers of same height are much lighter than a self-supporting tower. However, it requires much large space in plan to accommodate the guyed ropes.

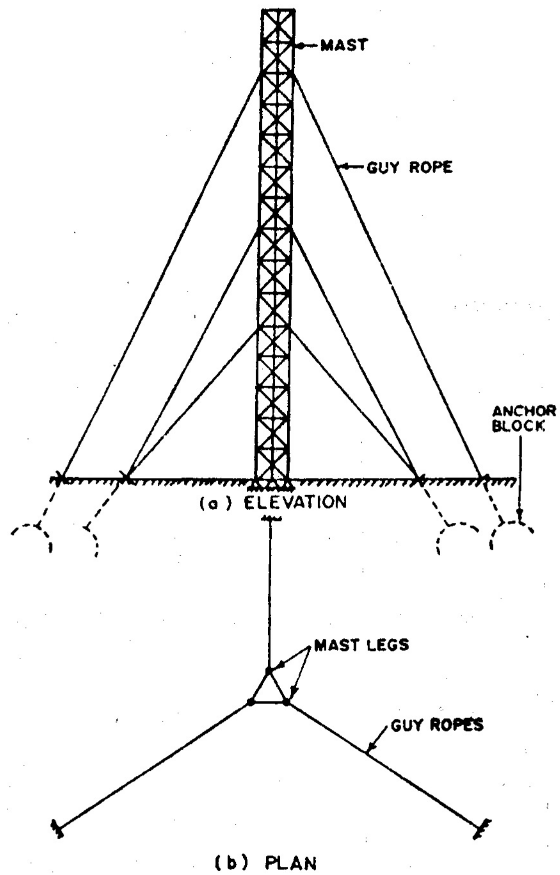


Fig: 1.4 Typical guyed tower

1.3 IMPORTANCE OF A MICROWAVE TOWERS

Over the last decade, as the government permitted the unleashing of several new communication technologies in the country, there has been a spurt in the use of towers for GSM (Global system for mobile), CDMA (Code division multiple access) , point to point and other applications. Since 1995, some 6000 towers have been created in the country to support the first two GSM operators in all the circles. With the imminent launch of the third and fourth operators, a similar number will be added in the short-term. The demand is likely to continue for many years to come as new networks, technologies make wireless the more viable option for data and voice networks and existing operators attempt to blanket the country with their coverage.

A number of advantages offered by the free-standing latticed steel tower, such as an ease of fabrication, transportation, and erection; have made the erection of these towers popular in diverse field conditions which pose widely different wind environment. Further, the heights of the steel tower have been going up with their application in the TV transmission system, heights above 200 m being no longer usual.

Hence the necessities of communication towers are also in great demand. So, it is highly essential to optimize the geometry of communication Towers.

1.4 STRUCTURAL CONFIGURATIONS

The corner of the self supported tower contains vertical or nearly vertical members are called legs or column members and they are main load bearing elements. The leg members are interconnected by bracings with or without horizontals which carry a nominal force. There are a number of different configurations that are commonly used in lattice towers and masts. In order to determine the buckling resistance of a lattice member it is necessary to first derive the appropriate geometric length of the member between intersection points providing restraint ,defined as the "system length"(effective length).The relevant slenderness ratio based on system length and appropriate radius of gyration is calculated and effective slenderness is determined appropriate to end condition**.** The main parts of the tower as shown in Fig 1.5 are:

(1) Leg members
(2) Primary bracing members
(3) Secondary members

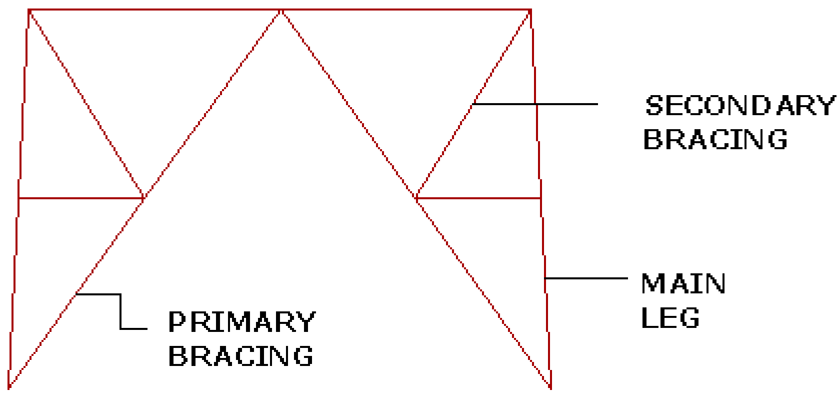


Fig 1.5 Panel showing components of tower

(1) Leg members

- Single members

Single angles, tubular section or solid rounds may be used for leg sections. The capacity of leg will depend on the pattern and the connection of bracings used to stabilize the leg. For legs or chords with axial compression load braced symmetrically in two normal planes in case of triangular structures, the slenderness should be determined from the system length between nodes.i.e intersections of bracings. The arrangement of double angle section in plan is shown in Fig 1.6.

- Compound members

Compound members for legs may be built up with two angles in cruciform section i.e. the section which are jointed together and then taken as fully composite if welded continuously. When intermittently connected the possible additional deformation due to shear should be taken into account by modifying the slenderness ratio.

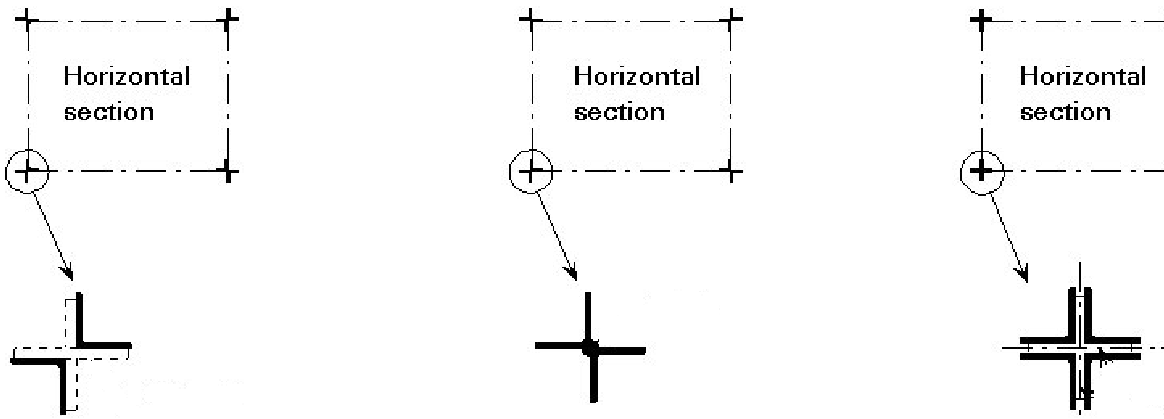


Fig 1.6 Single and double angle sections

(2) Primary bracing members

- Single lattice

A single lattice is commonly used where the loads are light and the length relatively short, as for instance near the top of towers or in the light guyed masts.(Figure 1.7a)

- Cross bracing system

Cross bracing system without secondary members

The load is equally split into tension and compression, both members are continuous. The members are adequately connected where they cross ,then the centre of connection may be considered as a point of restraint both transverse to and in the plane of the bracing (Figure 1.7b) .When the load is not equally split into tension and compression and provided both members are continuous.

Cross bracing system with secondary members

When secondary members are inserted, they reduce the system length of bracing members. Buckling should be checked over the system length on the rectangular axis for buckling to the bracing.

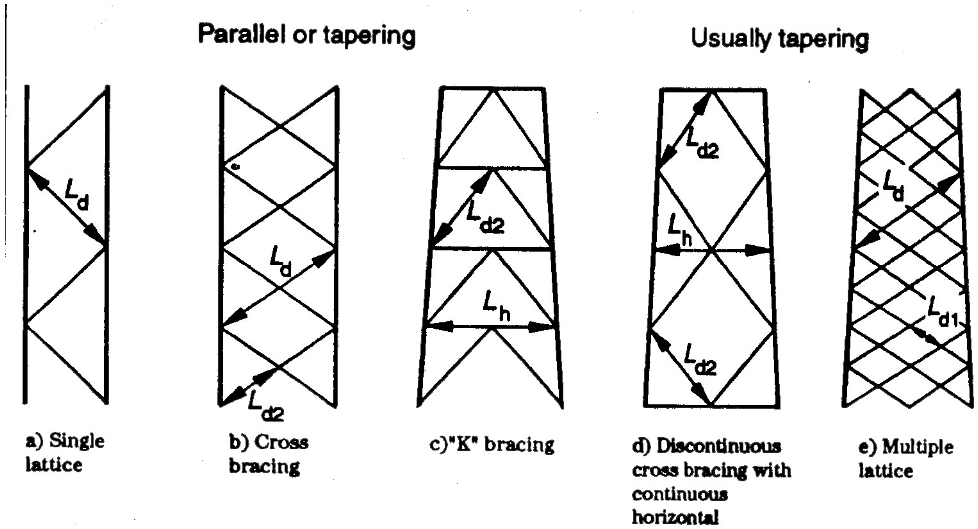
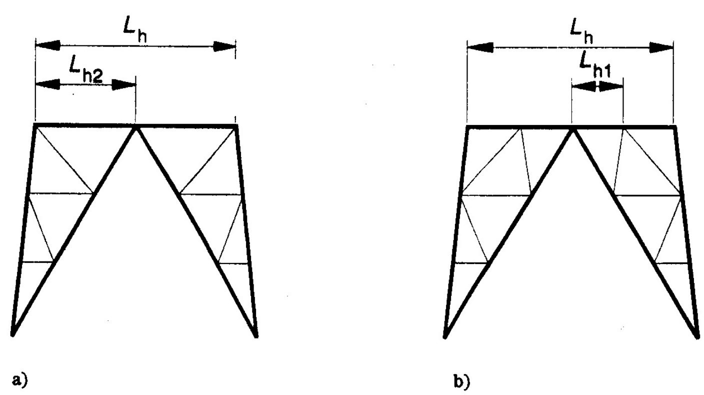


Fig: 1.7 typical primary bracing patterns

- K Bracing system

Such a bracing gives large head rooms and hence can be used in lower panels where large head room is required (Figure 1.8).The structure is statically determinate. However the length of diagonal is reduced. The system is suitable for towers of 50 m to 200 m height.



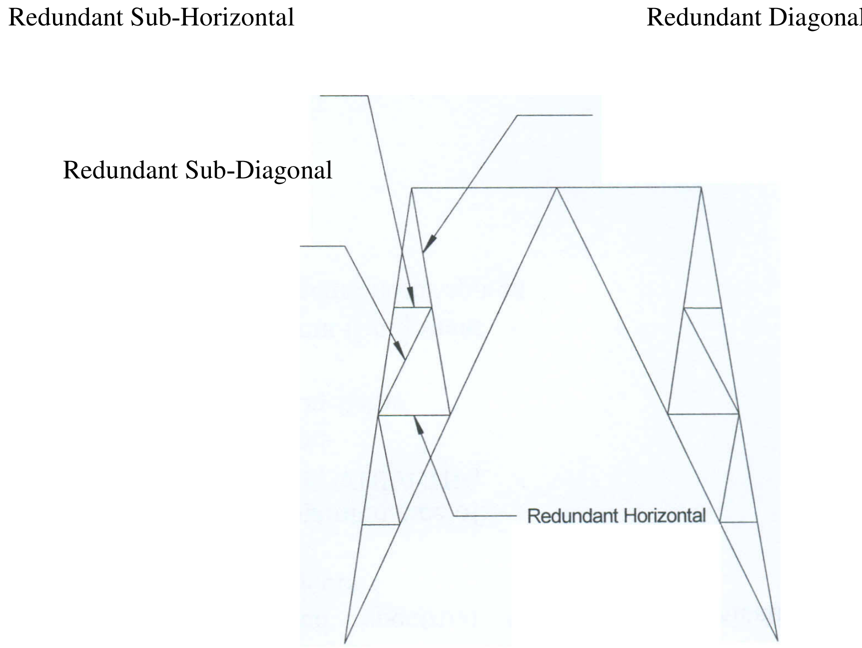


Fig: 1.8 Typical K type bracing

- Cranked K bracing

For large tower widths, a crank or a bend may be introduced into the main diagonals as shown in Fig 1.9. This has the effect of reducing the length and size of the redundant members but produces high stresses in the members meeting at the bend and necessitates fully triangulated transverse support at the joint. Diagonals and horizontals should be designed as k bracing, with the system lengths for diagonals based on their length to the knee joint.

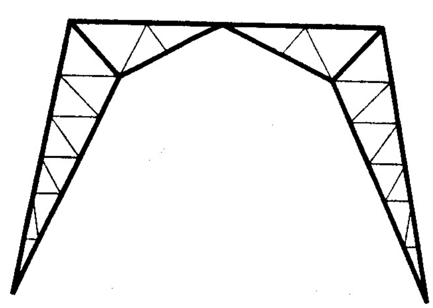


Fig: 1.9 Cranked K Bracing

1.5 OBJECTIVE OF STUDY

The preliminary aim is to study the behavior of tower subjected to different loads. The case study of a microwave steel tower supporting an antenna at its top is to be considered. Main objectives of study are as follows:

- To perform the static linear analysis of tower due to dead load and wind load respectively

- To perform the Dynamic analysis of an Microwave tower as per IS:875 (part-3) provisions

- To perform the seismic analysis of a tower according to IS:1893 (part-1) provisions and compare the results with static wind analysis

- To generate the model of tower in STAAD and analyze the structure

- To design the tower members according to the most critical condition out of wind and seismic

- To design of suitable foundation for the tower

- To prepare detailed sketches of connection detail of the tower

- To carry out parametric study of various alternatives on the basis of weight and deflection of tower by varying the base width and vertical profile of the tower and select the most optimum alternative

1.6 SCOPE OF WORK

Scope of work decided is as follows,

- To carry out static wind analysis and design of a 56 meter high square shape tower of 8 m base width and distributed in ten panels.

- To carry out dynamic analysis of the same tower using Gust response factor method and compare the results with the static analysis.

- To carry out seismic analysis of the tower and compare the results with static wind analysis.

- To prepare the model of the tower in STAAD 2007 an study the behavior according the the loads acting on tower

- To carry out foundation design and connection design for the microwave tower and prepare detailed sketches.

- To work on the economical aspect of tower by trying various alternatives considering parameters weight and deflection for the tower

Fig 1.10 shows the flow of work in which wind and seismic analysis and design pertaining to the critical condition is done. Various alternatives by varying the base width and vertical profile of tower is done.

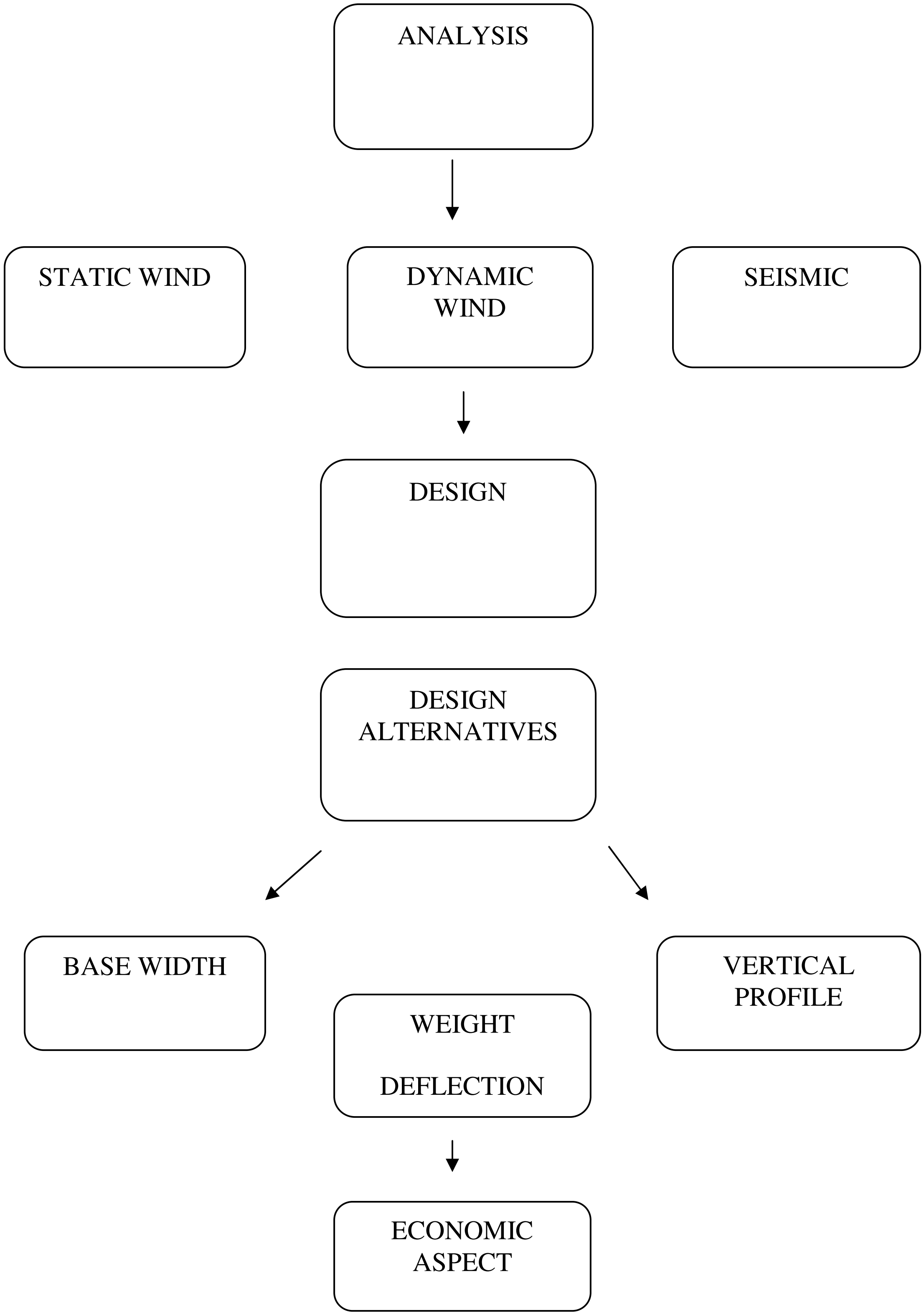


Fig 1.10 Flow of work

1.7 ORGANSIATION OF REPORT

The report has been divided in nine chapters

First chapter outlines the introduction for different types of towers and various structural configurations for towers.

Chapter two reviews the published literature on the wind analysis and seismic analysis of tower and corresponding codal provisions.

Chapter three gives the information of the microwave tower and antennas to be mounted on tower. It also gives the basic introduction about the components of tower.

Chapter four discusses the methodology of analysis of tower, in which static and dynamic analysis along with seismic analysis of a 56 m high tower is included.

Chapter five includes the design guidelines for the tower members and the sections obtained after the process of design. This chapter also contains information regarding types of foundation and design procedure for foundation for tower.

Chapter six gives use of software and shows the process of building the model and applying the loads.

Chapter seven contains information of parametric study of tower carried out by varying the base width and vertical profile of tower.

Chapter eight gives the discussion of results about manual approach, STAAD analysis and about various alternatives considered.

Chapter nine consist of details regarding conclusion and future scope of work.

2. LITERATURE SURVEY

2.1 GENERAL

The first stage for getting introduced to the microwave tower concept is through the literature survey. Literature survey is carried out to get more information for the static analysis and dynamic analysis of tower. The books, papers and standards referred explained all the factors related to the analysis and design of the towers.

2.2 LITERATURE SURVEY

2.2.1 Analysis of tower

- **Saraswat** [1] et al. discussed the design of the microwave tower subjected to the wind action with two different philosophies of Peak wind and Mean wind approach. Provisions of Indian codes are described in detail regarding computation of loads on antennas and tower body itself.

- **Shanmugasundaram** [2] et al. discussed the dynamic response characteristics of lattice tower supporting antenna and provided the comparison of codal provisions and the analytical results with the measured dynamic response. A full scale field experiment on a 52 m tall steel lattice tower has been carried out to measure wind characteristics and structural response. The GRF (Gust response factor) for base bending moment and top deflection have been evaluated using the measured structural response and compared with different codes.

- **Carill** [3] et al. discussed the influence of wind turbulence on the drag coefficients, which are the functions of tower solidity and used to calculate the wind forces. The interference of antenna dishes on wind forces of lattice towers is also discussed. The antenna model used and the arrangement of antennas on tower for testing are shown in Fig 2.1.

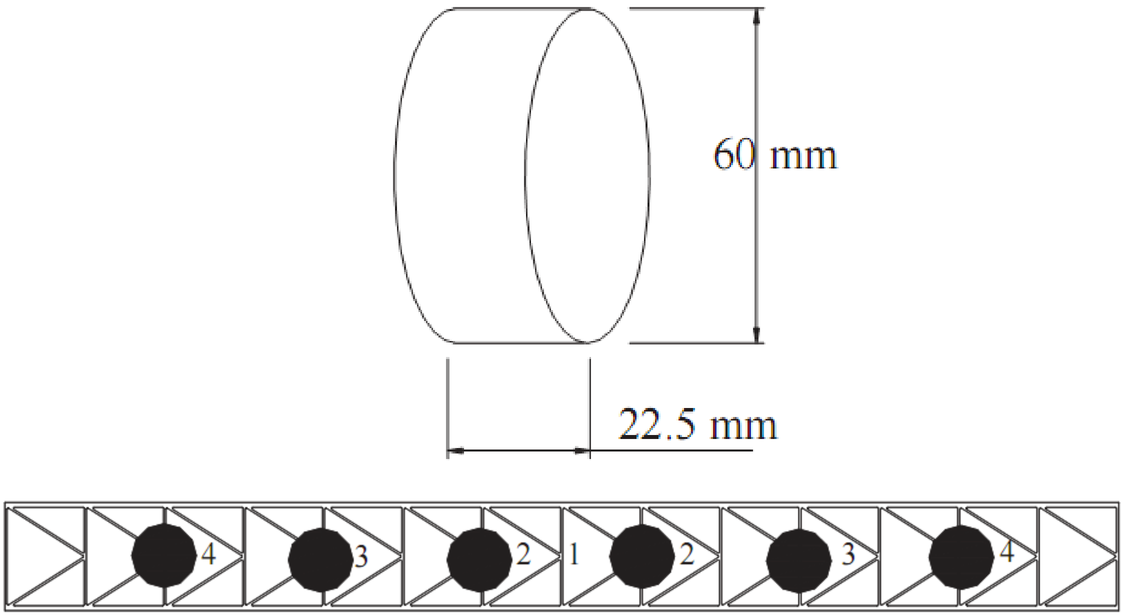
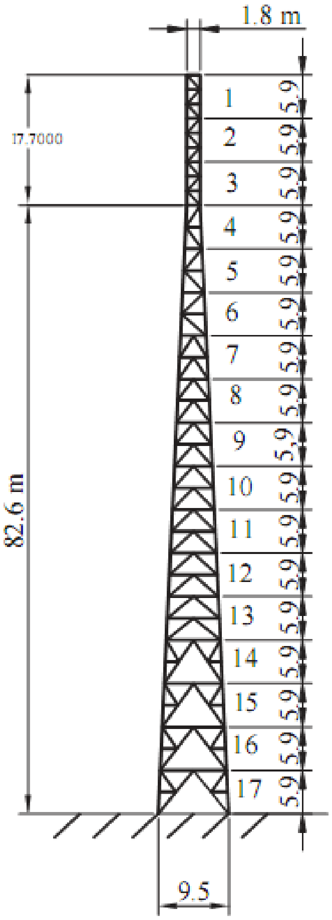


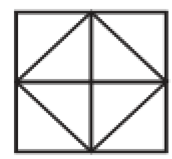
Fig 2.1 Model of microwave antenna disk and position of antenna disk on the model

Three section models of dimension as shown in table 2.1 were designed and constructed based on existing latticed tower of 100.3 m tall of 17 panels as shown in Fig 2.2

Table 2.1 Description of section models

Model 1	0.102x0.102x1.035 m
Model 2	0.102x0.204x1.035 m
Model 3	0.102x0.304x1.035 m





Horizontal bracing

Fig 2.2 lateral view of a 100.3 m tall tower

The wind incident angle, tower solidity, shielding effect and the influence of wind turbulence on drag coefficient were analyzed and the results were compared with different codes.

- **Holmes** [4] discussed the analysis of along wind dynamic response for free standing lattice tower. Effective static load distributions for the mean, background fluctuating, and resonant components of the load effects are derived.

  Equivalent static pressure distributions for the fluctuating and dynamic along-wind loads acting on lattice towers have been evaluated. An approximation to the distribution for the background loading can be used for both the shearing force and bending moment. This distribution, however, does depend on the height on the tower at which the load effects are being evaluated.

