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Review Article

Telecommunication Cell Tower Most Common Alternatives Overview

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Abstract: The main goal of this review is to decide which alternatives could be the most proper structural system type for 36 m tower height and practical specified deflection limit “Torsional Effect” at the tower top less than 0.5 degree to be used in a rural zone near of Budapest City, Hungary based on the most common control aspects (Aesthetical, Economical and Statical aspect) which influences the decision making and the selection process. According to that purpose a different tower types have been reviewed in detail (Lattice towers, Monopole and Guyed mast) in order to decide which alternatives could be selected for further investigations based on the limitations and requirements of the present case of study, where every alternative has it features, benefits and Specific limits of application. The resulted decision based on the presented study was that the most proper alternatives according to the specified information are the lattice towers (Square & Triangular) and the Monopole which deserve to be selected for further investigations.

Keywords: Cell Tower, Lattice, Monopole, Telecommunication, Antenna, Guyed Tower

1. Introduction

Telecommunication towers are used for communication purposes among people, require elevated antennas to effectively transmit and receive radio communications. In the absence of tall buildings that antennas can be mounted to, self-supporting, monopole and guyed towers tend to be the most economical choice for mounting antennas. These types of towers are generally lightweight in comparison to building a solid structure and are also easier to fabricate and erect. All the wireless communication, mobile networking, radio broadcasting and TV antennas are connected via these towers. Different heights are used in different places and purposes. They can vary from 15-60 m and sometime more if required. For example in the land areas towers are higher in hill area so 15 to 30 m high towers can be used but in land areas they are 30 to 60 m in height. There are different types of the telecommunication towers which are used i.e. monopole, self-supporting and guyed etc. The most used are the self-supporting towers in the field of telecommunication. Due to space constraints, towers in heavily developed areas tend to be self-supporting, monopole while towers in rural areas are often guyed. This paper represent the most relevant studies of cell Towers design where it could depend on the role of the considered aspects (Economical, Aesthetical, and Statical aspect).

2. General Review

In this paper the most common structural systems of the Telecommunications towers will be reviewed and described in details in order to reach the possible and the most suitable types of telecommunication cell towers that could be chosen for further investigations later. The different types of communication cell towers are based upon their structural action, cross section, type of sections used and on the placement of tower. A brief description is as given below:

2.1. Based on Cross Section of Tower

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

Triangular Lattice Towers have less weight but offer less stiffness in torsion. With the increase in number of faces, it's observed that weight of tower increases. The increase is 10% and 20% for square and hexagonal cross sections respectively. If the supporting action of adjacent beams is considered, the expenditure incurred for hexagonal towers is somewhat less, figure 1 [6].

2.2. Based on Structural Action

Towers are classified into three major groups based on the structural action. They are:

a) Self-supporting towers
b) Monopole
c) Guyed towers

Self-supporting towers

The towers that are supported on ground or on buildings are called as self-supporting towers. Though the weight of these towers is more they require less base area and are suitable in many situations. Most of the TV, MW, Power transmission, and flood light towers are self-supporting towers. In this paper the lattice cell towers as Triangular and Square towers will be investigated, as shown in figure 2(b) [5, 8, 9, 10].

Monopole towers

It is single self-supporting pole, and is generally placed over roofs of high raised buildings, when number of antennae required is less or height of tower required is less than 9m. It uses minimal space and resemble a single tube, requires one large foundation, typically not exceed 45 m height and the antennas are mounted on the exterior of the tower, as shown in figure 2(c) [5, 8, 9, 10, 11].

Guyed towers

Guyed towers provide height at a much lower material cost than self-supporting towers due to the efficient use of high-strength steel in the guys. Guyed towers are normally guyed in three directions over an anchor radius of typically 2/3 of the tower height and have a triangular lattice section for the central mast. Tubular masts are also used, especially where icing is very heavy and lattice sections would ice up fully. These towers are much lighter than self-supporting type but require a large free space to anchor guy wires. Whenever large open space is available, guyed towers can be provided. There are other restrictions to mount dish antenna on these towers and require large anchor blocks to hold the ropes, as shown in figure 2(a) [5, 8, 9].

Guyed Tower benefits

1. Ideal for heights over 60 m
2. Requires significant installation footprint to accommodate guy anchors
3. Has significant wind-loading capacity
4. Could be the cheapest choice in case of space availability for so high tower levels.

