PARAMETRIC OPTIMIZATION OF MOBILE TOWER BY VARYING LOADING CONDITIONS WITH DIFFERENT MATERIALS

Vinay Prabhakar[1], **Pradyumna Dashora** [2]

[1]*M.Tech. Student, Department of Civil Engineering, Pacific Academy Of Higher Education And Research University, Udaipur, Rajasthan, India*
[2],*Assistant Professor, Department of Civil Engineering, Pacific Academy Of Higher Education And Research University, Udaipur, Rajasthan, India*

***

***Abstract*:-** *Communication tower are used for transferring the signals. They are used by various mobile companies as well as for transferring the electricity or for power transmission. Recently various studies have been done on design of Telecommunication Towers, but in most studies the researchers have considered the effect of wind and seismic forces on the four legged self-supporting towers. However, no researches have provided the measure to overcome the effects of varying loading condition with different material. In this study, will be focusing on study of wind force on tower made of high steel tower with Lattice Steel Masts. In this study the mobile tower was developed using auto cad software. This 3D cad model developed was further imported into the ANSYS software where the model was subjected to the wind loads with the aid of different propositions used for development of the structure and further the structure was characterized using the software into smaller components. 3D solid elements were used for the development of the meshed model. This model was also subjected to different forms of material models in order to obtain an optimized structure. Material models of linear material properties were imported into the model and the contacts were defined as linear non separable structures so that the mobile structure behaves as a single structure and the load bearing capacity of the structure can be evaluated. The mobile tower is developed using different complex cross-sectional section cross bars/ties in order to maintain the connection between critical load carrying members and further they were connected in such a way the load transferred properly and each cross-member remains under safe condition with respect to maximum loads. In this case, the mobile tower is fixed at the bottom of the base with all degree of freedom as zero (means constant). After that the wind loads (taken from the reference paper) is applied on the structure to obtain the performance of the structure with respect to the loads. Further different forms of loads are applied on the structures and the structure is also varied with respect to enhancement in the thickness of the slabs used for developing the mobile structure in order to optimize the structure with respect to varying loading conditions. Taguchi method and ANOVA technique were used to optimize the results obtained in FEM analysis. In this study it was obtained that if the height increases then X bracing proves uneconomical and K bracing proves economical. Similarly, Magnitude of displacement in tower of smaller size is lower with same thickness on action of same intensity blast loads. Moreover, Magnitude of stresses in tower of smaller size is lower with same thickness on action of same intensity blast loads.*

Keywords: Mobile tower, ANSYS, Optimization, Varying wind/axial loads

1. INTRODUCTION

India already has over 750,000 cell phone towers and, according to estimates, will have more than quadruple that number by 2025, making it one of the fastest expanding telecommunication markets in the world. Telecommunication towers are tall structures that are typically used to support parabolic antennas that are utilized for microwave communication. They're also used to deliver radio and television signals to far-flung locations, and they're set up at a specified height. Self-supporting structures are classified as three-legged and four-legged space trussed structures. Self-supporting towers usually have a square or triangular design and are supported on the ground or on buildings. When compared to square towers, triangular towers generate smaller wind loads. [1].

Nonetheless, they are utilized exclusively for more modest levels of pinnacle because of hardships in joint itemizing and manufacture utilizing point segments. Nowadays there is a phenomenal ascent in the number of cross section towers because of a steadily expanding request in correspondence. Cross section towers are 3D space approaches that for configuration are customarily broke down as 2D supports. For security and economy, these plans should be all the more thoroughly investigated thinking about them as 3D edges. Media transmission pinnacles or cross section towers are arranged into three classifications that are Guyed poles, monopole and self-supporting pinnacles [2].

They function primarily as cantilever brackets and are designed to transport wind load as the primary source of natural burdens. Tremor-induced loads are generally ignored in plans, with the exception of basic designs used in high seismic-risk areas. The main cause of telecom tower failures throughout the world is concentrated energy winds, but there are still aspects to be targeted (HIW). The main issue is that estimating wind loads is challenging because it is based on a probabilistic technique. Several studies of telecommunication towers have been conducted, taking into account both the wind and seismic effects. The wind was used as the major force in the investigation, and the joint displacements were calculated using the Gust factor method.

Wind was taken as the main force for analysis and using the method of wind coefficient, joint displacement, chi force and peak stress were compared to find out the effect of difference in modeling strategy on force. design impact on truss. communication tower [3]. These towers require more steel but cover less base area, making them suitable for many situations. The availability of land, which meets the ideal conditions for transplanting in an urban environment, is very limited and there is no other option but to adopt roof towers with marginal adjustment in position but no right in height. This fact is mainly due to the fact that telecommunications towers built on the ground have performed well in previous earthquakes. However, roof-mounted towers respond to horizontal movement differently than those built on firm ground..

Communication towers are usually designed as a 3D truss, which is not an actual representation of the structure. In traditional stress calculations based on the analysis of linear elastic ideal trusses, the members are assumed to be concentrically loaded and connected by dowels.

Current requirements for large structures and road structures, such as bridges, are increasingly stringent, requiring larger elements that make them more expensive [1]. On the other hand, the current trend of large-scale project construction has made steel structure a major construction alternative [2]. However, reducing the amount of material in the manufacture of steel elements to make them lighter also makes them slimmer. This condition can affect the stability of the structure by possibly triggering local or global warping failures, in practice this is more severe when the structure is in a fire condition. [5]. Developments in the field of structural optimization began around the same time as FEM.

In the last decade, the telecommunication industry has radically changed from conventional operations and systems to data-driven applications, while its further evolution is a necessity in order to meet the new technologies of emerging smart home and smart city concepts. The telecommunications business has evolved dramatically in the previous decade, moving from traditional operations and systems to data-driven applications, and additional transformation is required to match the new technologies of upcoming smart home and smart city concepts. Given the growing demand for services in cities as a result of population growth, the necessity for more support structures to be built inside the urban framework is obvious. Existing telecommunication towers, on the other hand, may not have been intended to cope with such higher gravitational and lateral forces, resulting in a greater risk of overloading and damage, which could even lead to the collapse of such slender buildings due to the new loading framework.Furthermore, because these flexible constructions are subjected to fluctuating loads induced by dynamic wind effects on a constant schedule, fatigue damage is highly likely. As a consequence, exhaustion telecommunications masts and towers would need to be replaced [53].

A) APPLICATION COMMUNICATION TOWER

Communication tower are used for transferring the signals. They are used by various mobile companies as well as for transferring the electricity or for power transmission.

B) OBJECTIVE

I. To model the structure of mobile tower.

II. To analyze this structure by varying the different loading conditions.

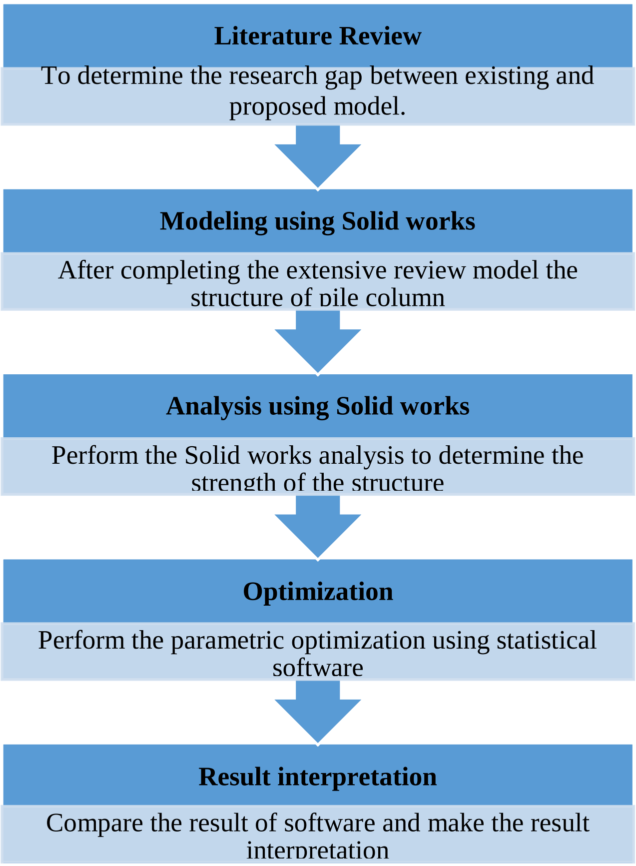
III. To optimize the loading parameter for enhancement of structure with different materials.

C) PROBLEM STATEMENT

In most of the recent studies as per the literature it was observed that more emphasis was given on the design of different types of telecommunication towers but the effects of winds and seismic loads on the structure with different forms of varieties as well as the optimization of the results in order to predict the best behaviour of the mobile tower structure. Furthermore, no researches have provided the measure to overcome the effects of varying loading condition with different material. In this study, will be focusing on study of wind force on tower made of high steel tower with Lattice Steel Masts

D) Research Methodology

Fig 1.1- Flow Chart



E) ORGANIZATION OF THE THESIS

This thesis is divided into seven chapters. An overview of these chapters is presented below.

Chapter 1

This is an introductory chapter which elucidates problem statement, objective, research methodology and organization of the thesis.

Chapter 2

This chapter deals with the literature survey for the presented work and examined a comprehensive background of other related research works contributed by many research papers and other referred journals related to the present work with recent research work going on worldwide and has assured the consistency of the work performed.

Chapter 3

This chapter deals with modelling of pile design analysis using Solidworks. This chapter contain design parameters for blasting analysis.

Chapter 4

This chapter deals with blast analysis using ansys or LS Dyna software. In this chapter the structure made by considering the different dimension and material is analysed by varying different operating parameters.

Chapter 5

This chapter deals with optimization of different operating and dimensional parameters to get the optimum result. This chapter deals with the detail analysis of design of experiments using Taguchi method.

2. LITERATURE REVIEW

As a key part of high voltage power transmission, the long span power transmission tower-line system is considered as the lifeline project in power engineering. Due to its large height-to-width ratio, latticed steel power transmission tower (except for 4-leg classical lattice towers of normal height) exhibits relatively low flexible bending stiffness. As common natural phenomena, ice and snow are threatening the operation of long span power transmission tower-line system, due to their accretion feature. The damages or failures occurring at the power transmission tower-line system due to ice or snow loads mainly involve three reasons [2]: (1) Overloading which is due to the increase of weight and windward area; (2) Unevenly ice-accretion or ice-shedding which would result in the damage of structural member; (3) Transmission line galloping which would result in the inclination or collapse of the whole tower. Based on the climatic conditions, additional considerationshould be taken into account in the design of power transmission tower-line system, especially for the progressive collapse analysis of power transmission tower-line system.

Jithesh Rajasekharan et al. (2014) designed the lattice tower for three heights of 30m, 40m and 50m with different types of bracings to study the effect of wind load on 4- legged lattice tower for wind zone V and VI using gust factor method. They also studied the seismic effect on the tower structures by carrying out the modal analysis and response spectrum analysis for zone II to zone V and concluded that the member stresses in bottom leg of XX braced tower are higher as compared to other tower models.

Siddesha. H (2010) presented the analysis of microwave antenna tower with Static and Gust factor method and compared the towers with angle and square hollow sections. The displacement at the top of the tower was considered as the main parameter. The towers with different configuration have also been analysed by removing one member present in the regular tower in lower panels. Square sections were found to be most effective for legs as compared to the angle sections.

A. Jesumi.et al. (2013)modelled five steel lattice towers with different bracing configurations such as the X-B, single diagonal, X-X, K and Y bracings for a given range of height. The heights of the towers are 40m and 50m with a base width of 2m and 5m respectively. The tower of height 40m has 13 panels and the tower of height 50m has 16 panels. 70-72% of the height is provided for the tapered part and 28-30% of the height is provided for the straight part of the tower. The towers have been analyzed for wind loads with STAAD Pro. V8i, to compare the maximum joint displacement of each tower. Optimized design has been carried out to estimate and to compare the weight of each tower. From the results obtained, Y bracing has been found to be the most economical bracing system up to a height of 50m.

Konno et al. (1973) presented the effects of earthquake loads on lattice telecommunication towers atop buildings and obtained the mode shapes, the natural frequencies, and the damping properties of such structures. Simulation of a stick model of the tower using lumped masses and a viscous damping ratio of 1% was used in their studies and observed that in some of the members, the forces due to earthquake were greater than those due to wind.

Bhatt et al. (2013) analysed two lattice towers of heights 18m and 40m by modelling them by three different structural idealizations namely, as 3D frame, 3D truss and as a hybrid of the two.It was found that the truss model gives representative values of axial forces /stresses in all members. However, the truss models underestimate the bending stresses because only the effect of out of plane bending has been considered in it. Either of the frame model or the hybrid model may be used for estimates of combined stresses for checking the design. In this study, it was found that the combined stresses necessitated the redesign of base members.

J.G.S. da Silva et al [3] carried out the Structural Assessment of current steel design models for guyed steel telecommunication towers for radio antenna by the finite element method in ANSYS using three different structural idealizations of the model. They recommended the adoption of the model with bracings made of truss elements. Sullins Eric James [11] on the

basis of the study of freestanding Kansas City tower (used as radio communication tower) analyzed using the ERITower software for wind and ice effects concluded that diagonal bracing tends to control the ability of the tower to withstand wind and ice loadings.