- **Loganathan** [5] gives the overview of analysis and design of steel transmission and communication towers. He discussed about tower configuration, tower testing method and impact of computers and gave the following steps for tower design:

    - Preliminary design-selection of the general configuration, overall geometry and initial component sizing
    - Estimation of loads including geometry and component size dependant loads
    - Analysis for component forces
    - Compute component resistances and check against corresponding forces. If inadequate, revise component sizes and repeat from step 2 until all the components are structurally adequate
    - Compute total cost of tower
    - Select different geometry and repeat steps 2 to 5 till a feasible optimal design is reached, considering availability of component sizes, joint detailing, fabrications, transport, assembly and erection

- **Dayaratnam** [6] discussed the introduction of various types of towers and basic structural configurations adopted. He also explained the computer program for analysis of the tower.

2.2.2 Seismic analysis of tower

- **Ghodatri** [7] et al. investigated the dynamic behavior of the self-supporting towers with four legs. The paper discussed the comparison of static analysis and the seismic analysis of the tower. These comparison results the necessity of considering the earthquake loads in tower analysis and design.

Details of 10 existing self-supporting telecommunication towers with heights varying from 18 to 67 m have been incorporated in the work.

- **McClure** [8] et al. discussed about the simple formulas proposed for the prediction of seismic response indicators such as maximum base shear, vertical dynamic reaction, and the overturning moment. These simplified predictors are applicable to towers of regular geometry and mass distribution and for which serviceability limits can be exceeded during strong motion.

2.2.3 Deflection of tower

- **Glanville** [9] et al. performed the full scale deflection measurement on 67 m steel frame tower and 233 m steel truss tower under wind loading. Measurements were acquired by various means and compared with along with theoretical estimates. The latter were obtained using a simple frequency domain prediction that incorporates some practical results and remarks. The dynamic cross-wind response of lattice towers were also measured and discussed.

  Full scale measurements were performed upon two lattice towers under wind loading. Commercially available accelerometers and He-Ne laser beam were used to measures background and resonant deflection of a 67 m steel frame communication tower. Cross-wind deflection was found to be approximately half the along-wind background deflection of the tower.

2.2.4 Modeling of tower

- Seetharaman [10] discussed the behavior of tower systems. Computer aided analysis of tower-like structure systems are presented and compared with conventional analysis procedures. In which matrix methods for tower analysis and computer modeling of the tower behavior are discussed. He stated that computer aided analysis provides a good insight into the stress distribution in various members due to external loadings.

2.2.5 Codal review of the lattice tower

- IS: 800-1984 provisions have been studied for the allowable stresses and design consideration for tower members, as no separate codes for the design of the lattice towers are available

- IS: 875-3 provisions are used for the static analysis and dynamic analysis of the towers.

- BS: 8100 (part 3) provisions have been studied for different structural configurations, for the assessment of the strength of the members and connection tools of towers etc.

2.3 SUMMARY

During an overview of the literature survey, various factors affecting the tower are studied and need for static and dynamic analysis are discussed. The available information for different positions of antennas are also studied. It is learnt that IS: 875-part-3 provides a methodology for working out equivalent load on a dynamically sensitive structure based on gust factor approach; this information is used in design procedure of microwave towers.

3. MICROWAVE TOWERS

3.1 GENERAL

In this chapter, a brief discussion is made about the use and the componenets of the tower. Necessity of the Microwave towers and the components details of the structure are also discussed.

Microwave towers are highly repetitive and therefore their designs have to be commercially competitive. Substantial savings in the materials and the total cost can be achieved through selection of efficient structural configuration and rational and optimum designs without compromising on the safety and reliability of towers.

3.2 WHY USE OF MICROWAVE TOWERS

Microwave towers are typically, tall structures designed to support antennas for telecommunications and broadcasting including television. They are almost the tallest man made structure. Due to expansion of communication a necessity of microwave towers has been extended. Here are some points which led to the advantage of microwave self-supported towers over guyed towers,

- Microwave towers occupy less ground space than guyed towers. Therefore guyed towers can be provided only where large space is available.
- Microwave towers can carry more dead weight resulting from the mounted antennas, platform, climbing ladders etc.
- A lot more designs and alternatives are possible by varying its base width, vertical profile, panel heights etc.
- In addition construction of Self supported towers is much simpler than other more conventional towers and maintenance is found to be very safe.
- Self supported towers are also flexible for future expansion.

3.3 TYPES OF ANTENNA

An antenna is a device that is made to efficiently radiate and receive radiated electromagnetic waves. There are certain important characteristics that should be considered when choosing an antenna for communication as follows:

- Antenna radiation and patterns
- Power gain
- Directivity
- Polarization

There are many different types of antennas, characterized mainly by its shape and working frequencies as follows:

- Dipole antennas
- Multiple element dipole antennas
- Yagi antennas
- Flat panel antennas
- Parabolic disk antennas
- Slotted antennas

The use of flat panel antenna as in Fig 3.1 is intensive due to its light weight characteristics. The different panel antennas working on different frequencies are described below.

The data for different types of antenna working on different frequencies are taken from A-INFO telecommunication antenna source book. Information about dimension and the weight of antennas are also referred. Some flat panel antennas dimension and weight are tabulated in table 3.2.

Table 3.1: Working frequencies of panel antennas

OPERATING FREQUENCIES	COMMONLY USED FOR HEIGHTS
900 MHz	45m, 75m, 120m
1800 MHz	30m, 53m, 77m
1900 MHz	29m, 50m, 75m
2100 MHz	28m, 48m, 70m

Fig: 3.1 Typical view of Flat panel antennas

Flat panel antennas are configured in patch type format and physically in the shape of square or rectangular. Flat panel antennas are quite directional as they have most of power radiated in one direction in both vertical and horizontal planes. Flat panel antennas can be made to have varying amount of gain based construction. This can provide excellent directivity and considerable gain. As shown in table 3.2, panel antennas are manufactured in different dimensions and they are available of about 20 kg weight.

Table 3.2: List of different types of panel antennas

Dimension (mm)	Weight (kg)
780 x 360 x 210	6
1720 x 360 x 210	10
2260 x 340 x 210	14
2560 x 340 x 200	15
2260 x 380 x 260	16
2760 x 380 x 260	18
2500 x 380 x 260	16
780 x 260 x 210	7
1720 x 360 x 210	10.5
2260 x 360 x210	13.5
2760 x 360 x 210	16.5
1720x 380 x 260	14
2760 x 380 x 260	20

Antenna Mounting Frames

Frames for mounting antennas on towers or masts shall be designed in consideration with the type of tower structure, the type, weight and size of the antenna.

They are made from galvanized steel, stainless steel or aluminum. There are no welded parts. All joints are implemented with bolts and nuts.

Some basic designs exist for certain tower structural forms. Side mount with multiple antennas in straight panel is shown in Fig 3.2, and in trapezoidal antenna is shown in Fig 3.3.

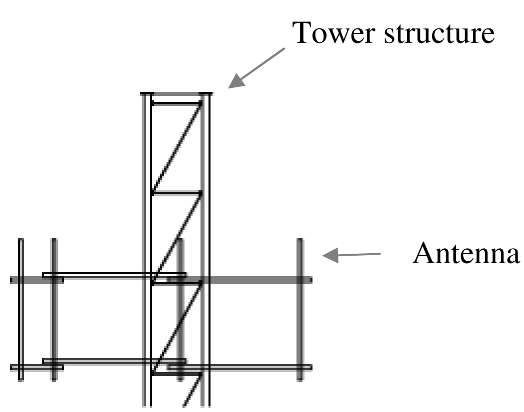


Fig 3.2 Side mounts with multi antennas

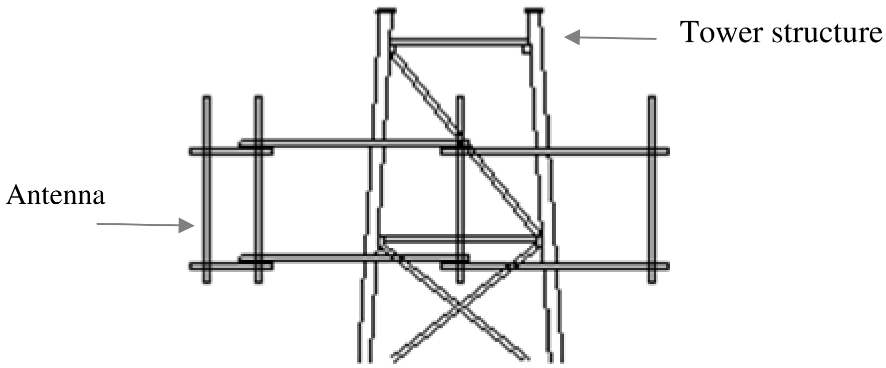


Fig 3.3 Antenna mounted on a self supported tower

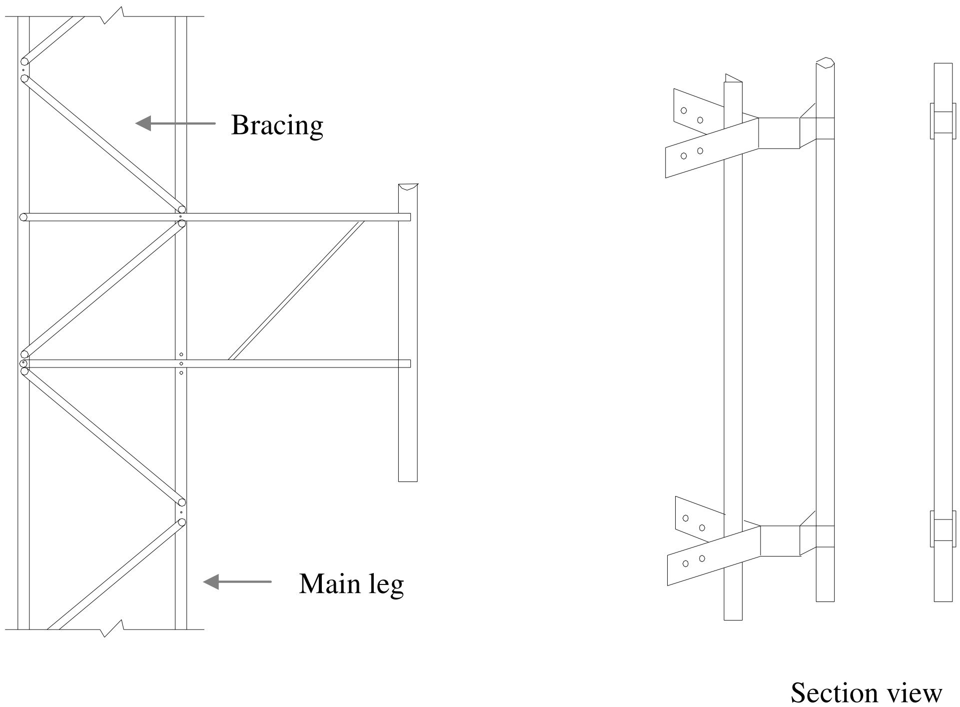


Fig3.4: Typical view of side antenna mounts

Fig 3.4 shows the single antenna mount on straight panel and the section view of the connection of antenna.

3.4 COMPONENT DETAILS

The typical communication tower is constituted of the following components:

a) Panels

Any lattice tower is the assembly of different panels. The panel is the component of tower, which consists of legs, diagonals, horizontals and redundant (secondary bracings) as shown in Fig 3.7. The height of the panel should be decided based on truncation requirements and economical design criteria. Generally for rooftop towers panel's height is in the multiples of 3 m and for green field towers the panel heights is in the multiples of 5 m to 10 m.

b) Legs

The corner vertical members of the tower are called Legs or column members and are main load bearing elements as shown in Fig 3.7. Generally angular sections are used for legs. If single sections are adequate starred angles or double starred angles are also used. Tubular sections are also used for legs, generally for triangular towers. Tubular sections are economical when the loading on the tower is less. The leg members behave in compression or tension depending on direction of wind.

c) Diagonals/Bracing

The inclined members which are connected to the legs diagonally are called diagonals. The loads from the legs will be transferred to the diagonals by shear diagonals which have both compression and tension.

d) Horizontal

The straight members which inter connects the legs horizontally are called horizontals. The horizontals carry shear/torsion. However, these horizontals are provided to support ladders and where the platforms are to be supported. The legs are interconnected by diagonals and horizontals. The horizontals are also subjected to bending at the platforms due to imposed live loads.

e) Redundants

The secondary members which are provided to reduce the slenderness ratio of the main members are called redundant as shown in Fig 3.5. The redundants carry nominal stresses and generally are designed from slenderness ratio criteria.

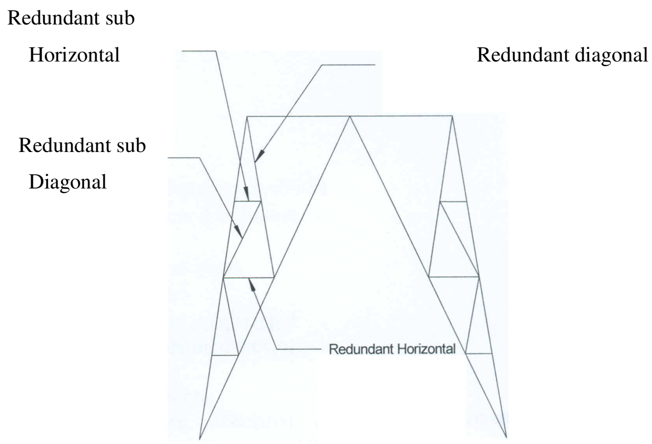


Fig 3.5 Tower members showing redundant

f) Gusset plates

Gusset plates are used to interconnect the members. For angular towers gusset plates are provided when it is not possible to accommodate number of bolts required for a connection in the space available on the members. For hybrid towers (legs tubular and bracings angular) gusset plates are required to connect secondary members to the legs. Gusset plates are also required to reduce the secondary stresses introduced due to the bracing members. Economically it is essential to avoid gusset plates as far as possible.

The other external components of the microwave communication tower are as listed below:

- Back marks
- Cover plates
- Cleats
- Climbing ladder
- Anchor plates
- Stiffners
- Flange plates
- Bracket members
- Stitch plates
- Base plates
- Railings
- Aviation lamps

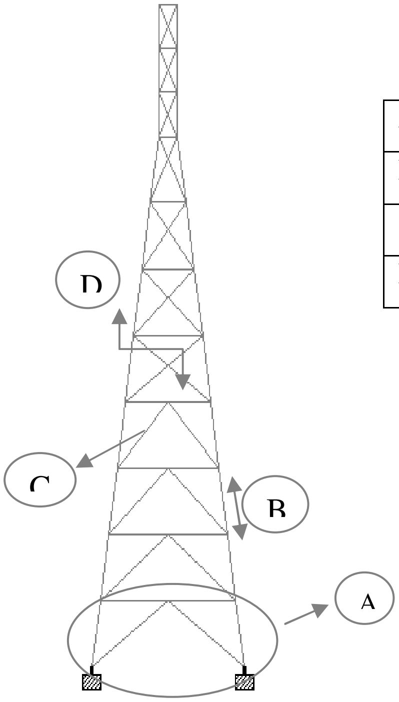
Fig 3.6 shows the typical view of the step bolts on tower, lightening arrester and climbing ladder on the tower respectively.

Lightening arrester

Step bolts

Climbing ladder

Fig 3.6 Typical external components of tower



A	Panel in tower
B	Main legs of tower
C	Diagonal/bracings of tower
D	Horizontal bracings

Fig 3.7 Components of tower

3.5 DESIGN PARAMETERS FOR A MICROWAVE TOWER

The designs of following parameters are important for a communication tower:

a) Tower type

Tower type is selected by users such as self-supporting tower or guyed type depending upon the area available at the location of tower.

b) Height of tower

The height of tower is based on the height required for antenna line of sight. Generally height of microwave tower varies from 9 m to 200 m (as shown in Fig 3.8).

c) Base width of tower

The centre to centre spacing between the tower footings i.e., base width at concrete level from the centre of gravity of the corner leg angle to that of the adjacent corner leg angle.

The width depends upon the magnitudes of the physical loads imposed upon the towers by antennas, wind loads and height of application of the load from ground level. Towers with larger base widths result in low footing cost and lighter leg members at the expense of longer bracings members.

The base width of the tower is determined from the formula

$$B = K (M)^{1/2} \quad \text{... (3.1)}$$

Where,

B = base width (m)

M = overturning moment (kN-m)

K = a constant

Value of K varies from 1.35 to 2.5

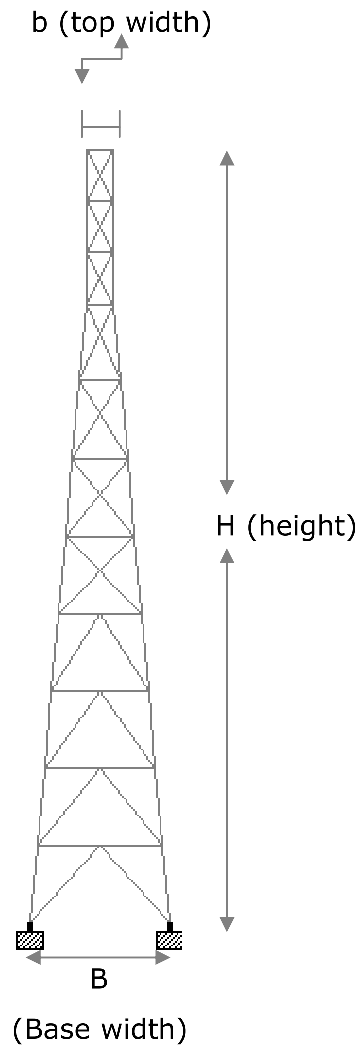


Fig 3.8 Dimensions of a tower

d) Top width of tower

Top width of tower is depending on the cross-section of tower and positions and width of ladder. If the tower is having a triangular cross-section and the ladder is inside the top width of tower should be fixed so as to accommodate the ladder inside. Generally the width of tower varies from 400 mm to 500 mm (as shown in Fig 3.8).

If the tower is having a square section to accommodate the ladder inside the top width of tower should be minimum 1500 mm to 1750 mm.

e) Geographic location of tower

Depending on the geographical location of the tower the basic wind speed is to be considered to calculate the wind load on the tower. Clause 5.2 of IS-

875(part-3) gives the basic wind speed applicable to 10 m high above mean ground level.

f) Design of tower members

Factor of safety adopted in the design of members have a great bearing on the cost of the structure and are chosen so that the structure proves economical as well as reliable.

However from reliability and serviceability consideration for microwave communication tower factor of safety is taken as 1.2 to 1.5.

g) Angle of twist and sway

From the serviceability consideration according to TIA/EIA-222-F standards (structural standards for steel antenna towers and antenna supporting structures) the angle of twist and sway should not be greater than 0.5 degrees.

h) Type, numbers and location of antennae

The type of antenna and their numbers and height at which these antennas are to be mounted has a greater impact in the cost of tower.

4. METHODOLOGY FOR ANALYSIS

4.1 GENERAL

Tower analysis, involves the determination of the behavior in terms of its internal stress resultants and displacements. All types of externally applied loads and their combinations, plus the dead weight of the tower are required in the solution for its analysis. Static analysis for towers is necessary because they are remarkably light in weight and flexible as compared to old massive masonry structures. Towers are also subjected to dynamic wind action due to the randomly varying wind action. When the natural frequency of the tower becomes adequately low, consideration of dynamic wind action on it becomes important.

4.2 TYPES OF LOAD

The towers are invariably analyzed as trusses, the loads are applied at the joints and the members are designed as tie or struts.

Following are various types of loads acting on tower:

- Gravity loads
  - (1) Weight of members
  - (2) Weight of platforms, railings, ladders, lifts etc.
  - (3) Weight of antennas, instrument, appliances etc.
  - (4) Weight of gussets and secondary bracings etc.
  - (5) Live loads

- Lateral loads
  - (1) Wind loads
  - (2) Seismic loads

- Erection loads

The following load combination is adopted during calculating of the maximum member stresses:

- Dead load + Wind load (wind parallel and diagonally to tower) … 4.1

The combination adopted for the design of tower members is adopted as stated in eqn 4.1. Dead load consists of the weight of antennas and the other ancillaries attached to the tower and wind load acting on the members of the tower.

The gravity loads are almost fixed, since these are dependant on structure design. Wind and the seismic loads are the most important of all and often control the design. The seismic load may not be critical as the mass of structure is not heavy and near the ground. Live loads on the tower are negligible when compared with other loads.

4.3 WIND LOAD ANALYSIS

Wind effect on structures can be classified as "static" and "dynamic". Wind effects are only dynamic in nature but generally these dynamic effects are expressed in terms of equivalent static load. Wind causes a random time-dependant load, which can be seen as a mean plus a fluctuating component. Structures will experience dynamic oscillations due to the fluctuating component of wind.

Static wind effect primarily causes elastic bending and twisting of structures. The dynamic effects of the wind are either periodic forces such as due to vortex shedding, flutter, Galloping and Ovailing or non-periodic such as turbulent buffeting.

There are two different approaches for designing the structures subjected to wind excitation.

(1) Static analysis by force coefficient method

(2) Dynamic analysis by Gust factor method

In the analysis of towers, the greatest uncertainty is associated with the insufficient knowledge of wind loads. Therefore a static linear three dimensional structural analysis is sufficient for the most of the time. Dynamic analysis of self supporting lattice towers are rarely necessary unless there are special circumstances such as high masses at top, which are used as the viewing platform or circular solid sections.

The following assumptions are made while performing analysis of communication towers:

- All members of bolted type frame work are pin-connected in such a manner that the members carry axial loads only
- Shear is distributed equally between the two members of a double web system
- Shear is carried by the diagonal member under tension in a Pratt system with members design for tension only, the others members being inactive
- Plan members at levels other than those at which external loads are applied or where the leg slope changes, are designated as redundant members
- Any face of the tower subjected to external loads lies in the same plane, so far as the analysis of the particular face is concerned
- The member on all the four faces of the tower share loads equally
- Vertical loads placed symmetrically and dead weight of the structure is shared equally by the four legs.

4.3.1 Force coefficient method

In this approach wind speed averaged over a short duration of few seconds is measured at every hours of day at a standard height of 10 m. From these values maximum speed in a year is determined which is the peak wind of the year. The yearly maximum wind recorded over a number of consecutive years are considered for computing extreme values , that is the peak wind speed for various return periods. These are also known as Basic wind speed (V_z).

Calculation of wind forces

Lattice towers are subjected mainly to two types of loads:

- Gravity loads due to self weight etc.
- Lateral loads due to wind action

The wind acting at the panel points have two effects as shown in Fig 4.1

- Horizontal shear effect due to lateral load
- Vertical force due to moment of lateral force

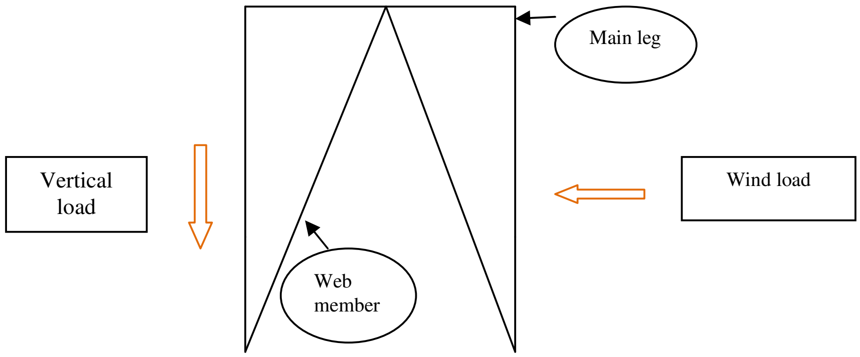


Fig 4.1 Wind effects on a tower

The lateral load due to wind is mainly resisted by web members while the gravity loads and the vertical force due to wind moments are resisted by chords or leg members.

The stepwise procedure for calculating wind force on tower by force coefficient method is as follows:

1. Design wind speed at any height (z)

The value of basic wind speed for tower as per IS: 875 (clause 5.3) is modified to obtain the design wind speed by using different modification factor and computed as:

$$V_z = V_b \times K_1 \text{ X } K_2 \text{ X } k_3 \qquad \dots (4.2)$$

Where,

K_1 = risk coefficient

K_2 = terrain category, height and structure size factor.

K_3 = Topography factor

2. Design wind pressure at any height (z)

The design wind pressure at any height above the mean ground level is obtained by following relationship:

$$Pz = 0.6\ Vz^2 \qquad \dots (4.3)$$

Where,

P_Z = Design wind pressure at height z in N/m^2

V_Z = design wind speed at height z in m/sec

3. Solidity ratio (δ) and force coefficient (C_f)

C_f is the net wind force coefficient which depends upon the solidity ratio (δ) of the tower. The values of force coefficients for square tower and Triangular tower are tabulated in Table 4.1.