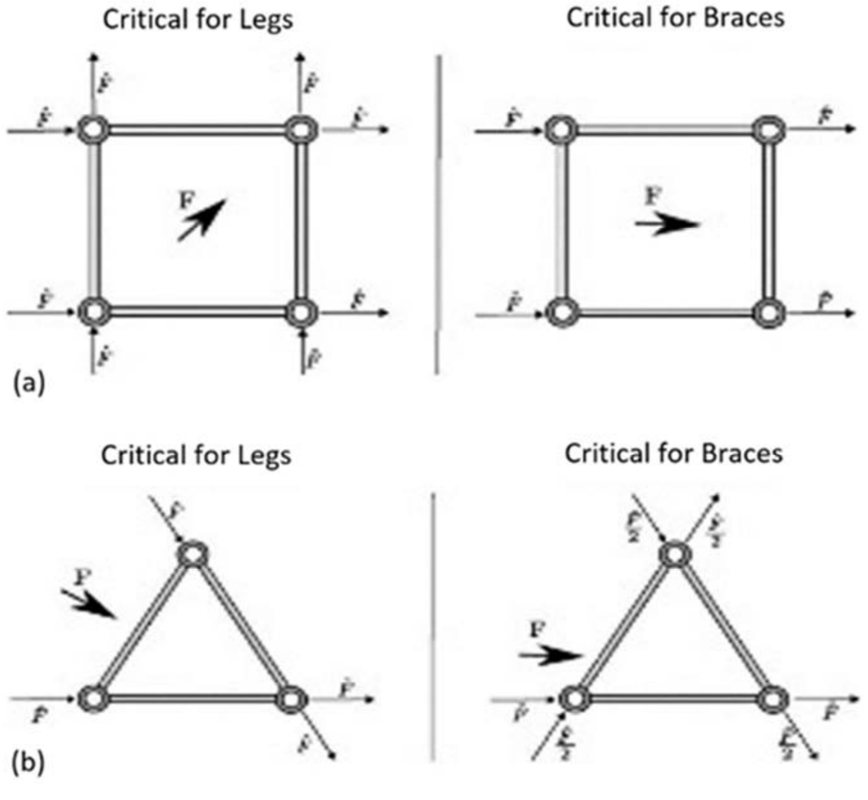

Figure 1. *Triangular and Square Tower.*

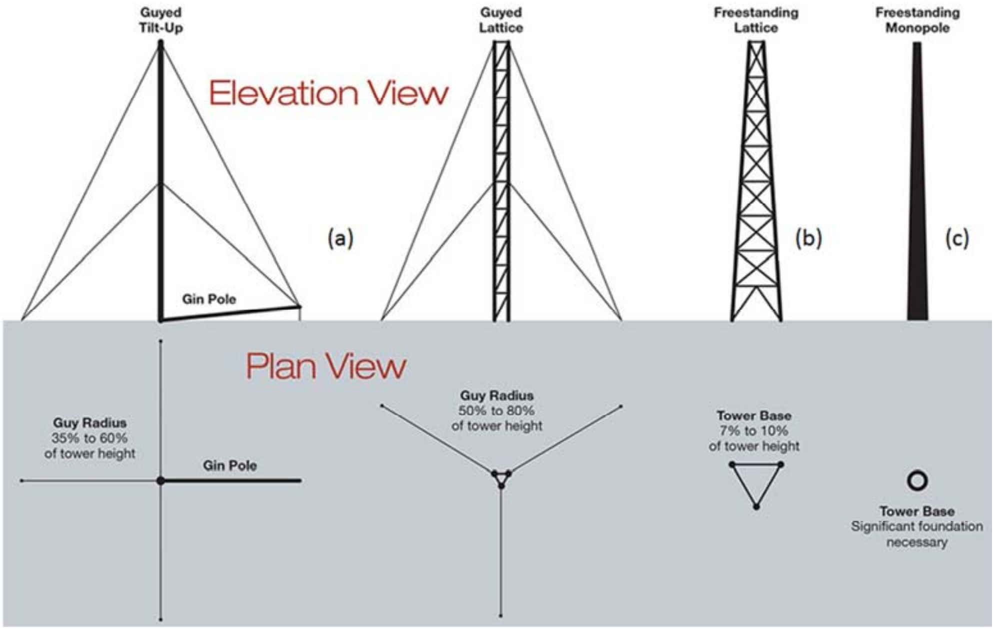

Figure 2. *Types of telecommunication towers.*

2.3. Based on Type of Material Sections

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

2.4. Based on the Placement of Tower

Based on this placement, Communication towers are classified as follows, as shown in table 1 [7].

Table 1. *Towers classifications according to the placement.*

	Green Field Tower	Roof Top Tower
Erection	Erected on NG. with suitable foundation	Erected on Existing building with raised columns and tie beams.
Height	30 – 200 m	9 - 30 m
Location	Rural Areas	Urban Areas
Economy	Less	More

3. Towers Bracing System Types

Once the width of the tower at the top and also the level at which the batter should start are determined, the next step is to select the system of bracings. The following bracing systems are usually adopted for telecommunication towers as below, as shown in figure 3 [7, 10].

1. Single web system (a)
2. Double web or Warren system (b)
3. Pratt system (c)
4. Portal system (d)
5. Offset or staggered system (e)

Single web system: It comprises either diagonals and struts or all diagonals. This system is particularly used for narrow-based towers, in cross.

Arm girders and for portal towers type, figure 3 (a).

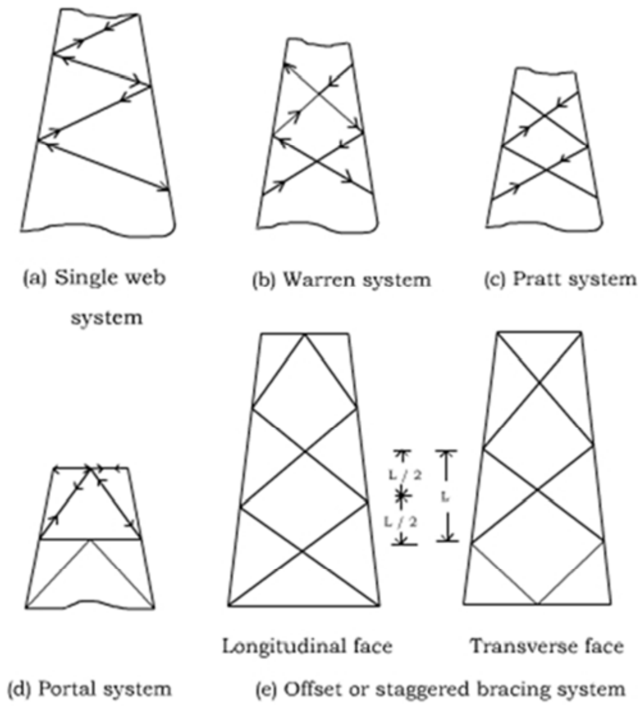

Figure 3. *Bracing Systems.*

Portal system: The diagonals are necessarily designed for both tension and compression, as shown in figure 3 (d) [7], therefore, this arrangement provides more stiffness than the

Pratt system. The advantage of this system is that the horizontal struts are supported at mid length by the diagonals. Like the Pratt system, this arrangement is also used for the bottom two or three panels in conjunction with the Warren system for the other panels.

4. Review for the Relevant Studies and Existing Tower Structures

4.1. Triangular & Rectangular Towers

In (4th of February 2017) Turkey, A. M. Tah, Kamiran M. Alsilevanai, Mustafa Özakça [1, 8, 9], represented in their study of "Comparison of Various Bracing System for Self-Supporting Steel Lattice Structure Towers" where they dealt with the effectiveness of various bracing systems used in lattice towers. 7 types of bracings used in 4-legged rectangular based self-supporting telecom towers and 4 types of bracings used in 3-leg triangular self-supporting telecommunication towers are analyzed, as shown in figure 4 [1]. The investigated bracing systems are K, KD, Y, YD, D, XB and X-bracing, figure 5 [1]. This study has focused on identifying the economical bracing system for a given range of tower heights. Towers of height 40 to 60 m for telecommunication have been analyzed under critical loads such as wind and earthquake loads. The load cases include diagonal wind has been found to be most critical cases for towers. The performance of various bracing system has been identified and reported.