W.Q. Jiang [9] showed that accurate prediction of the structural capacity of lattice towers under different failure modes is very important for accurate assessment of the reliability of transmission lines and power grids, and for design of efficient failure containment measures.

The full-scale transmission lattice tower tests showed that the analysis results grossly underestimate the measured deflections, which might be as large as three times the theoretical linear elastic deflections.

Al-Hussein et al. [9] conducted a study that explored an approach to complement simulation with visualization. It reduces difficulties stemming from lack of proficiency in generating detailed simulations, a skill that engineers are not typically trained for. They claimed that the integration of a 3D model and lifting event simulations is helpful in validating lifting plans. However, users of this system have to rely on visuals to identify the conflicts associated with each simulated event.

All design domains created are based on the geometry of a steel lattice self-supported tower located in Greece designed to resist wind as well as seismic actions (Fig. 2.3). The 19 m height four-legged loom features square on plan configuration, partially-tapered vertical profile and triangular shaped tip to allow antenna fitting. This structure represents a conventional geometry for lattice telecommunication towers and will be referred in this work to as original tower UA.Three 2D distinct geometries were formed, all based on the perimeterlines of the tower UA; (i) a fully-tapered (FT), (ii) a fully-straight (FS), and (iii) a partially-tapered (PT) (Fig.2.3). The analysis of the domain to produce the most consistent and realistic outcomes shall be considered in the creation of the novel skeleton.

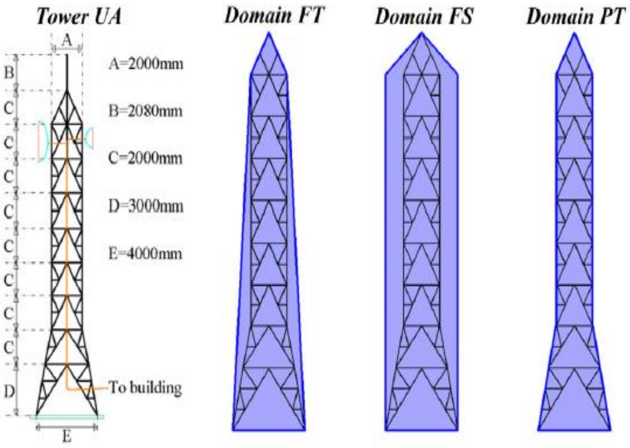


Fig.2.1.Developing the 2D domains based on the geometry UA.

Moreover, as for the boundary conditions, it is assumed that the fullbase width of the 2D designed domains is fixed during the optimisationstudies. This is to identify the exact location of stress paths generatedwithin the domain, and hence estimate the number and location ofcolumns required in a single tower face. Providing fixities only at thebase corners would force the analysis to distribute the material from thetop to the bottom two fixities of the domain, and therefore, potentiallylimit the number of columns. On the other hand, to reduce the computationaltime of the 3D domain OA and provide a more coherentresult, fixities are provided only at the base corners.Furthermore, the initial analyses are performed considering pointloads applied only at the top and at the locations of secondary horizontalbracing members as currently used in the topology of UA model (Fig. 2.2). It is noticed that by altering the loading scenario to a highermagnitude of distributed load, it has improved the outcome of theanalyses by providing coherent and ideal topology layouts.

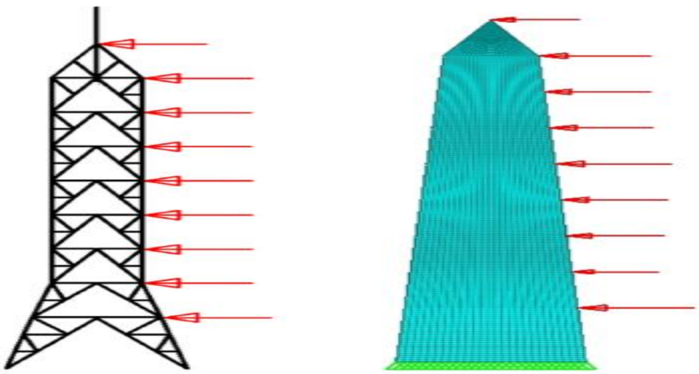


Fig. 2.1 Initial loading scenario used for the OA.

Based on Fig. 3, the reduction of the element thickness and volume fraction constraint improved the clarity of the solution with fewer core elements replacing smaller and thinner dense elements (i.e., struts and ties). Analysis C clearly indicated the stress paths required by a single tower face to withstand the loading scenario presented in Fig. 7. In addition, it is confirmed that the final output is dependent on the specified loading scenario and support conditions.

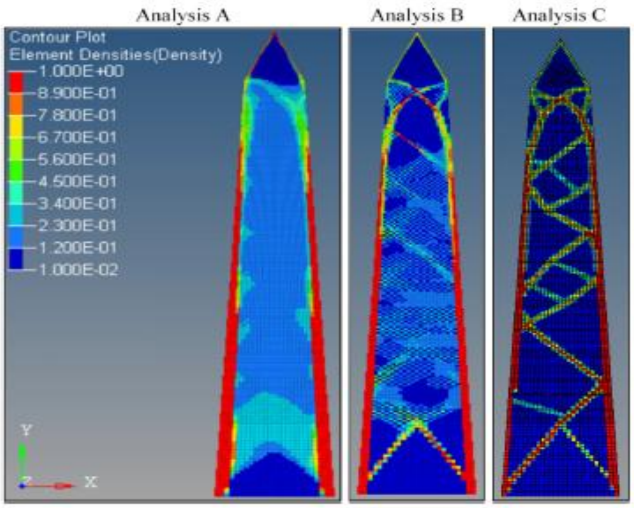
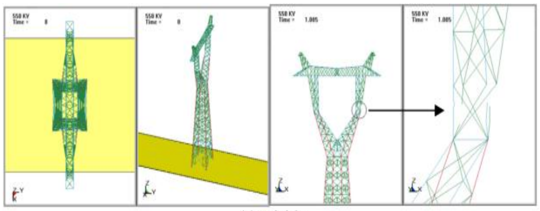
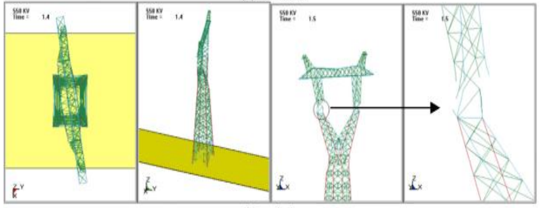


Fig. 2.3 progressively improving OA output by altering parameters

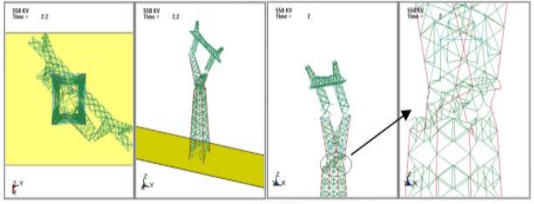
Fig. 2.4 shows the progressive collapse process of the tower. At 0.06 s, the unbalanced torsion due to line fracture leads to the fracture of a few structural members which connect the middle V-part and upper part of the tower, as shown in Fig. 2.4(a). Then, the middle V-part starts to twist. The structural members which connect the middle V-part and upper part of the tower on the other side also fracture at 1.5 s. Thanks to the fixed supports of tower bottom, the bottom part of the tower remains steady under torsional force. At 2.0 s, the connecting components between middle V-part and bottom part of the tower all fracturewhich leads to the whole collapse of the tower. As shown in Fig. 2.4(d), the upper part of the tower breaks into pieces from the middle V-part firstly, followed by the breakage between the middle V-part and bottom part of the tower. As shown in Fig. 17(e), the collapse of the tower in the simulation shows a similar pattern with the collapse of the real wine-cup shape tower. The bottom part of the tower normally would not collapse, only if the failure happens at the tower feet connections.



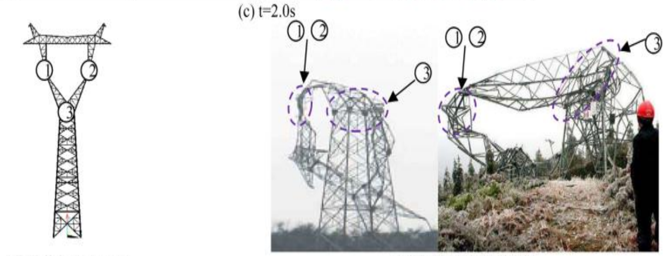
(a) t=0.06s



(b) t=1.5s



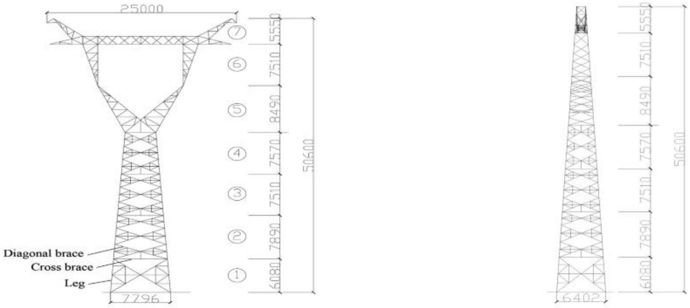
(c) t=2.0s



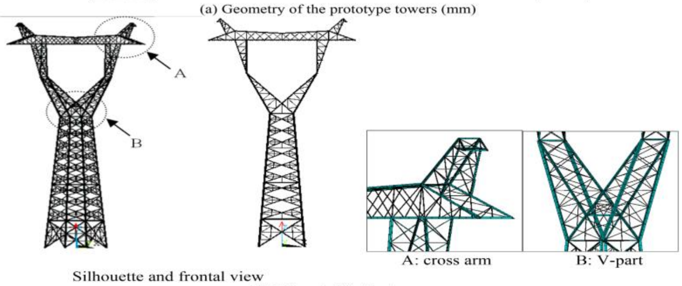
(d) Failure sequence (e) Real tower collapse

Fig.2.4 Progressive collapse simulation.

The finite element (FE) model of the prototype tower is developedby using ANSYS as shown in Fig. 5(a). BEAM188 3D beam element withuser-defined profile is used to simulate leg members and cross-bracemembers. This element works best with the default choice in solutioncontrol. 3D uniaxial tension-compression truss element LINK8 withoutbending capacity is used to simulate diagonal-brace members. Materialnonlinearity and large deflection capabilities are available for bothBEAM188 and LINK8 elements. Both ends of the braces are assumed tobe pinned. The stress-strain relationship of steel is shown in Fig. 5(b).According to the investigation of power transmission tower collapseaccidents, the failure normally would not occur at the leg feet connection,except that the bolts used to fix leg feet are sabotaged.Therefore, the bottom end of the model is fixed. Geometric nonlinearityis considered in the finite element model.

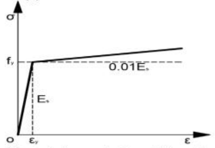


(a) Geometry of the prototype towers (mm)



Silhouette and frontal view

(b) FE model for the tower



(c) steel stress-strain relationship

Fig.2.5 Prototype towers and the finite element model.

The existing tower, model UA, has a partially tapered form, with thecolumn member sections changing at three different heights (Fig. 6).Symmetries are maintained on both the axes in the plan view, making it simple for the designer in estimating and assigning the possible loads acting on the tower structure. Angle sections are used in the entire model. The profile is considerably simple, where the first bay is 3 m in height, followed by 8 bays of 2 m height each. This is owing to the bracings that are equally spaced. There are a total of 428 members altogether comprising of columns and bracings at varying levels. The column sections change at three different heights, while the bracing member sections remain the same in the entire model.

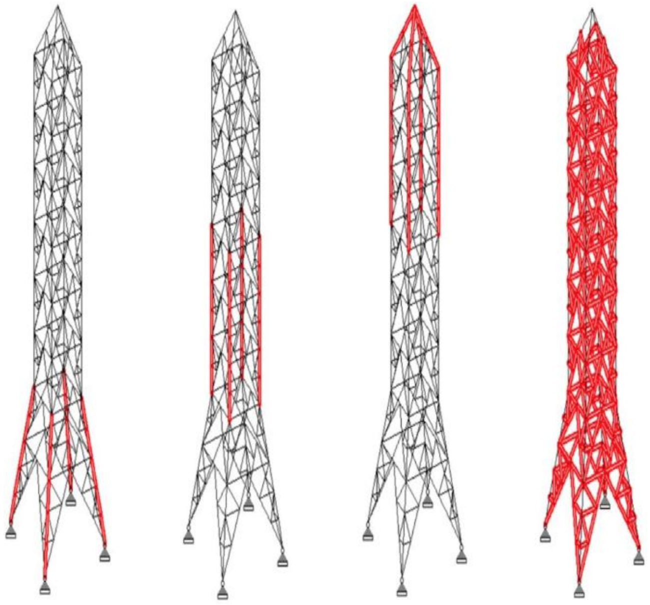


Fig 2.6 Sections groups considered in Model UA.