Solidity ratio (δ) for the towers varies from 0.05 to 0.5 and it is assumed in the beginning of the design. After designing the members the solidity ratio is compared with the actual solidity ratio. Different solidity ratios for triangular and square towers as per IS: 875(table 30) is as follows:

Table 4.1 Force coefficients

Solidity ratio δ	C_f	
	Square towers	Equilateral triangular towers
0.05	4.0	3.3
0.1	3.8	3.1
0.2	3.3	2.7
0.3	2.8	2.3
0.4	2.3	1.9
0.5	2.1	1.5

4. Wind force in panels of the tower

Wind force in the panel of tower is calculated by (IS: 875 clauses 6.3)

$$F = A_e \times C_f \times P_z \quad \text{... (4.4)}$$

Where,

F = along wind load on structure at any height z corresponding to area Ae

C_f = net wind force coefficient, which depends upon solidity ratio (δ)

δ = Solidity ratio

A_e= effective frontal area

P_z = Design wind pressure at any height z

Force coefficient C_f for latticed tower depends on flow regime, solidity ratio, and shape of structural members.

When wind blows into the corner of the tower, the maximum load may be taken as 1.2 times the load for wind blowing against the face, irrespective of solidity ratio of the panel.

5. Distribution of wind load on the joints.

The wind force acting on the panels and the weight are distributed to the legs and the braces to all joints connecting the elements. Computation of loads at different joint are carried out panel by panel.

4.3.2 Gust Factor method

Any building or structure which satisfies either of the following two criteria shall be examined for dynamic effects of wind (IS: 875 clause 7.1)

- Building and closed structures with a height to minimum lateral dimension ratio of more than about 5.0 OR

- Building and closed structures with natural frequency in the first mode less than 1.0 Hz.

Wind force calculation

Along wind load on a structure on a strip area Ae at any height z is given by:

$$F_z = C_f \times A_e \times P_z \times G \qquad \text{... (4.5)}$$

Where,

F_z = along wind load on the structure at any height z corresponding to strip area Ae

C_f = force coefficient

A_e = effective frontal area considered for the tower at height z

P_z = design pressure at height z due to hourly mean wind obtained as eqn 4.3

G = Gust factor

$$G = 1 + gfr\sqrt{\left[B(1+\varnothing^2) + \frac{SE}{\beta}\right]} \qquad \text{... (4.6)}$$

g_f = peak factor defined as the ratio of the expected peak value to the root mean value of a fluctuating wind component and is calculated by the graphs plotted between g_f and the height as shown in Fig. 4.2 as per IS:875(part 3)fig 8, page 50

r = roughness factor which is dependent on the size of the structure in relation to the ground roughness

B = background factor indicating a measure of slowly varying component of fluctuating wind load as shown in Fig. 4.3 as per IS : 875(part 3) page 50

S = size reduction factor to include the effect of aerodynamics to take care size of structure IS: 875(part 3) fig 10,page 51 is calculated from graph given in Fig. 4.4. The size reduction factor S depends upon the frequency of the structure.

E= measure of available energy in the wind stream at the natural frequency of the structure. Its shows the level of energy or intensity of the wind pressure and is calculated by the curve IS: 875(part 3)fig 11,page 52 given in Fig. 4.5 depending upon the frequency of the structure.

β = damping coefficient of the structure

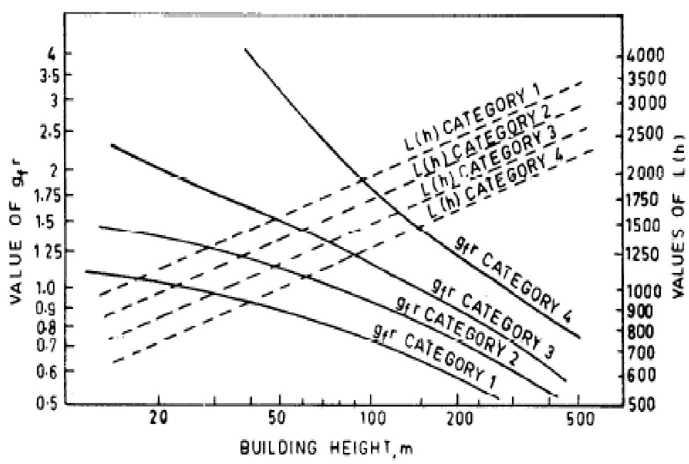


Fig. 4.2 Values of $g_f r$ and L (h)

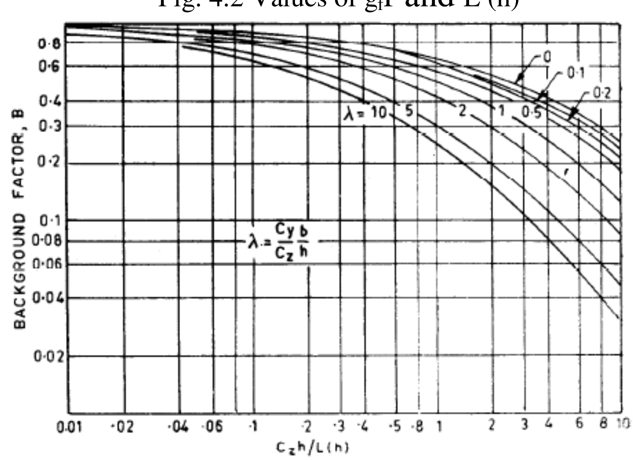


Fig. 4.3 Background Factor, B

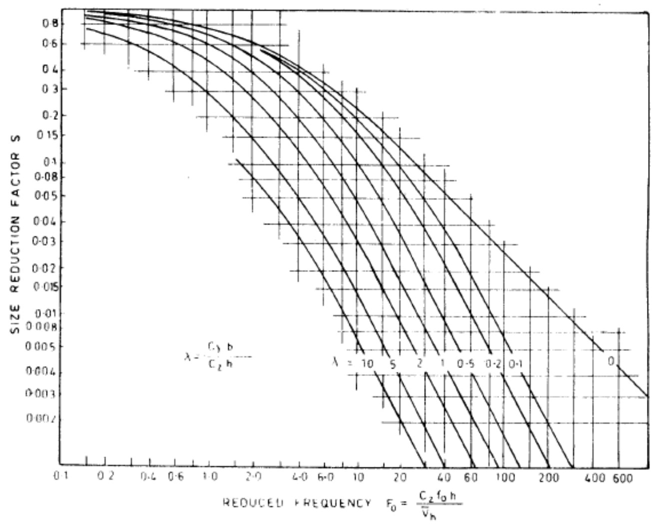


Fig. 4.4 Size reduction factor, S

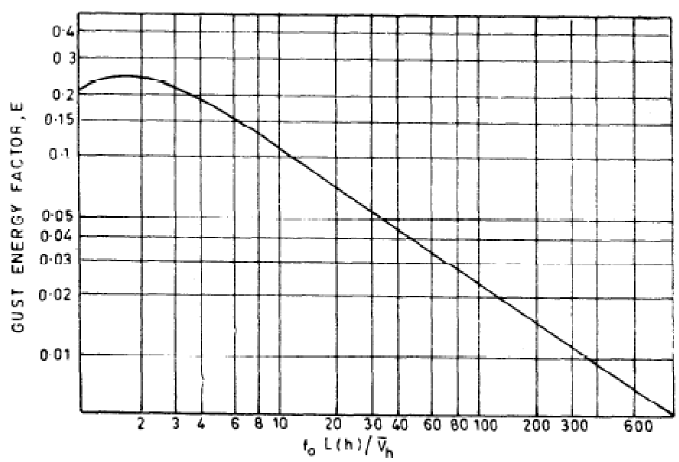


Fig. 4.5 Gust energy Factor, E

4.4 STATIC AND DYNAMIC WIND ANALYSIS OF A TOWER

Wind analysis of 56 m height Microwave tower with a hemi-spherical dome type antenna mounted on its top is done. The relevant data of the tower is as mentioned below:

- Height of tower = 56 m
- Base width = 8 m
- Top width = 2 m
- Number of panels = 10 panels
- Height of straight portion = 36 m
- Height of tapering portion = 20 m
- No of panels in straight portion = 6panels @ 6 m each

- No of panels in inclined portion = 4 panels @ 5 m each
- Disc (hemi-spherical dome) size = 4 m radius
- Position of antenna = 56 m(at top)
- Basic wind speed = 44 m/s (wind zone 3)
- Location of tower = Mumbai

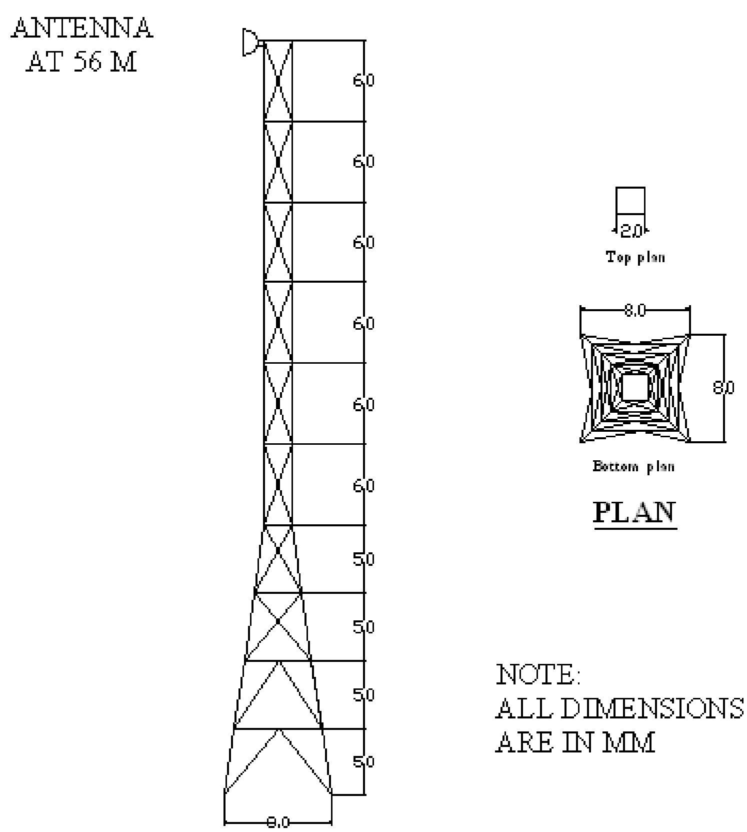


Fig 4.6 View of a 56m tower

Analysis is been carried out, the final forces are calculated at the joints of tower and then transferred to the member as axial force. Finally the comparison is made between the forces calculated by static and dynamic analysis.

The numbering of joints, the numbering of members, sketches of tower with the position of antenna are given in Fig 4.6 and 4.7 respectively.

The towers is analyzed for three basic loads and are as follows:

- Self weight
- Superimposed load from antenna
- Wind loads,
  - (a) Wind parallel to face of the tower
  - (b) Wind diagonal to the tower

1. **Design wind speed**

- Basic wind speed = V_b = 44 m/s as the tower is located in Mumbai is obtained from Fig.1 IS: 875 -part 3
- Risk coefficient (K_1) = 1.05
  It is calculated from table 1 of IS: 875 (page 11).Tower is an important structure and its mean probable design life is considered as 100 years from that factors (K_1) is taken as 1.05.
- Terrain, height and structure size factor (K_2)

  It is calculated from table 2 of IS: 875 (page 12). The various k_2 factor at different height are being computed in table 4.2.
- Topography coefficient = K_3 = 1.0
  Topography factor incorporates the topographic features around the structure as the tower is located in open land it is taken as 1.0 from IS: 875 part3 clause 5.3.3.1.

2. **Design wind pressure(P_z) at different panels height**

The design wind pressure at different panel height can be calculated can be calculated by eqn 4.2

$$P_z = 0.6\ V_z^2$$

➢ Design wind pressure for panel-1
  Height of panel = 53 m

Wind pressure is found at the centre of the panel to distribute the equal wind pressure on the panel. So it is found at the mid height of first panel that is 53 m.

K_2 = 1.26

(As the tower height is between 50 to 100 m it lies in Class C type therefore K_2 factor for terrain category 1 from table 2 of IS: 875 part-3 is 1.26).

From eqn 4.2,

$$V_z = V_b \times K_1 \times K_2 \times k_3$$
$$= 44 \times 1.05 \times 1.26 \times 1.00$$
$$= 58.212 \text{ m/s}$$

Design wind pressure at 53 m for panel 1

$P_z = 0.6\ V_z^2$

$= 0.6 \times 58.212^2$

$= 2033.1\ N/m^2$

The calculated design wind pressure for different panels at different height (z) is computed in table 4.2.

3. Solidity ratio (δ) and force coefficient (C_f)

The solidity ratio of panel-1 in which gross area is

$$A_g = 2.0 \times 6.0$$

$$= 12\ m^2$$

Assuming section ISA 150 x150 x10 for leg members, ISA 60 x 60 x 6 bracings and ISA 50 x50 x6 for horizontal members. The solid flat plate obstruction area is,

$$A_o = \text{area of legs} + \text{area of braces}$$

Where,

Area of legs of ISA 150 x150 x10 of length as 6.0 m as shown in Fig 4.8

Area of legs = 2 x 6 x 0.15

= 1.8 m^2

Area of braces of ISA 60 x 60 x6 of length 6.234 m

Area of braces = 2 x 6.234 x 0.06

= 0.74 m^2

Obstructed area A_o= 1.8 + 0.74

= 2.54 say 2.6 m^2

Therefore solidity ratio = obstructed area/ gross area

= 2.6 / 12

= 0.216

Add extra 15% for secondary bracings, etc

Therefore total effective area A_e = 15% of 2.6 m^2 + 2.6 m^2

= 0.39 + 2.6

= 2.99 m^2

Solidity ratio = 0.23

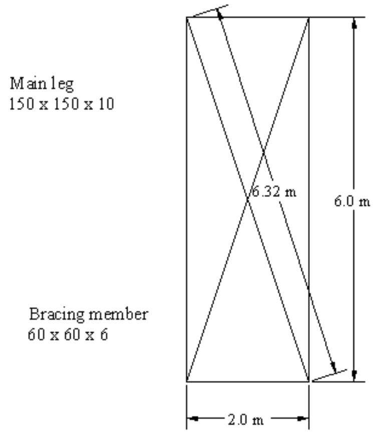


Fig.4.8 Dimensions of panel -1

Force coefficient (C_f) for the solidity ratio 0.23 is interpolated from table 4.1 for square towers. Panel by panel values of force coefficient and solidity ratio are tabulated in Table 4.2.

Table 4.2 Design wind pressure (Pz) at different panel height

PANEL NO FROM TOP	HEIGHT(M)	K_2	P_z (N/m^2)	SOLIDITY RATIO	C_f
1	53	1.26	2033	0.23	3.15
2	47	1.24	1970	0.23	3.15
3	41	1.22	1905	0.23	3.15
4	35	1.21	1875	0.23	3.15
5	29	1.19	1810	0.35	2.55
6	23	1.18	1785	0.39	2.8
7	17.5	1.16	1725	0.26	3.0
8	12.5	1.12	1605	0.20	3.3
9	7.5	1.02	1330	0.15	3.55
10	2.5	1.0	1280	0.12	3.7

To calculate the wind force entire structure is distributed in 3 parts as follows:

- Wind force on antenna
- Wind force in panel 1to 6
- Wind force in panel 7 to 10

The purpose of dividing entire structure in three parts is that panel 1 to 7 is of same dimension and straight, same as for panel 7 onwards as they all are inclined. Therefore calculation for panel 1 in straight portion and calculation for panel 7 in inclined portion is carried out here.

4. Wind force on antenna

Wind load on dish are from eqn 4.4

$F = C_f \times A_e \times P_z$ (force to be calculated on the front face of antenna)

Wind coefficients (C_f) for antennas (IS: 875:part3, fig6) as shown in Fig 4.9. C_f = 1.4 from fig 4.9 as antenna is considered as hemispherical bowl

$A_e = (\Pi/4)\ r^2$ (r = 2m of the antenna)

   = 3.14 m^2

P_z= 2.0 kN/m^2 (from table 4.2)

Wind force = 9.2 kN

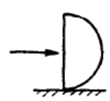
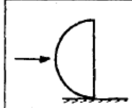
SIDE ELEVATION	DESCRIPTION OF SHAPE	C_f
	CIRCULAR DISC	1·2
	HEMISPHERICAL BOWL	1·4
	HEMISPHERICAL BOWL	0·4

Fig 4.9 Wind force coefficient for solid shapes mounted on surface

5. Wind force in the panels of the tower

When wind force acts on tower, following sequence is adopted for the distribution of force in members:

- Wind force on tower
- Force distributed to the panel of the tower
- From the panel the force is distributed to the joint in the panels
- From the joint the force is transferred to the members

Force in panel 1 is calculated by eqn 4.4

$F = A_e \times C_f \times P_z$

= 2.99(as per calculated in point 3) x 3.15 x 2.033 (from table 4.2)

= 19.14 kN

Similar axial forces in the panels of tower from 1 to 10 are tabulated in Table 4.3 as below:

Panel from top	Static wind force(kN)
1	19.14
2	18.55
3	17.89
4	17.65
5	17.81
6	21.49
7	19.91
8	19.31
9	17.21
10	17.87

Table 4.3 Static wind forces in panels of a tower

Detailed calculation for the wind force in panel 1 is shown below:

- Wind force calculated on legs of members (150x150x10) of the panel-1

  Height of midpoint of panel 1 = 53 m

  Properties of the section:

  Length of legs = L = 6 m

  Width of legs = B = 150 mm

  Weight = 22.8 kg/m (from steel table)

  W_1 = weight of legs = 4 (6) (0.228) =5.47 kN say 5.5kN

  Number of legs exposed to wind N = 2

  Obstruction area = NLB = 2 x (6) x (0.15) = 1.8 m^2

  Add 15% of gusset plates

  Total obstructed area = 2.07 mm^2

  F_1 = Wind load = Ae x C_f x P_z (from table 4.2)

  = 2.07 x 3.15 x 2.033

  = 13.25 kN say 14 kN (forces on legs only)

- Wind force on the brace members of the panel-1

  Number of braces = 8

  Number of obstructing braces = 2

Section of braces = ISA 60 x 60 x 6 (see Fig 4.6)

L = 6324 mm

Width of leg = B = 60 mm

Weight = 5.4 kg/m (from steel table)

Weight of braces = W_b = 8 x (6.324) x (0.054) = 2.7 kN

Wind obstruction = A = 2. LB

Wind load on braces as per eqn 4

$F = C_f \times A_e \times P_z$

Where C_f = 3.15 from table 4.2

A_e= 2 x 6.324 x 0.06

= 0.7588 m^2

P_z= 2.033 kN/m^2 from table 4.2 for panel 1

Therefore F = [(3.15) x {2 x (6.324) x (0.06)} x (2.033)] = 4.85 kN (force on braces)

The weight and wind load on legs and braces are to be distributed to all the joints connecting the elements. There are 8 joints connecting 4 braces of which only two braces provide the obstruction.

Total loads from the legs and braces are

Weight = W = 5.5(wt from legs) + 2.7(wt from braces) = 8.2 kN

Wind load = F = 14 (wind load on legs) + 4.85(wind on braces) = 18.85 kN

Loads on each joint with 15% extra for gussets and distributed on the eight joints of the tower

W = 8.2 x (1.15) / 8

= 1.18 kN

F = 18.85 x (1.15) / 8

= 2.70 kN

Total forces at each of joints 1 to 4 from legs, braces and horizontals are:

W = 1.18 + 0.3(Wt of horizontal section 50x50x6) = 1.48 kN

F = 2.70 + 0.1(wind load on horizontal section 50x50x6) = 2.8 kN

The above calculations are for the loads (kN) on tower joints for panel 1. The calculation of wind load and dead load for other panels are carried out similarly and are given in Table 4.4.

Table 4.4 Loads (kN) on tower joints

Joint no's	Panel from Top	Level (m)	Load(kN)/joint	
			Nodal Wind load	Nodal dead load
1,2 (dish)	1	56	4.6	4.5
1,2,3,4	1	56	2.96	1.48
5,6,7,8	2	50	4.9	2.36
9,10,11,12	3	44	4.7	2.80
13,14,15,16	4	38	4.9	3.24
17,18,19,20	5	32	5.1	3.84
21,22,23,24	6	26	5.7	6.04
25,26,27,28	7	20	6.5	6.91
29,30,31,32	8	15	7.3	6.28
33,35,37,39	9	10	8.3	7.48
41,43,45,47	10	5.0	9.1	9.35
49,51,53,55	10(bottom)	0.0	4.6	4.81

6. Forces in members of panel 1

Forces in the members are four three conditions:

(a) Only self weight acting on tower

(b) Only weight of dish acting on tower

(c) Wind load condition

i. Wind parallel to the tower

ii. Wind diagonal to the tower

Forces in the members through all the conditions are mentioned below.

(a) Load condition 1: Only self weight acting (Fig. 4.10)

Self weight of panel 1 (as from Table 4.4)

W = 4 [1.48]

= 5.92 kN

By symmetry forces in braces = 0 (see Fig 4.10)

Nd

$F_{31} = F_{41} = 0$, where F_{ij} means forces in i th member and in j th load

$F_{21} = F_{11} = Wg/4 = 1.48$ kN (compression).

Where,

F_{11} = Force in leg member 1 with load condition 1 as shown in Fig 4.10

F_{21} = Force in leg member 2 with load condition 1

F_{31} = Force in brace members 3 for load condition 1

F_{41} = Force in brace member 4 for load condition 1

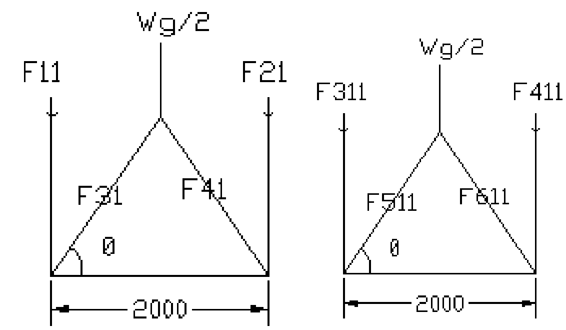


Fig. 4.10 Load condition 1 (only self weight acting)

(b) Load condition 2: Weight of dish (Fig. 4.12)

Weight of dish = Ws = 9 kN and is acting at eccentricity with front columns at = e = 0.5 m (Fig 4.11)

The axial force and the bending moments caused by the load are:

$P = Ws = 9$ kN

$M_z = W_s (e + a)$

$= 9 (0.5 + 1)$

$= 13.5$ kN

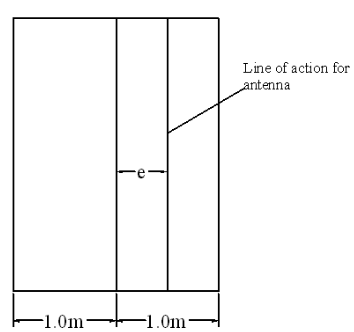


Fig 4.11 Eccentricity in an antenna

Equilibrium in the horizontal direction of the section gives;

$$(F_{32} + F_{42}) \cos\theta = 0$$

$$F_{32} = - F_{42}$$

Where $\theta = 71.56^{\theta}$

Θ = angle between horizontal and bracings

$\text{Tan}\Theta = 6/2 = 3$

Therefore $\Theta = \text{Tan}^{-1}(3) = 71.56$

Equilibrium in the vertical direction gives;

$$F_{12} + F_{22} + P/2 = 0$$

$$F_{12} + F_{22} = -P/2 = -4.5 \text{ KN} \qquad \dots (4.6)$$

The equilibrium of moments about the point of intersection of bracings gives;

$$(F_{12} \times a/2 - F_{22} \times a/2) + M_z/2 = 0$$

$$F_{12} - F_{22} = -M/a = -6.75 \text{ KN/m} \qquad \dots (4.7)$$

Addition of values in eqn (4.6) and (4.7) gives

F_{12} = - 5.625 KN (Comp)

F_{22} = 1.125 kN (tension)

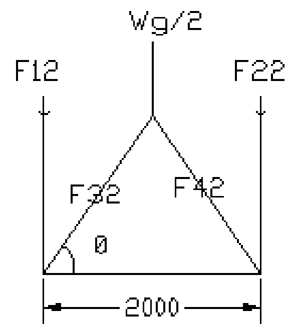


Fig. 4.12 Load condition 2(weight of dish)

(c) Wind load condition:

As seen in fig 4.13 two wind load condition is to be considered wind parallel and diagonal to plane of truss to find the critical wind force on tower. The forces through both the condition is found below:

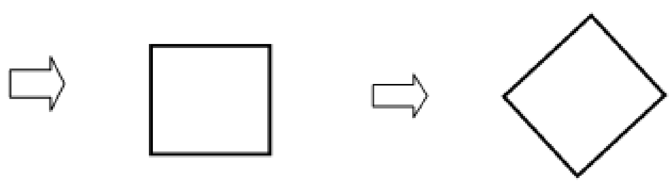


Fig.4.13 Wind parallel and diagonal to plane of truss

- Wind parallel to the plane of truss

The tower is considered as consisting of two parallel plane trusses and the wind load is equally shared by both. The wind force on panel 1 and dish with their distances is shown in Table 4.5

Total Wind force on dish = 9.2 kN (as calculated earlier in $4^{th\ point}$), as it is distributed on two joints therefore force on one joint = 9.2/2

= 4.6 kN

Wind load on panel = 2.96 kN (from table 4.4)

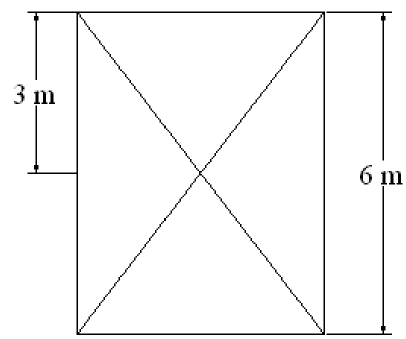


Fig 4.14 Typical view of panel 1

The moment of force is taken at half of panel 1 that is 3 m from top as shown in fig 4.14. The forces and the moment about distance 3m is shown in table 4.5.