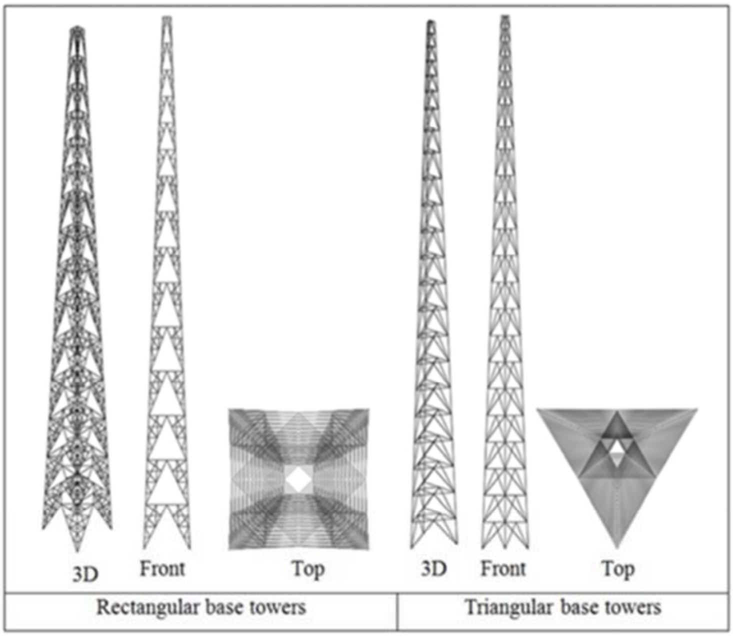

Figure 4. *Geometry of telecom Tower.*

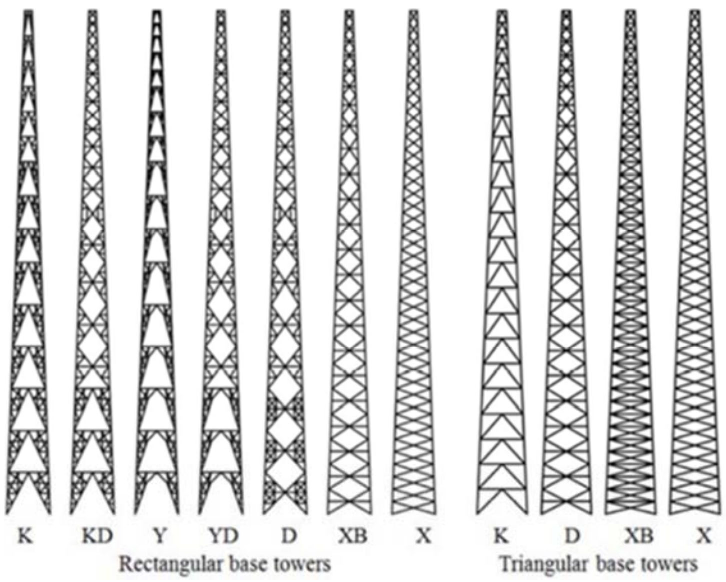

Figure 5. *Towers with different base and bracing systems.*

4.1.1. Bracing Systems

In the current study, Seven different types of bracing system consist of K, KD, Y, YD, D, XB and X are considering for rectangular base telecommunication towers with a height of 60, 50 and 40 m. Four different bracing systems consist of K, D, XB and X-bracing for triangular base telecommunication towers are also studied. Figure 5 [1] illustrates both rectangular and triangular base towers with different bracing patterns.

4.1.2. Numerical Analysis

The steel telecommunication tower design is not a straight forward process, but an interactive compromise between many factors, which must ultimately satisfy basic strength requirements. Generally, in structural analysis, the actual complex structure and loading are modelled numerically, using several simplifying assumptions. On the other hand, the most commonly used tower geometries, when the truss solution is adopted, possess structural mechanisms that compromise the assumed structural behaviour. In present study, structural analysis based on a less conservative solution, for the steel tower design considering all the actual structural forces and moments. A modelling strategy combining three-dimensional beam and truss finite elements is proposed. In tower models the main members such as legs use beam elements while the bracing system utilizes truss elements. The linear and nonlinear analyses of tower are carried out for obtaining the performance of bracing systems. The TOWER program used in this study to evaluate the structural performance of bracing system. The towers have been modelled in 3D using TOWER program.

4.1.3. Design Loads

Various types of loads have been calculated accurately depending on the design parameters. The gravity loads are almost fixed, since these are dependent on the structural design. In the load calculation the wind plays a vital role. The correct assessment of wind will lead to proper load assessment and reliable design of tower structure. Maximum wind pressure is the chief criterion for the design of lattice towers.

4.1.4. Load Combinations

Differing external loads acting simultaneously on the supports of towers are combined to load cases in an adequate manner. These combinations of actions need to comply with the requirements concerning reliability, security and safety. The load cases should take care of all loading conditions to be expected during construction and during the whole life period of towers such that damage will be unlikely. In many standards for telecom towers are distinguished between normal and exceptional.

4.1.5. Results Summary

i. Rectangular Based Results

The linear and nonlinear analyses results are presented in Table 2 [1]. In both linear and nonlinear analyses the critical loading is obtained for X and XB structure type for 60, 50 and 40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°) is critical, for XB and YD with the same failure containment load case is critical. All critical members are in compression. The weight of towers designed based on linear and nonlinear analyses are the same, table 2. Minimum weight of tower for 60 m, 50 m and 40 m heights obtained for the KD and YD bracing system, On the other hand, the heaviest tower for 60 m, 50 m and 40 m heights obtained for the XB and Y bracing system. The best performance for linear analysis of sway values in degree is obtained for X and XB bracing for 60 m, 50 m, and 40 m heights are 0.84, 0.52 and 0.56 degrees. The worst performance is return to KD and D bracing with value 1.2 degrees for 60 m, for 50 m height is obtained for Y bracing system is 1.09 degree and for 40 m height YD and D bracings have a value of 0.73 degrees.

Table 2. *Linear and nonlinear analysis results of rectangular base towers.*

Type	H (m)	SFF (Hz)	Weight (N)	Linear Analysis				Non-linear Analysis			
				% MEU	El. Type	OOP (cm)	Sway (deg)	% MEU	El. Type (deg)	OOP (cm)	Sway
	60	1.2262	75032.6	97.58	Leg	64.6	1.17	98.01	Leg	65.26	1.18
K	50	1.7565	56188.8	93.93	Brac.	41.87	0.93	95	Leg	42.26	0.94
	40	2.2696	39495.9	94.71	Leg	28.64	0.71	94.65	Leg	28.88	0.71
	60	1.1339	60868.1	99.01	Brac.	67.37	1.2	99.34	Leg	67.85	1.21
	50	1.784	46352.5	96.08	Brac.	38.82	0.82	95.67	Brac.	39.02	0.82
KD	40	2.2804	31003.7	96.19	Brac.	27.35	0.69	95.5	Brac.	27.47	0.7
	60	1.197	90008.9	97.12	Brac.	65.83	1.16	N. G.	N. G.	N. G.	N. G
	50	1.60641	59951.7	97.15	Leg	41.73	1.09	96.98	Leg	42.17	1.11
Y	40	2.0876	43606.4	95.58	Leg	29.41	0.73	95.42	Leg	29.75	0.74
	60	1.1349	62683.1	97.4	Brac.	64.81	1.17	97.62	Leg	65.3	1.18
	50	1.7833	47033.9	98.9	Leg	38.87	0.8	98.9	Leg	39.1	0.81
YD	40	2.2802	30876	98.06	Brac.	26.71	0.68	97.33	Brac.	26.83	0.68
	60	1.1359	63642.3	98.79	Brac.	66.4	1.2	98.44	Brac.	66.83	1.2
	50	1.7847	47585.4	97.05	Leg	40.05	0.83	97.35	Leg	40.23	0.84
D	40	2.0626	32702.9	94.22	Leg	28.25	0.73	95.38	Leg	28.39	0.74
	60	1.2214	145853	99.06	Leg	53	0.98	99.55	Leg	53.29	0.99
	50	1.756	131852	93.86	Leg	28.34	0.55	94.22	Leg	28.43	0.55
XB	40	2.2693	33057.7	93.42	Leg	23.52	0.56	93.74	Leg	23.6	0.56
	60	1.1503	122436	93.6	Leg	51.12	0.84	91.91	Leg	51.45	0.84