The real tower is positioned at an elevated base of 20 mm above the existing ground level. As shown in Fig. 7 there are two antennas of different sizes, having radius 1.2 m and 0.6 m, facing different directions on the tower. Looking at the plan and section, the positions of the antennas can be known. Their positioning is fixed and cannot be changed as they are directed towards the satellite that transmits signals to these receivers. Orientation is critical, hence the antennas shall be fixed at the exact height and direction as per the recommendations from the service provider. The live load is considered as 1.5 kN. By engineering judgement, this is the most onerous condition, and hence will display the worst condition for which the tower will be designed.

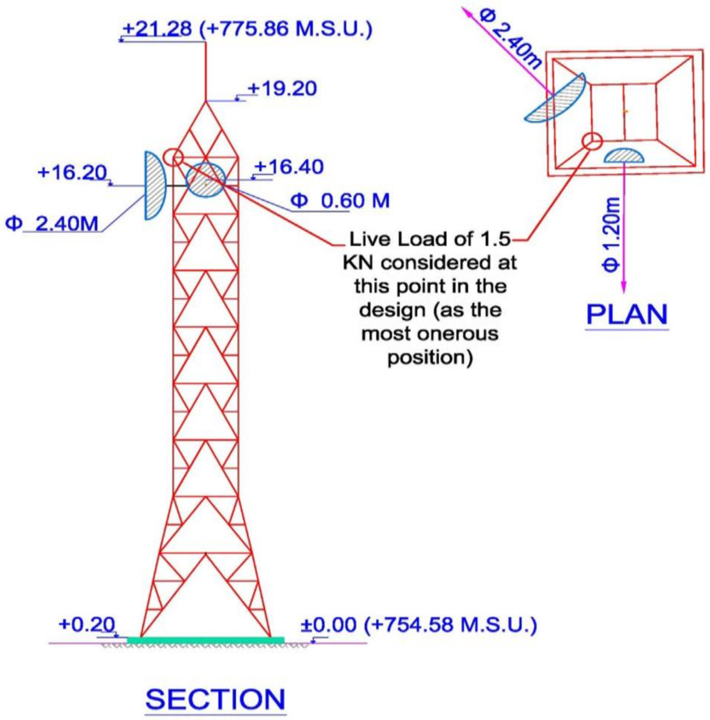
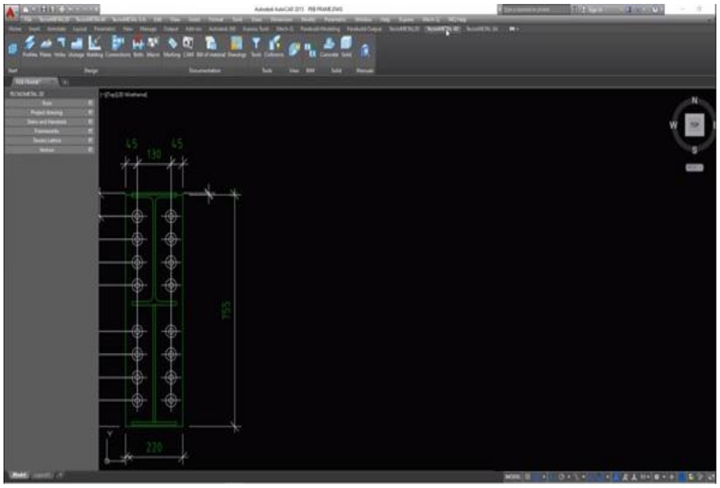


Fig.2. Existing tower location - plan and section.

3. MODELLING OF TELECOMMUNICATION TOWER

Modelling of telecommunication tower is perform in AutoCAD software. In this software with the aid of specific keywords like definition of point, line connections further section development were used to specify the development of a mobile tower.



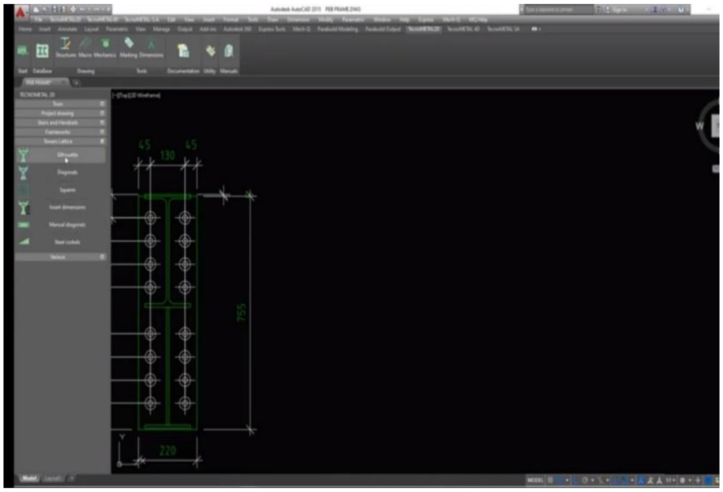
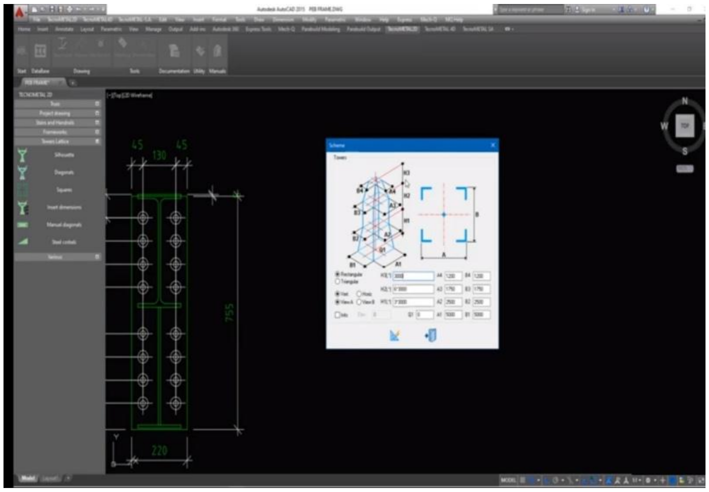


Fig. 3.1 Lines and Points defined in tower



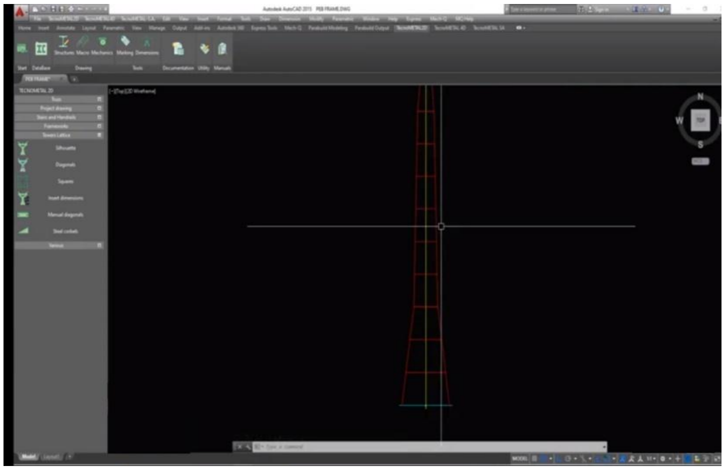
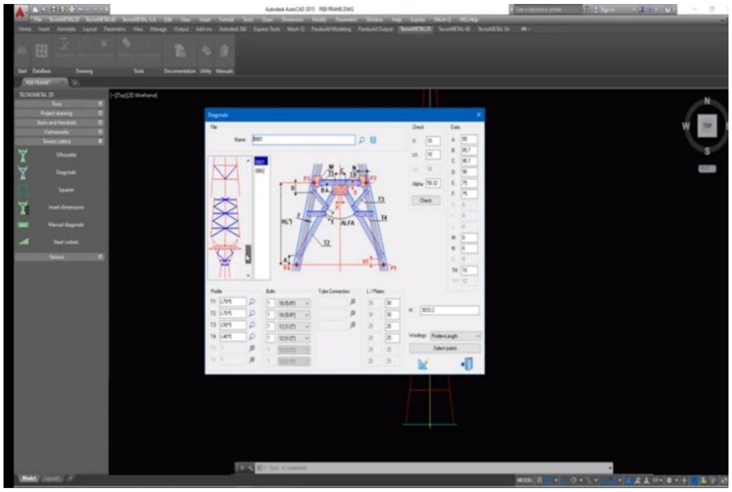
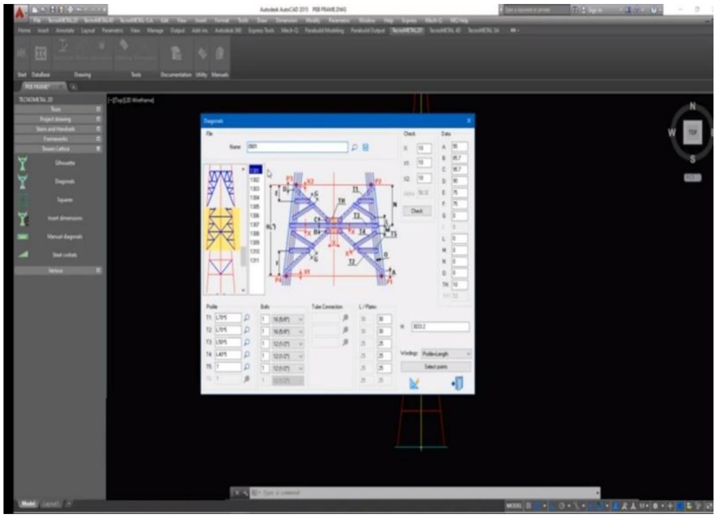


Fig. 3.2Sections defined in mobile tower





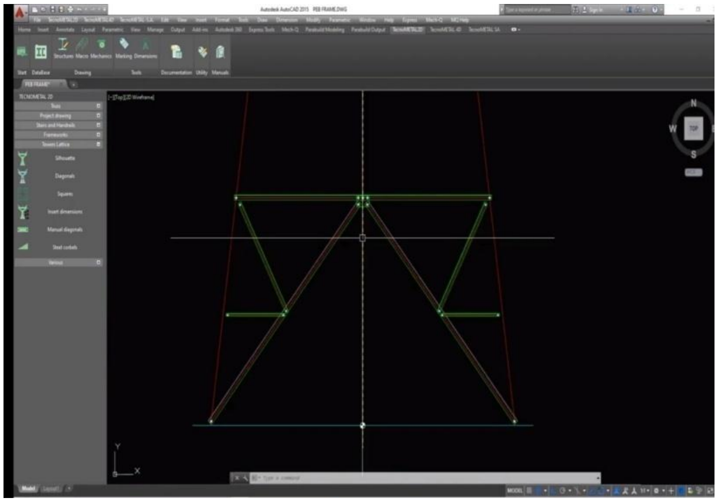


Fig 3.4Critical section development in mobile tower

In Fig 4.4 the mobile tower is developed using different complex cross-sectional section cross bars/ties inorder to maintain the connection between critical load carrying members and further they were connected in such a way the load transferred properly and each cross-member remains under safe condition with respect to maximum loads

