Influence of modeling in the response of steel lattice mobile tower under wind loading

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ABSTRACT

These days there is an unprecedented rise in the number of lattice towers due to an ever increasing demand in communication. Lattice towers are 3D space frames that for design are conventionally analyzed as 2D trusses. For safety and economy, these designs need to be more rigorously analyzed considering them as 3D frames. In this study, two lattice towers of heights 18m and 40 have been analyzed by modeling them by three different structural idealizations namely, as 3D frame, 3D truss and as a hybrid of the two. The wind has been taken as the primary force for the analysis and using Gust factor method, the joint displacements, member forces and maximum stresses have been compared to find out the effect of the difference in the modeling strategy on the design forces acting on a latticed communication tower. 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 re-design of base members.

Keywords: Lattice communication towers; Structural idealizations; 3DFrame model; 3DTruss model; Hybrid model; Wind analysis; Gust Factor Method.

1. INTRODUCTION

India has more than 250,000 cell phone towers at present and according to an estimate is expected to have mobile towers just double this number by 2015, making it one of the fastest growing telecommunication market. In most parts of the world similar development in the telecommunication can be observed. The rate at which this growth in communication is gathering momentum proportionality is increasing the demand for the production of steel telecommunication towers.

The communication towers are often designed as 3D trusses, which is not the actual representation of the structure. In the traditional stress calculations based on linear elastic ideal truss analysis, members are assumed to be concentrically loaded and pin-connected.

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.

2. WIND LOADS AND ANALYSIS OF LATTICE TOWERS

For evaluating the dynamic response of lattice towers, Indian code of standard (and most of other codes worldwide) recommends the use of GFM or Gust effectiveness factor method (GEFM). The structural loads produced by wind gusts depend on the size, natural frequency and damping of the structure in addition to the inherent wind turbulence. These loads are applied as the equivalent static loading on the structures. The gust factor is a function of wind, terrain and structural characteristics.

According to the GFM given in IS: 875(part-3), 1987, the along wind load on a structure on a strip area (A_e) at a height (z) is given by [8]:

$$F_z = C_f * A_e * p_z * G$$

Where,

C_f	=	Force coefficient of the structure,
A_e	=	Effective frontal area considered for the structure at height z,
p_z	=	Design pressure at height z due to hourly mean wind obtained as $0.6V_z^2$ (N/m^2)
G	=	Gust factor

All notations are as per IS: 875 (part-III).

The GFM considers wind forces as equi-static forces, therefore not truly accounting for the dynamic effects of the wind. However, it is well accepted practice to take into account the dynamic effects of the wind for slender and open structures like tall mobile towers by adopting GFM.

3. MATERIAL AND METHODOLOGY

In this study an 18m and a 40m high ground supported latticed mobile tower, located at a highly developed area in New Delhi (NCR) were modeled in STAAD.Pro 2007 to obtain the structural response of a ground supported latticed mobile tower. The present analysis involves the modeling of the tower as

a) Rigid frame,
b) Truss and
c) Combined truss and frame.

The angle section tower members conform to IS:802 (Part1/Sec1)-1995 and IS:802(Part1/Sec2)-1992. The wind effects over the steel towers were the main horizontal loads considered in the structural analysis.

4. STRUCTURAL SYSTEM IDEALIZATIONS

The three models adopted in this study are discussed below in brief:

a) Model I or Rigid Space Frame model: In this case, members were considered as rigid jointed members.

b) Model II or Space Truss model: In this model, the tower was idealized as a space truss with all members taken to be hinged permitting in-plane rotation.

c) Model III or Combined or Hybrid model: In this model, the main leg members were rigid jointed, while the bracings were considered to be hinged.

The member sections and other details are as follows:

1) Ground Supported Mobile tower (18 m)

Height of tower	18m
Height of straight portion at top of tower	15m
Height of inclined portion	3m
Base width	1.8m
Top width	1m
No. of 1.5m high panels	2 NOS
No. of 3.0m high panels	5 NOS

Antenna particulars:

a) 6 numbers of $0.26*2.6m^2$ CDMA antennae weighing 20 kg each, at a height of 17m from the base.

b) 2 numbers of 0.3m diameter Microwave antennae weighing 25 kg, at a height of 15m from the base.