Table 4.5 Wind loads and distances

Panel no	Dish	1
Force(kN)	4.6	2 (2.96)
Distance(m)	3	3

The sum of the horizontal forces up to panel 1 from table 4.4

$$Q_1 = Q_{s1} + Q_{t1} \qquad \text{... (4.8)}$$

$$= 4.6 + 2\ (2.96)$$

$$= 10.52 \text{ kN}$$

Where,

Q_1 = Sum of the horizontal forces

Q_{s1} = Wind forces on dish

Q_{t1} = Wind forces on tower

M_1 = Moments due to load on dish +moments due to load on tower

M_{s1}= Moment due to load on dish at the centre of panel

= 4.6 x 3

= 13.8 kNm

M_{t1} = Moment due to load on tower

$= (2.96 \times 3) \times 2$

$= 17.76kNm$

$M_1 = M_{s1} + M_{t1}$ … (4.9)

$= 13.8 + 17.76$

$= 31.56$ kNm

By Horizontal and vertical equilibrium of forces, considering the symmetry of structure and loading, the forces are computed as below:

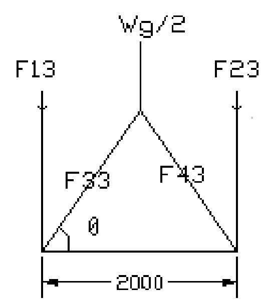


Fig. 4.15 Forces in legs and braces

$$F_{13} = \frac{M_1}{a} = \frac{M_{s1}}{a} + \frac{M_{t1}}{a} \quad \text{… (4.10)}$$

Where M_{s1} and M_{t1} values are taken from eqn 4.9

a = 2 (width of panel 1)

$= (13.8/2) + (17.76/2)$

$= 15.78$ kN

The equilibrium of section in horizontal direction gives,

$(F_{33} - F_{43}) \cos\theta = 10.52$ kN

Where $\Theta = 71.56$

$\cos\theta = 0.31$

$F_{33} - F_{43} = 33.93$ kN

From symmetry of the structure and loading, as from Fig 4.15.

$F_{33} = - F_{43}$ and $F_{33} - F_{43} = 33.93$

$F_{33} = 33.93/2$

$= 16.96$ kN

$F_{13} = 15.78$ kN (tension) (as from eqn 4.10)

$F_{23} = - 15.78$ kN (comp)

$F_{33} = 16.96$ kN (tension)

$F_{43} = -16.96$ kN (comp)

- Wind parallel to the diagonal

  Since the legs are upright, the horizontal force is resisted by the braces and the forces in the braces are equal and opposite.

  The bending moment is resisted by pair of legs as seen in Fig 4.16 and forces in the other two legs which lie along the diagonal about which the moment is zero

Forces in the leg is given by,

$$F_{14} = \frac{M_{s1}}{a} + \frac{1.7M_{t1}}{a} \qquad \text{... (4.11)}$$

F_{14} = (13.8/2) + (1.7x 17.76/2)

= 21.99 kN

F_{34} = - 21.99 kN (comp)

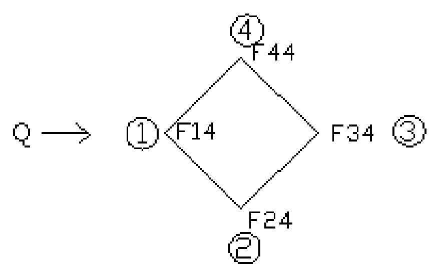


Fig. 4.16 Forces when wind blowing diagonally

The forces in the brace of panel 1 is given by following eqn

$$F_d = \frac{1}{8}\left[\frac{Q_{s1}}{\cos\theta} + \frac{2.4\sqrt{2}Q_{t1}}{\cos\theta}\right] \qquad \text{... (4.12)}$$

Where,

F_d = Force in each brace

Q_{s1} = wind force on dish = 4.6 kN (as per eqn 4.8)

Q_{t1} = wind force on tower = 2 x 2.96 = 5.92 (as per eqn 4.8)

Cosθ = 0.31 (where Θ = 71.56)

From eqn 4.12

$F_d = 1/8[(4.6/0.31) + (2.4 \times \sqrt{2} \times 5.92)/0.31]$

$= 10.16$ kN

The horizontal force is resisted by the braces and the forces in the braces are equal and opposite. The forces are resolved in horizontal plane and then parallel to the diagonal.

Final forces (kN) in panel 1 are as tabulated as given in table 4.6. Final design forces in table 4.5 are calculated as per eqn 1.

Table 4.6 Forces (kN) in panel - 1

Load condition	Leg member (1,2,3,4)		Braced member (5,6,7,8,9,10,11,12)		Horizontal (A_1, A_2, A_3, A_4)	
	Tension	Comp	Tension	Comp	Tension	Comp
1 Dead load						
Tower	-1.48	1.48	0	0	-0.35	0.35
Dish	1.13	5.63	0	0	-	-
Total	-0.35	7.11	-	-	-0.35	0.35
2 Wind load parallel to plane						
Tower	8.88	8.88	16.96	16.96	-4.9	4.9
Dish	6.9	6.9	-	-	-	-
Total	15.78	15.78	16.96	16.96	-4.9	4.9
3 Wind load parallel to diagonal						
Tower	15.09	15.09	10.16	10.16	-	-
Dish	6.9	6.9	-	-	-	-
Total	21.99	21.99	10.16	10.16	-	-
Design forces (eqn 4.1) [W.L(3) +D.L(1)]	21.64	29.1	16.96	16.96	4.9	4.9

From Table 4.6 it is stated that for the design forces for main legs wind diagonal to plane is critical condition and for braces wind parallel to plane is the critical condition.

7. Force in members of panel 7

Panels 1 to 6 are straight panels in the tower and the inclined portion starts from the panel 7, hence the force calculation for the panel 7 is as follows:

(a) Load condition 1: Only self weight acting

Nodal dead load up to 7^{th} panel specified in table 4.3 is added here to find out the total weight up to panel 7.

W_g = 4 [1.48 + 2.36 +2.80 + 3.24 +3.84 + 6.04 +6.91]

= 106.68 kN

By symmetry forces in braces = 0

$F_{31} = F_{41} = 0$ (where F ij = forces in i th member and in j th load)

$F_{21} = F_{11}$ = Wg/4 =106.68/4= 26.67 kN (compression).

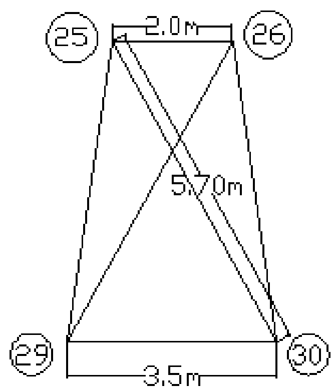


Fig.4.17 Panel 7

(b) Load condition 2: Weight of dish

The calculation in condition 2 with only dish weight acting on it is same as calculated above. Therefore final forces as calculated above for panel 1 in 6^{th} point for condition "b" and it is also notified in table 4.5 are,

$F_{(73)2}$ = - 5.625 KN (Comp)

$F_{(74)2}$ = 1.125 kN (tension)

(c) Wind load condition:

- Wind parallel to the plane of truss

The sum of the nodal wind forces up to 7th panel specified in table 4.3 is added here to find out total horizontal wind force up to panel 7.

$Q_1 = Q_{s1} + Q_{t1}$

= 4.6 + 2 (2.96 + 4.9 + 4.7 + 4.9 + 5.1 + 5.7 + 6.5) (from table 4.4)

= 73.2 kN

M_1 = moments due to the load on dish + moment due to load on tower

Moment due to wind load on each panel is calculated about the centre of panel 7 as follows, wind force with panel distance is tabulated in table 4.7.

Table 4.7 Wind load and distances

Panel no	Wind force (kN)	Dist(m) from centre of panel 7
Dish	4.6	38.5
1	2.96	38.5
2	4.9	32.5
3	4.7	26.5
4	4.9	20.5
5	5.1	14.5
6	5.7	8.5
7	6.5	2.5

$M_1 = M_{s1} + M_{t1}$

= 4.6 (38.5) + 2 [(2.96 x 38.5) + (4.9 x 32.5) + (4.7 x 26.5) + (4.9 x 20.5) + (5.1 x 14.5) + (5.7 x 8.5) + (6.5 x 2.5)]

= 1438.5 kNm

Therefore,

M_{s1} = 177.1 kNm

M_{t1} = 1261.4 kNm

a = Avg width of trapezoidal panel = 2.75

Forces in the legs as per eqn 4.10,

$$F_{(73)3} = \frac{M1}{a} = \frac{Ms1}{a} + \frac{Mt1}{a}$$

= (177.1/2.75) + (1261.4/2.75)

$= 64.4 + 458.6$

$= 523$ kN

By Horizontal and vertical equilibrium of forces, considering the symmetry of structure and loading, the forces in braces computed as below:

The equilibrium of section in horizontal direction gives,

The members $F_{77 \text{ and }} F_{78}$ braces are inclined with Θ with horizontal so they can be resolve in horizontal direction.

$(F_{77} - F_{78}) \cos \theta = 73.2$ kN

$F_{77} - F_{78} = 133.09$ kN

Where $\cos\theta = 2.75 / 5$

$= 0.55$

$\Theta = 56.63^{\theta}$

From symmetry of the structure and loading as seen in Fig 4.18,

As the tower is symmetrical about vertical axis forces in the braces will be equal and opposite.

$F_{77} = - F_{78}$ and $F_{77} - F_{78} = 133.09$

$F_{77} = 133.09/2$

$= 66.54$ kN

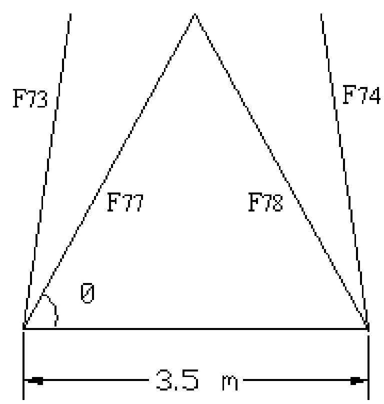


Fig 4.18 Forces in panel 7

Forces in legs are,

$F_{(73)3} = 523$ kN (tension)

$F_{(74)3} = - 523$ kN (comp)

$F_{(77)3} = 66.54$ kN (tension)

$F_{(78)3} = -66.54$ kN (comp)

- Wind parallel to diagonal of truss

The calculation will be same as calculated above for panel 1 and forces in legs of members are found by eqn 4.11

$$F_{(73)4} = \frac{M_{s1}}{a} + \frac{1.7M_{t1}}{a}$$

M_{s1} = 177.1 kN (as calculated earlier)

M_{t1} = 1261.4 kN

a = avg width of panel 7 = 2.75m

$F_{(73)4}$ = (177.1/2.75) + (1.7x 1261.4/2.75)

= 844.17 kN

$F_{(74)4}$ = - 844.17 kN (comp)

The forces in braces are given by eqn 4.12 from as follows

$$F_d = \frac{1}{8}\left[\frac{Q_{s1}}{\cos\theta} + \frac{2.4\sqrt{2}Q_{t1}}{\cos\theta}\right]$$

Where,

Q_{s1} = 4.6kN (wind force on dish)

Q_{t1} = 68.5 (wind force on panel up to panel 7)

Cosθ = 0.55 (where Θ = 55.36)

Therefore forces in brace,

F_d = 1/8[(4.6/0.55) + (2.4 x √2 x 68.5/0.55)

F_d = 53.96 kN

F_{77} = 53.96 kN (tension)

F_{78} = - 53.96 kN (comp)

Final forces in panel 7 are tabulated below in table 4.8 and maximum axial forces in all panels of the tower for main legs and bracings are tabulated in Table 4.9 and table 4.10 respectively.

Table 4.8 Forces (kN) in panel - 7

Load condition	Leg member (73,74,75,76)		Braced member (77,78,79,80 81,82,83,84)		Horizontal (G_1,G_2,G_3,G_4)	
	Tension	Comp	Tension	Comp	Tension	Comp
1 Dead load						
Tower	-26.72	26.72	0	0	-0.35	0.35
Dish	1.13	5.63	0	0	-	-
Total	-25.59	32.35	-	-	0.35	0.35
2 Wind loads parallel to plane						
Tower	458.6	458.6	66.54	66.54	6.5	6.5
Dish	64.4	64.4	-	-	-	-
Total	523	523	66.54	66.54	6.5	6.5
3 Wind loads parallel to diagonal						
Tower	779.77	779.77	53.96	53.96	-	-
Dish	64.4	64.4	-	-	-	-
Total	844.17	844.17	53.96	53.96	-	-
Design forces eqn 4.1 [W.L(3) + D.L(1)]	818.58	876.52	66.54	66.54	6.5	6.5

Final design forces for panel 1 and 7 are tabulated in table 4.6 and 4.8. As by similar process forces in all remaining panels are found out. Table 4.9 and 4.10 shows the final design forces for the panels. Tables are prepared by the final values for wind parallel and diagonal to the tower.

Table 4.9 Axial forces (kN) in main legs of a tower

Panel from Top	Member No	Dead load (1)	Wind parallel to plane of truss (2)	Wind parallel to diagonal of truss (3)	Design force (1) + (3) (W.L+D.L)	
					(T)	(C)
1	1 to 4	7.11	15.78	21.99	21.64	29.1
2	13 to 16	9.47	57.9	97.74	95.03	107.21
3	25 to 28	12.27	130.2	197.17	191.66	209.4
4	37 to 40	15.51	279.96	407.7	398.95	423.21
5	49 to 52	19.4	362.4	634.71	622.07	654.11
6	61 to 64	24.13	535.9	933.5	915.2	957.6
7	73 to 76	32.35	523	844.17	818.58	876.52
8	85 to 88	38.58	427.73	741.28	709.46	779.86
9	97 to 100	45.8	402.99	696.72	657.68	742.52
10	109 to 112	55.15	395.29	682.17	633.78	737.32

Table 4.10 Axial forces (kN) in bracings of a tower

Panel from Top	Member No	Wind parallel to plane of truss	Wind parallel to diagonal of truss	Design Forces
1	5 to 12	16.96	9.93	16.96
2	17 to 24	31.25	21.5	31.25
3	29 to 36	46.45	34.31	46.45
4	41 to 48	62.25	48.24	62.25
5	53 to 60	78.70	68.43	78.70
6	65 to 72	95	76	95
7	77 to 84	66.54	53.80	66.54
8	89 to 96	68.59	55.88	68.59
9	101 to 108	124.28	101.86	124.28
10	113 to 120	136.22	112.19	136.22

- **Dynamic Analysis by Gust Factor method**

Force in panel due to the wind action is given by eqn 4.4

$$F_z = C_f \times A_e \times P_z \times G$$

Where,

P_z= design wind pressure at height z due to the hourly mean wind obtained as $0.6 \times V_z^2$ N/m^2 and V_z is given be eqn 4.2

$V_z = V_b \times K_1 \times K_2 \times K_3$

V_b = 44 m/s (basic wind speed at Mumbai)

K_1= 1.05 (from IS: 875)

K_3= 1.0

K_2= 0.9336 (from table 33 of IS: 875 for height 53 m)

$V_z = V_b \times K_1 \times K_2 \times K_3$

$= 44 \times 1.05 \times 0.9336 \times 1.0$

$= 43.13$ m/s

Design wind pressure at height 53 m

$P_z = 0.6 \times V_z^2$

$= 0.6 \times 43.13^2$

$= 1116$ N/m^2

C_f = 3.15(force coefficient for panel 1 as calculated above in table 4.2)

A_e = Effective frontal area of obstructed legs and braces

= 1.8 (area of legs) + 0.74 (area of braces)

= 2.54 m^2 + (15% for gusset plates)

= 2.99 m^2

G = gust factor is calculated as stated in sec 4.3.2 by eqn 4.6 as below,

$$G = 1 + g_f r \sqrt{\left[B(1+\varnothing^2) + \frac{SE}{\beta} \right]}$$

$g_f r$ = 0.9 (it is interpolated from graph as shown in Fig 4.2 depending upon the terrain category)

L(h) = 820 (a measure of turbulence length scale from graph as shown in Fig 4.2 depending upon the category $g_f r$ and building height)

B = background factor depending upon $\lambda = \frac{c_y}{c_z}\frac{b}{h}$ and $\frac{c_z h}{L(h)}$ (from Fig 4.3)

Where,

c_y = lateral correlation constant which may be taken as 10 in the absence of more precise load data

c_z = longitudinal correlation constant = 12

b = 8.0 m (breadth of structure normal to wind stream)

h = 56 m (height of tower)

$$\lambda = \frac{c_y}{c_z}\frac{b}{h} \qquad \text{... (4.13)}$$

= 10 x 8/12 x 56

= 0.119

$$\frac{c_z h}{L(h)} = 12 \times 56/820 \qquad \text{... (4.14)}$$

= 0.819

Where,

C_z = 12

h = height of tower 56 m

L(h) = From fig 4.2 = 820

Now as per values of eqn 4.13 and 4.14 with the values of λ from fig 4.3 we find the background factor B = 0.7

S = size reduction factor depends upon $\lambda = \frac{c_y}{c_z}\frac{b}{h}$ and $F_0 = \frac{C_z f_0 h}{v}$

Where,

The three dimensional model was prepared in staad and natural frequency and time period of tower was find out.

f_0 = natural frequency of tower

= 1.188 Hz (STAAD output)

Time period of tower = 0.842 sec (STAAD output)

F_0 = 12 x 1.18 x 56 / 43.13

$= 18.38$ Hz

Therefore from reduced frequency F_0 and from fig 4.4 size reduction factor "s" is interpolated for the values of "λ"

Therefore from Fig 4.4 the value of S is carried out as 0.25.

E = Gust energy factor depend upon $\frac{f_0 L(h)}{V}$

$$\frac{f_0 L(h)}{V} = 1.188 \times 820 / 43.13 \quad \text{… (4.15)}$$

$$= 22.58$$

The value of E from Fig 4.4 is computed as 0.075.

Therefore value of G from eqn 4.4 is 1.837

Force in panel 1

$$F_z = C_f \times A_e \times P_z \times G$$

$$= 3.15 \times 2.99 \times 1.116 \times 1.837$$

$$= 19.30 \text{ kN}$$

Similar the dynamic wind forces in panels 1 to 10 are tabulated as below in Table 4.11.

Table 4.11 Dynamic wind forces in panels of a tower

Panel from top	Dynamic wind force (kN)
1	19.30
2	18.75
3	18.66
4	17.52
5	18.47
6	20.78
7	20.44
8	17.85
9	18.51
10	19.88

The panel by panel force is calculated and tabulated in Table 4.12.

Table 4.12 Dynamic wind pressure (kN) at different panels of a tower

Panel from Top	Height (m)	K_2	V_z m/s	P_z N/m^2	Force(kN)
1	53	0.9336	43.13	1.116	19.30
2	47	0.9225	42.61	1.089	18.75
3	41	0.9075	41.92	1.054	18.66
4	35	0.8925	41.23	1.019	17.52
5	29	0.88	40.65	0.991	18.47
6	23	0.859	39.68	0.944	20.78
7	17.5	0.835	38.57	0.892	20.44
8	12.5	0.80	36.96	0.819	17.85
9	7.5	0.78	36.036	0.779	18.51
10	2.5	0.78	36.036	0.779	19.88

4.5 SEISMIC ANALYSIS OF A TOWER

It is generally recognized that in the latticed telecommunication towers wind effects, and combinations of wind and dead load effects are more likely to govern the design than are earthquake effects. Microwave towers in particular must obey very stringent serviceability criteria, usually specified in the terms of tilt and twist limits. Although the tower structure may appear sound after an earthquake, localized permanent deformations, especially in attachments of heavy antennas to mounts, may render it unserviceable. Seismic analysis of such structures is not symmetrically done in practice, and satisfactory performance in past earthquakes has demonstrated that it is not always necessary. The force attracted by a structure during a seismic disturbance is a function of ground acceleration and the properties of the structure. Following are some of important factors on which seismic force is dependent;

- Stiffness of the tower
- Damping characteristics of the tower
- Probability of an earthquake occurring at a particular site of tower
- Importance of the tower based on the failure
- Foundation characteristics

- **Seismic performance levels**

Earthquake –resistant design precautions vary depending on seismicity of tower locations, including the potential amplification effects of special geotechnical conditions, and tower performance level defined by the owner of the tower.

- Life safety

A telecommunication tower designed for the life safety should not collapse in the failure mode that is direct threat to life safety. This performance objective should apply without exception to all towers located in areas of human occupancy, with special attentions paid to towers supported on building's rooftops.

- Interrupted serviceability

A telecommunication tower designed for interrupted serviceability is not required to be fully serviceable during the strong motion, but it should not sustain any damage that would make it unserviceable immediately or shortly after the earthquake has occurred.

- **Calculation of seismic force for a 56 m high tower**

The procedure for calculating the seismic force on the different panels of the tower is as follows:

Tower location: Mumbai

- Tower is designed and checked for stability for 5 times design horizontal seismic coefficient A_h as per IS 1893:1984

Where,

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} \qquad \textbf{...}\ (4.16)$$

Z = zone factor for the maximum considered Earthquake and service life of a structure in a zone. Mumbai is in zone 3 therefore it is taken as 0.16. The factor 2 in the denominator is to reduce the maximum considered Earthquake zone factor to the factor for design basis Earthquake

I = Importance factor depending upon the functional use of structure and is 1.5 for tower (IS 1893:part1)

R= response reduction factor, depending upon the perceived seismic damage performance of the structure and is taken as 4.0 from IS 1893:part1

S_a/g = Average response acceleration coefficient based on the soil condition and natural period and damping of the structures

The three dimensional modeling of tower was done in staad and analysis and design was also carried out. The approximate fundamental natural time period of vibration (T) in seconds is from output of STAAD.

Natural frequency of tower = 1.188 Hz

Time period of tower = 0.842 sec

For towers, the weight W of the structure is less in comparison with the buildings. The natural period such that Sa/g value is quite low. Because the mass of the tower is low and the Sa/g value is also less, the resultant earthquake force is quite small as compared to the wind force normally considered for Indian conditions. Thus earthquake seldom become a governing design criteria.

Assuming soil condition as medium soil sites, S_a/g from IS 1893: part 3

$$\frac{S_a}{g} = \frac{1.36}{T} \qquad \text{... (4.17)}$$

$$\frac{S_a}{g} = 1.61$$

As from eqn 4.16,

A_h = (0.16/2)/(1.5/4)/1.61

= 0.048

Design horizontal seismic coefficient (A_h) = 0.0483

A_h = 5 x 0.0483

= 0.2415

- Design seismic base shear (V_B)

The total design lateral force or design seismic base shear (V_B) along any direction can be determined by following be expression:

$$V_B = A_h \times W \qquad \text{... (4.18)}$$

Where,

W = Seismic weight of tower

Total weight of tower = 314.4 kN (Section found out)

= 314.4 + 9 (weight of antenna from the data)

= 323.4 kN

$V_B = A_h \times W$

$= 0.241 \times 323.4$

$= 75.63$ kN

- Design of lateral force

The design base shear (V_B) is distributed along the panel height as per following expression from IS: 1893.

$$Q_i = V_b \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \qquad \text{... (4.19)}$$

Where,

Q_i = Design lateral force in panel i

W_i = seismic weight of panel

h_i = height of panel i measured from base and

n = number of panels in the tower

- Design lateral force for panel 1

Height of panel -1 (h_1) = 53 m

Weight of panel -1 = 1.39 kN

Main leg (110 x 110 x 8 @ 13.4kg/m) = 6 x 13.4 = 80.4 kg

Bracing (70 x 70 x 8 @ 8.3 kg/m) = 6.324 x 8.3 = 52.48 kg

Horizontal (50 x 50 x 6 @ 4.5kg/m) = 2 x 4.5 = 9 kg

Therefore weight of panel 1 = 80.4 + 52.48 + 9 = 141.8 kg

= 1.39 kN

From eqn 4.19,

$Q_i = 75.63 \times 3904.51 / 28797.7$

$= 10.25$ KN

Similar the seismic forces in panels 1 to 10 are tabulated in Table 4.13

Table 4.13 Seismic forces in panels of a tower

Panel from top	Seismic force(kN)
1	10.25
2	9.83
3	11.57
4	14.19
5	12.13
6	10.64
7	4.40
8	1.62
9	0.90
10	0.10

The design lateral forces for each panels are computed such as and listed in Table 4.14.