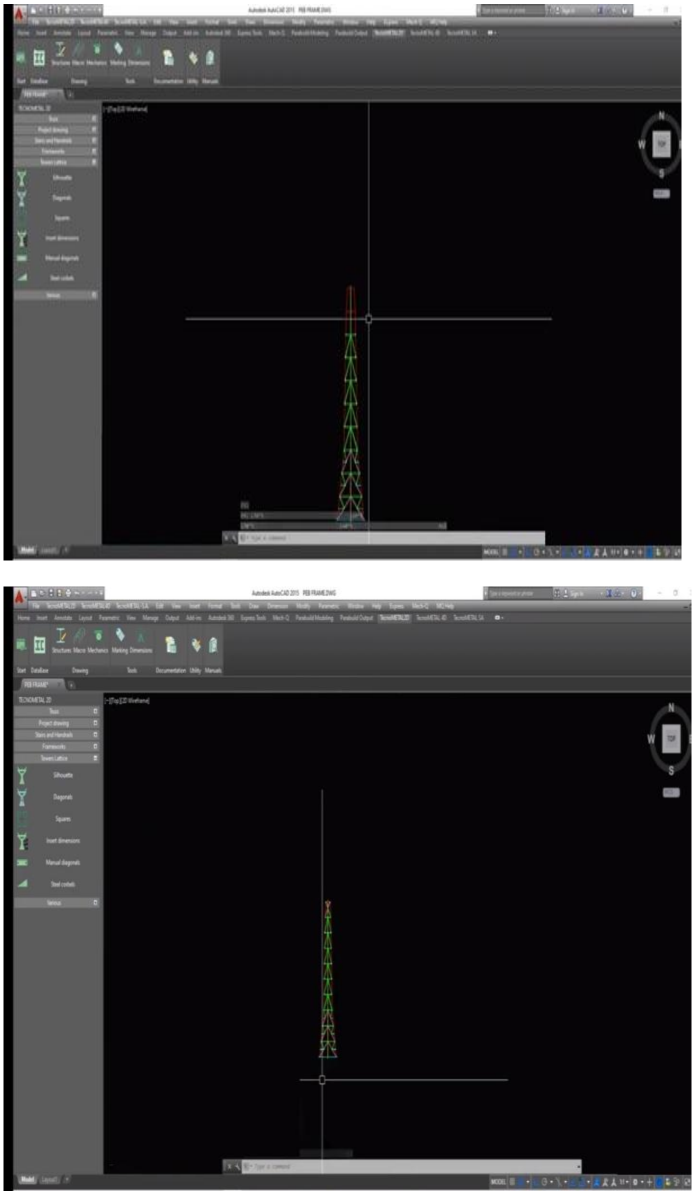


Fig. 3.5 Final developed mobile tower

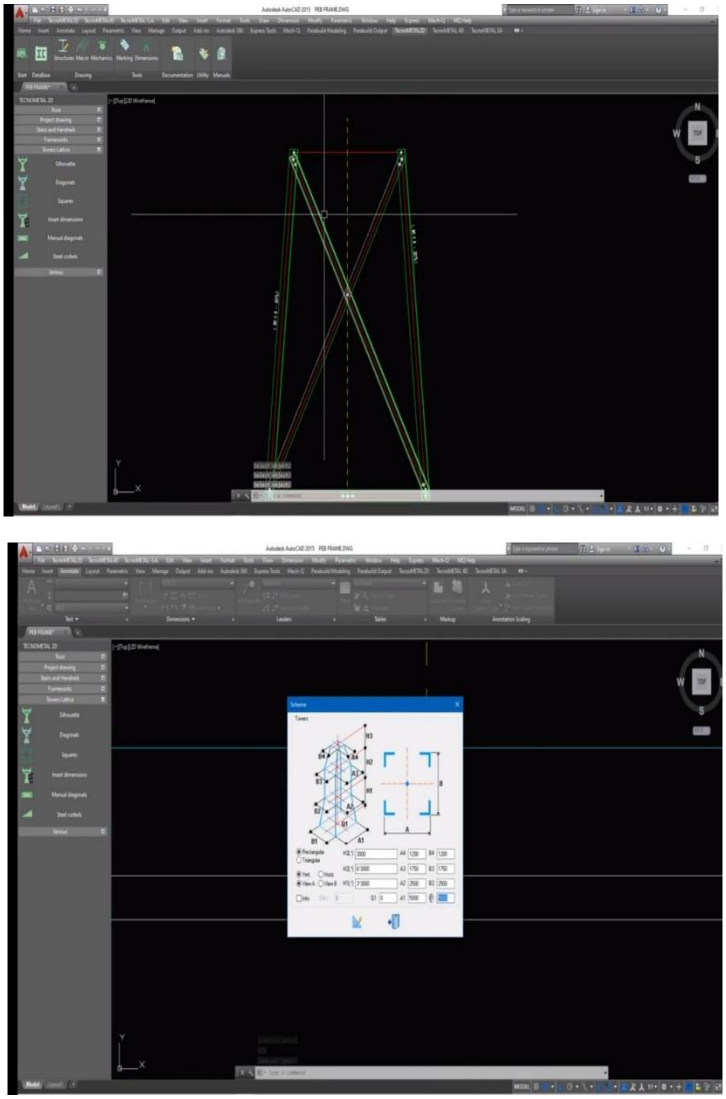


Fig 3.6 Different cross-members in mobile tower

Using the finite element method, the tower bar was modeled as a 3D truss and 3D beam element to maintain the dynamic properties of the structural model. The main parameters that affect the natural frequency of the branch tower, and the vibration modes associated with them. The main purpose of the modeling strategy adopted was to study the structural behavior of the branch towers and prevent the development of disturbing structural mechanisms that could lead to uneconomical or unsafe structures. The towers (50m, 70m, 90m) examined in this study have a truss-like shape with a square cross section. The hot-rolled angle sections, connected by bolts, form the main structure as well as the brace system.

Prestressed cables support key structures that must be under constant tension. Some of these cables are connected to specific rod sets that are placed to improve the torsional stiffness of the system. This study took into account the vertical loads acting (the weight of structures, stairs, antennas, cables, etc.). The main horizontal load was the effect of the wind on the tower. These horizontal loads were calculated according to the method described in Brazilian code NBR 6123 (NBR 6123, 1988) and applied to the nodes of the branch tower. Two wind load cases related to actions perpendicular and diagonal to the towers face were considered in this analysis. Theadopted guy prestress loads were in accordance to the valuesdescribed in the Canadian Code CSA S37-94 (CSA S37-94, 1994).

4. ANALYSIS OF TELECOMMUNICATION TOWER

Tower modeling has caused problems related to the loss of continuity in parts of the structure due to the presence of hinges connected to 3D finite truss elements. The apex angle (main landing gear) is usually connected by three bolts, so the design assumptions are strict. Truss connections are considered flexible, so truss elements are used to model the angle at which a moment break occurs. On the other hand, using only one screw for diagonal or horizontal diagonal connection creates another discontinuity. In-plane movement can be considered flexible, but out-of-plane movement ignores the twist and bend continuity that exists in the structure. To overcome this problem, dummy bars, without mass and with low axial stiffness, were incorporated to the structure. In this process every new bar represents the suppression of an internal degree of freedom. Although the towers have only four bars in the horizontal plane defined by a typical transversal section it is still necessary to add a fifth dummy bar to create two isostatic triangles. If this restraint was not considered a simple structural mechanism collapse would occur. Another reason for using these bars is to improve the structure torsion stiffness. All the abovementioned aspects allied to all the difficulties associated with the investigated tower geometry and to the truss finite element characteristics highlight the fact that the traditional truss design is not the bestrecommended methodology to be used. It should be stressed that the large number of dummy bars, adopted to enable the structural analysis to be performed, is the major disadvantage of this structural modelling strategy

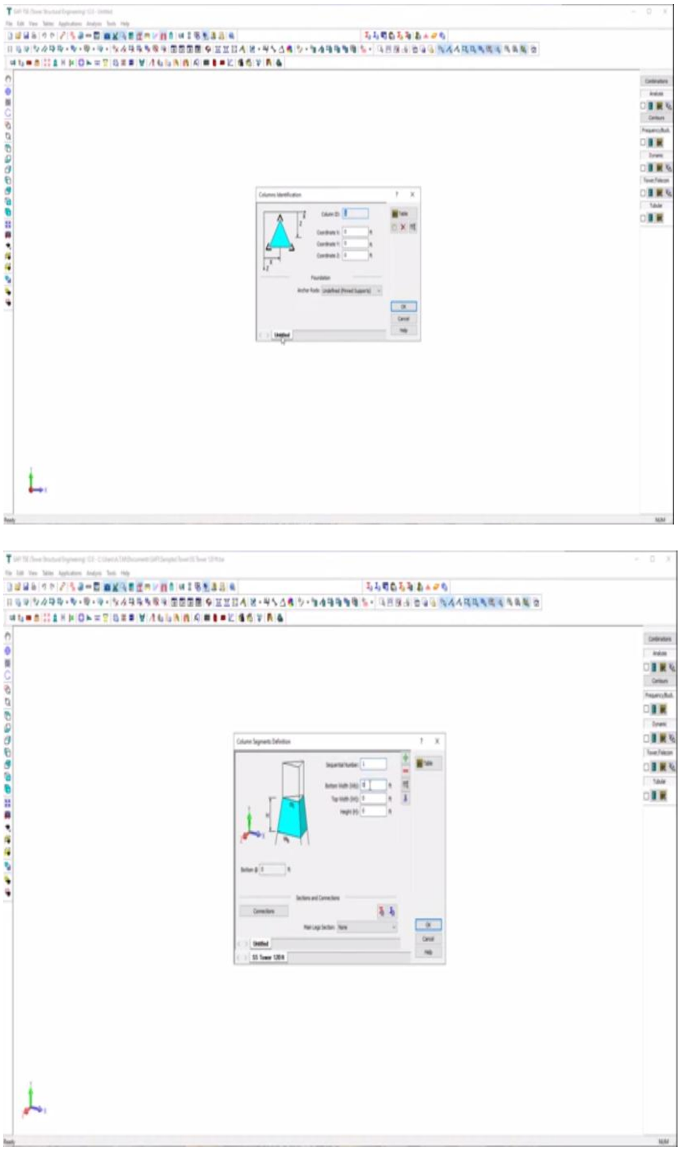
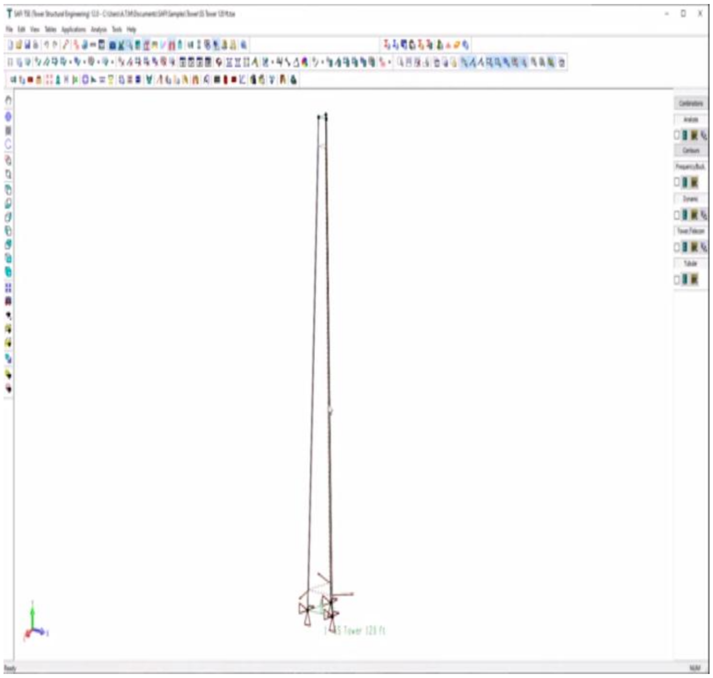


Fig 4.1.Development of Truss frame for mobile tower

In fig 4.1 the mobile tower of different height 50 m, 60 m and 70 m respectively was defined top be developed a FEM model. The developed FEM model was prepared using Triangular and Quad shaped elements of specific shapes and sizes. The core elements were the transformed into different layers to obtain different size of towers.



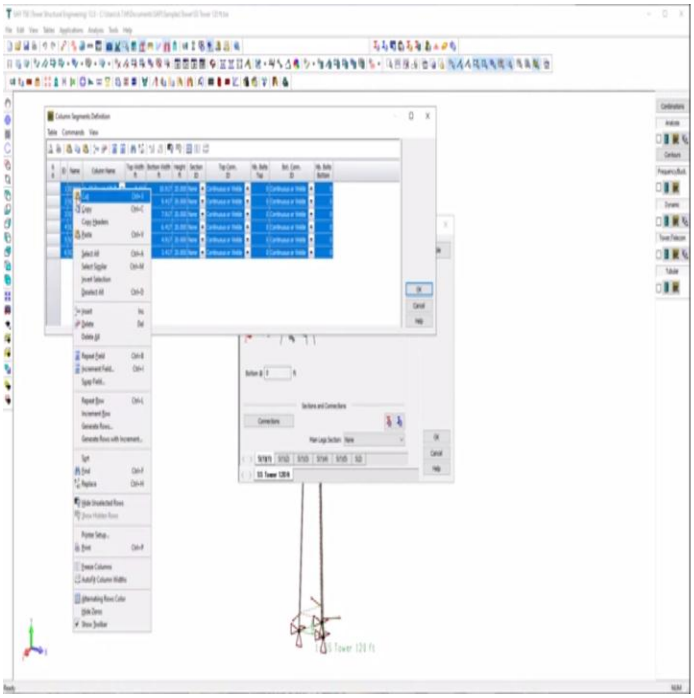


Fig 4.2 Assigning Boundary conditions

In order to solve a FEM problem the most critical thing to be taken into account is that the whole structure has to be properly constrained. The structure developed has to define fully with respect to its boundary condition and application of loads. In this case, the mobile tower is fixed at the bottom of the base with all degree of freedom as zero (means constant). After that the wind loads (taken from the reference paper) is applied on the structure to obtain the performance of the structure with respect to the loads.