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c) 1 Microwave antenna of 1.2m diameter weighing 77 kg, and 1 Microwave antenna of 0.6m diameter weighing 45 kg, at a height of 15m from the base.

Other tower accessories' particulars:

A platform weighing 0.82kN/m provided at a height of 16m from the tower base.

2) Ground Supported Mobile tower (40m)

Height of tower	40m
Height of straight portion at top of tower	30m
Height of inclined portion	10m
Base width	4m
Top width	2m
No. of 5m high panels	8 NOS

Antenna particulars:

a) $0.26*2.6m^2$ CDMA antennae weighing 20 kg each, at a height of 38m from the base.

b) 0.3m diameter Microwave antennae weighing 25 kg, at a height of 35m from the base.

c) 1.2m diameter Microwave antenna weighing 77 kg, and 0.6m diameter Microwave antenna weighing 45 kg, at a height of 35m from the base.

Other tower accessories' particulars: A platform weighing 0.82kN/m provided at a height of 36m from the tower base. The member details of towers at various levels are shown in table 1 and 2.

Table 1: Member Details of 18 m Angle section Lattice Mobile Tower

Sl No	Elevation	Member Description	Section
1	0-6	Leg Member	ISA100X100X10
		Bracings	ISA 65X65X5
2	6-18	Leg Member	ISA 80X80X8
		Bracings	ISA 45X45X5