Table 4.14 lateral force (kN) in panels of a tower

Panel from Top	W_i (kN)	h_i(m)	$W_i h_i^2$	Q_i(kN)	Base shear(kN)
1	1.39	53	3904.51	10.25	10.25
2	1.694	47	3742.06	9.83	20.08
3	2.62	41	4404.42	11.57	31.65
4	4.41	35	5402.5	14.19	45.84
5	5.49	29	4617.09	12.13	57.97
6	7.66	23	4052.14	10.64	68.61
7	5.47	17.5	1675.18	4.40	73.01
8	3.95	12.5	617.18	1.62	74.63
9	6.09	7.5	342.56	0.90	75.53
10	6.49	2.5	40.56	0.10	75.63
Total	45.264	-	28797.75	-	75.63

5. DESIGN

5.1 GENERAL

The Microwave Antenna towers under consideration are made from steel angles. The various members of the tower are to be designed for resultant axial force induced in them due to various loadings. All the members of the lattice masts resist load by tension and/or compression and also angle is the better choice for lattice structure because its shape makes it easy to connect one angle to the other. The chapter contains the information about the procedure for design of members which are under axial force. The information about the connection design and the class of bolts used is specified. In this chapter different types of foundation used for towers are been explained. Some guidelines for the connection of the member are been explained. This chapter also discusses about the slenderness ratio limitations, Minimum thickness of members adopted as per code requirements.

5.2 DESIGN PROCEDURE

5.2.1 Compression members

The tower members carrying axial compression only (without bending) are called compression members. The design of a compression member is a trial and error procedure. It depends on the length, end conditions and loads, the gross area of cross-section depend upon the permissible stress σ_{ac} which in turn depends upon l/r ratio. The design is done in following steps,

- Given the actual length and end conditions, find the effective length L_{eff}
- Depending upon end condition find allowable stress and assume value of stress
- Then find the actual cross section area required
- From the steel tables, select a suitable section having the above area. Find the minimum radius of gyration r_{min} for this section
- Compute $\lambda = L_{eff} / r_{min}$
- Select the value of σ_{ac} corresponding to the value of slenderness ratio(λ)
- Strength of an axially loaded compression member

The maximum compressive axial load which can be permitted on a compression member, is given by

$$P = \sigma_{ac} \times A \qquad \ldots 5.1$$

Where,

P = Axial compressive load (kN)

σ_{ac} = Permissible stress in axial compression (MPa)

A = Cross- sectional area of the member (mm^2)

5.2.2 Tension members

The tower members carrying axial tension only are called tension members. The design of a member, subjected to axial tension is done in following steps:

- Knowing the axial pull and permissible value of σ_{at} calculate the net cross-sectional area required
- Take a suitable section making allowance of bolt holes
- Find the actual A_{net} for the section by making deduction for bolted holes.

  A tension member is designed for its net sectional area at the joint. When a tension member is spliced or joined to a gusset plate by rivets or bolts, the gross sectional area is reduced by rivet holes. The type of sections used as tension members are discuss below,
- Single angle connected by only one leg to gusset plate

  Here, the net effective area is given by-

$$A_{net} = A_1 + A_2 \times K_1 \qquad \ldots 5.2$$

Where,

A_1 = net sectional area of the connected leg

A_2 = gross cross-section area for the unconnected leg

$$K_1 = \frac{3A_1}{3A_1 + 3A_2} \qquad \ldots 5.3$$

- Double angle connected back to back to same side of gusset plate
- Here, the net effective area is given by eqn 5.2 and the value of K_1

$$K_1 = \frac{5A_1}{5A_1 + 5A_2} \qquad \ldots 5.4$$

- If the Anet is greater than the selected section area section is o.k.

5.3 SELECTION OF MATERIAL

5.3.1 Use of hot rolled steel sections

Since towers are manufactured in factory environment and have to be assembled at site, the ease of transport and assembly during tower erection are equally important points for consideration. So far, the practice is overwhelmingly in favour of the use of hot rolled angle steel sections in the design of towers.

5.3.2 Minimum flange width

Minimum flange widths for bolts as per IS: 802 (part-II) tables 1 of different diameter are given below:

Table 5.1 Minimum flange width for bolts

BOLT DIAMETER (mm)	FLANGE WIDTH (mm)
12	40
16	45
20	50
24	60

5.3.3 Minimum thickness of members

As per IS-802 minimum thickness of galvanized members shall be as follows:

- For legs 5 mm
- For other members 4 mm

5.3.4 Grades of steel

Generally two grades of steel i.e., mild steel (MS) and high tensile (HT) steel are used in the fabrication of microwave towers.

5.4.4 Slenderness ratio limitation (l/r)

Slenderness ratio as per IS: 800-1984 table 3.1

Table 5.2 Slenderness ratio limitations

Type of member	L/r
Carrying loads resulting from dead loads and superimpose loads.	180
Carrying loads resulting from wind or seismic forces only provided the deformation of such members does not adversely affect the stress in any part of the structure.	250
Normally acting as a tie in a roof truss but subject to possible reversal of stress resulting from action of wind	350
Tension member in which a reversal of direct stress due to loads other than wind or seismic forces occur	180
Compression flange of beam	300
Tension members (other than pretensioned members)	400

5.4 SECTION DESIGN

- **Design of main legs (1 to 4) of panel 1**

The compressive and tensile force values are carried out from Table 4.9 in chapter 4 for panel 1.

Compressive force = 29.1 kN

Tensile force = 21.64 kN

Length = 6.0 m

Bolt diameter = 20 mm

Effective length = 6.0 x 1.0

= 6.0 m

Allowable stress = 0.8 x σ_{ac}

Assuming σac = 60 N/mm^2

Therefore cross-section area required = 29.1 x 1000 / 60

= 485 mm^2

Try ISA 110 x 110 x 8 mm

Properties of the section:

Area = 1702 mm^2

Thickness = 8 mm

r_{min}= 33.8 mm (minimum radius of gyration)

Now, slenderness ratio (λ) = L_{eff} / r_{min}

= 6000 / 33.8

= 177.51 < 180 (o.k.)

The value of allowable compressive stress is now found out by interpolation from table 5.1 in IS: 800-1984

Therefore allowable compressive stress σ_{ac} = 33.99 N/mm^2

$\sigma_{allowable}$ = 0.8 x 33.99

= 27.192 N/mm^2

Permissible load = stress x area

= 27.192 x 1702

= 46280 N

= 46.28 kN > 29.1 kN (safe)

Check for the tension force

The force values are taken from Table 4.9 from chapter 4

Tension force = 21.64 kN

Bolt diameter = 20 mm

Assuming tensile stress σ_{at} in steel = 150 N/mm^2

Now gross diameter of bolt hole = 20 + 1.5

= 21.5 mm

Net effective area from eqn 5.3

$A_{net} = A_1 + K A_2$

A_1 = net sectional area of the connected leg

A_2 = gross cross-section area for the unconnected leg

$K_1 = \frac{3xA_1}{3xA_1+A_2}$ (only one leg connected to gusset plate)

Where,

A_1 = (110 – 21.5 – 8/2) x 8

= 676 mm^2

A_2 = (110 – 8/2) x 8

$= 848\ mm^2$

$K_1 = [(3 \times 676) / (3 \times 676) + (848)]$

$= 0.705$

Therefore effective net are required

$A_{net} = A_1 + K\ A_2$

$= 676 + (0.705) \times 848$

$= 1273.84\ mm^2$

Strength of member = $A_{net} \times \sigma_{at}$

$= 1273.84 \times 150$

$= 191076$ N

$= 191.076$ kN > 21.64 kN (safe)

- **Design of bracings (5 to 12)**

The force values are taken from Table 4.10 in chapter 4.

Compressive force = 16.96 kN

Tensile force = 16.96 KN

Length = 6.324 m

Bolt diameter = 20 mm

Effective length = 0.85 x L

= 0.85 x 6324

= 5.375 m

Allowable stress = $\sigma_{ac} \times 1.0$

Assuming $\sigma_{ac} = 60\ N/mm^2$

Therefore cross-section area required = 16.9 x 1000 /60

$= 282.66\ mm^2$

Try ISA 70 x 70 x 8

Properties of the section:

Area = $1058\ mm^2$

Thickness = 8 m

r_{min}= 21.2 mm

Now, slenderness ratio (λ) = L_{eff} / r_{min}

= 5437 / 21.2

= 256.46

The value of allowable compressive stress is now found out by interpolation from table 5.1 in IS: 800-1984

Therefore allowable compressive stress σ_{ac} = 18 N/mm^2

Permissible load = stress x area

= 18 x 1058

= 1904 N

= 19.04 kN > 16.96 kN (safe)

Check for the tension force

The values of force are taken from Table 4.10 from chapter 4

Tension force = 16.96 kN

Bolt diameter = 20 mm

Assuming tensile stress σ_{at} in steel = 150 N/mm^2

Now gross diameter of bolt hole = 20 + 1.5

= 21.5 mm

Net effective area from eqn 5.3

$A_{net} = A_1 + K A_2$

A_1 = net sectional area of the connected leg

A_2 = gross cross-section area for the unconnected leg

$$K_1 = \frac{3xA_1}{3xA_1+A_2}$$

Where,

A_1 = (70- 21.5 – 8/2) x 8

= 356 mm^2

A_2 = (70 – 8/2) x 8

= 528 mm^2

K1 = [(3 x 356/ (3 x 356) + (528)]

= 0.40

Therefore effective net are required

$A_{net} = A_1 + K A_2$

= 356 + (0.40) x 528

$= 567.2\ mm^2$

Strength of member = $A_{net} \times \sigma_{at}$

$= 567.2 \times 150$

$= 85080\ N$

$= 85.080\ kN > 16.96\ kN$ (safe)

- **Design of horizontal members (A_1 to A_4)**

The value of forces is taken from table 4.6 from chapter 4

Compressive force = 4.9 kN

Tensile force = 4.9 kN

Length = 2.0 m

Bolt diameter = 20 mm

Effective length = 2.0 x 1.0

= 2.0 m

Allowable stress = $0.8 \times \sigma_{ac}$

Assuming $\sigma ac = 60\ N/mm^2$

Therefore cross-section area required = 4.9 x 1000 / 60

$= 81.66\ mm^2$

Try ISA 50 x 50 x 6 mm

Properties of the section:

Area = $568\ mm^2$

Thickness = 6 mm

r_{min}= 15.1 mm (minimum radius of gyration)

Now, slenderness ratio (λ) = L_{eff} / r_{min}

= 2000 / 15.1

= 132.45 < 180 (o.k.)

The value of allowable compressive stress is now found out by interpolation from table 5.1 in IS: 800-1984

Therefore allowable compressive stress $\sigma_{ac} = 56.5\ N/mm^2$

$\sigma_{allowable} = 0.8 \times 56.5$

$= 45.2\ N/mm^2$

Permissible load = stress x area

= 45.2 x 568

= 25673.6 N

= 25.67 kN > 4.9 kN (safe)

Check for the tension force

The force values are taken from Table 4.6 from chapter 4

Tension force = 4.9 kN

Bolt diameter = 20 mm

Assuming tensile stress σ_{at} in steel = 150 N/mm^2

Now gross diameter of bolt hole = 20 + 1.5

= 21.5 mm

Net effective area from eqn 5.3

$A_{net} = A_1 + K A_2$

A_1 = net sectional area of the connected leg

A_2 = gross cross-section area for the unconnected leg

$K_1 = \dfrac{3xA_1}{3xA_1+A_2}$ (only one leg connected to gusset plate)

Where,

$A_1 = (50 - 21.5 - 6/2) \times 6$

$= 153\ mm^2$

$A_2 = (50 - 6/2) \times 6$

$= 282\ mm^2$

$K_1 = [(3 \times 153) / (3 \times 153) + (282)]$

$= 0.206$

Therefore effective net are required

$A_{net} = A_1 + K A_2$

$= 153 + (0.206) \times 282$

$= 211.09\ mm^2$

Strength of member = $A_{net} \times \sigma_{at}$

= 211.09 x 150

= 31663.5 N

= 31.66 kN > 4.9 kN (safe)

- **Section design for panel 7**

Design of main legs

The compressive and tensile force values are carried out from Table 4.9 in chapter 4 for panel 1.

Compression force = 876.52 kN

Tension force = 818.58 kN

Length = 5.0 m

Bolt diameter = 20 mm

Effective length (l_{eff}) = 5.0 x 0.85

= 4.25 m

Allowable stress = 1.0 x σ_{ac}

Assuming stress = 60 N/mm^2

Area required = (876.52 x 1000)/ 60

= 14608.66 mm^2

Area to be provided by one angle = 14608.8/2

= 7304 mm^2

Try ISA 2L 200 x 200 x 25

Properties of section:

Area = A = 9380 mm^2

I_{xx} = 3436.3 x 10^4 mm^4

I_{yy} = 3436.3 x 10^4 mm^4

C_{xx} = C_{yy} = 58.8 mm

I_{xx} for two angles = 2 x I_{xx}

= 2 x 3436.3

= 6872.6 x 10^4 mm^4

I_{yy} for two angles = 2[I_{yy} + A(C_{yy} +gusset /2)^2]

= 2 x [3436.3 x 10^4 + 9380 (58.8 + 26/2)^2]

= 9671.9 x 10^4 mm^4

I_{min} = I_{xx} = 3436.3 x 10^4 mm^4

r_{min} = $\sqrt{}$(I_{min} / A)

= $\sqrt{}$(3436.3 x 10^4)/ (2 x 9380)

= 42.79 mm

Slenderness ratio = λ = l_{eff} / r_{min}

= (4.25 x 1000)/ 42.79

= 99.32 < 180 (OK)

The value of allowable compressive stress is now found out by interpolation from table 5.1 in IS: 800-1984

Therefore allowable compressive stress σ_{ac} = 80 N/mm^2

Permissible load = (80 x 2 x 9380)/(1000)

= 1500.8 kN > 876.52 kN (not economical)

Try ISA 2L 200 x 200 x 15

Properties of section:

Area = A = 5780 mm^2

I_{xx} = 2197.7 x 10^4 mm^4

I_{yy} = 2197.7 x 10^4 mm^4

C_{xx} = C_{yy} = 54.9 mm

I_{xx} for two angles = 2 x I_{xx}

= 2 x 2197.7

= 4395.4 x 10^4 mm^4

I_{yy} for two angles = 2[I_{yy} + A(C_{yy} +gusset /2)^2]

= 2 x [2197.7 x 10^4 + 5780 (54.9 + 16/2)^2]

= 8969.00 x 10^4 mm^4

I_{min} = I_{xx} = 4395.4 x 10^4 mm^4

r_{min} = √(I_{min} / A)

= √(2197.7 x 10^4)/ (2 x 5780)

= 43.60 mm

Slenderness ratio = λ = l_{eff} / r_{min}

= (4.25 x 1000)/ 43.60

= 97.47 < 180 (OK)

The value of allowable compressive stress is now found out by interpolation from table 5.1 in IS: 800-1984

Therefore allowable compressive stress σ_{ac} = 82.53 N/mm^2

Permissible load = (82.53 x 2 x 5780)/(1000)

= 954.04 kN > 876.52 kN (OK)

Check for tension force

Tension force = 818.58 kN

Assuming tensile stress in steel = σ_{at}

$= 150\ N/mm^2$

Area required = (818.58 x 1000)/150

= 5457.2 kN

Area to be provided by single angle section = 5457.2/2

$= 2728.6\ mm^2$

Gross diameter of bolt hole = 20 + 1.5 = 21.5 mm

As the angles are on the both sides of gusset plates net effective area

Net effective area = A_{net} = 2[A – deduction for bolt holes]

= 2 x [5780 – (21.5 x 16)]

$= 10915\ mm^2$

Permissible load = (10915 x 150)/1000

= 1637 kN > 818.58 kN (OK)

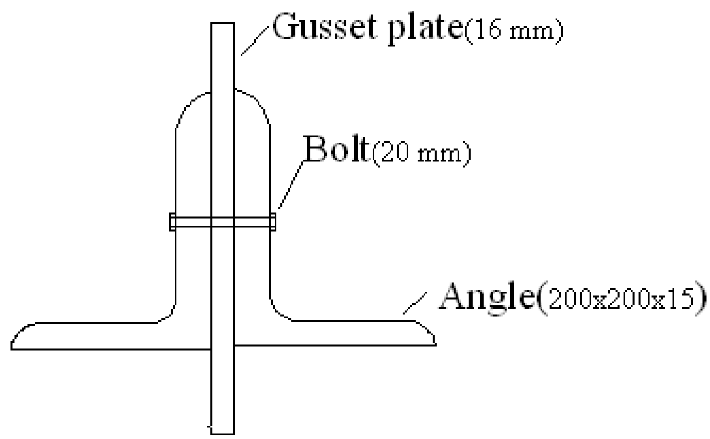


Fig 5.2 View of double angle section

Similar process is carried out for all member design of panel 1 to 10 and section designed is been tabulated in table 5.3.

Table 5.3 Section provided in different panels

Panel from Top	Member Numbers	Axial force (kN)	Length(m)	Provided section	
				Designation	No
1	`1 to 4(main leg)	29.1	6	ISA 110 x 110 x 8	1
	5 to 12 (bracings)	16.96	6.324	ISA 70 x 70 x 8	1
	A1 to A4 (horizontal	4.9	2	ISA 50 X 50 X 6	1
2	13 to 16 (main leg)	107.21	6	ISA 110 X 110 X 12	1
	17 to 24(bracings)	31.25	6.324	ISA 80 X 80 X 6	1
	B1 to B4 (horizontal)	4.9	2	ISA 50 X 50 X 10	1
3	25 to 28 (main leg)	209.40	6	ISA 130 X 130 X 15	1
	29 to 36 (bracings)	46.45	6.324	ISA 90 X 90 X 10	1
	C1 to C4 (horizontal)	4.9	2	ISA 50 X 50 X 6	1
4	37 to 40 (main leg)	423.21	6	ISA 150 X 150 X 15	2
	41 to 48 (bracings)	62.25	6.324	ISA 100 X 100 X 10	1
	D1 to D4 (horizontal)	5.1	2	ISA 50 X 50 X 6	1
5	49 to 52 (main leg)	654.11	6	ISA 150 X150 X 18	2
	53 to 60 (bracings)	78.70	6.324	ISA 100 X 100 X 12	1
	E1 TO E4 (horizontal)	5.7	2	ISA 50 X 50 X 6	1
6	61 to 64 (main leg)	957.6	6	ISA 200 X 200 X 18	2
	65 to 72 (bracings)	95	6.324	ISA 110 X 110 X 12	1
	F1 to F4 (horizontal)	6.5	2	ISA 50 X 50 X6	1
7	73 to 76 (main leg)	876.52	5	ISA 200 X 200 X15	2
	77 to 84 (bracings)	66.54	5.76	ISA 100 x 100 x 10	1
	G1 to G4 (horizontal)	7.3	2	ISA 50 X 50 X 6	1
8	85 to 88 (main leg)	779.86	5	ISA 150 X 150 X 18	2
	89 to 96 (bracings)	68.59	6.60	ISA 100 x 100 x 12	1
	H1 to H4 (horizontal)	8.3	3.50	ISA 50 X 50 X 6	1
9	97 to 100 (main leg)	742.52	5	ISA 200 X 200 X 12	2
	101 to 108 (bracings)	124.28	8.28	ISA 130 X 130 X 15	1
	I1 to I4 (horizontal)	9.1	5	ISA 50 X 50 X 6	1
10	109 to 112 (main leg)	737.32	5	ISA 200 X 200 X 12	2
	113 to 120 (bracings)	136.22	9.67	ISA 130 X130 X 15	1
	J1 TO J4 (horizontal)	9.1	6.50	ISA 50 X 50 X 6	1

5.5 CONNECTION DESIGN

Tower structures are usually bolted type. The diameter of bolts shall not be less than 12 mm. Bolts used for erection of microwave lattice towers shall be of diameter 12, 16 and 20 mm. The length of bolts shall be such that the threaded portion does not lie in the plane of contact of members. Connections are to be designed for the relevant shear and bearing stresses and the class of bolts used. There is no restriction on the number of bolts.

The design axial forces, angle size with the bolts provided for the main legs of tower are shown in Table 5.4. Same information about the Bracings and horizontal members of the tower is shown in table 5.5 and 5.6 respectively.

The sketch of detailing of tower is shown in fig 5.1. In connection the diameter of bolt is used about 12, 16 and 20 mm. The bolts used for connection are of property class 4.6 class and the maximum permissible stress as per IS: 800-1984 table 8.1.

Table 5.4 Design of bolts of tower main leg members using 4.6 class bolts

Panel no	Angle size	Dia of bolts(mm)	Max force(kN)	Bearing force(kN)	Shearing force (kN)	Min force value(kN)	No of bolts reqd	Bolts prov
1	110x110x8	16	29.1	52.5	24.052	24.05	1.2	2
2	110x110x12	16	107.21	73.5	24.052	24.05	4.45	6
3	130x130x15	16	209.4	84	24.052	24.05	8.70	10
4	2l 150x150x15	20	423.21	103.2	72.61	72.61	5.8	6
5	2l 150x150x18	20	654.11	129	72.61	72.61	9.0	10
6	2l 200x200x18	20	957.6	129	72.61	72.61	13.18	14
7	2l 200x200x15	20	876.52	103.2	72.61	72.61	12.07	12
8	2l 150x150x18	20	779.86	129	72.61	72.61	10	10
9	2l 200x200x12	20	742.52	103.2	72.61	72.61	10	10
10	2l 200x200x12	20	737.32	90.3	72.61	72.61	10.15	10

Bolt calculation for panel 1:

Maximum axial force = 29.1 kN

Section assigned = ISA 110 x 110 x 8

Dia of bolts = 16 mm

Strength of bolts in shearing and bearing

As per IS: 800-1984 Table 8.1,

Maximum permissible stress in bolts

Shear (ζ_{vf}) = 100 Mpa

Bearing (σ_{pf}) = 300 Mpa

1. Strength of bolts in single shear = $100 \times \Pi/4 \times d^2$

   $= 100 \times \Pi/4 \times 17.5^2$

   = 24052 N = 24.052 kN

2. Strength in bearing 10 mm plate = 17.5 x 10 x 300

   = 52500 N = 52.500 kN

Minimum strength value of bolts from 1 and 2

Therefore minimum value = 24.052 kN

No of bolts required = max axial force / min rivet value

= 29.1 / 24.052

= 1.20

Therefore provide 2 no of bolts

Similar process for bolt calculation is carried out for all the members for panels 1 to 10.

Table 5.5 Design of bolts of tower bracing members using 4.6 class bolts

Panel no	Angle size	Dia of bolts(mm)	Max force(kN)	Bearing force (kN)	Shearing force(kN)	Min force value(kN)	No of bolts reqd	Bolts prov
1	70x70x8	12	16.96	40.5	14.31	14.31	1.1	2
2	80x80x6	12	31.25	32.4	14.31	14.31	2.1	2
3	90x90x10	12	46.45	48.6	14.31	14.31	3.2	4
4	100x100x10	16	62.25	63	24.052	24.05	2.5	4
5	100x100x12	16	78.70	73.5	24.052	24.05	3.2	4
6	110x110x12	16	95	52.5	24.052	24.05	3.9	4
7	100x100x10	16	66.54	52.5	24.052	24.05	2.7	4
8	100x100x12	16	68.59	63	24.052	24.05	2.8	4
9	130x130x15	16	124.28	52.5	24.052	24.05	5.1	6
10	130x130x15	16	136.22	63	24.052	24.05	5.6	6

Table 5.6 Design of bolts of tower horizontal members using 4.6 class bolts

Panel no	Angle size	Dia of bolts(mm)	Max force(kN)	Bearing force (kN)	Shearing force(kN)	Min force value(kN)	No of bolts reqd	Bolts prov
1	50x50x6	12	4.9	24.3	14.31	14.31	0.3	2
2	50x50x6	12	4.9	24.3	14.31	14.31	0.3	2
3	50x50x6	12	4.9	24.3	14.31	14.31	0.3	2
4	50x50x6	12	5.1	24.3	14.31	14.31	0.3	2
5	50x50x6	12	5.7	24.3	14.31	14.31	0.3	2
6	50x50x6	12	6.5	24.3	14.31	14.31	0.4	2
7	50x50x6	12	7.3	24.3	14.31	14.31	0.5	2
8	50x50x6	12	8.3	24.3	14.31	14.31	0.5	2
9	50x50x6	12	9.1	24.3	14.31	14.31	0.6	2
10	50x50x6	12	9.1	24.3	14.31	14.31	0.6	2

5.6 TOWER FOUNDATION

The stability of tower depends both on the strength as well as stability of foundations. The foundation for tower is designed for the following forces and moments:

- Downward load on leg
- Uplift load on leg
- Horizontal thrust
- Over turning moments

Generally the load acting on the top of footing is inclined, and this inclined load is resolved into the vertical and horizontal (lateral) components.