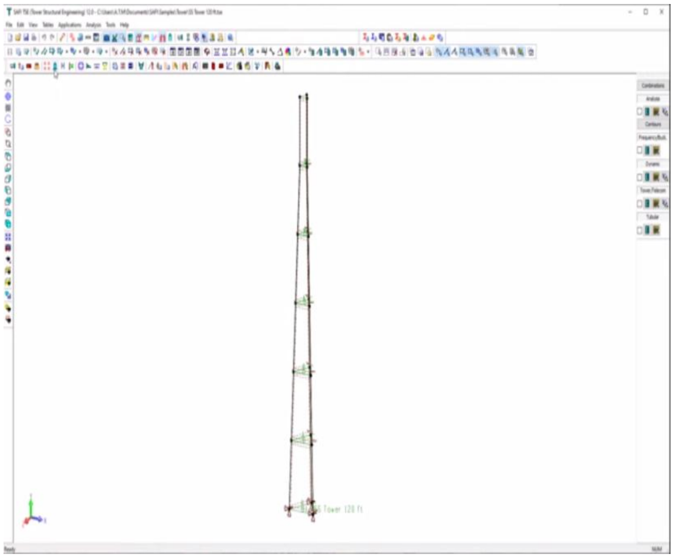
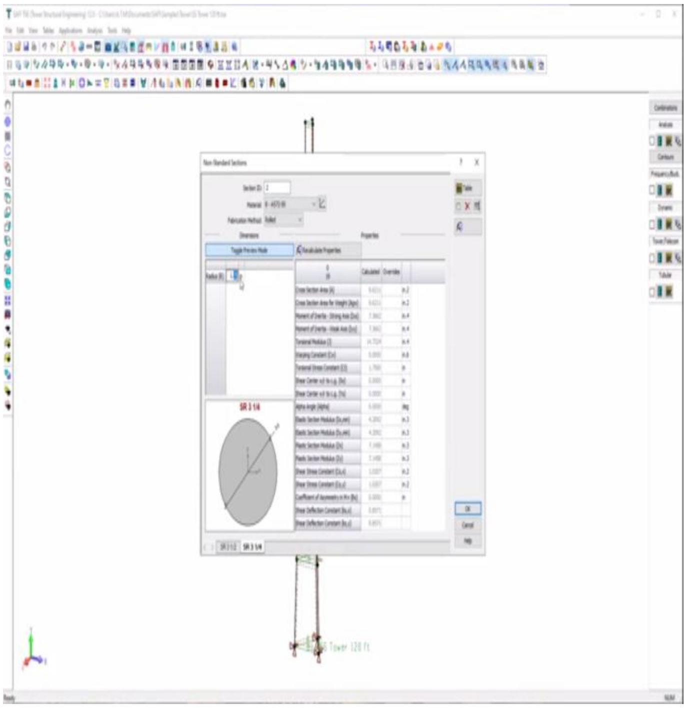


Fig. 4.3 Applied border conditions and material assignments



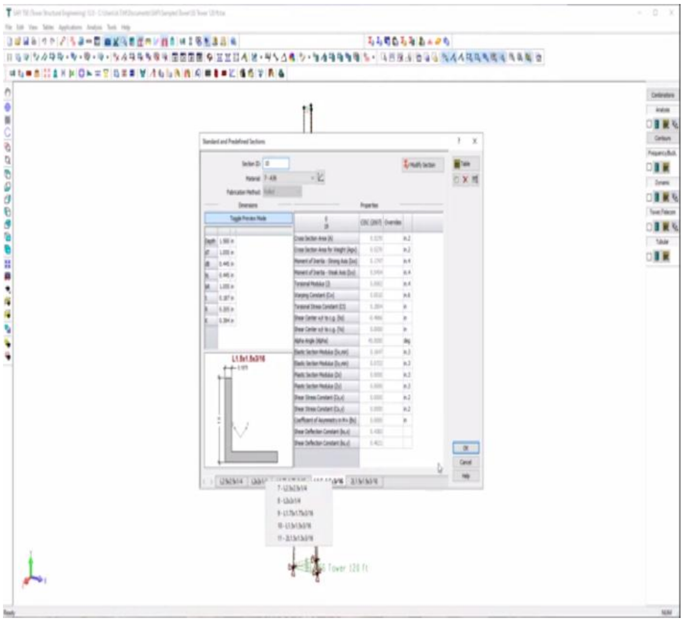
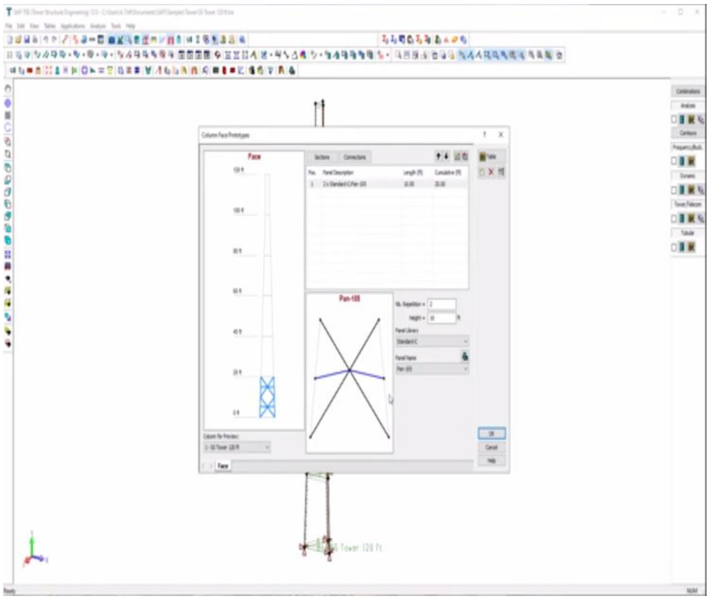
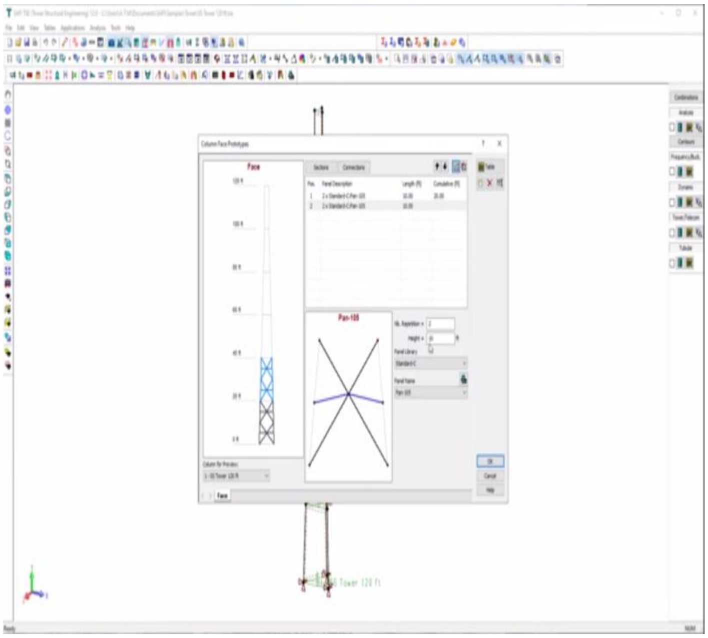
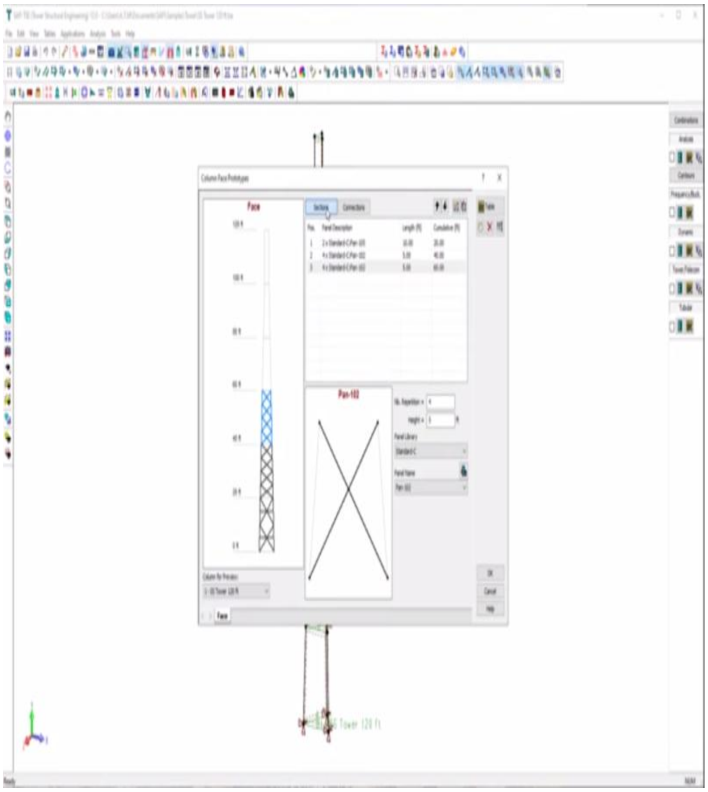


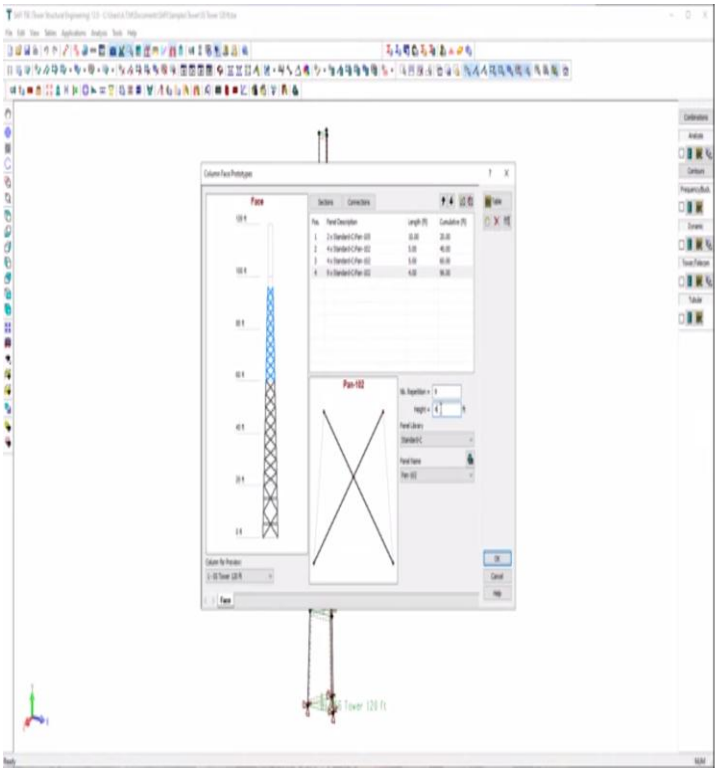
Fig.4.4.Assignment of Section to the chosen elements

In Fig. 4.4, it is indicated that the mobile tower is developed with aid of different elements and all the elements developed will require defining the cross-section of the elements to obtain the displacement and the stresses in the structure. Therefore, in this case the sections of different elements are defined with respect to different conditions of the load action upon it and also the position the element is placed.









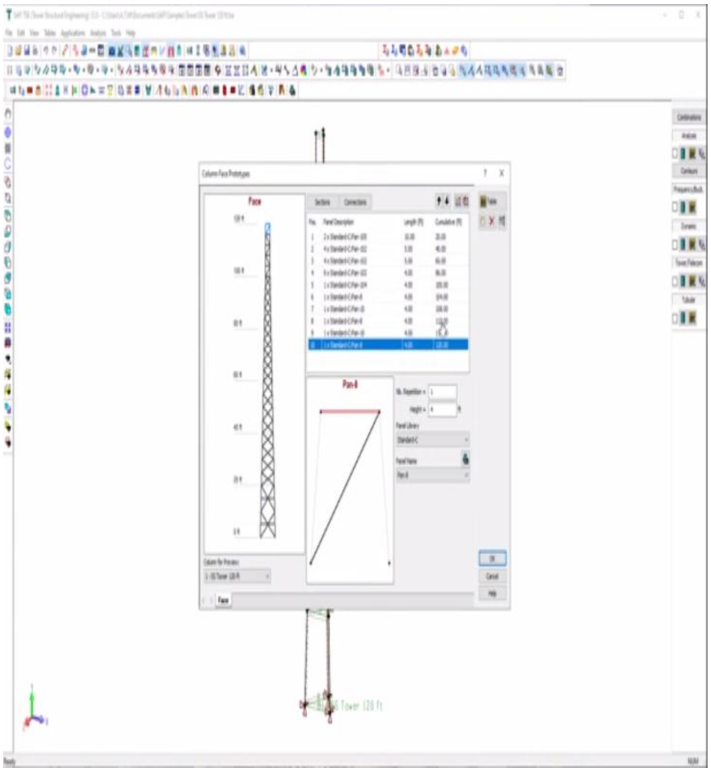
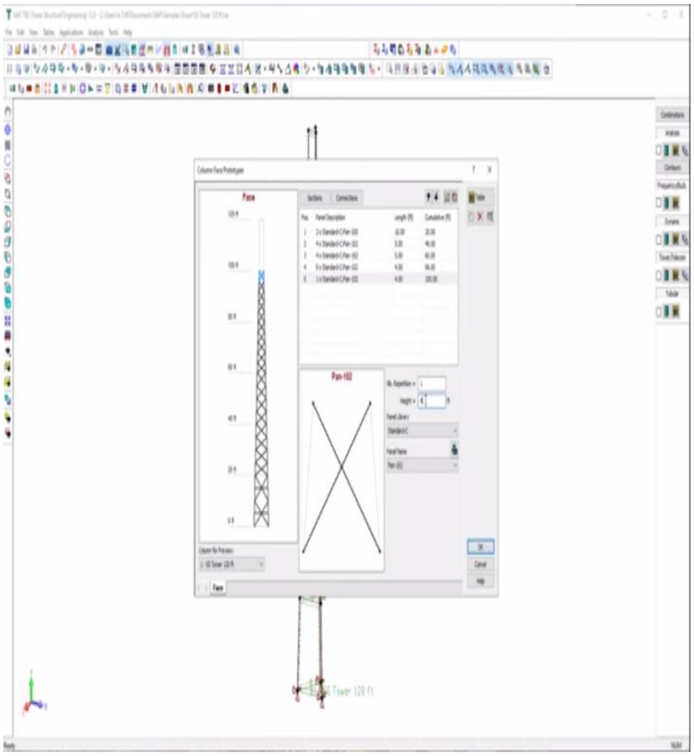