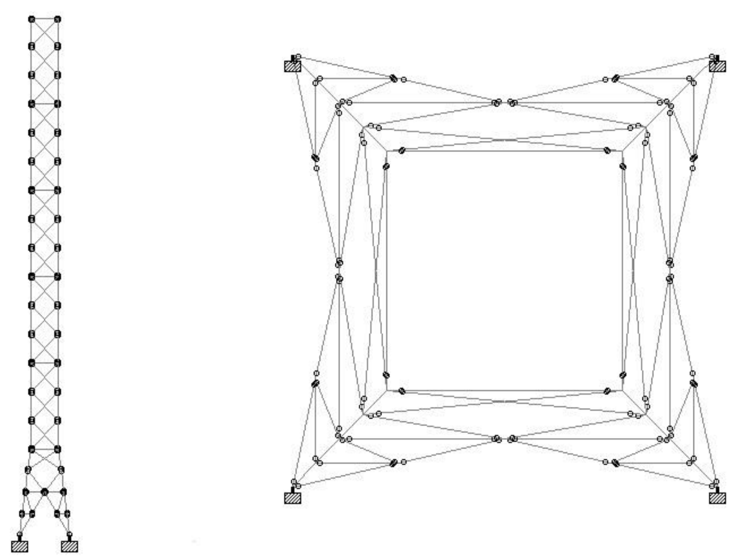

FIGURE 1: 18m tower a) Geometric layout b) Top view

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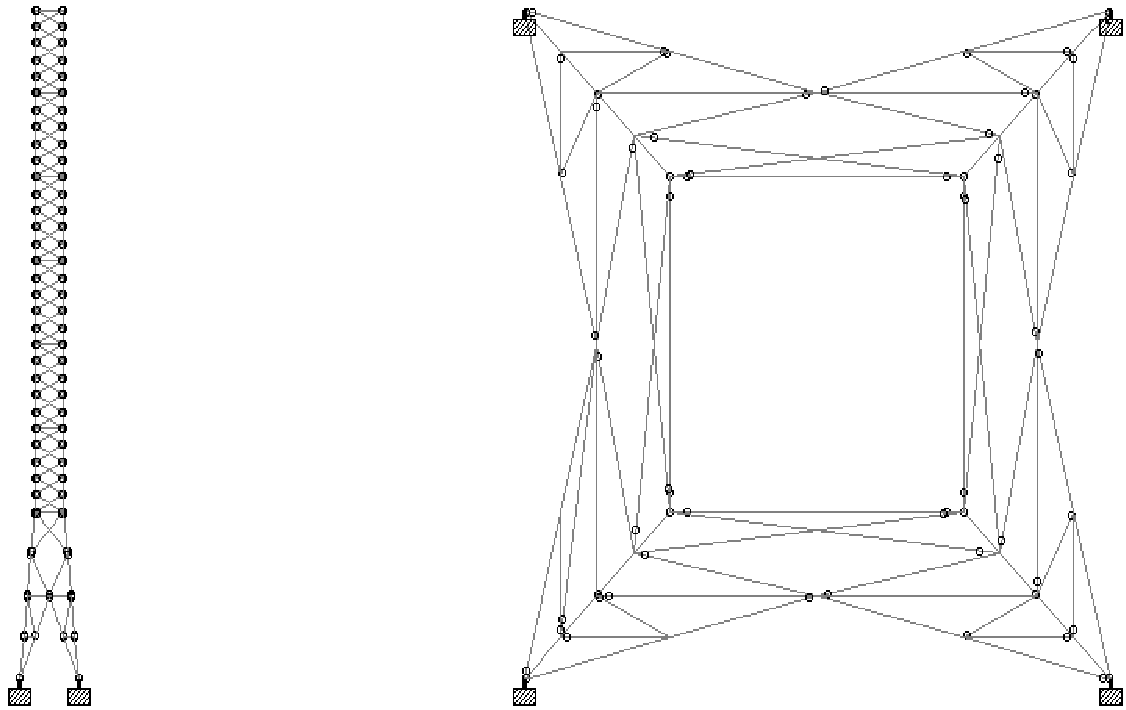

FIGURE 2: 40m tower a) Geometric layout b) Top view

Table 2: Member Details of 40 m Angle section Lattice Mobile Tower

Sl. No.	Elevation	Member Description	Section
1	0-5	Leg Member	ISA100X100X10
		Bracings	ISA 65X65X5
2	5-10	Leg Member	ISA100X100X10
		Bracings	ISA 65X65X5
3	10-40	Leg Member	ISA 80X80X8
		Bracings	ISA 45X45X5

For the calculation of the wind loads by the gust factor method, the following parameters were considered [8]:

Wind zone = Zone IV, Basic wind speed, V_b = 47m/s, Risk Coefficient factor, k_1*= 1.07
*Considering design life of 100 years.
Topography factor, k_3 = 1.00

Table 3: Wind loads acting on 18m tower

Elevation(m)	k_2’	Design Velocity(m/s)	Design Pressure(N/m^2)	Panel Load(N)
18	0.24	12.07	87.41	983.25
15	0.24	12.07	87.41	1933.06
12	0.24	12.07	87.41	1933.06
9	0.24	12.07	87.41	1933.06
6	0.24	12.07	87.41	1976.00
3	0.24	12.07	87.41	2071.00

Table 4: Wind loads acting on 40m tower

Elevation(m)	k_2’	Design Velocity(m/s)	Design Pressure(N/m^2)	Panel Load(N)
40	0.395	19.86	236.65	4987.40
35	0.365	18.35	202.00	9654.70
30	0.34	17.10	175.45	8345.70
25	0.29	14.58	127.55	7756.20
20	0.24	12.07	87.41	6823.60
15	0.24	12.07	87.41	6823.60
10	0.24	12.07	87.41	7006.50
5	0.24	12.07	87.41	7010.00

5. ANALYTICAL RESULTS

Table 5: Comparison of Joint displacements (mm): 18m tower

Elevation	Node No.	Frame Model (I)	Truss Model (II)	Hybrid Model (III)
18	84	44.05	44.70	44.37
9	48	12.97	13.21	13.20
3	25	0.71	0.70	0.70