Type	H (m)	SFF (Hz)	Weight (N)	Linear Analysis				Non-linear Analysis			
				% MEU	El. Type	OOP (cm)	Sway (deg)	% MEU	El. Type (deg)	OOP (cm)	Sway
X	50	1.7441	109958	89.26	Leg	28.34	0.52	88.58	Leg	28.43	0.52
	40	2.2448	38507.9	96.74	Leg	24.79	0.61	97.32	Leg	24.87	0.62

ii. Triangular Based Tower

The linear and nonlinear analyses results are presented in table 3 [1]. In both linear and nonlinear analyses the critical loading is obtained for D and XB structure type for 60, 50 and 40 m height. Failure containment loads case 1.2D (dead load)+1.6Wo (wind load applied to tower with an angle of 90°) is critical for 60 m height D bracing system and for 50 m height D bracing system. For 40 m height Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) for XB bracing system. X and K bracing the failure containment load case for 60 and 50-40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 90°) is critical for 60 m height X bracing system and Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) are critical for both 50 and 40 m height of K bracing system, table 3. All critical members are in compression. The weight of towers designed based on linear and nonlinear analyses are the same. According to table 3 [1], the minimum weight of tower for 60 m and 40 m heights are obtained for X bracings and for 50 m height is obtained for XB bracing system on the other hand, the heaviest tower for 60 m, 50 m and 40 m heights obtained for the K bracing system. The best performance for linear analysis of sway values in degree is all obtained for K bracing for 60 m, 50 m and 40 m heights is 0.31, 0.2 and 0.24 degree. The worst performance is return to X bracing with value 0.52 and 0.31 degree for 60 m and 40 m heights. XB bracing has a value of 0.4 degree for 50 m height.

4.1.6. Study Conclusions

The result of the linear and nonlinear analysis clarify that bracing systems used in tower show different structural behaviour. Such as, if the height of the tower is less than 50 m the difference between linear and nonlinear analysis is negligible. The smallest weight is obtained at KD and YD type bracing systems for rectangular base towers. In case of triangular base towers X and XB type bracing systems give the smallest weight design. The best performance according to sway values are obtained at X and XB type bracing system for rectangular and K-bracing system for triangular base towers.

Table 3. *Linear and nonlinear analysis results of triangular base towers.*

Type	H (m)	SFF (Hz)	Weight (N)	Linear Analysis				Non-linear Analysis			
				MEU %	El. Type	OOP (cm)	Sway (deg)	MEU %	El. Type	OOP (cm)	Sway (deg)
K	60	1.1785	242048.3	95.99	Leg	18.22	0.31	96.63	Leg	18.40	0.32
	50	1.6582	207310.9	96.37	Leg	10.51	0.20	96.84	Leg	10.59	0.20
	40	1.8397	78979.2	96.27	Leg	10.66	0.24	96.57	Leg	10.66	0.25
D	60	1.1786	97460.8	85.85	Brac.	31.39	0.49	85.14	Brac.	31.66	0.49
	50	1.6947	59835.2	87.69	Brac.	14.15	0.28	85.99	Brac.	14.22	0.28
	40	1.9285	41423.7	79.21	Leg	15.80	0.40	79.63	Leg	15.88	0.40
XB	60	1.1391	82423.8	94.13	Brac.	28.05	0.47	90.08	Brac.	28.25	0.47
	50	1.7006	56489.1	82.43	Brac.	19.79	0.40	82.87	Brac.	19.88	0.41
	40	1.9123	44851.6	77.32	Leg	14.73	0.35	72.94	Leg	14.80	0.35
X	60	1.1255	73716.2	97.64	Brac.	35.45	0.52	97.77	Brac.	35.70	0.53
	50	1.6780	61513.5	83.00	Brac.	18.01	0.31	82.78	Brac.	18.09	0.31
	40	2.0448	34669.0	94.43	Leg	18.68	0.42	94.78	Leg	18.75	0.42

4.2. Square Lattice Tower

In (2010) India, Siddesha H., represented in his research of "Wind Analysis of Microwave Antenna Towers" [2, 8, 9], as shown in figure 6 of open latticed steel towers with angle sections are commonly used in microwave antenna towers presents the static and gust factor method (GFM). The comparison is made between the tower with angle and square hollow section. The displacement at the top of the tower is considered as the main parameter. The analysis are also done for different configuration by removing one member as a present in the regular tower at lower panels.

Figure 6. *Square lattice tower with angle sections.*

4.2.1. Modeling and Analysis of Tower

The modeling and analysis of tower have been done by using ANSYS software. The members of the tower are modeled by using BEAM 188 element. Several authors have done the experimental and analytical investigations by using various finite element software's.

4.2.2. Material Properties

The most widely used commercial structural material low carbon steel (C14) with Density 7870kg/m^3, Tensile strength (yield) 415 MPa, Modulus of Elasticity 200 GPa, has been Selected for the study. The chemical composition of the section used in the present analysis.

4.2.3. Tower Configuration and Sections

40 m height tower of square in plan have been considered which is having a base width of 4 m reduces to 1.91 m at the top [2]. The analysis has been done for the following sections in regular tower configuration for the entire tower as shown in Figure 7 [2]. The sections adopted for this configuration are as below,

a) A tower with Leg and bracing members as angle Sections (L-A & B-A).
b) A tower with Leg members as Square Hollow section and bracing members as Angle Sections (L-S & B-A).
c) A tower with Leg and bracing members as Square Hollow Sections (L-S & B-S).

The total weight of the tower kept nearly constant for all these sections. The wind load has been calculated using static method and GFM. The calculated values have been applied on the tower. The analysis is also done for different configuration with different sections at bottom first, Second and both the panels. The remaining bracings in panels (that is from 3rd to 14th) are kept constant in terms of configuration and sections as in regular tower. The sections adopted for leg members are similar as explained above, but for Bracing member the sectional dimensions were changed. In the present work X, X Horizontal, X and M bracing have been used.