Fig.4.5 Different Cross tie members



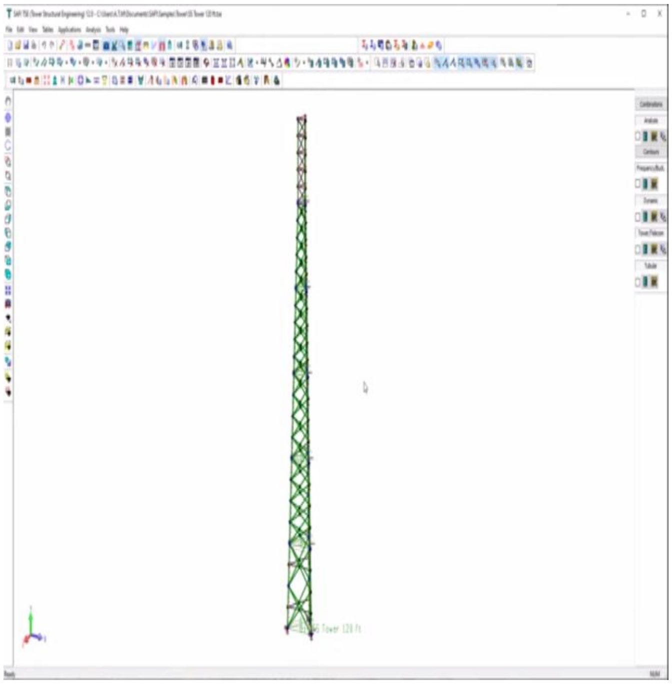
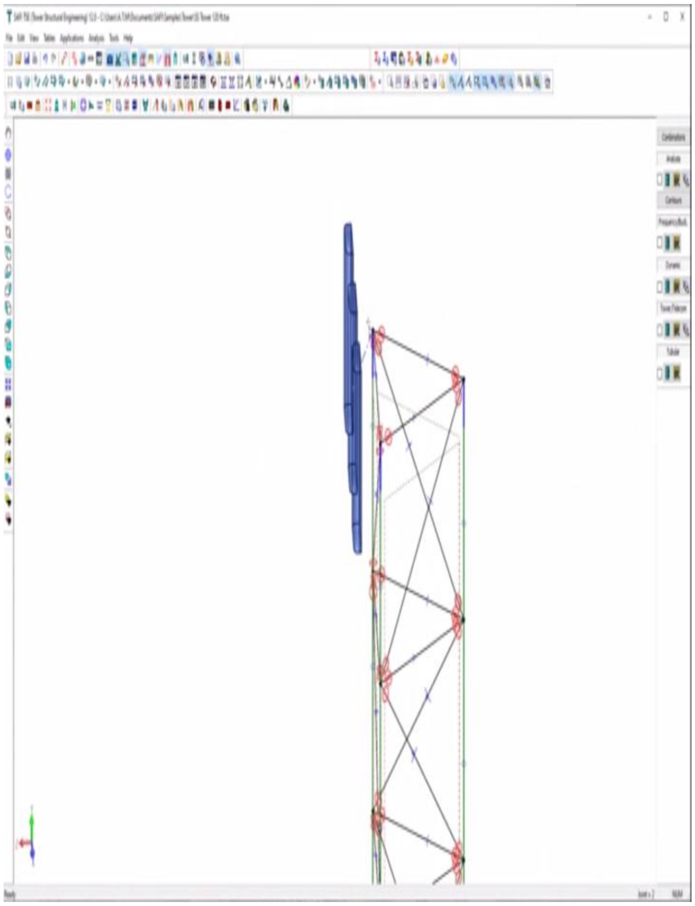


Fig.4.6 Mobile tower with complete cross tie members

Different mobile towers were designed with respect to the height of the tower but irrespective of that the model of the structure developed remains more of less the same. Cross member ties are specifically used with the definition of proper glued contacts so the contact definition within the surfaces remains the structures to act as a single structure and appropriate load transfer.



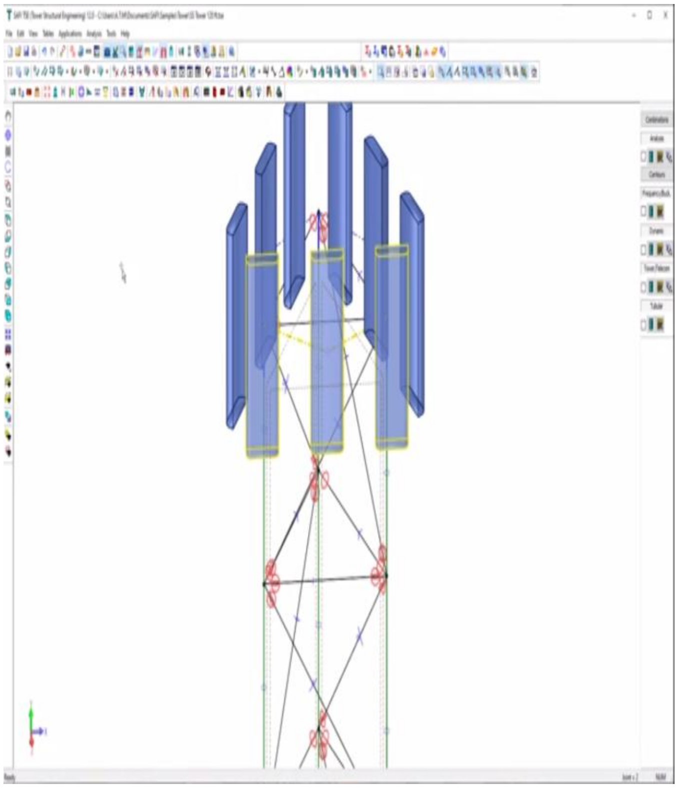
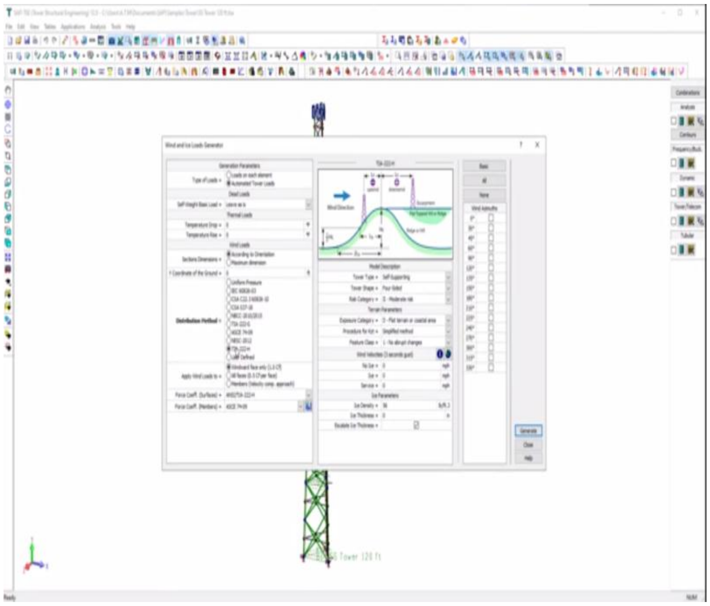