Table 6: Comparison of Member Stresses (MPa) at 3m and base: 18m tower

Elevation	Member Type	Member Number	Axial Stresses			Combined Stresses		
			I	II	III	I	II	III
3m	Leg	30	-40.61	-41.13	-40.78	-65.82	-57.05	-66.17
	Diagonal	37	-8.55	-9.28	-8.53	-11.15	-9.68	-9.41
	Horizontal	25	1.35	1.37	1.33	8.41	10.20	9.20
0m	Leg	57	-31.10	-33.08	-32.30	-58.10	-33.10	-32.66

Table 7: Comparison of Joint displacements (mm): 40m tower

Elevation	Node No.	Model I	Model II	Model III
40m	36	701.79	702.73	702.26
20m	20	187.39	187.80	187.63
10m	12	25.78	25.94	25.90
5m	5	26.61	39.83	30.65

Table 8: Comparison of Member Stresses (MPa) at 5m and base: 40m tower

Elevation	Member Type	Member Number	Axial Stresses			Combined Stresses		
			I	II	III	I	II	III
5m	Leg	38	-190.66	-192.75	-191.66	-253.71	-247.65	-247.37
	Diagonal	446	-9.53	-9.27	-10.56	-87.38	-9.34	-10.64
	Horizontal	464	8.77	8.78	8.77	31.24	22.81	17.34
0m	Leg	37	-190.79	-192.75	-191.57	-253.62	-194.05	-195.11

6. RESULTS:

On the basis of the above tabulated values following observations can be made:

1) The displacement pattern as indicated in table 5 & table 7 conform to expected prototype behavior wherein the rigid frame model indicates the least displacements at each level in contrast to the larger displacements exhibited by the truss model.
2) In the 18m tower, the deviations of the displacements in the rigid frame and truss models from the hybrid at the top are 0.72% and 0.74%. At 3m, the differences between the frame model and truss model from the hybrid model are 1.4% and 0%.
3) In the 40m tower, the deviations of the displacements in the rigid frame and truss models from the hybrid at the top are 0.07% and 0.07%. At 10m, the differences between the frame model and truss model from the hybrid model are 0.46% and 0.15%.
4) Axial stress patterns conform to the prototype behavior wherein the greatest axial stresses are exhibited by the truss model, both for the representative members and leg member at the base. Deviation in leg, diagonal and horizontal members in 18m and 40m tower are given in table 9 and 10.

Table 9: Deviation in stresses in typical representative members: 18m tower

Member		Difference (Model I & III)		Difference (Model II& III)	
		Axial Stress	Combined Stress	Axial Stress	Combined Stress
3m	Leg	0.41%	0.53%	0.86%	14%
	Diagonal	0.23%	18.50%	9%	3%
	Horizontal	1.5%	9%	3%	11%
0m	Leg	4%	78%	2.4%	1.35%

Table 10: Deviation in stresses in representative members: 40m tower

Member		Difference (Model I & III)		Difference (Model II& III)	
		Axial Stress	Combined Stress	Axial Stress	Combined Stress
10m	Leg	0.10%	1.10%	0.24%	9.70%
	Diagonal	1.26%	12.27%	0.68%	0.20%
	Horizontal	0.14%	104.25%	1.16%	9.24%
5m	Leg	0.52%	2.56%	0.57%	0.12%
	Diagonal	10%	721.20%	12.20%	12.20%
	Horizontal	0%	80%	0.11%	31.50%
0m	Leg	0.41%	30%	0.62%	0.54%

7. CONCLUSIONS

On the basis on the present study, following conclusions can be drawn:

The wind analysis results showed that irrespective of the tower height modeling strategy does not significantly affect the displacement pattern, particularly maximum lateral displacement at the top of the tower. Truss model, in general, reflects the lower bound on stresses, irrespective of height, due to dominance of the axial stresses. The bending components normal to the plane of the element are of a lower order. The prototype as fabricated has members which are likely to be subjected to in-plane and out of plane moments. The frame idealization, hence, provides a better estimate of the design forces. Deviations, if any, are easily accounted for by the conventionally adopted factor of safety. The combined model involves more rigorous analysis, whereas the frame model is the safest to adopt due to highest stresses. As the tower height increases, the difference in the stresses among the different idealizations do change, but the generic trend remains the same.

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