The lateral and the longitudinal loads, acting at a greater height cause large overturning moments, which are to be resisted by the foundation with a minimum factor of safety of three.

5.6.1 Types

The different type of foundation as per Dayaratnam [6] depends upon the type of loads acting on tower are as follows:

Table 5.7 Classification of foundation for a tower

Class of foundation	Type of loading	Types of foundation
Class A	Heavy uplift with light shear	Enlarged base or individual footing under each leg
Class B	Heavy overturning moments with light vertical load and shear	With or without enlarged base or piles
Class C	Heavy downward load	Enlarged base Under-reamed piles Group of piles

5.6.2 Design procedure

The design of any foundation consists of following two parts:

1. Stability analysis

Stability analysis aims at the possibility of failure of foundation by tilting, overturning, uprooting and sliding due to load intensity imposed on soil by foundation being in excess of the of the ultimate capacity of the soil.

a) check for bearing capacity

The total downward load at the base of footing consists of:

- compression per leg derived from tower design
- Buoyant weight of concrete below ground level

Thus, the maximum soil pressure below the base of foundation (toe pressure) depends upon the vertical thrust (compression load) on the footing and moments at the base level due to the horizontal shears and other eccentric loadings. Under both action of down thrust and moments, the soil pressure below the footing is not uniform and maximum toe pressure 'P' on the soil is determined from the following equation:

$$P = \frac{W}{B^2} + \frac{M_T}{Z_T} + \frac{M_L}{Z_L} \qquad \text{... 5.6}$$

Where,

W = Total vertical downward thrust including the weight of footing

B = dimension of footing base

M_t and M_L = moments at the base of footing about the transverse and longitudinal axes of the footing

Z_T and Z_L = section modulii of a square footing

b) Check for uplift resistance

The resistance to uplift is considered to be provided by

- Buoyant weight of foundation
- Weight of soil volume contained in the inverted frustum of cone on the base of the footing as shown in fig 5.2.

The ultimate resistance to uplift is given by:

$$U_p = W_s + W_f \qquad \text{... 5.7}$$

Where,

W_s = weight of soil in the frustum of cone

W_f = weight of foundation

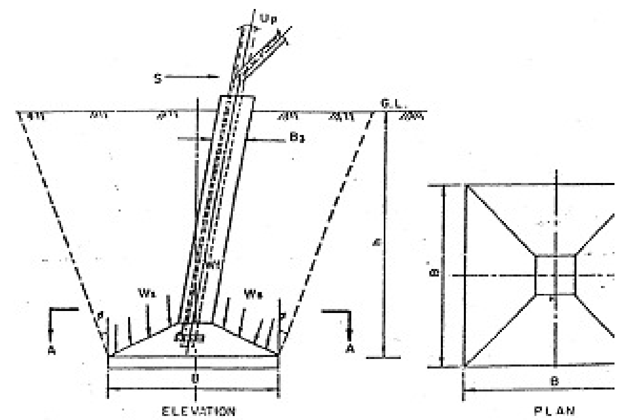


Fig 5.3 Resistance against uplift

c) Check for side thrust

When the lateral load acting, the column will act as a cantilever beam free at the top and fixed at the base and supported by the soil along its height. Stability of a footing under a lateral loads depends on the amount of passive pressure mobilized in the adjoining soil as well as the structural strength of footing as shown in fig 5.3.

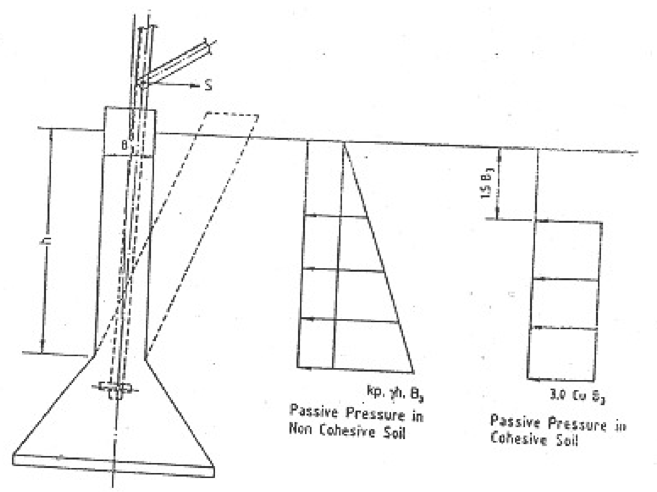


Fig 5.4 pressure diagram for soil

d) Check for overturning

Stability of the foundation against overturning is checked by following criteria:

- The foundation over-turns at the toe
- The weight of footing acts at the centre of base
- Part of earth cone standing over the heel causes the stabilizing moment

(2) Structural design of foundation

Structural design of concrete foundation comprises the design of following:

- Column
- Base slab/pyramid/block

a) Design of column

- The column is designed for maximum bending moments due to the side thrust
- Combined uplift and bending determines the requirement of longitudinal reinforcement in the column

- Stub angle is used to connect the last angle of panel and the foundation

Structural detailing of column

- In any column that has the larger cross-sectional area than that required to support the load, the minimum percentage of steel is based on the area of concrete required to resist the direct stress
- The minimum number of longitudinal bars provided in a column shall be four in square column and six in circular column
- The bars shall not be less than 12 mm in diameter
- Where longitudinal reinforcement is required in strength, nominal longitudinal reinforcement not less than 0.15% of the cross-sectional area is provided

- The spacing of stirrups/ lateral ties shall not be more than the least of the following distances:
  (1) The least lateral dimension of column
  (2) Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied.
  (3) Forty-eight times the diameter of the transverse stirrups/lateral ties
- The diameter of the polygonal links or lateral ties shall be not less than one-fourth of the diameter of the largest longitudinal bar, no less than 6 mm

b) Structural design of base slab

The base slab in R.C.C. spread foundations is single stepped or multi stepped. The design of concrete foundations is done as per limit state method of design given in IS: 456-2000

Codal stipulation for design of R.C.C. foundations

The important codal stipulation for concrete foundation considered in design is explained below:

- Footings shall be designed to sustain the applied loads, moments, forces and the induced reactions and to ensure that any settlement which may occur shall be as nearly uniform and the bearing capacity of soil is not exceeded
- Thickness to the edge of footing in reinforced concrete footings shall not be less than 15 cm (5 cm lean concrete plus 10 cm structural concrete). In case of plain concrete footing thickness of the edge shall be not less than 5 cm

- Bending moment

- The bending moment at any section shall be determined by passing through the section of a vertical plane which extends completely across the footing and computing the moment of the forces acting over the entire area of the footing on the side of the said plane
- The greatest bending moment to be used in the design of an isolated concrete footing which supports a column and should be computed at the face of the column and at the thickness where width/ thickness changes

- Shear and bond

The shear strength of footing is governed by the more severe of the following two conditions:

- The footing acting essentially as a wide beam, with a potential diagonal crack extending in a place across the entire width; the critical section for this condition shall be assumed as a vertical section located from the face of the column at a distance equal to the effective depth of the footing
- Two-way action of the footing, with potential diagonal cracking along the surface of truncated cone or pyramid around the concentrated load

5.6.3 Design

The foundation is of class A type as per table 5.7 with individual footing under each leg.

a) Working foundations loads from tower support reactions

Compression = 737.32 kN

Uplift = 633.78 kN

Transverse thrust = 137.32 kN

Longitudinal thrust = 137.32 kN

Transverse thrust and longitudinal thrust is carried out by the wind force acting on the tower which is carried out by adding the wind load acting on each panel joints.

Transverse thrust = 2(4.6 + 2.96 + 4.9 + 4.7 + 4.9 + 5.1 + 5.7 + 6.5 +7.3 + 8.3 + 9.1 + 4.6)

= 137.2 kN

b) Soil properties

Soil properties are tabulated in Table 5.8

Table 5.8 Soil properties

Type of soil	Dry unit weight	Frustum of cone	Max allow bearing capacity
Hard cohesive soil	1800 kg/m^3	30°	400 kPa

c) Concrete properties

Grade of concrete: 25 N/mm^2

d) Uplift resistance

Uplift resistance for the foundation is carried out by eqn 5.7

$$U_p = W_s + W_f$$

Where,

W_s = weight of soil

W_f = weight of footing

To find the uplift pressure depth of foundation is to be carried out by eqn 5.8

$$H = \frac{p}{w}\left[\frac{(1-\sin\theta)}{(1+\sin\theta)}\right]^2 \qquad \text{… 5.8}$$

Where,

H = depth of foundation

p = downward load

w = Unit weight of soil = 1800 kg/m^3 (Table 5.8)

= (1800 x 9.81)/1000

= 17.65 kN/m^3

H = (737.32/17.65) x [(1- sin30) / (1 + sin30)]^2

= 4.59 m

Provide total depth of 4.6 m

Area of footing A_f = 1.1 p / safe bearing capacity of soil

Therefore A_f = 1.1 x 737.32/ 400

= 2.02 m^2

Provide 2.8 m x 2.8 m square footing

B_f = 2.8 m

L_f = 2.8 m

Volume of soil in the depth of foundation is carried out by,

$$V_u = \frac{H}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right] \quad \text{... 5.9}$$

Where,

A_1=A_2 = B^2 + 4 X B X H tanΘ + ∏ H^2 $\tan^2$Θ

Where,

B = width of footing

H = height of footing

Θ = frustum of cone

A_1=A_2= $(2.8)^2$ + ((4 x 2.8 x 4.6 x tan(30)) + ((∏ x 2.8^2 x $\tan^2$ (30))

= (7.84) + (29.74) + (8.21)

= 45.79 m^2

Therefore volume of soil from eqn 5.9,

V = 4.6/3 [45.79 + 45.79 +√ (45.79 x 45.79)

= 210.63 m^3

Unit weight of soil = 17.65 kN/m^3

Therefore weight of soil W_s= 17.65 x 210.63

= 3717.61 kN

Volume of footing = 0.756 + 0.693 + 1.568 + 0.975 (Volume of the footing as seen in Fig 5.5)

Volume of footing = 3.99 m^3

Weight of concrete = 235.44 kN/m^3

Therefore weight of footing W_f = 235.27 x 3.992

= 939.19 kN

Uplift resistance U_p = W_s + W_f

= 3717.61 + 939.19

= 4656.8 kN > (737.32 x 1.5 = 1105.9 kN)

e) Check for bearing capacity

Max allowable bearing capacity of soil = 400 kN/m^2

The maximum toe pressure "p" on soil can be determined by

$$P = \frac{W}{B^2} + \frac{M_T}{Z_T} + \frac{M_L}{Z_L}$$

W = Total vertical downward load + Over weight of footing

W = 737.32 + 939.19

= 1676.51 kN

M_t = Moment at the base of footing about transverse axes of loading

Transverse thrust = 137.32 kN

Height (h) = 4.6 m (total height of foundation)

Moment at the base of footing = 631.6 kNm

M_l = Moment at the base of footing about longitudinal axes of loading

Longitudinal thrust = 137.32 kN

Height (h) = 4.6 m (total height of foundation)

Moment at the base of footing = 631.6 kNm

Z_t = Z_l = Section modulli of the square footing

= 1/6 (B^3)   where B = 2.8 m

= 3.65 m

Therefore pressure at toe = P = {(1676.51 /2.8 x 2.8) + (631.6/3.65) + (631.6/3.65)}

= 27.27 + 173.041 + 173.041

= 373.35 kN < 400 kN (safe)

Factor safety obtained = 1.07

Factor of safety required = 1.0

f) Check for overturning

Moment due to uplift = 633.18 x B/3

= 590.96 kNm

W_s = Weight of soil = 3717.61 kN

W_f = Weight of footing = 939.19 kN

Resisting moment due to soil = {(W_s/2) x (5B/6)}

= {(3717.61 /2) x (5 x 2.8/6)}

= 4331 kNm

Resisting moment due to Concrete = (W_c X B/3)

= (939.19 x 2.8/3)

= 876.57 kNm

Total resisting moment = 4331+ 876.57

= 5207 kN > 590.96 kNm

g) Design of column

Compressive force P_u= 737.32 kN

Assume the reinforcement percentage p = 2.5

p/f_{ck}= 1.5/25 d^i = 70 mm (cover)

= 0.06   D = 700 mm (depth of column)

$P_u/f_{ck}bd$ = 737.32 x10^3/25x700x700 d^i/D = 0.1

= 0.06

Chart 69 of sp-16 is used for d^i/D = 0.1 f_y = 415 N/mm^2

From chart,

$M_{ux1}/f_{ck}bd^2$ = 0.12

M_{ux1} = M_{uy1} = 1029 kNm

M_{ux}= 137.32 x 3.45 M_{uy}= 137.32x 3.45

= 473.7 kNm   = 473.7kNm

As per cl. 38.6 of IS-456:1978

$[M_{ux}/ M_{ux1}]^{\alpha n} + [M_{uy}/M_{uy1}]^{\alpha n} < 1.0$ ... 5.10

$^{\alpha n =}$ 1.0 for tension with bending

[473.7 /1029] + [473.7 /1029]

[0.46] + [0.46]

0.92 < 1.0 hence O.K

A_{st} =1.5 x 70 x 70/100 = 73.50 cm^2

Provide 20 bars of 22 mm dia will give 76.02 cm^2

Transverse reinforcement

Diameter of lateral ties shall not be less than one-fourth of the diameter of the largest longitudinal bar and no less than 6 mm.

Therefore diameter of lateral ties = 22/4 = 5.5 mm

Hence provide lateral ties of 6mm

Spacing of stirrups/lateral ties shall not be more then the least of the following distances:

1. The least lateral dimension of column = 700 mm
2. Sixteen times the smallest diameter of the longitudinal reinforcement
   = 16 x 28
   = 448 mm
3. Forty-eight times the diameter of the transverse stirrups/lateral ties
   = 48 x 6
   = 288 mm

   Therefore provide 15 nos of 6 mm lateral ties @ 290 mm c/c.

h) Design of footing

Depth of footing from bending moment consideration

Ultimate moment at column face parallel to x- axis

$M_{ux} = W_u \times B_f \times C_x^2/2$ $\quad\quad$ $C_y = C_x = (L_f - d_f)/2$

$= 137.32 \times 2.8 \times 1.075^2/2$ $\quad\quad$ $= (2.8 - 0.650)/2$

$= 222.16$ kNm $\quad\quad$ $= 1.075$

Required effective depth for bending about x-x axis

$$d_x = \sqrt{\frac{M_{ux}}{R_{u\max} b_1}} \quad \text{... 5.11}$$

M_{ux} = 222.16 kN/m

R_{umax} = 3.45

b_1 = Width at the top of footing

= b + 2e

= 700 + 2 x 50

= 800 mm

Therefore from eqn 5.11 effective depth

$d_x = \sqrt{(222.16 \times 10^6)/(3.45 \times 800)}$

d_x = 283 mm

Ultimate moment at column face parallel to y-axis

$M_{uy} = W_u \times B_f \times C_x^2/2$

$= 737.32 \times 2.8 \times 1.075^2/2$

$= 1192.89$ kNm

Required effective depth for bending about y-axis

$$d_y = \sqrt{\frac{M_{uy}}{R_{u\max} D_1}} \qquad \text{… 5.12}$$

$M_{uy} = 1192.89$ kNm

$R_{umax} = 3.45$

D_1 = Length of footing at top with 50 mm level projections

= D + 2e

= 650 + 2 x 50

= 750 mm

therefore effective depth from eqn 5.12

$d_y = \sqrt{(1192.89 \times 10^6)/(3.45 \times 750)}$

$d_y = 678$ mm

Provide total depth of 680 mm

Effective cover for bottom steel = 50 mm

Effective depth for bending about x-axis d_{eff} = 650-50-10

= 620 mm

i) Check the depth of footing for two-way shear

Critical section is taken at $d_{eff}/2$ from the periphery of column

Perimeter at critical section = 2{700 + 700 + (2 x 620)}

= 5280 mm

Effective depth d_2 at the critical section

X_1 = (2800 – 700 -100)/2

= 1000 mm

Y_1 = 680 – 200

= 480 mm

$Y_2 = (d_{eff}/2 - e)\, Y_1/X_1$

= (620/2 – 50) x 480/1000

= 124.8 say 125 mm

d_{eff} = 680-125–50 (effective depth at critical section)

$= 505$ mm

Area resisting shear = A_2 = 5280 x 505

= 2666400 mm^2

Shear resisted by concrete

$\zeta_{uc} = 0.25\sqrt{f_{ck}}$ (shear strength of concrete)

$= 0.25\sqrt{25}$

$\zeta_u = 1.25$ N/mm^2

$\zeta = k_s\, \zeta_{uc}$ (from IS 456:2000 clause 40.2.1.1)

= 1 x 1.25

= 1.25 N/mm^2

$V_{uc2} = \zeta_{uc}$ x A_2

= 1.25 x 2666400/1000

= 3333 kN

Design shear V_{ud} = 737.32x [(2800 x 2800) – (700 + 620) x (700 + 620)] x10^{-6}

= 3295 kN

j) Area of steel

$$A_{sx} = \frac{0.5 f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6xM_{ux}}{f_{ck} b_1 d_x^{\,2}}}\right) b_1 d_x \qquad \text{… 5.13}$$

A_u = (0.5 x 25/415)[1- √1- (4.6 x 222.16 x 10^6)/(25 x 800 x 678^2)] x 800 x 678

A_{stx} = 1028.33 mm^2

$A_{stx.min}$ = [0.85 x 800 x 620] /415

= 1015.9 mm^2

Provide 14 nos of 10 mm dia bars (1099.55 mm^2)

$$A_{sy} = \frac{0.5 f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6xM_{uy}}{f_{ck} D_1 d_y^{\,2}}}\right) D_1 d_y \qquad \text{… 5.14}$$

A_u = (0.5 x 25/415)[1- √1- (4.6 x 1192.89 x 10^6)/(25 x 750 x 678^2)] x 750 x 678

A_{sty} = 6083 mm^2

$A_{sty.min}$ = [0.85 x 750 x 620] /415

= 952.40 mm^2

Provide 20 nos of 20 mm dia bars (6283.18 mm^2)

Spacing between bars = (2500 – 150 – (20 x 20)]/20-1

= 118.42 mm

k) Detailing of footing

The detailed sketch of foundation is shown in fig 5.4.

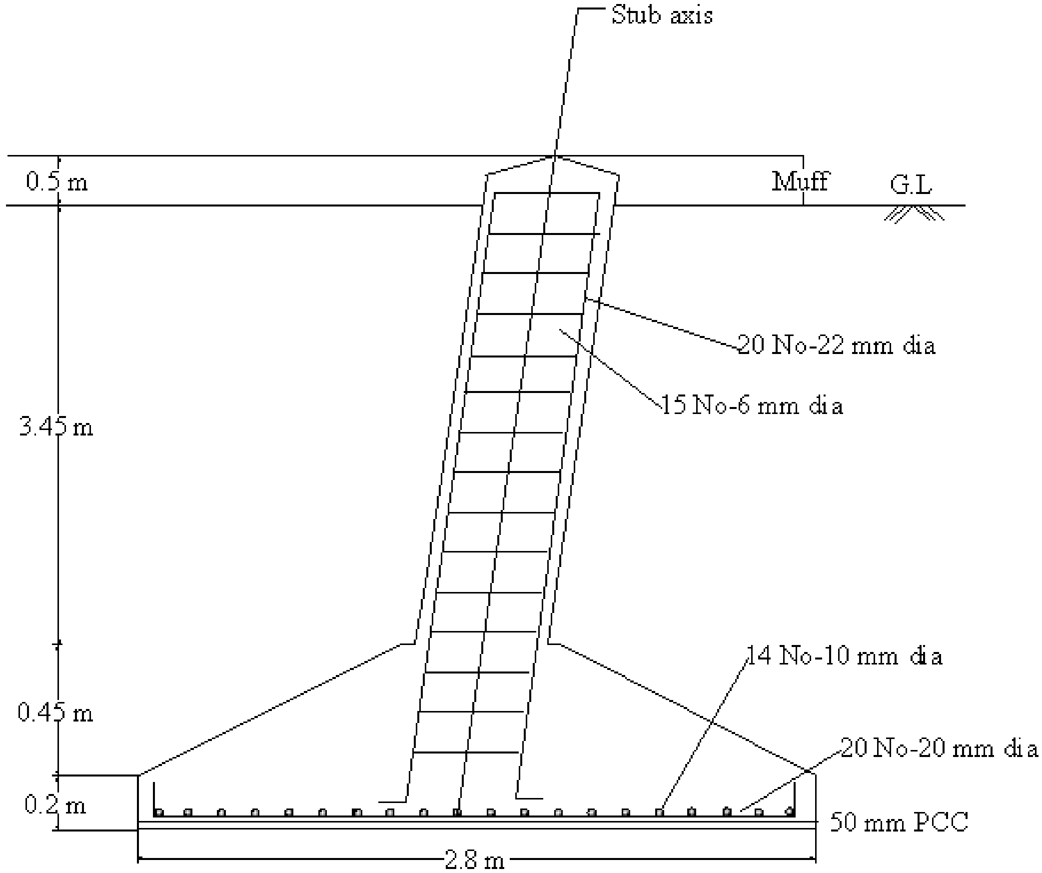


Fig 5.5 Detailing of footing

l) Check for one way shear for bending about y-axis

$D_1 = Y_1 - (d_y - e)\, y_1/x_1$

$= 480 - (678 - 50) \times 480/1025$

$= 185.91$ mm

$Y_1 = D_f - D_{fmin}$

$= 680 - 200$

$= 480$ mm

$X_1 = C_y - e$

$= 1075 - 50$

$= 1025$ mm

Width of footing at critical section = B_2

$B_2 = D + 2d_y$ ... 5.15

$= 700 + 2 \times 678$

$= 2056$ mm

Area of footing at critical section

A_y = (2056 + 2800) D_1/2 + (200 -50) x 2800

= 871389 mm^2

Design shear stress from IS 456(table 19) ζ_{ucy} = 0.5604 N/mm^2

Shear resisted by concrete = V_{ucy} = 0.5604 x 871389/1000

= 488.32 kN

Design shear = V_{udy} = W_u x L_f x (C_y – d_y) … 5.16

Where W_u = upward factored soil pressure

= (1.5 x 737.32)/5

= 221.19 kN/m^2

Design shear = 221.19 x 2800 x (1075 – 620) x 10^{-6s}

= 281.79 kN

Therefore V_{ucy} > V_{udy} (safe).

6. USE OF SOFTWARE

6.1 GENERAL

Engineering design is a process of evolution-it is a process of planning and decision making in order to generate useful information to ensure safety. This applies to all design situations and computer-aided design thus involves the use of computer systems to improve communication, information flow and decision making. Therefore the use of STAAD.pro 2007 for design problem is chosen here. STAAD.Pro is the most popular structural engineering software product for 3D model generation, analysis and multi-material design.

6.2 STATIC LINEAR ANALYSIS OF A TOWER

The procedure for doing a nonlinear static analysis consists of following tasks:

- Building the model
- Applying the loads
- Analysis
- Reviewing the results

To computerize a structural model, it is essential to locate certain points in the structural system-to represent the overall behavior of the system. If the tower is modeled as a beam, specific points on the beam are chosen and they are assigned certain degrees of freedom. In the case of 2-D and 3-D models, the joints are taken as the nodes and they are assigned degrees of freedom.

Normally, towers are triangulated structural systems of primary members such as leg members and bracings and, redundant members are called secondary members. Primary members forming the triangulated system (three dimensional truss) carry the loads from their point of application down to the foundation. Secondary members provide intermediate bracing points to the primary members and thus reduce the unbraced lengths of primary members. In a first order analysis the forces in the secondary members are considered as zero.

6.2.1 Behavior of tower systems

The tower structural system, as a whole has a tendency to undergo large displacements and it is also prone to buckling. The loads on the microwave tower are mainly its self weight and antenna weight, and portion of wind acting on the body of tower. Therefore considering the behavior of towers, they can be modeled as one-, two- or three dimensional systems.