Figure 7. *Views of Regular Microwave Tower 3D.*

4.2.4. Boundary Conditions and Loading

All the tower configurations used are assumed as rigidly connected at the base and all degrees of freedom at the bottom nodes are restrained. Figure 8(a) [2] shows the Panels considered for the calculation of wind loads. Figure 8(b) [2] shows the variation of wind loads at different panels. For the calculation of wind loads by static method the following parameters were considered as per IS: 875 (part 3) -1987. Wind speed- 55m/s, Risk coefficient (k_1) -1.08, Terrain, height and structure size factor (k_2) category 2 and class B (assumed), Topography factor (k_3) -1 (assumed).

4.2.5. Study Conclusions

The analysis of microwave antenna tower with different sections and configurations were done for wind loads. The following conclusions may be drawn from the analytical results.

1). Square hollow sections can be used more effectively in leg members in comparison with the angle sections in regular tower under static and GFM.
2). Square hollow Sections used in bracings along with the leg members do not show much reduction of

displacement compared to tower with SHS sections used in Leg members under static and GFMs.

3). X and M bracing in square hollow Sections for legs and bracings at the lower first Panel shows a maximum reduction of displacement compared to the regular tower with angle sections under static and GFMs.

4). X and M bracing in SHS for legs and bracings at the lower first panel shows a maximum reduction of displacement in comparison with the tower with SHS for legs and bracings in lower second, lower first and second Panels with different configurations in both static and GFM.

The antenna loads have been calculated as 3 m diameter paraboloid type antenna without radome is considered in the analysis. It is assumed that the antenna is mounted at a height of 40 m (that is top of the tower) on one of the leg member facing normal to the direction of wind. The wind incidence angle for the antenna is assumed as zero degree. The gust factor is taken as unity. The wind force along the direction of the wind is obtained as 25044.06 N. This antenna load is used in both the methods and is applied for all other configurations.

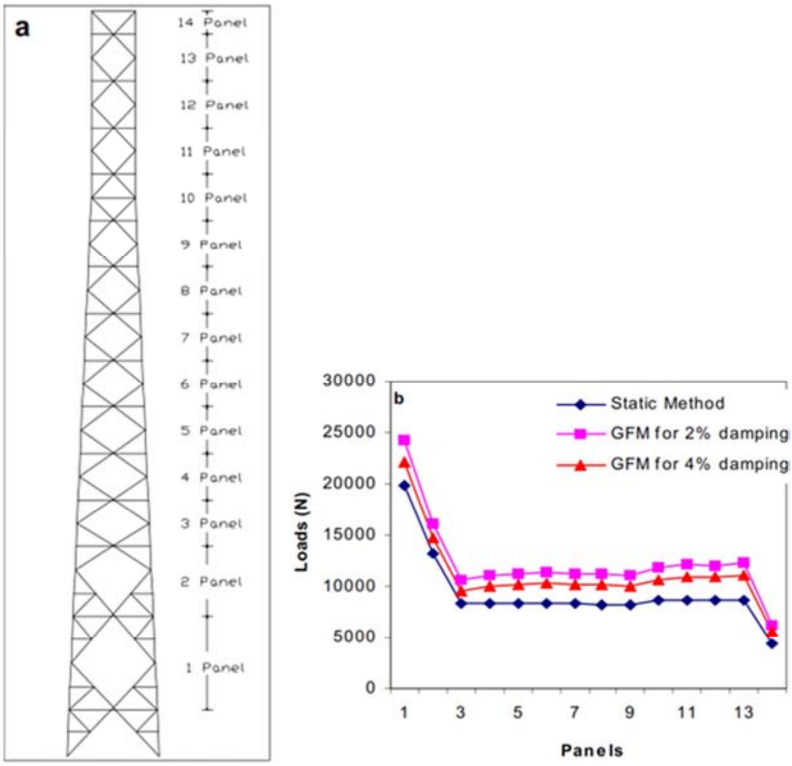

Figure 8. Wind load calculation.

a. Panels of regular microwave tower,
b. Variation of loads at different Panels

4.3. Monopole Tower

In November (2015) Riya Joseph & Jobil Varghese, represented a study about "Analysis of Monopole Communication Tower" [3, 8, 9, 13], figure 9. Where Modern telecommunication structures are essential to the present society. The emergence of new technologies creates demand for additional facilities and introduction of new elements into the cities. Their vast selection of communications poles is designed and manufactured for durability, wear, corrosion resistance, and visual appeal. Monopoles are polygonal sectioned and hot dip galvanized hollow steel structures. All accessories for onsite assembly are bolted, consequent body sections are either slip jointed or bolted. Base plates, flanges and accessories are welded to the sections. Monopole towers can support all the equipment, antennas and utilities similar to that of the conventional lattice tower.

Figure 9. Monopole Communications Tower.

4.3.1. Structure Modelling

ANSYS software have been used for the structural analysis. Tower is to be designed in such a way that the antennas can be placed at certain elevations. Signal transmission should not be obstructed in any case. Tapered tubular tower with diameter increasing towards the base have been applied [3].

4.3.2. Tower Material

ASTM 572 have been used, Table 4 [3], where is most commonly used material in towers. It is a high strength, low alloy steel that finds its best application where there is need for more strength per unit of weight.

Table 4. *Material Properties.*

ASTM 572 STEEL	
Modulus of Elasticity	2.1 x 10^{11} N/m^2
Yield stress	350 MPa
Poisson's Ratio	0.3
Density	7850 kg/m^3

4.3.3. Tower Dimensions

The monopole to be modeled is a tubular steel pole of 40m height. The main shaft of the monopole is having the shape of a 20 sided regular polygon. Diameter of the shaft is 900 mm at the bottom and 500mm at the top. Thickness of the section is adopted in such a way that the analysis results in minimum deflection without increasing the volume and cost of material.

4.3.4. Boundary Conditions

The supporting conditions of the tower was assumed that the tower is rigidly attached to the ground, fixed – free boundary condition is applied i.e. tower is fixed at the base and free at the top.

4.3.5. Meshing

To achieve high accuracy, the meshing of the element should be fine as possible, Figure 10 [3].

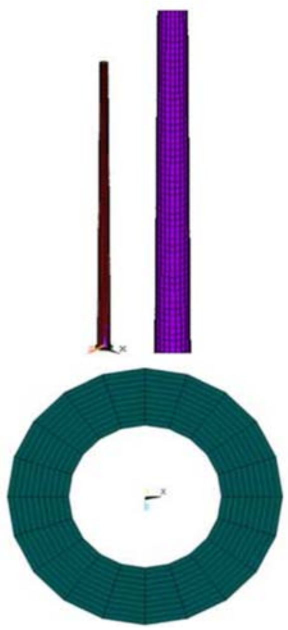

Figure 10. *Meshed & Top View of the Model.*

4.3.6. Loading

For the analysis of the communication tower wind and seismic loads are considered along with antenna loads. The forces exerted on a structure by wind depend on the size and shape of the structural members in the path of wind and the speed on which the wind is blowing. The wind force acting on any structure is the sum of wind forces acting on its individual parts. The design wind speed is calculated taking into account the terrain type, height of the structure, topography, and risk level for the structure.