Fig. 4.7 Complete mobile tower with attachments



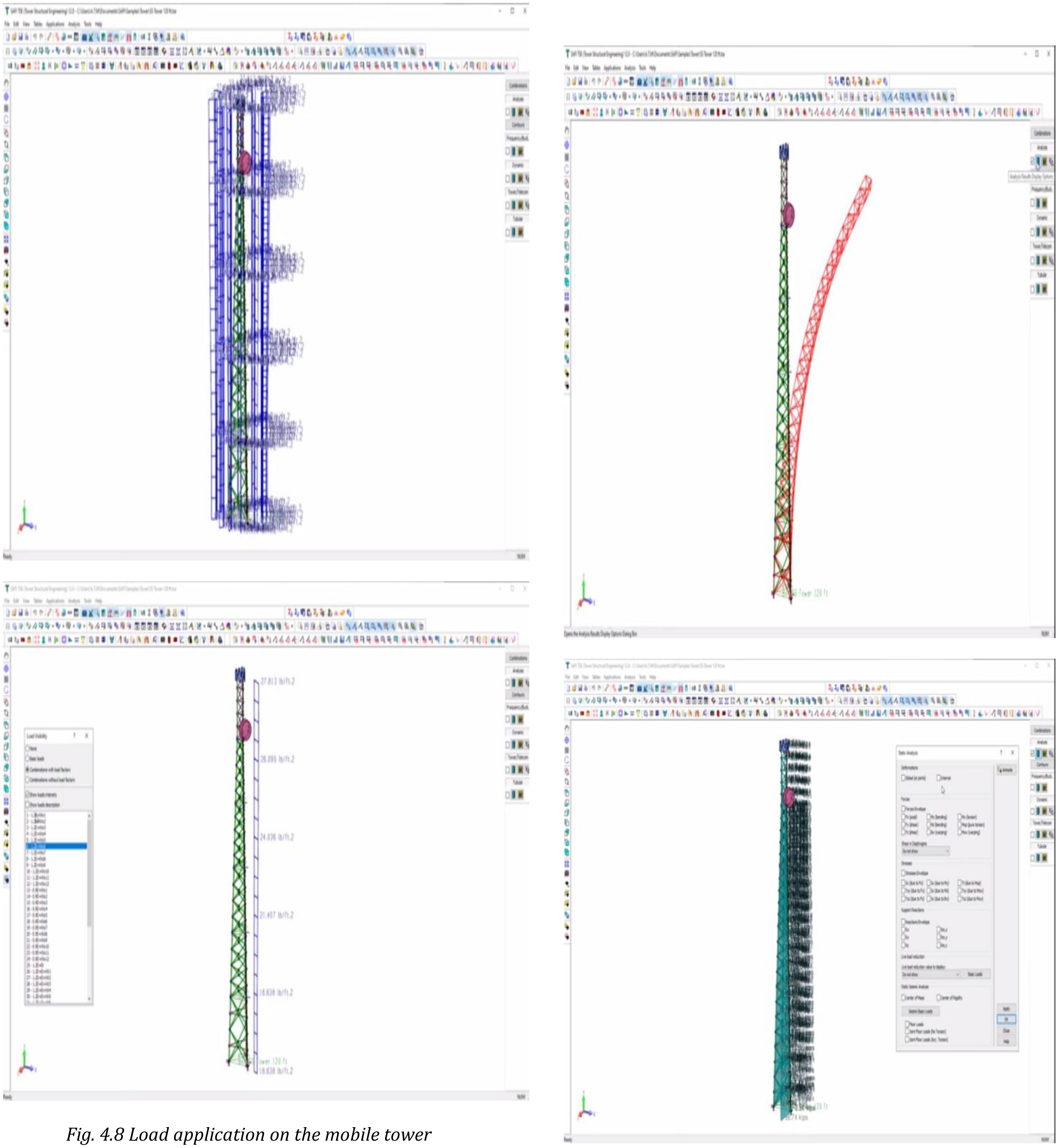


Fig. 4.8 Load application on the mobile tower

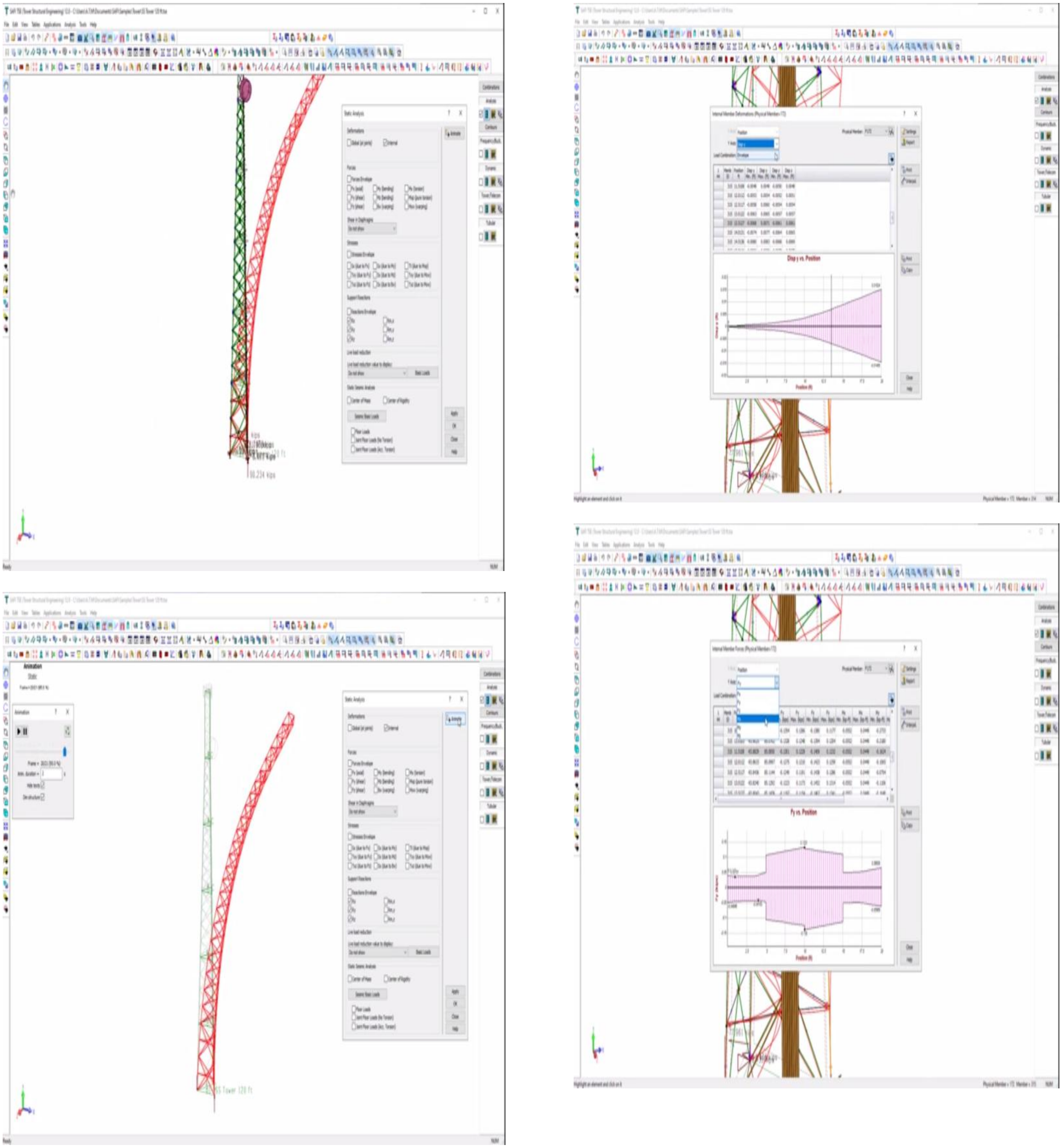


Fig 4.9 Vibration analysis of towers with height 50 m, 60 m and 70 m respectively

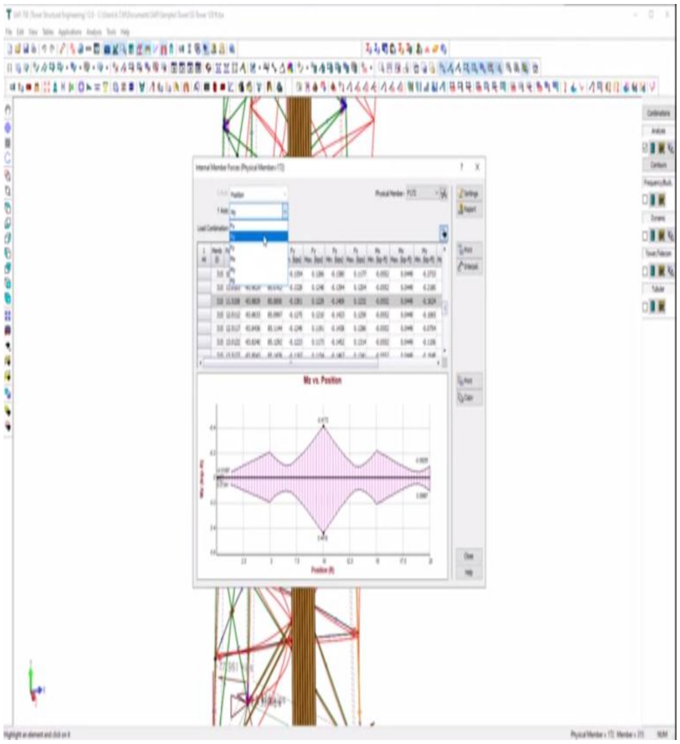
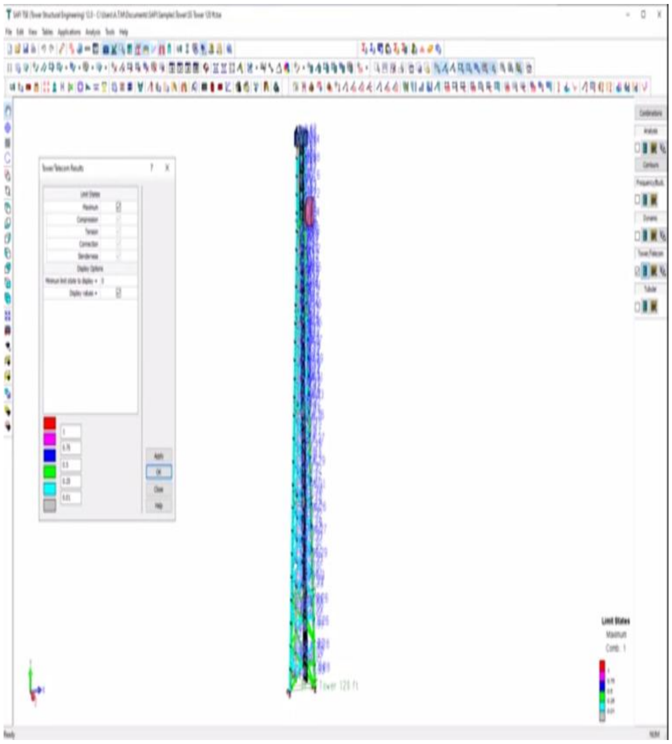
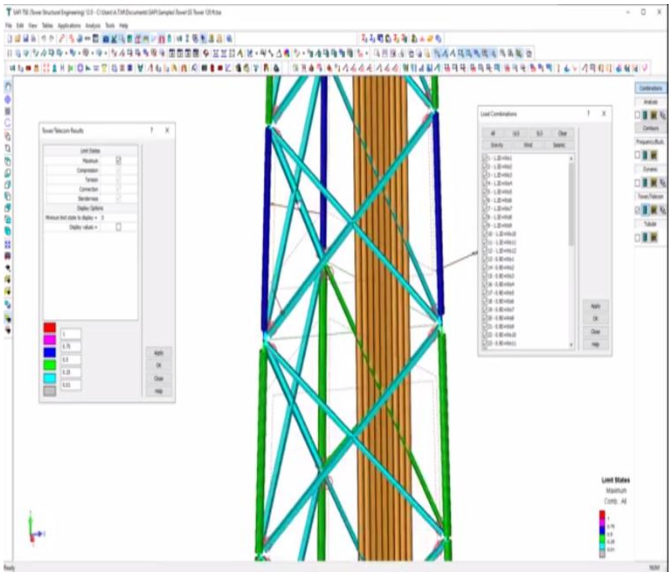


Fig. 4.10 Different modes of vibration on 50 m mobile tower





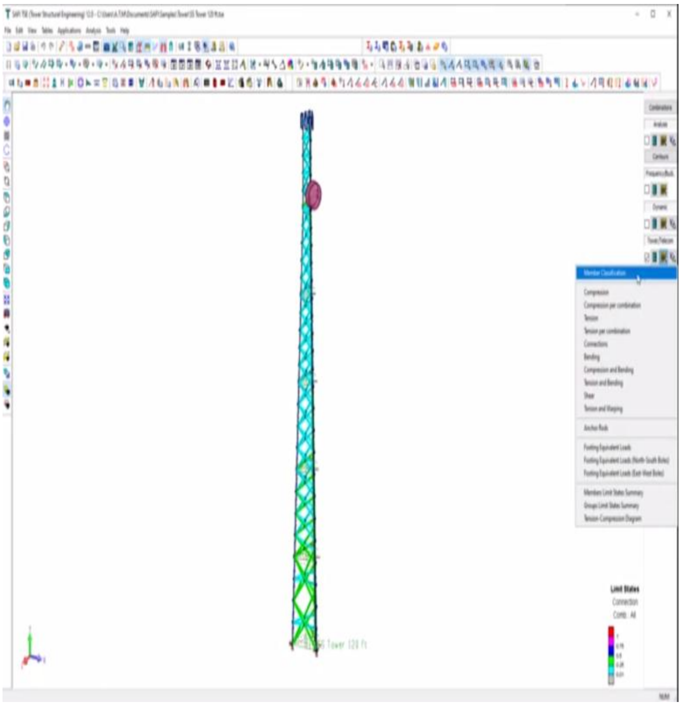


Fig 4.11 Final Response shown

The impact loads considered in this analysis are self-weight and two wind load cases. In these cases, horizontal wind loads are applied perpendicularly and diagonally to the face of the piers. The transverse wind load was calculated according to the procedure described in the Brazilian code NBR 6123 (NBR 6123, 1988) and was applied to the steel nodes of the tower. The results of static analysis of cell towers for towers with bars are studied (heights 50 m, 70 m and 90 m), according to the three structural models mentioned above. The maximum values of stress and transverse displacement are presented and compared.

5. PARAMETRIC OPTIMIZATION OF TELECOMMUNICATION TOWER

The start line for each production process/product optimization was Taguchi's optimization philosophy. However, this philosophy is being global criticized because of its incapability to resolve multi goal optimization problem. To remove this, it was observed in the literature that utility of Grey relational analysis, desirability feature approach, software theory, TOPSIS, fuzzy inference system (FIS), and most important issue analysis (PCA) [33-35], in my opinion incorporated with Taguchi method. The most important motto is to transform more than one targets into an equal unmarried goal feature; that may sooner or later be optimized via way of means of Taguchi method. However, those techniques depend on a few assumptions.

TAGUCHI'S S/N RATIO FOR PERFORMANCE EVALUATION

There is a loss feature which describes the deviation from the target (preferred level) and in addition converted into S/N ratio. The converted S/N ratio is likewise described as first-rate assessment index. The least variant and the choicest layout are acquired through reading S/N ratio. The better the S/N ratio, the greater solid the attainable first-rate. It additionally reduces the sensitivity of the machine overall performance to supply of variant.

There are three S/N ratios of common interest for optimization of static problems:

Nominal-the-best/target-the-best

In this approach, the closer to the target value, the better and the deviation is quadratic

The formula for these characteristics is

$$\frac{S}{N} = -10\log\frac{y}{S_y^2}$$

Lower-is-better (LB)

The lower is better approach held when a company desires smaller values.The formula for these characteristics is;

$$\frac{S}{N} = -10\log\frac{1}{n}\sum y^2$$

Higher-is-better (HB)

It is required when a manufacturer desires higher values of a characteristic. The formula for these characteristics is;

$$\frac{S}{N} = -10\log\frac{1}{n}\sum\frac{1}{y^2}$$

Here, y is the average of observed values; S_y^2 is the variance of observation and N is the number of observations

However, Taguchi method is considered only for single-objective optimization problems. It cannot be utilizedfor getting the single optimal setting of process parameters considering more than one performance parameter.