- One dimensional behavior

Here, the tower is treated as a cantilever beam with varying cross-sectional properties along the height. The properties of the beam are calculated assuming that the tower legs are the flanges of an equivalent I- section. The sectional properties of the equivalent I-section-moment of inertia and cross-sectional area are used to find out the forces in the tower due to bending and axial load effects. But in this simple idealization, the effect of bracing and torsion is included in approximate manner. This results in disproportionate distribution of loads on to various members producing either an underdesign or an overdesign.

- Two dimensional behavior

Even though one-dimensional beam model gives an idea of the general behavior of towers, the spatial nature of the tower requires a more realistic modeling to obtain better prediction of tower response. The real 3-D structure is conveniently reduced to a 2-D structure. The loads are also resolved along the planes. These assumptions lead to conservative and hence uneconomical structure.

- Three dimensional behavior

In a three-dimensional idealization, the spatial nature of the structure is used and hence all loads are allowed to act simultaneously. The participation of members is considered for axial, bending and torsional effects respectively.

6.3 GENERATING THE MODEL

This step includes the three dimensional modeling of tower in which it requires the information about the parameters of the tower given below as shown in Fig 6.1.

- Total height of tower
- Top width of tower

- Base width of tower
- No of panels along the height of the structure

After the parameters are defined, generation of model with the help of structure wizard is done as shown in Fig 6.2

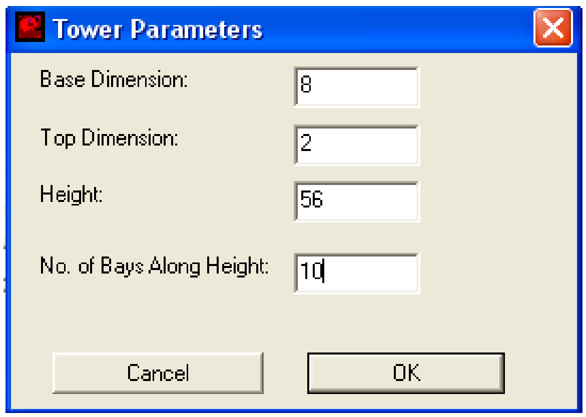


Fig 6.1 A typical view of a tower parameter box

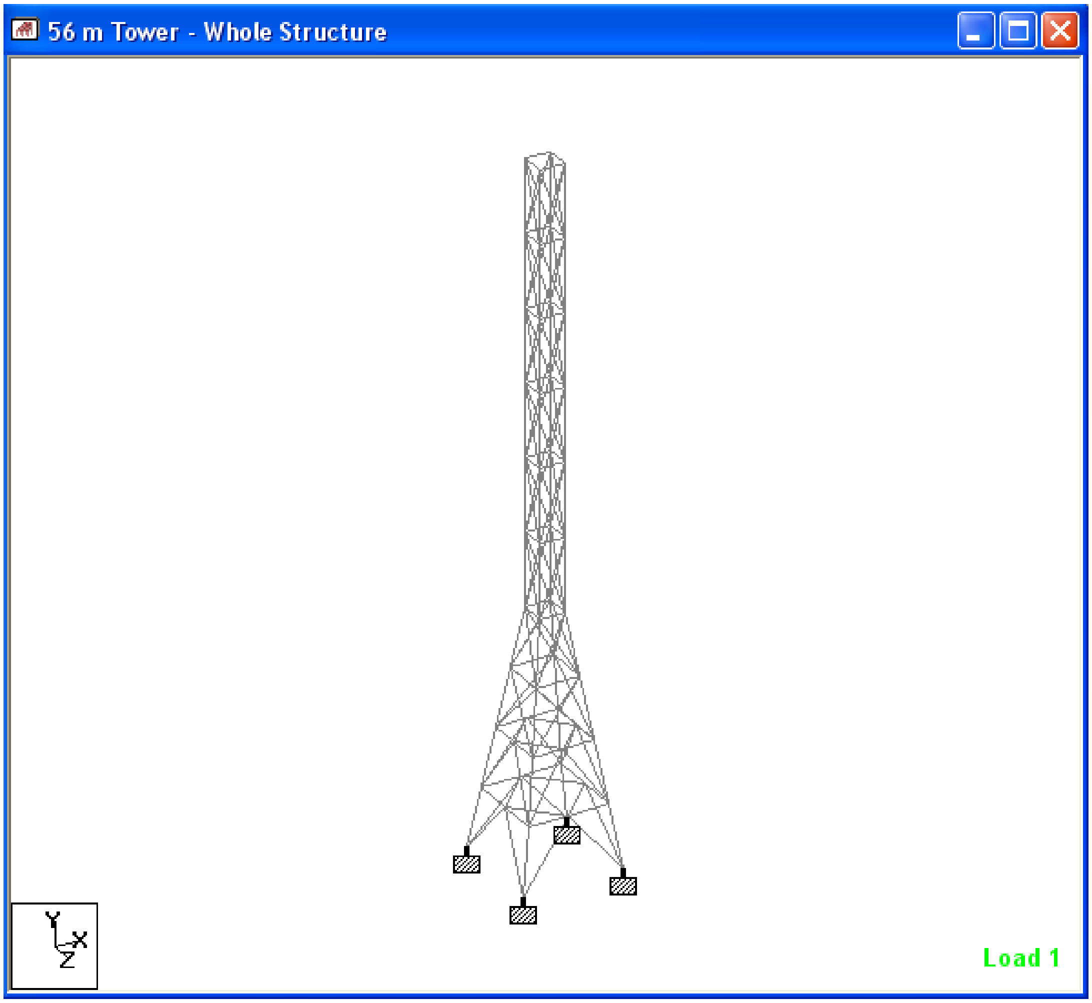


Fig 6.2 Whole view of a structure

After the generation of model general properties to be assigned such as,

- Material
- Specification
- Boundary condition
- Section properties

After completing the model preparation the above stated properties are specified and it is run for analysis and a 3-D view of tower model as shown in Fig 6.3 is obtained.

6.4 APPLYING THE LOADS

Tower like systems are subjected to various kinds of loading. They are both functional and accidental. The functional loads are weight and wind forces acting on the tower. Accidental loads are rarely in nature and sometimes occur due to the pull of wind.

After generating model sections are assumed as per solidity ratio and assigned to all the panels of the tower, and all the members are specified as truss members.

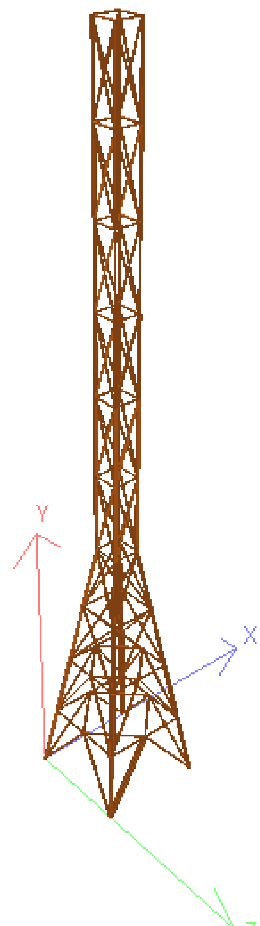


Fig 6.3 3-D model view of a tower

The main loading factor affecting on the design tower is wind load and the weight of the tower. Wind load is calculated depending upon the obstructed area of the tower per panel and the load is distributed between the joints intersecting along the direction of wind as shown in Fig 6.4. The following load cases and load combination are taken to know the behavior of the tower system:

- Dead load of tower
- Wind acting parallel and diagonally on tower
- Dead load and wind acting on antenna

Final Load combination adopted (dead load +wind load) for wind acting parallel and diagonal to tower

Figure 6.4 shows the wind load acting on the tower. Wind load is calculated by the solidity ratio and is divided into four obstructing joints. Dead load on the tower is applied by considering the dead loads at the node. For the consideration of wind acting diagonally loads are to rotate at 45° angle and then apply on the tower. The load acting on antennas is calculated by knowing the dimension and weight of antenna. Generally, the weight of flat panel antennas is very low due to its small size and therefore it is negligible. The wind load on antenna is found out by obstructed area of antenna and distributed among the node where it is attached.

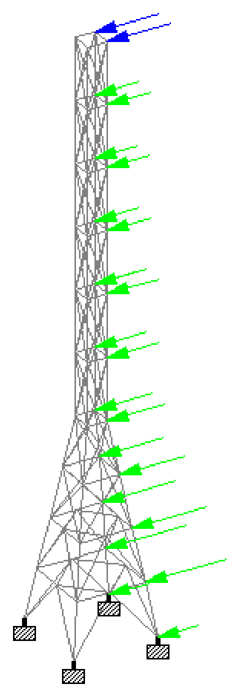


Fig 6.4 Typical view of wind load acting on a tower

The wind load calculated in the chapter 4 on the tower is tabulated in table 6.1 which states the joint load due to wind force acting on it.

6.5 ANALYSIS

After generating the model and assigning all the parameters the software performs the analysis. STAAD also performs pushover analysis, cable analysis etc. The tower resists loads by axial effects (tension and compression), bending, shear and torsion. Since the towers are generally made up of structural angles, are joined by field bolting, the systems have very flexible connections resulting in rotation of joints. This makes the structure responds as a pin-jointed system enable it to carry the axial force i.e. compression and tension.

6.6 REVIEWING THE RESULTS

Results from the linear static analysis mainly consist of nodal displacement, stresses in beams, modes shapes and reaction forces.

The panel wise displacement values of tower is in table 6.2 for the condition of wind blowing parallel and diagonally to the tower.

Wind blowing parallel to tower: 332 mm (from staad)

Wind blowing diagonally to tower: 330 mm (from staad)

The displacement values are taken from staad and it is at the top. The panel wise displacement of tower is tabulated in table 6.1.

Table 6.1 Maximum displacement of a tower

Panel no	Displacement (mm)	
	Wind blowing 0°	Wind blowing 45°
1	332	330
2	239	269.5
3	220	210
4	158	154.3
5	107	104.8
6	63.2	63.5
7	28.75	32.4
8	14.16	15.34
9	3.6	3.68
10	0.051	0.16

A permissible value for 56 m is taken as 300 mm as per BS: 8100 and critical condition for deflection is wind blowing parallel to the tower.

After the analysis is complete, the design of section is also done. Before starting the design following parameters are to be assigned:

- Yield strength of steel

- Slenderness ratio limitations
- Permissible ratio of allowable stress

The axial forces in the main legs and bracings are tabulated in table 6.3 and 6.4 which are generated after modeling the tower analyzing it.

After designing the sections are optimized and table 6.5 shows the results of the angle section given by the Staad for the panels 1 to 10.

Table 6.2 Axial forces in members of a main legs

Panel from Top	Member No	Wind blowing at 0°	Wind blowing at 45°
1	1 to 4	10.65	14
2	13 to 16	35.41	55
3	25 to 28	72.09	131
4	37 to 40	117	248
5	49 to 52	167	392
6	61 to 64	262	590
7	73 to 76	320	453
8	85 to 88	308.9	578
9	97 to 100	311.0	544
10	109 to 112	307.89	563

Table 6.3 Axial forces in bracings of a tower

Panel from Top	Member No	Wind blowing at 0°	Wind blowing at 45°
1	5 to 12	14.83	18
2	17 to 24	25.11	41
3	29 to 36	33.92	66
4	41 to 48	47.10	94
5	53 to 60	58.74	142
6	65 to 72	51.17	146

7	77 to 84	65.43	153
8	89 to 96	156.11	72
9	101 to 108	102.13	24
10	113 to 120	17.67	26

Table 6.4 Section provided in different panels

Panel from Top	Members Numbering	Axial force (kN)	Length (m)	Provided section	
				Designation	No
1	`1 to 4(main leg)	14	6	ISA 130 x 130 x 8	1
	5 to 12 (bracings)	18	6.324	ISA 130 x 130 x 8	1
	A1 to A4 (horizontal	2	2	ISA 90 X 90 X 6	1
2	13 to 16 (main leg)	55	6	ISA 150 X 150 X 10	1
	17 to 24(bracings)	41	6.324	ISA 80 X 80 X 6	1
	B1 to B4 (horizontal)	7	2	ISA 70 X 70 X 6	1
3	25 to 28 (main leg)	131	6	ISA 150 X 150 X 15	1
	29 to 36 (bracings)	66	6.324	ISA 90 X 90 X 10	1
	C1 to C4 (horizontal)	17	2	ISA 90 X 90 X 8	1
4	37 to 40 (main leg)	248	6	ISA 130 X 130 X 12	2
	41 to 48 (bracings)	94	6.324	ISA 100 X 100 X 10	1
	D1 to D4 (horizontal)	33	2	ISA 50 X 50 X 6	1
5	49 to 52 (main leg)	392	6	ISA 130 X 130 X 15	2
	53 to 60 (bracings)	142	6.324	ISA 100 X 100 X 12	1
	E1 TO E4 (horizontal)	52	2	ISA 50 X 50 X 6	1
6	61 to 64 (main leg)	590	6	ISA 150 X 150 X 12	2
	65 to 72 (bracings)	146	6.324	ISA 130 X 130 X 8	1
	F1 to F4 (horizontal)	66	2	ISA 100 X 100 X 6	1
7	73 to 76 (main leg)	453	5	ISA 130 X 130 X15	2
	77 to 84 (bracings)	153	5.76	ISA 130 x 130 x 8	1
	G1 to G4 (horizontal)	37	2	ISA 100 X 100 X 8	1
8	85 to 88 (main leg)	578	5	ISA 130 X 130 X 12	2
	89 to 96 (bracings)	72	6.60	ISA 150 x 150 x 10	1
	H1 to H4 (horizontal)	40	3.50	ISA 50 X 50 X 5	1
9	97 to 100 (main leg)	544	5	ISA 150 X 150 X 12	2
	101 to 108 (bracings)	24	8.28	ISA 130 X 130 X 8	1
	I1 to I4 (horizontal)	31	5	ISA 55 X 55 X 5	1
10	109 to 112 (main leg)	563	5	ISA 150 X `150 X 12	2
	113 to 120 (bracings)	26	9.67	ISA 150x150x10	1
	J1 TO J4 (horizontal)	31	6.50	ISA 70x70x5	1

7. PARAMETRIC STUDY

7.1 GENERAL

This chapter deals with the study conducted on the tower by varying the base width and vertical profile of the tower. From design point of view, deflection of tower and weight of tower are important parameters to be considered. In this chapter study is carried out on two aspects by varying vertical profile of the tower and by varying the base width of the tower. The description of the alternatives is tabulated in Table 7.2.

7.2 DETAILS OF TOWERS

Details of antenna used on the tower in different alternatives are given in Table 7.1.

Table 7.1 Antenna details used on a tower

Frequency	900 MHz
Dimension(mm)	2760 x 380 x 260
Weight (kg)	20 kg

The electrical and mechanical specification of antenna along with sketch and details is shown in Fig 7.1. The details of the antenna are taken from A-INFO telecommunication antenna source book.

In all alternatives total height of tower is uniform as 56 m. In Table 7.2 alternatives 1 to 7 are of 11 panels. Here top three panels are of 3.73 m height and bottom 8 panels are of 5.6 m height respectively.

Alternatives 8 to 14 have 12 panels, with top six panels are straight of 3.73 m each and bottom six panels are tapered with 5.6 m height respectively.

The aim of this parametric study is to see the effect of varying dimension on the deflection and weight of tower respectively.

Fig 7.1 Specifications for 900 MHz flat panel antenna

Specification of 900 MHz antenna:

Modal no jxtxjz-900-90-17D

Frequency range: 870-960 MHz

Gain (dbi): 17

Polarisation: ± 45°

Impedance (ohms): 50

Lightning protection: direct ground

Maximum input power (W): 500

Radiating element material: Aluminum

Radome color: gray

Dimension (mm): 2640 x 295 x 156

Packing size(mm): 2760 x 380 x 260

Weight (kg): 20 kg

Operating temperature (°C) = -40 - +70

Reposition temperature: -55 - +80

Maximum wind speed (km/h): 210

Fig 7.2 gives 14 alternatives with different base width and vertical profile respectively. The description of all alternatives is given in table 7.2.

Table 7.2 Parameters for different alternatives of a tower

Alternatives	Height(m)	No of panels	Base width(m)	Top width	Uniform portion		Tapering portion	
					No	Total Ht	No	Total Ht
Manual	56	10	8	2	6	36	4	20
1	56	11	16	2	3	11.2	8	44.8
2	56	11	14	2	3	11.2	8	44.8
3	56	11	12	2	3	11.2	8	44.8
4	56	11	10	2	3	11.2	8	44.8
5	56	11	8	2	3	11.2	8	44.8
6	56	11	6	2	3	11.2	8	44.8
7	56	11	4	2	3	11.2	8	44.8
8	56	12	16	2	6	22.4	6	33.6
9	56	12	14	2	6	22.4	6	33.6
10	56	12	12	2	6	22.4	6	33.6
11	56	12	10	2	6	22.4	6	33.6
12	56	12	8	2	6	22.4	6	33.6
13	56	12	6	2	6	22.4	6	33.6
14	56	12	4	2	6	22.4	6	33.6

7.3 RESULTS OF VARIOUS ALTERNATIVES

Different alternatives 1 to 14 of different base widths are analyzed in staad and designed manually for the following load cases:

- Dead load of tower
- Wind load at 0° on tower

- Dead load and wind load on antenna

Final load combination adopted = Dead load + wind load

The designed sections and corresponding weight of tower are tabulated below from table 7.3 to 7.16. The tables contains weight only of sections as the weight of antenna is earlier included during analyzing the tower.

Table 7.3 Section details for alternative 1 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	1	60x60x5	4.5	1	50x50x6	4.5	219.78
2	3.73	1	70x70x6	6.3	1	55x55x8	6.4	310.99
3	3.73	1	75x75x8	8.9	1	65x65x6	5.8	329.51
4	5.74	1	90x90x10	13.4	1	75x75x10	11	867.2
5	5.74	1	100x100x12	17.7	1	80x80x10	11.8	1097.39
6	5.74	1	110x110x12	19.6	1	80x80x10	11.8	1255.2
7	5.74	1	110x110x15	24.2	1	90x90x10	13.4	1617.9
8	5.74	1	150x150x12	27.2	1	90x90x6	8.2	1136.8
9	5.74	2	100x100x12	17.7	1	90x90x10	13.4	1717.48
10	5.74	2	110x110x12	19.5	1	90x90x12	15.8	2045.64
11	5.74	2	110x110x15	24.2	1	100x100x12	17.7	2498.8
Total weight = 13096 + 1613.2 (horizontal) = 14709.2 kg (14.70 ton)								13096kg

Calculation for panel 1:

Section assigned for main legs (60 x 60 x 5) @ 4.5 kg/m

Total no of main legs = 4

Length = 3.73 m

Total weight (main legs) = 4 x 3.73 x 4.5

= 67.14 kg

Section assigned for bracings (50 x 50 x 6) @ 4.5 kg/m

Total no of bracings = 8

Length = 4.24 m

Total weight (bracings) = 8 x 4.24 x 4.5

= 152.64 kg

Total wt of main legs and bracings = 152.64 + 67.14

= 219.78 kg

Section assigned for horizontal bracing 50 x 50 x 6 @ 4.5 kg/m

Total horizontal length = 71 m

Total weight of horizontal section = 4 x 89.5 x 4.5

= 1613.2 kg

Total base shear = 95.67 kN (wind)

Maximum top deflection = 75.33 mm

Fundamental frequency = 1.826 Hz

Time period = 0.548 sec

All the above values are from STAAD.

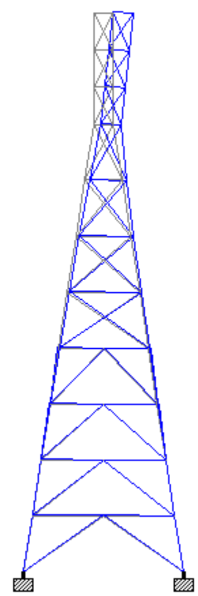


Fig 7.3 First mode shape of a tower

Table 7.4 Section details for alternative 2 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	310.99
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.51
4	5.70	4	90x90x10	13.4	8	75x75x10	11	858.12
5	5.70	4	100x100x12	17.7	8	65x65x6	5.8	731.56
6	5.70	4	110x110x12	19.6	8	75x75x10	11	1156
7	5.70	4	110x110x15	24.2	8	80x80x10	11.8	1419.2
8	5.70	4	130x130x8	15.9	8	80x80x10	11.8	1059.1
9	5.70	8	100x100x6	9.2	8	90x90x6	8.2	936.4
10	5.70	8	100x100x8	12.1	8	90x90x8	10.8	1280
11	5.70	8	100x100x12	17.7	8	90x90x10	13.4	1771.9
Total weight = 10072 + 1454.08 (horizontal) = 11526 kg (11.52 ton)								10072 kg

Table 7.5 Section details for alternative 3 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	310.99
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.51
4	5.67	4	90x90x8	10.8	8	80x80x0	11.8	832.06
5	5.67	4	90x90x10	13.4	8	75x75x10	11	905.81
6	5.67	4	100x100x10	14.9	8	80x80x10	11.8	1057.3
7	5.67	4	110x110x12	19.6	8	80x80x12	14	1397.6
8	5.67	4	110x110x15	24.2	8	75x75x6	6.8	928.5
9	5.67	8	100x100x6	9.2	8	75x75x6	6.8	818.21
10	5.67	8	100x100x12	17.7	8	75x75x8	8.9	1357.4
11	5.67	8	110x110x8	13.4	8	75x75x10	11	1332.04
Total weight = 9489 + 1295.04 (horizontal) = 10784 kg (10.78 ton)								9489 kg

Table 7.6 Section details for alternative 4 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	6.4	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	5.8	310.99
3	3.73	4	75x75x8	8.9	8	65x65x6	13.4	329.51
4	5.64	4	90x90x10	13.4	8	90x90x10	13.4	961.4
5	5.64	4	100x100x6	9.2	8	90x90x10	9.2	917.1
6	5.64	4	110x110x8	13.4	8	100x100x6	9.2	832.2
7	5.64	4	110x110x12	19.6	8	100x100x6	11	1021.5
8	5.64	8	100x100x6	9.2	8	75x75x10	10.8	995.81
9	5.64	8	100x100x8	12.1	8	90x90x8	13.4	1142.05
10	5.64	8	110x110x8	13.4	8	90x90x10	13.4	1376.4
11	5.64	8	110x110x12	19.6	8	90x90x10	6.4	1690.45
Total weight = 9797.1 + 1136(horizontal) = 10933.1 kg (10.93 ton)								9797.1kg

Table 7.7 Section details for alternative 5 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	310.99
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.51
4	5.63	4	90x90x12	15.8	8	80x80x10	11.8	930.6
5	5.63	4	100x100x12	17.7	8	80x80x12	14	1117.6
6	5.63	4	110x110x12	19.6	8	90x90x10	13.4	1172.31
7	5.63	4	110x110x15	24.2	8	90x90x12	15.8	1463.8
8	5.63	8	100x100x10	14.9	8	90x90x8	10.8	1216.18
9	5.63	8	100x100x12	17.7	8	90x90x8	10.8	1357.7
10	5.63	8	110x110x10	16.5	8	90x90x10	13.4	1459.1
11	5.63	8	110x110x12	19.6	8	90x90x10	13.4	1621.3
Total weight = 11198 + 977 (horizontal) = 12175 kg (12.17 ton)								11198 kg

Table 7.8 Section details for alternative 6 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	310.99
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.51
4	5.61	4	90x90x8	10.8	8	65x65x10	9.4	696.12
5	5.61	4	90x90x12	15.8	8	90x90x10	13.4	1022.82
6	5.61	4	100x100x10	14.9	8	90x90x12	15.8	928.76
7	5.61	4	110x110x12	19.6	8	80x80x12	14	1193.88
8	5.61	8	110x110x15	24.2	8	90x90x8	10.8	1605.9
9	5.61	8	130x130x8	15.9	8	90x90x8	10.8	1242.7
10	5.61	8	130x130x10	19.7	8	90x90x8	10.8	1421.6
11	5.61	8	130x130x12	23.4	8	90x90x12	15.8	1852.2
Total weight = 10824 + 817.9 = 11641.9 kg (11.64 ton)								10824 kg

Table 7.9 Section details for alternative 7 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	310.99
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.51
4	5.60	4	90x90x10	13.4	8	80x80x12	14	978.46
5	5.60	4	100x100x10	14.9	8	90x90x8	10.8	867.41
6	5.60	4	110x110x12	19.6	8	100x100x8	12.1	824.198
7	5.60	4	110x110x15	24.2	8	100x100x12	14.9	1306.59
8	5.60	8	130x130x8	15.9	8	80x80x12	14	1383.02
9	5.60	8	130x130x10	19.7	8	90x90x6	8.2	1289.62
10	5.60	8	130x130x12	23.4	8	90x90x10	13.4	1708.07
11	5.60	8	130x130x15	28.9	8	90x90x12	15.8	2079
Total weight = 11296 + 658.8 (horizontal) = 11954.8 kg (11.95 ton)								11296 kg