4.3.7. Gust Factor Method

The wind load on monopole is calculated based on IS: 875 (Part 3) - 1987. The following design parameters are used for calculating the wind loads:

Basic Wind Speed: 33 m/s

a) Risk coefficient kl=1.06
b) Terrain Category: 2, Class: B
c) Topography factor k3=1.0

The wind Loads as in table 5 [3]

Table 5. *Wind Loads.*

Height (m)	V_b (m/s)	k_2	V_z (m/s)	P_z (kN/m^2)	G x P_z (kN/m^2)
40	38	1.125	45.32	1.23	2.46
36	38	1.125	44.91	1.21	2.42
32	38	1.105	44.5	1.19	2.38
28	37	1.09	42.75	1.09	2.18
24	37	1.07	41.97	1.06	2.12
20	36	1.05	40.07	0.96	1.92
16	36	1.026	39.15	0.92	1.84
12	36	0.996	38	0.87	1.74
8	33	0.98	34.28	0.71	1.42
4	33	0.98	34.28	0.71	1.42

4.3.8. Antenna Load

Both the pole and lattice structures are subjected to same antenna loads and the deflection behavior is compared. There are 4 nos. of GSM antennae of size 2.6m x 0.3m and 4 nos. of CDMA antennae of size 2.5m x 0.26m. The wind load due to these antennae on the pole and lattice structure is calculated based on the exposed area of the antenna, Table 6 [3].

Table 6. *Antenna Loads.*

Item	Quantity	Size (m)	Weight (kg)	Location from base (m)	Total load (kN/m^2)
CDMA	2	0.26 x 2.5	20	40	0.615
CDMA	2	0.26 x 2.5	20	36	0.615
GSM	2	0.3 x 2.6	25	34	0.641
GSM	2	0.3 x 2.6	25	28	0.641

4.3.9. Study Conclusions

These have smaller plan dimension and are composed of only few components. These are more economical considering the cost of land. Structure was modeled in ANSYS. Load calculations were done as per IS codes. Gust factor method was adopted in order to include the dynamic effects.

Displacements and stresses were obtained within the permissible limits. Variation in the results with change in thickness was studied. Wind effect was studied by analyzing the same structure to an increased wind load. Towers of two different heights were taken for the study.

4.4. Guyed Mast Tower

In April – June (2007), Marcel Isandro R. De Olivera and his structural team, represented a study about the "Structural Analysis of Guyed Steel Telecommunication Towers for Radio Antennas" [4, 8, 9, 12], figure 11 proposes an alternative structural analysis modelling strategy, based on a less conservative model combining 3D beam finite elements in the main structure and 3D truss elements in the bracing system and eliminating the use of dummy bars present in the traditional analysis, Further comparisons of the two above mentioned methods and another design alternative only using 3D beam finite elements on three existing guyed steel telecommunication towers (50m, 70m and 90m high) are described. The comparison is focused on the tower structural response in terms of displacements, bending moments, stresses, natural frequencies, and buckling loads.

Figure 11. *Telecommunications Guyed tower.*

4.4.1. The Structural Modelling

Several authors have contributed with theoretical and experimental investigations to access the best modelling strategy for steel telecommunication towers. The main purpose of the adopted modelling strategies was to investigate the structural behavior of guyed steel towers, preventing the occurrence of spurious structural mechanisms that could lead to uneconomic or unsafe structure. The towers investigated in the present paper (50m, 70m and 90m), have a truss type geometry with a square cross section. Hot rolled angle sections connected by bolts compose the main structure as well as the bracing system. Pre-stressed cables support the main structure, which must be always in tension. Some of these cables are linked to a specific set of bars arranged to improve the system torsional stiffness. The geometry configuration of the three guyed towers are depicted in Figure 12 [4].

This investigation considered as acting vertical loads: structure self-weight, stairs, antennas, cables, etc. The steel tower wind effects were the main horizontal loads. These horizontal loads were calculated according to the procedures described on the Brazilian code NBR 6123 (NBR 6123, 1988) and applied to the guyed tower nodes. Two wind load cases related to actions perpendicular and diagonal to the towers face were considered in this analysis. The adopted guy pre-stress loads were in accordance to the values described in the Canadian Code CSA S37-94 (CSA S37-94, 1994). A lateral view of the tower and its corresponding idealized structural model are presented in Figure 13 [4] It can be shown that the idealized structural model cannot only be represented by 3D truss finite elements.

All the above-mentioned 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 best-recommended 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. The used 3D truss finite element is presented in Figure 14 [4]. This structural modelling strategy is characterized by the use of 3D beam finite elements with rigid connections. The adopted beam finite elements presented six degrees of freedom per node associated with translation and rotation displacements in space, respectively, as shown in Figure 14.

When all the structural modelling strategies investigated are compared, the beam element modelling is the easiest to use. This conclusion is mainly justified by the fact that the adoption of dummy bars to prevent possible mechanisms is not required in the beam modelling strategy. Another advantage is the computational model uniformity since all the adopted bars are represented by a single finite element type (3D beam). Despite all the mentioned structural modelling advantages the model final results should be carefully checked. This is due to the fact that in principle, the rigid connections adopted in this strategy can lead to some disturbing and/or spurious effects, especially when the tower critical buckling loads are considered.