Taguchi's experimental approach has been followed to minimize the experiment trails, and L16 orthogonal array has been used to perform the experimental runs. Besides, ANOVA has been performed to find out the significance of process variables. Axial Load, Bending Strength and Thickness are taken as input parameters and maximum displacement as a Output parameters

Taguchi Array	L16(4^3)
Factors:	3
Runs:	16

Table 1.Design of experiment L16 orthogonal array

Sr. No	Axial Load	Bending Strength	Thickness	Maximum Displacement
1	100	5	0	9.5
2	100	7.5	2	9
3	100	10	4	8.5
4	100	15	6	8
5	200	5	2	13.5
6	200	7.5	0	12.5
7	200	10	6	11.5

8	200	15	4	10
9	300	5	4	21
10	300	7.5	6	19.5
11	300	10	0	17.6
12	300	15	2	15.4
13	400	5	6	20
14	400	7.5	4	24.3
15	400	10	2	22.6
16	400	15	0	20.3

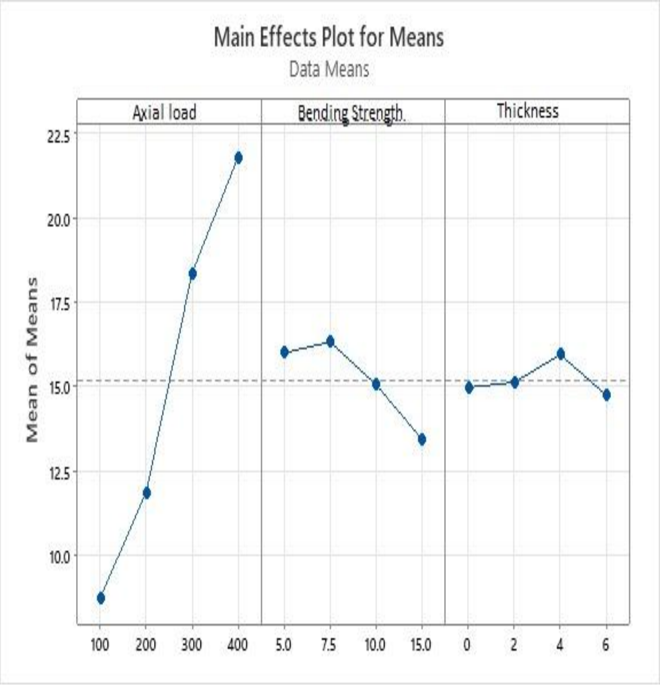
Table 2 Response Table for Signal to Noise Ratios

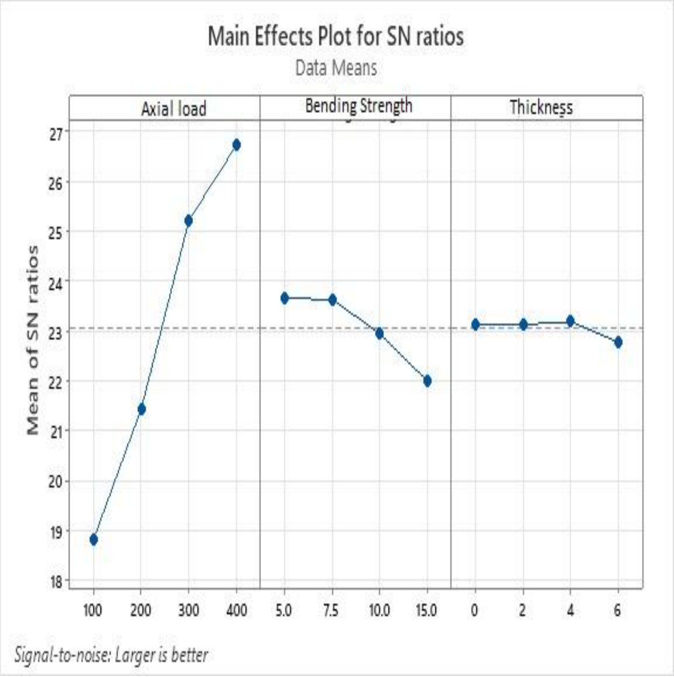
Larger is better

Level	**Axial Load**	**Bending Strength**	**Thickness**
1	18.82	23.66	23.14
2	21.44	23.63	23.13
3	25.23	22.95	23.19
4	26.74	21.99	22.77
Delta	7.92	1.67	0.41
Rank	1	2	3

Table 3 Response Table for Means

Level	**Axial Load**	**Bending Strength**	**Thickness**
1	8.750	16.000	14.975
2	11.875	16.325	15.125
3	18.375	15.050	15.950
4	21.800	13.425	14.750
Delta	13.050	2.900	1.200
Rank	1	2	3



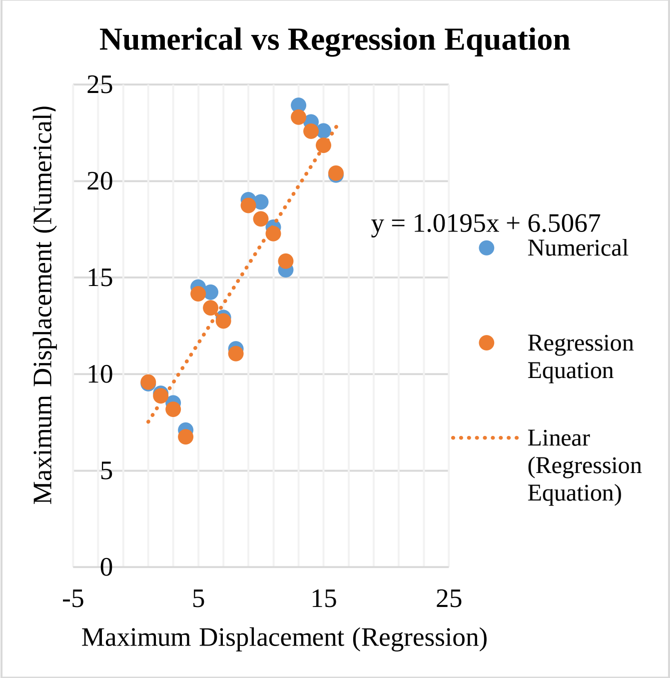


Regression Equation

Maximum Displacement = 6.45 + 0.04565 Axial Load - 0.287 Bending Strength + 0.007 Thickness

Table 4 -Maximum Displacement Numerical vs Regression equation

S.No	Maximum Displacement	
	Numerical	Regression Equation
1	9.5	9.58
2	9	8.87
3	8.5	8.17
4	7.1	6.75
5	14.5	14.15
6	14.24	13.42
7	12.92	12.75
8	11.30	11.05
9	19.03	18.73
10	18.92	18.03
11	17.6	17.27
12	15.4	15.85
13	23.92	23.31
14	23.05	22.58
15	22.6	21.85
16	20.3	20.40



Coefficients

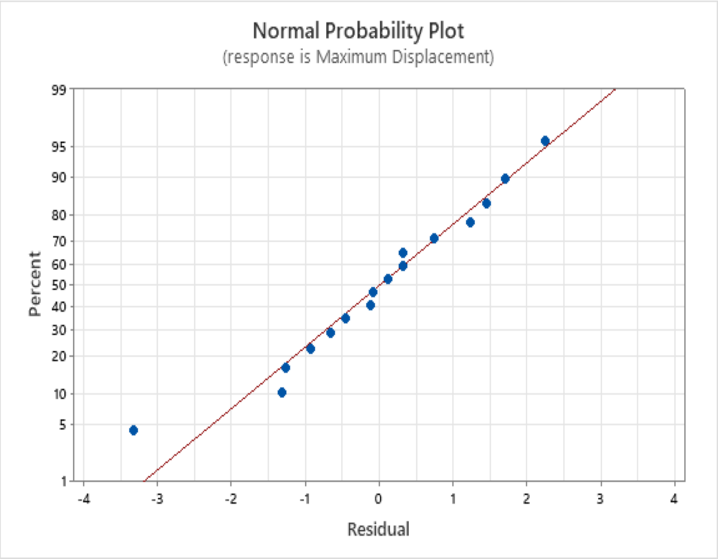
Term	Coef	SE Coef	T-Value	P-Value	VIF
Constant	6.45	1.45	4.45	0.001	
Axial Load	0.04565	0.00344	13.29	0.000	1.00
Bending Strength	-0.287	0.104	-2.76	0.017	1.00
Thickness	0.007	0.172	0.04	0.966	1.00

Model Summary

S	R-sq	R-sq(adj)	R-sq(pred)
1.53652	93.88%	92.35%	87.60%

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Regression	3	434.789	144.930	61.39	0.000
Axial Load	1	416.784	416.784	176.54	0.000
Bending Strength	1	18.000	18.000	7.62	0.017
Thickness	1	0.004	0.004	0.00	0.966
Error	12	28.331	2.361		
Total	15	463.120			



Prediction

S/N Ratio	Mean
27.4211	22.375

6. CONCLUSIONS

- The developed method in this study is less conservative than traditional methods. The proposed methodology, as it uses 3D bean truss model for modelling the whole mobile structure relevant to wind and seismic loads.
- The mobile tower with different heights of same configuration namely 50 m, 60 m and 70 m were developed using FEM technique to obtain the natural displacement and stresses developed using different load sets.
- For a height of 50 m tower resting on the building X bracing is economical in the form of deflection at the top of the tower
- If the height increases then X bracing proves uneconomical and K bracing proves economical.
- Magnitude of displacement in tower of smaller size is lower with same thickness on action of same intensity blast loads.
- Magnitude of stresses in tower of smaller size is lower with same thickness on action of same intensity blast loads
- Intensity of displacement in tower design combination showed an increase with the enhancement in the intensity of blast loads. The maximum displacement of 50 mm happens at 2495 N load while at 195 N load the maximum displacement was 12 mm.
- ANOVA analysis is also carried to analyse the parametric performance of Mobile tower design.
- Taguchi analysis is performed for analysis of the parameters of Mobile tower design

REFERENCES

1. A.N. Nayak and Dr.S.K. Bhattacharya. Behavior of joints with rectangular and square hollow sections. Journal of the Institution of Engineers, Civil Engineering Division. 78:116-122, Nov 1997.

2. Berding Daniel and Charney Finley A. The effect of modeling parameters on the wind drift of steel frame buildings. 2007 Structures Congress: New Horizons and Better Practices, ASCE.

3. Da Silva J.G.S. et al. Structural assessment of current steel design models for guyed steel telecommunication towers for radio antenna. Journal of Constructional Steel Research. 61:1108-1134, 2005.

4. Gomathinayagam S.et al. Dynamic Response of Lattice Tower with Antenna under Wind Loading.Journal of the Institution of Engineers (India). 87: 37-43, 2000.

5. Gucuyen Engin et al. Effect of changes on joint connections of steel lattice towers due to environmental loads. International Journal of Engineering and Industries. 2(11), 2011.

6. Harikrishna P. et al. Analytical and Experimental Studies on the Gust Response of a 52m Tall Steel Lattice Tower under Wind Loading. Computers and Structures, 70:149-160, 1999. Structural Engineering Research Centre, Chennai.

7. Jiang W.Q. et al. Accurate modeling of joint effects in lattice transmission towers. Engineering Structures. 33:1817–1827, 2011.

8. Sullins Eric James. Analysis of radio communication towers subjected to wind, ice and seismic loadings. MS notion, (2006) Faculty of the Graduate School of the University of Missouri,Columbia.

9. Cello F.Carril Jr., Nicholas Isyumov, Reyolando M.L.R.F Brazil (2003) "Experimental Study of The Wind Forces on Rectangular Lattice Communication Towers with Antennas" Journal of Wind Engineering and Industrial Aerodynamics, vol.91 pp. 1007-1022.

10. Ananias Kudzys (2006). "Safely of Power Transmission Line Strictures' under Wind and lee Storms" Engineering Structure, vol.28 pp.682-689

11. NitinBhosale, Prabhat Kumar, PandeyA.D.[1], (2012): "Influence of host structure characteristics on response of rooftop telecommunication towers" , International journal Civil and Structural Engineering, Volume 2,Issue No 3, 2012, ISSN NO. 0976 -4399.

12. Ananias Kudzys (2006). "Safely of Power Transmission Line Strictures' under Wind and lee Storms" *Engineering Structure,* vol.28 pp.682-689

13. NitinBhosale, Prabhat Kumar, PandeyA.D.[1], (2012): "Influence of host structure characteristics on response of rooftop telecommunication towers" , International journal Civil and Structural Engineering, Volume 2,Issue No 3, 2012, ISSN NO. 0976 -4399.

14. Grekavicius L, Hughes JA, Tsavdaridis KD, Efthymiou E. Novel morphologies of aluminium cross-sections through structural topology optimisation techniques. In: The 13th International Aluminium Conference (INALCO 2016). 21–23 September 2016, Naples, Italy; 2016b.

15. Groen JP, Sigmund O. Homogenization-based topology optimization for high-resolution manufacturable microstructures. Int J Numer Meth Eng 2018;113(8):1148–63.

16. Hajirasouliha I, Pilakoutas K, Moghaddam H. Topology optimization for the seismic design of truss-like structures. Comput Struct 2011;89(7–8):702–11.

17. Holmes JD, Banks RW, Roberts G. Drag and aerodynamic interference on Microwave dish antennas and their supporting towers. J Wind Eng Ind Aerodyn 1993;50:263–70.

18. Jiang C, Jia H. Evolutionary based intelligent algorithm for topology optimizationof structure. Sixth international conference on intelligent systems design and applications.California: IEEE; 2006. p. 897–902.

19. Kefal A, Sohouli A, Oterkus E, Yildiz M, Suleman A. Topology optimization ofcracked structures using peridynamics. Continuum Mech Thermodyn2019;31(6):1645–72.

20. Kentli A. Topology optimization applications on engineering structures. IntechOpenpublications; 2019.

21. Nielsen MG, Støttrup-Andersen U. Advantages of using tubular profiles for telecommunication structures 31 August/2 September Quebec City. London: Taylor & Francis Group plc; 2006. p. 45–51.

22. Nicolaou A, Tsavdaridis KD, Efthymiou E. Topology optimisation study for the design of lattice towers. In; The 9th Greek National Steel Structures Conference. 5–7 October, 2017, Larissa, Greece; 2018.<http://eeme.ntua.gr/proceedings/9th/Papers/063_PAP_Nicolaou.pdf.

23. Noii N, Aghayan I, Hajirasouliha I, Kunt MM. A new hybrid method for size and topology optimization of truss structures using modified ALGA and QPGA. J Civ Eng Manage 2016. ISSN 1392-3730.

24. ISO 12494. 2001. Atmospheric Icing of Structures. International standard ISO 12494:2001 (Annex D).