Table 7.10 Section details for alternative 8 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	311.07
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.48
4	5.74	4	90x90x8	10.8	8	70x70x10	10.2	596.54
5	5.74	4	100x100x6	9.2	8	70x70x10	10.2	559.42
6	5.74	4	100x100x10	14.9	8	75x75x5	5.7	541.4
7	5.84	4	110x110x10	16.5	8	90x90x12	15.8	1212.04
8	5.84	4	110x110x10	16.5	8	90x90x12	15.8	1389.04
9	5.84	4	130x130x12	23.4	8	80x80x10	11.8	1233.83
10	5.84	4	150x150x12	27.2	8	80x80x10	11.8	1395.2
11	5.84	8	100x100x12	17.7	8	80x80x12	14	1824.8
12	5.84	8	100x100x12	17.7	8	80x80x12	14	1927.8
Total weight = 11540 + 1340.48 (horizontal) = 12880.4 kg (12.88 ton)								11540 kg

Table 7.11 Section details for alternative 9 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	311.07
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.48
4	3.73	4	90x90x8	10.8	8	70x70x10	10.2	595.67
5	3.73	4	100x100x6	9.2	8	70x70x10	10.2	558.68
6	3.73	4	100x100x10	14.9	8	75x75x5	5.7	537.74
7	5.78	4	110x110x10	16.5	8	80x80x12	14	1101.5
8	5.78	4	110x110x10	16.5	8	100x100x12	17.7	1453.3
9	5.78	8	100x100x8	12.1	8	80x80x8	9.6	736.65
10	5.78	8	100x100x12	17.7	8	80x80x8	9.6	990.5
11	5.78	8	100x100x12	17.7	8	80x80x10	11.8	1189.8
12	5.78	8	110x110x12	19.6	8	80x80x10	11.8	1304.55
Total weight = 9328.7 + 1226.8(horizontal) = 10555.5 kg 10.55 ton								9328.7kg

Table 7.12 Section details for alternative 10 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	311.07
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.48
4	3.73	4	90x90x8	10.8	8	70x70x10	10.2	593.08
5	3.73	4	100x100x6	9.2	8	70x70x10	10.2	556.38
6	3.73	4	100x100x10	14.9	8	75x75x5	5.7	534.24
7	5.72	4	110x110x10	16.5	8	80x80x12	14	1086.4
8	5.72	4	110x110x10	16.5	8	100x100x12	17.7	1401.2
9	5.72	8	100x100x8	12.1	8	80x80x6	7.3	942.596
10	5.72	8	110x110x8	13.4	8	80x80x6	7.3	1029.4
11	5.72	8	110x110x12	19.6	8	80x80x8	9.6	1485
12	5.72	8	110x110x12	19.6	8	80x80x10	11.8	1675.6
Total weight = 10164 + 1113.28(horizontal) = 11277.28 kg (11.27 ton)								10164 kg

Table 7.13 Section details for alternative 11 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	311.07
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.48
4	3.73	4	90x90x10	13.4	8	70x70x10	10.2	650.42
5	3.73	4	100x100x6	9.2	8	70x70x10	10.2	555
6	3.73	4	100x100x10	14.9	8	75x75x5	5.7	531.86
7	5.68	4	110x110x10	16.5	8	80x80x10	11.8	963.88
8	5.68	4	110x110x15	24.2	8	90x90x12	15.8	1423.2
9	5.68	8	100x100x6	9.2	8	75x75x8	8.9	872.94
10	5.68	8	100x100x8	12.1	8	80x80x6	7.3	947.36
11	5.68	8	100x100x12	17.7	8	80x80x8	9.6	1350.2
12	5.68	8	110x110x15	24.2	8	80x80x10	11.8	1811.3
Total weight = 9966 +999.6(horizontal) = 10965.6 kg (10.96 ton)								9966 kg

Table 7.14 Section details for alternative 12 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	311.07
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.48
4	5.74	4	90x90x10	13.4	8	70x70x10	10.2	648.28
5	5.74	4	100x100x6	9.2	8	70x70x10	10.2	553.53
6	5.74	4	100x100x10	14.9	8	75x75x5	5.7	529.44
7	5.64	4	110x110x15	24.2	8	100x100x12	17.7	1416.7
8	5.64	4	110x110x15	24.2	8	100x100x12	17.7	1483.29
9	5.64	8	100x100x12	17.7	8	80x80x6	7.3	758.4
10	5.64	8	110x110x8	13.4	8	80x80x8	9.6	791.5
11	5.64	8	110x110x8	13.4	8	80x80x10	11.8	927.2
12	5.64	8	110x110x10	16.5	8	80x80x10	11.8	1023.56
Total weight = 8992+ 886.08(horizontal) = 9878.08 kg (9.87 ton)								8992 kg

Table 7.15 Section details for alternative 13 tower configuration

Panel	Height(m)	Section details						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	311.07
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.48
4	3.73	4	90x90x12	15.8	8	70x70x10	10.2	701.16
5	3.73	4	100x100x12	17.7	8	70x70x10	10.2	743.87
6	3.73	4	130x130x12	23.4	8	75x75x5	5.7	719.37
7	5.62	8	100x100x8	12.2	8	110x110x8	13.4	1200.286
8	5.62	8	100x100x12	17.7	8	110x110x8	13.4	1477.58
9	5.62	8	110x110x8	13.4	8	80x80x10	11.8	1165.02
10	5.62	8	110x110x10	16.5	8	90x90x12	15.8	1510.312
11	5.62	8	110x110x12	19.6	8	90x90x12	15.8	1666.14
12	5.62	8	130x130x10	28.9	8	80x80x10	11.8	1829.8
Total weight =11873 + 772.48(horizontal) = 12645 kg = 12.64 (ton)								11873 kg

Table 7.16 Section details for alternative 14 tower configuration

Panel	Height(m)	Section details`						Wt (legs + braces) kg
		No	Legs	W(kg/m)	No	Braces	W(kg/m)	
1	3.73	4	60x60x5	4.5	8	50x50x6	4.5	219.78
2	3.73	4	70x70x6	6.3	8	55x55x8	6.4	311.07
3	3.73	4	75x75x8	8.9	8	65x65x6	5.8	329.48
4	3.73	4	90x90x12	15.8	8	70x70x10	10.2	699.9
5	3.73	4	100x100x12	17.7	8	70x70x10	10.2	742.46
6	3.73	4	130x130x12	23.4	8	75x75x5	5.7	717.5
7	5.60	8	100x100x8	12.1	8	110x110x8	13.4	1186.35
8	5.60	8	100x100x12	17.7	8	110x110x12	19.6	1754.14
9	5.60	8	110x110x2	19.6	8	90x90x12	15.8	1611.2
10	5.60	8	130x130x10	19.7	8	100x100x12	17.7	1706.44
11	5.60	8	130x130x15	28.9	8	100x100x12	17.7	2128.74
12	5.60	8	150x150x15	33.6	8	90x90x12	15.8	2213.12
Total weight = 13620 + 658.88(horizontal) = 14278.88 kg (14.27 ton)								13620 kg

Table 7.17 Preliminary trials for optimizing configuration

ALTERNATIVES	BASE WIDTH (m)	TOP WIDTH (m)	TAPERING PORTION (m)	UNIFORM PORTION (m)	TOTAL WEIGHT (Ton)	DEFLECTION (mm)
Manual	8	2	20	36	32.04	332
1	16	2	44.8	11.2	14.7	75.33
2	14	2	44.8	11.2	11.52	117
3	12	2	44.8	11.2	10.78	128.34
4	10	2	44.8	11.2	10.93	247.8
5	8	2	44.8	11.2	12.17	365
6	6	2	44.8	11.2	11.64	559.8
7	4	2	44.8	11.2	11.95	758.8
8	16	2	33.6	22.4	12.88	407
9	14	2	33.6	22.4	10.55	477
10	12	2	33.6	22.4	11.27	415
11	10	2	33.6	22.4	10.96	474
12	8	2	33.6	22.4	9.87	573
13	6	2	33.6	22.4	12.64	761
14	4	2	33.6	22.4	14.27	990

There are two governing parameters which require consideration while optimizing the configuration of tower for a prescribed height.

- Total weight of tower

It is obvious that, the total weight of the tower should be minimum from the economic point of view.

- Maximum deflection

In microwave towers, the deflection may become important design criteria because; excessive antenna deflection caused by high wind load may result in signal loss or distortion.

For the towers of 56 m height studied in this work, a maximum permissible deflection of 300 mm as per BS: 8100 is considered adequate. The deflections of the alternatives are taken from STAAD and they are due to the wind blowing parallel to the tower.

Thus, the problem of optimization reduces to minimization of both these parameters. It is seldom possible to arrive at a configuration for which both total height as well as the maximum deflection will be minimum. Here an attempt has been made to arrive at an configuration for which the deflection will be minimum and at the same time total weight will not be much in excess of the minimum value. Important parameters of all the cases studied in the preliminary trials are tabulate in table 7.18 respectively.

It is been summaries that as the tapering portion increase weight also reduces and also deflection. A typical 4 no's of flat panel antennas are mounted on the tower each contains 20 kg weight. An alternative 1 to 4 which has base width 16 m to 10 m respectively are in limited value of deflection and in comparison weight is also less. The manual problem with base width 8 m and tapering portion only 20 m and uniform portion of 36 m is not economical and also not in the permitted deflection value. The most economical is alternative 3 which have less weight and permitted deflection.

7.4 SUMMARY

This chapter deals with the economic aspect of the tower by considering parameters of weight of the tower and deflection respectively. An attempt is made to study the effect of base width variation on parameters of the tower.

8. DISCUSSION OF RESULTS

8.1 GENERAL

Static analysis and Dynamic wind analysis of tower is carried out using the methodology as explained in Chapter 4. Chapter 5 and 6 give the results of the tower analyzed manually and using of software respectively. Chapter 7 consists of the results of alternatives for tower configuration. This chapter contains information about comparative assessment of results of analysis and design of towers covered in various chapters as discussed above respectively.

8.2 MANUAL ANALYSIS AND DESIGN OF TOWERS

The forces at the joint are computed assuming:

- Member weights are equally distributed to the two joints connecting the member
- No load is applied at the middle K-brace joint but allocated to column joint
- Dead and wind loads are increased by 15% at each joint to account for gusset plates and bolts
- The loads and wind loads of members are equally distributed to the connecting joints

The tower is analyzed for three basic loads and they are:

- Self weight
- Superimposed load from antenna
- Two wind loads,
  - (c) Wind parallel to face of the tower
  - (d) Wind diagonal to the tower

Static analysis by force coefficient method and dynamic analysis by gust factor method has been carried out respectively. Salient observations and discussion of the results of analysis are presented subsequently.

- ➢ When the wind load acting diagonally to plane of the tower, the bending moment is resisted by the pair of legs and forces in other two legs lying along the diagonal about which the moment is considered as zero.
- ➢ The axial force in leg is higher in case when the wind is acting parallel to diagonal compared to when the wind is acting parallel to the plane

- The axial force in the bracings are observed more when the wind blows parallel to the tower compared to when the wind is blowing diagonally to the tower
- Dynamic force due to the wind are found more or less same as static force due to effect of wind
- It has been observed that wind analysis governs the tower design as the force due to seismic analysis is lesser compared to wind force as given in table 8.1

- **Comparison of lateral forces(kN) on tower**

  The final lateral forces acting on the tower as calculated in earlier chapter are compared in Table 8.1.

Table 8.1 Comparison of lateral forces (kN) acting on the panels

Panel from top	Static wind force(kN)	Dynamic wind force (kN)	Seismic force(kN)
1	19.14	19.30	10.25
2	18.55	18.75	9.83
3	17.89	18.66	11.57
4	17.65	17.52	14.19
5	17.81	18.47	12.13
6	21.49	20.78	10.64
7	19.91	20.44	4.40
8	19.31	17.85	1.62
9	17.21	18.51	0.90
10	17.87	19.88	0.10

- **Graphical representation of lateral force (kN)**

Fig 8.1 shows the graphical representation of the comparison of static and dynamic analysis, which shows the force comparison of lateral force for the panels. It can be observed that the dynamic force is more or less same as the static force. In top three panels of tower, dynamic force is higher than the static force for around 15m from top.

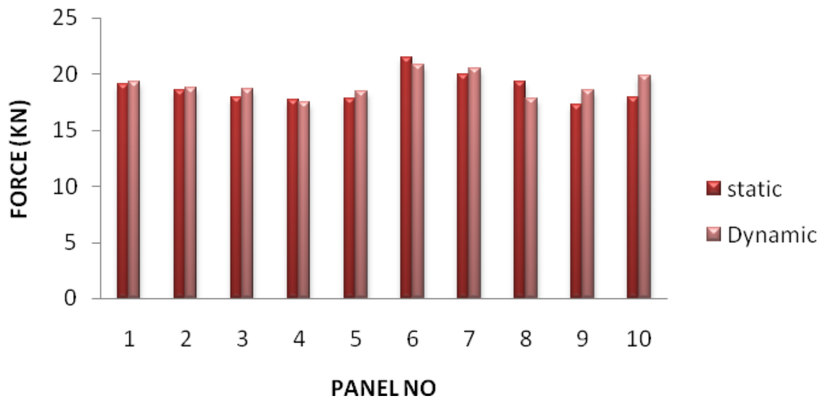


Fig. 8.1 Comparison of static and dynamic lateral force (kN)

- **Comparison of wind lateral force (kN) and Seismic force (kN)**

Fig 8.2 shows the wind and seismic force comparison in each panel which indicates that wind analysis gives the higher amount of force acting on the panels.

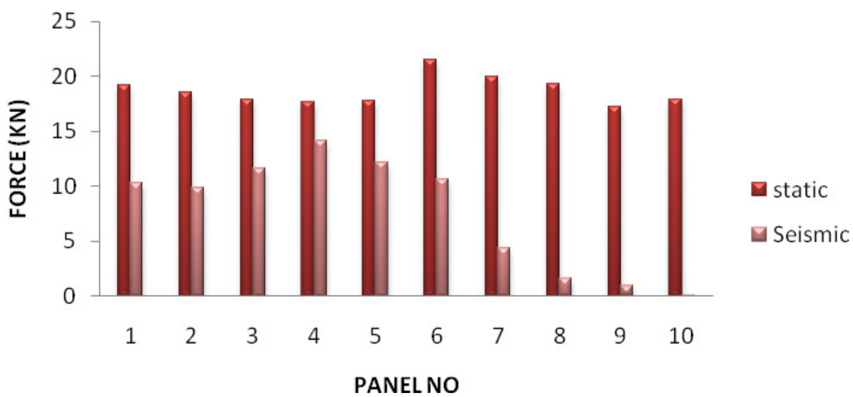


Fig 8.2 Comparison of lateral static wind and seismic force

Fig 8.3 shows the comparison of the wind and seismic force acting on the tower. Seismic force increases up to the forth panel, subsequently reduces and becomes very less at the bottom. Compare to wind static and dynamic forces, seismic forces are very less. Hence it can be observed that the present case of tower design is governed by the wind forces.

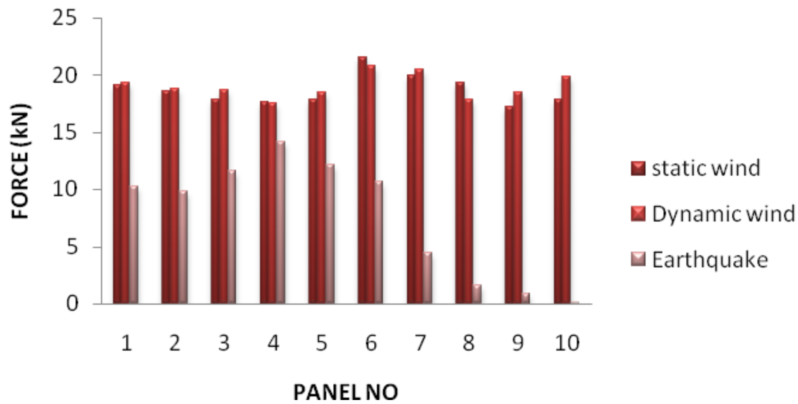


Fig 8.3 Comparison of lateral forces acting on a tower

8.3 ANALYSIS AND DESIGN OF TOWERS USING STAAD

The same tower of 56 m height analyzed manually is also by the software and results are compared. Process of analysis carried out is discussed in chapter 6.The deflection profile of the tower obtained for the wind blowing parallel and diagonally is compared as shown in Fig 8.4. The maximum deflection of the tower is 332 mm and in both the cases deflection tends to be same.

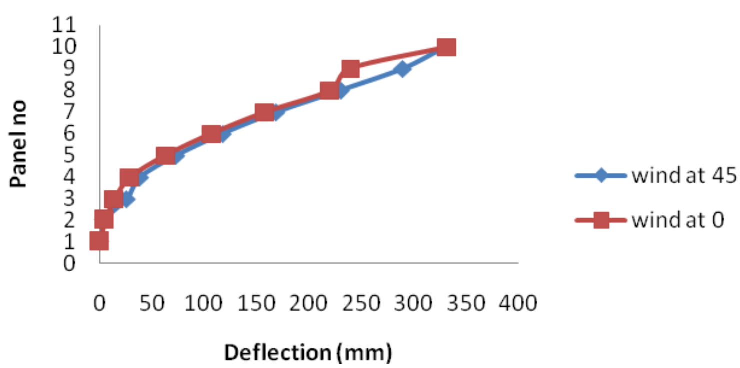


Fig 8.4 Deflection comparison due to wind

The comparison of the axial force in the main legs of members when the wind is blowing parallel or diagonally to the tower is shown in table 8.2 and for bracing members in table 8.3 respectively. The observations on some salient features of results are as follows:

- The maximum axial force in the panels is observed at the location where the trapezoidal portion of the tower converts into the straight portion
- The manual method gives the higher axial forces in the member in comparison of software results

- The maximum force in the bracings are observed at location when the trapezoidal portion converts in to the straight portion
- Wind blowing parallel to the tower is observed as critical condition for bracing design

Table 8.2 Comparison of axial forces (kN) in main legs of a tower

Panel no	Wind blowing at 0°		Wind blowing at 45°	
	Manual	From Staad	Manual	From Staad
1	15.78	10.65	21.99	14
2	57.9	35.41	97.74	55
3	130.2	72.09	197.17	131
4	279.96	117	407.7	248
5	362.4	167	634.71	392
6	535.9	262	933.5	590
7	523	320	844.17	453
8	427.73	308.9	741.28	578
9	402.99	311.0	696.72	544
10	395.29	307.89	682.17	563

Table 8.3 Comparison of axial forces(kN) in bracings of a tower

Panel no	Wind blowing at 0°		Wind blowing at 45°	
	Manual	From Staad	Manual	From Staad
1	16.96	14.83	9.93	18
2	31.25	25.11	21.5	41
3	46.45	33.92	34.31	66
4	62.25	47.10	48.24	94

5	78.70	58.74	68.43	142
6	95	51.17	76	146
7	66.54	65.43	53.80	153
8	68.59	156.11	55.88	72
9	124.28	102.13	101.86	24
10	136.22	17.67	112.19	26

8.4 PARAMETRIC STUDY

By studying the values of deflection and weight for different alternatives from Table 7.17, the following points are observed for a same tower of 56 m height:

- The tower behaves essentially as a cantilever structure fixed at the base (ground level). The deflection goes on reducing as the height of tapering portion increases. At the same time there is reduction in deflection as the base width increases

- In first 7 cases listed above the proportion of tapering portion to uniform portion is 4: 1 therefore it is observed that there is a decrease in deflection with increase in base width. The stiffness of structure is increased because of its slanting legs which slant more with increase in base width making the tower more sensitive for vertical deflections compared to horizontal deflections.

- As can be observed from Fig 8.5, Alternatives 1 to 7 in which tapering portion is more compared to the alternatives 8 to 14 for different base width, the weight is also less. As the base width increase from 4 m, weight decreases for about 14 m base width but again it increases at the 16 m width.

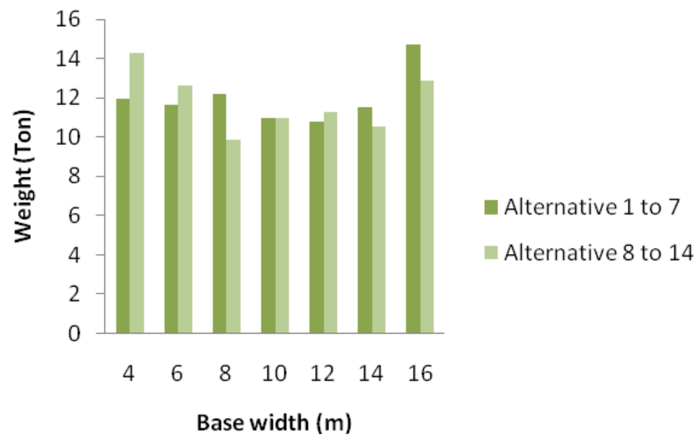


Fig 8.5 Weight comparison of tower for different base Width

- The deflection of tower increases with the increase in uniform portion. As given in Fig 8.6, an alternative 1 to 7 has lesser deflection compared to alternatives 8 to 14 as the height of uniform portion is more.

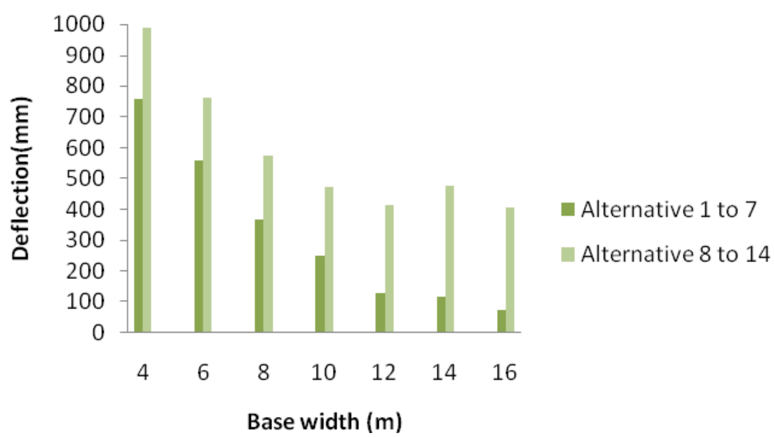


Fig 8.6 Deflection comparison of tower for different base Width

- Alternatives 1 to 4 are the most suited configurations for the 56 m height tower. It has been summarized that from the economical point of view, base width of 10 m to 14 m is most suited by keeping the ratio of uniform portion to trapezoidal portion of 4:1. The most economical is alternative 3 which has less weight and permissible deflection.

9. CONCLUSIONS AND SCOPE FOR FUTURE WORK

9.1 SUMMARY

Due to the expanding communication system, a large number of lattice towers to support cellular antennas are extensively used. The main features in designing the tower is the height of tower as due to the antenna mounted on top of tower for receiving and transmitting the signals. There are two main parameters, deflection of the tower and the weight of tower which are to be optimized for the economical condition. The various configurations are to be adopted and analyzed for different comparison of forces.

The need to design a lattice tower for resonant dynamic response due to wind load arises when the natural vibration frequency of the structure is low enough to be excited by the turbulence in the natural wind. However, in the design of lattice towers, there is an apparent need for broadening the basis of design to include the load effects, such as top deflection, bending moment and shear force. Due to relative small weight of structure of these towers and having wide-area components at top of them like dishes, the main and considerable load for the design is generally wind load. But with the increase in height and also taking into the account slenderness of the structure, the seismic loads are also considerable in such structures.

9.2 CONCLUDING REMARKS

A methodology is described in detail for analysis and design of antenna towers. The following conclusions are drawn from the numerical examples attempted:

- Wind is the most predominant condition for designing of the tower, but with increase in the number of antennas and weight of tower, the effect of earthquake load is increased
- A comparison between the results obtained from the linear static analysis from the dead load and wind load and output results obtained from dynamic analysis reveal that the values obtained from wind load is more compared to earthquake load, the earthquake loads in the panel are much less than wind forces

- Wind blowing parallel to the tower is observed as critical condition for bracing design and wind blowing parallel to the diagonal condition is critical condition for design of main legs
- The maximum force in the bracings and main legs are observed at location when the trapezoidal portion converts in to the straight portion
- As the tapering portion increases, the weight of tower and total deflection at top also reduce, which further lead to the economical design of the tower.
- The manual method gives the higher axial forces in the members compared of software results.

9.3 FUTURE SCOPE OF WORK

Static analysis and dynamic analysis is conducted here for a typical tower with the provision of IS 875: part-3. The typical tower analysis can also be carried out with IS: 875 Draft code. Parametric study can be carried out by varying the width of tower and evaluating economic section. Computer program can be prepared for the optimization of the tower geometry. Seismic analysis of tower can be repeated for a tower with increasing number of antenna fixed to the tower and results can be compared.

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