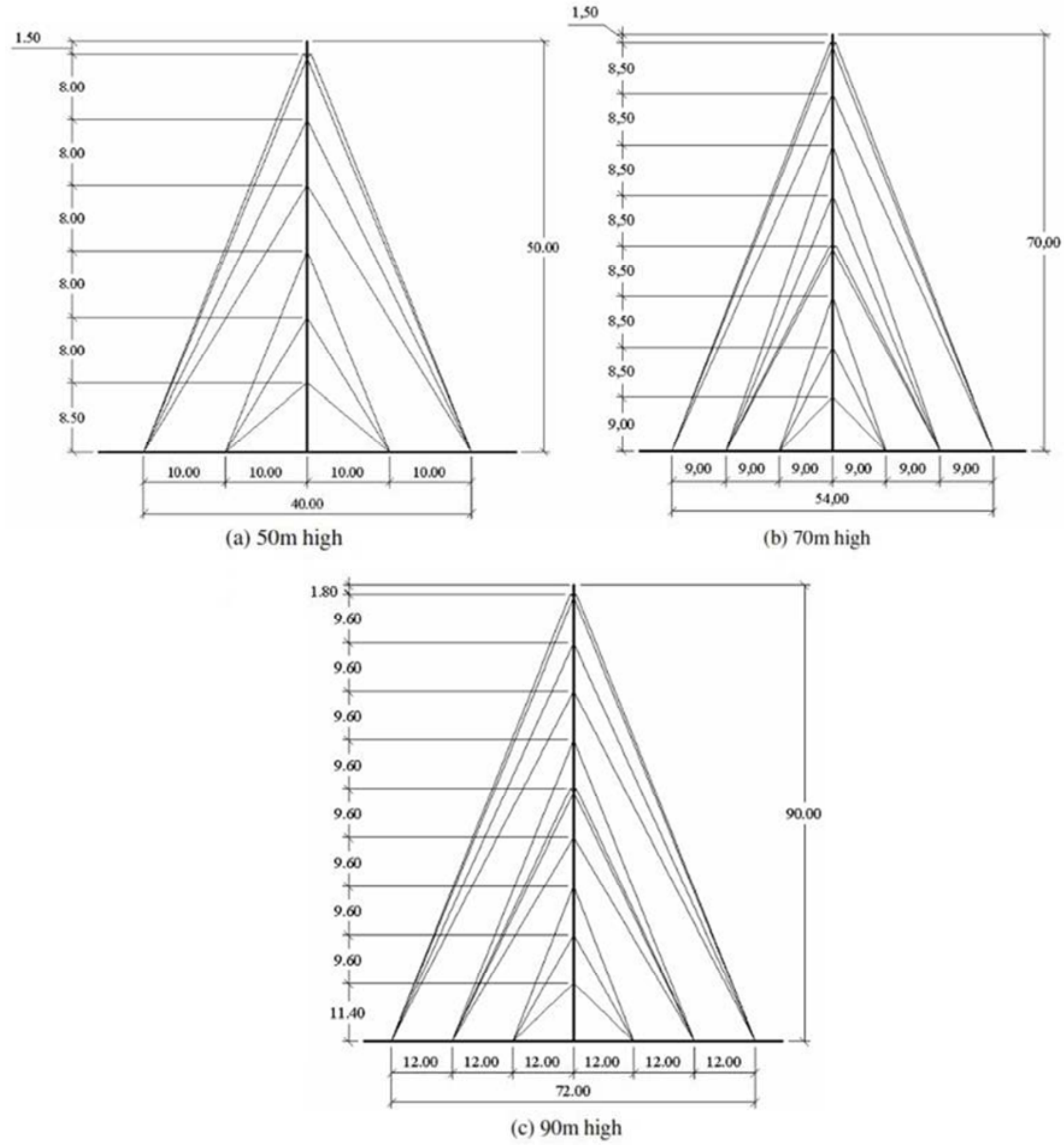

Figure 12. *Towers basic geometric data.*

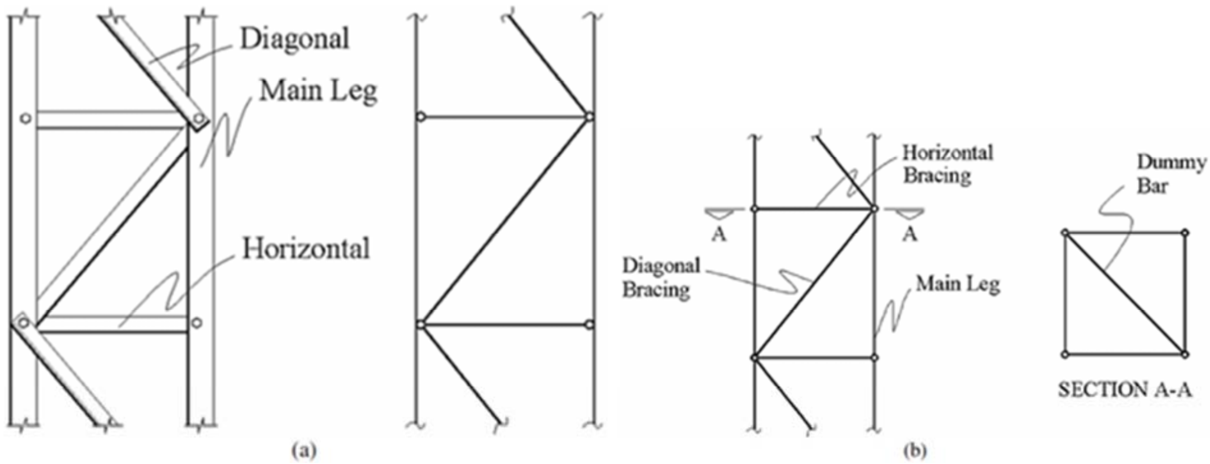

Figure 13. *(a) Tower main structure and; (b) associated idealized model and dummy bars location.*

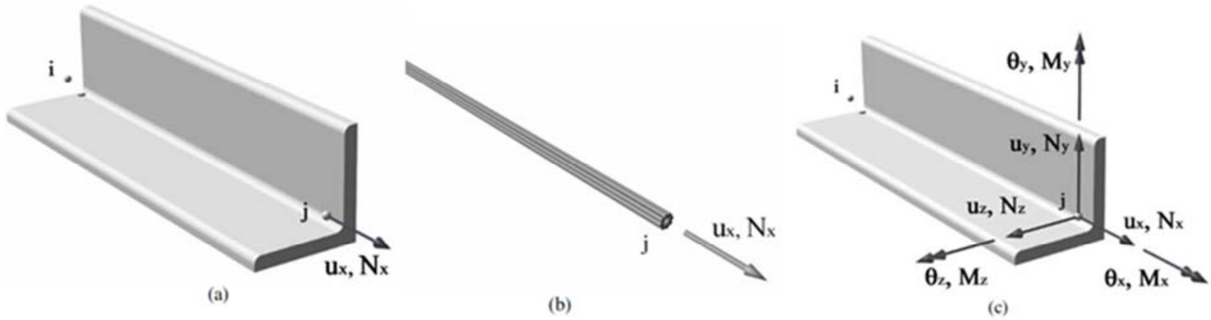

Figure 14. *(a) Adopted spatial truss, (b) cable (tension only) and (c) spatial beam finite elements.*

4.4.2. Static Analysis

Table 7 [4] present linear static analysis results for the investigated guyed towers (50m, 70m and 90m high), according to the 3 earlier mentioned structural models. Maximum values of stresses and horizontal displacements are presented and compared.

The maximum stress points are depicted for the mixed beam and truss element model considering the perpendicular wind load case. The maximum stresses, caused mainly by bending effects, were associated, in all cases studied, to the towers base members. On the other hand, the lateral displacements values were generally not significantly changed when the simple truss model (Strategy I), the beam model (Strategy II) or the combined beam and truss model (Strategy III) were considered, Tables 7 to 9 [4].

Table 7. *High steel towers maximum stresses and horizontal displacements.*

Modeling Strategies: I- Truss Element II – Beam Element III – Combined Beam and Truss Element					
Models	**Tower Height (m)**	**Perpendicular Wind**		**Diagonal Wind Direction**	
		$\sigma_{max.}$ **(MPa)**	U_{max} **(mm)**	$\sigma_{max.}$ **(MPa)**	U_{max} **(mm)**
I		83.8	0.049	78.6	0.025
II		344.7	0.049	318.4	0.026
III	50	357.0	0.049	330.3	0.024
Difference (I and III)		76.5%	None	76.2%	3.8%
Difference (II and III)		3.4%	None	3.6%	7.7%
I		74.7	0.089	66.4	0.039
II		411.6	0.093	378.9	0.044
III	70	425.7	0.093	392.2	0.044
Difference (I and III)		82.5%	None	83.1%	None
Difference (II and III)		3.3%	None	3.4%	None
I		83.4	0.090	74.2	0.041
II		388.8	0.099	360.7	0.049
III	90	398.5	0.099	369.9	0.049
Difference (I and III)		79.1%	9.1%	79.9%	16.3%
Difference (II and III)		2.4%	None	2.5%	None

Table 8. *High steel towers natural frequencies.*

Modeling Strategies: I- Truss Element; II – Beam Element; III – Combined Beam and Truss Element						
Models	**Tower Height (m)**	**Natural Frequencies f_{0i} (Hz)**				
		f_{01}	f_{02}	f_{03}	f_{04}	f_{05}
I		3.420	3.420	4.203	4.203	5.360
II	50	4.142	4.142	5.124	5.124	5.504
III		2.609	2.698	2.698	2.731	4.000
I		2.616	2.616	3.783	3.783	4.233
II	70	3.016	3.016	4.225	4.225	4.781
III		3.015	3.015	4.222	4.222	4.779
I		2.497	2.497	3.151	3.151	3.420
II	90	2.903	2.903	3.634	3.634	3.812
III		2.902	2.902	3.633	3.633	3.806

Table 9. *High steel towers buckling loads.*

Modeling Strategies: II – Beam Element III – Combined Beam and Truss Element							
Models	**Tower Height (m)**	**Wind Direction**					
		Perpendicular			**Diagonal**		
		Buckling Loads λ_{0i}					
		λ_{01}	λ_{02}	λ_{03}	λ_{01}	λ_{02}	λ_{03}
II	50	10.114	10.306	11.164	10.566	11.063	11.142
III		5.520	5.859	6.118	10.526	10.570	10.630
II	70	14.568	14.810	16.085	15.676	16.079	16.205
III		11.245	11.499	11.648	13.350	13.501	13.522
II	90	11.066	11.121	11.946	11.617	12.354	12.440
III		8.3721	8.4425	8.5999	9.2856	9.3311	9.3617

4.4.3. Stability Analysis

Table 9, presented the first three buckling load factor for the investigated tower structures [4]. The stability analysis considered the load actions related to perpendicular and diagonal wind load combinations and the last two already mentioned finite element modelling strategies (beam elements and mixed beam and truss elements). As expected the results clearly indicated the significant influence of the bracing system finite element modelling strategy over the tower critical loads. Critical loads evaluated according to second methodology (beam elements) are substantially higher than the proposed combined strategy. The lower critical factors are always associated with the perpendicular wind load case, which can be associated with the instability failure control. These buckling loads are not associated with usual design practice and, if adopted, could lead to unsafe structures.

4.4.4. Study Remarks

This paper proposes an alternative structural analysis modelling strategy, based on qualitative and quantitative comparisons, for guyed steel towers. The proposed methodology less conservative than traditional analysis methods, uses a combined solution of three-dimensional beam and truss finite element to model the structural behavior of 3D tower structures under several loading conditions. Generally, in all the cases studied the maximum stress values for the structural tower modelling based on the three investigated methodologies were significantly modified. On the other hand, the lateral displacement values were not significantly changed when the usual truss model, the beam model or the combined beam and truss model were considered. Based on the difficulties found in the analyzed guyed steel towers, present in current engineering design practice, and corroborated by the nature of the 3D truss finite element, an analysis only using this element cannot be indicated. This method also implies in the utilization of a great number of dummy bars to prevent the occurrence of structural mechanisms. This fact increases the amount of work to model the structure and generates a potential error source if the rigidities and/or number of dummy bars were not properly considered.

5. Conclusions

According to the reviewed alternatives of previous studies and/or constructions related to the Telecommunications towers as have been summarized in this paper and concluded several more common used types of towers in the field of telecommunications like the self-supporting towers (Triangular, Square and rectangular based towers), Monopole towers and Guyed mast towers where the decision could be taken according to that in order to select the most proper alternatives that could be investigated later to serve specific aim and purpose. The aim of the study is to investigate the possible towers type relevant to the design and specified location requirements, where a telecommunication tower need to be designed and allocate in a rural zone in Hungary near of the Capital city Budapest with fixed height 36 m required and the construction area availability depending on the design situation and recommendations based on the Eurocode regulations taking into account three important aspects (Economical, Statical and Aesthetical aspects) with a deflection limit at the top of the tower to be not more than 0.5 degree as torsional effect.

The alternative based on the mast guyed tower has a good valuation in case of the high towers for more than 50 m height to be more suitable and economical solution, but according to the present required tower height 36 m it would be not the best solution especially for the large construction area required and several footing to be able to support the tower cables which make it non conservative solution, Therefore this alternative could not be investigated in case of limited area and small heights.

The alternative based on the Monopole tower Concept has a relatively low score. This is mainly due to the architectural value, technical, assembly, transporting and the manufactural difficulties adding to that the cost of plate as a shell element material. The relatively large foundation required and the foundation difficulties therefore, this alternative could be investigated from Aesthetical point of view.

The alternatives of the Triangular lattice tower and the Square lattice tower more or less the same could be elaborated and investigated by applying the preliminary design process according to the Eurocode requirements and specifications.

The triangular based tower with CHS sections would be used and K-bracings system because it has the most efficient sway resistance. The main advantage of the Triangular lattice tower structure is the resemblance to the architectural design and the smallest construction area require.

The apparently more efficient stiffness of the Square lattice tower due to the usability of 4 legs to resist the applied actions compensates this difference where angle cross sections would be used and single web bracing system (V-bracing).

According to the summarized conclusion of the previous studies where it proofs that in case of the Lattice towers with less than 50 m height the nonlinear analysis could be negligible, then according to the present case of tower height for 36 m height only linear analysis could be considered.

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