International Journal of Creative and Open Research in Engineering and Management
ISSN: **3108-1754 (Online)**
Volume 02 Issue 06 June-2026 | Impact Factor: 3.5

Structural Analysis and Design of a 220 kV Single-Circuit Transmission Line Tower Using STAAD.Pro under Wind and Seismic Loading Conditions

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How to Cite this Article:

Madhav, B. (2026). Structural Analysis and Design of a 220 kV Single-Circuit Transmission Line Tower Using STAAD.Pro under Wind and Seismic Loading Conditions. International Journal of Creative and Open Research in Engineering and Management, <i>02</i>(6). https://doi.org/10.55041/ijcope.v2i6.128

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Abstract—
The transmission tower plays an essential role in electrical power generation and transmission since it supports the conductor and provides clearances at long spans. As the need for power increases, structural integrity of such towers becomes increasingly critical. The aim of this paper is to carry out analysis and design of a 220 kV self-supporting single-circuit lattice steel transmission tower using the software STAAD.Pro. In order to perform such analysis, wind loads have been determined by referring to IS 875 (Part 3) – Wind zone III (wind speed = 44m/s). This gives us the total horizontal wind load as 130.35 kN. Also, seismic forces have been determined according to IS 1893 – seismic zone II, giving a value of design base shear as 12.43 kN. With an acceptable nodal displacement of 6.20 mm at the tower tip (with the maximum allowable value being 290 mm or H/100), leg members will experience the highest compressive load of 615 kN. The member utilization factor is 0.82, and all other members have lower utilization factors, showing structural adequacy. Modal analysis showed that the first natural frequency is 5.00 Hz (T=0.20 s), meaning the tower is stiff enough without any possibility of resonance during service. It was observed that the wind load controlled the design parameters as compared to the seismic load. The designed tower structure has been proven to meet the requirements of IS 802, IS 875 (Part 3), IS 1893, IS 800, among others.

Keywords— Transmission Tower; STAAD.Pro; Wind Load Analysis; Seismic Analysis; Modal Analysis; Response Spectrum Analysis; IS 875; IS 1893; IS 802; Lattice Steel Structure; 220 kV

I. INTRODUCTION

Electricity is a basic requirement for any industrialized economy. The huge growth in power production capability in the developing countries of India and others has posed heavy requirements on high voltage power transmission lines. Tower structures like the lattice towers carrying the conductors spanning hundreds of kilometers are one of the most complex yet often overlooked civil engineering structures. Failure in such structures will have severe impacts in the form of large-scale power shortages.

In general, the transmission tower is a slender and tall three-dimensional space frame structure whose lateral load behavior is inherently different compared to the conventional building. The loading conditions that the transmission tower needs to resist are gravity load due to its weight, concentrated loads from the conductors and ground wires, aerodynamic loads from wind actions on tower as well as the conductor itself, inertial forces due to earthquakes, and thermal effects. Out of these various loading conditions, wind loads have been the most significant factor in India for many years.

The 220 kV voltage grade has an important role to play within the power system of India, since it is the major bulk transmission line used both for inter-state and intra-state operations to transmit power generated at power stations to regional substations for subsequent transmission to consumers' end-use points. Structures built for this voltage require special design features based on clearance criteria, conductor spans, and load carrying capacity. Notwithstanding their extensive use, finite element analysis studies of 220 kV single circuit lattice structures are not abundant in the existing literature.

Software packages such as the one used in STAAD.Pro provide facilities to conduct accurate analysis in the third dimension of structures, which is important because of the complex coupled behavior of the structure. These packages allow users to carry out analyses in three dimensions under static and dynamic loading conditions.

The research solves the problem through the development of an original parametric model of a self-supporting single-circuit 220 kV transmission line tower in STAAD.Pro. Loads due to wind action and seismic effects have been computed using first principle calculations as per IS 875 (Part 3) and IS 1893, respectively, over ten independent load cases and further combined using five code-prescribed load combinations. Response parameters such as nodal displacement, member axial forces, member utilisation ratio, support reactions, natural frequencies, and response spectrum base shear are then computed and evaluated in comparison to their respective permissible values.

1.1 Scope and Limitations

This current study involves 3D modelling, static analysis with consideration for wind load and seismic action, mode shape analysis, dynamic analysis based on response spectra method, and checking of members in compliance with Indian Standards. The purposeful exclusion of foundation analysis, non-linear analysis, conductor galloping, icing, soil-structure interaction, progressive collapse evaluation, and erection phase analysis are seen as areas for further investigation.

II. LITERATURE REVIEW

There has been continuous interest in the structural behavior of transmission towers, spurred on by two factors: the increasing network expansion and the occurrence of catastrophic failures associated with high wind. In the review below, the pertinent research threads driving the current study will be consolidated.

There is consensus within the literature that wind loading governs the design of the lattice towers. It has been proven via field measurements and simulation modeling that the lateral displacement reaches a peak value at the top of the tower and that the axial compressive forces in leg members reach peak levels when design wind loads are applied. Dynamic response induced by wind forces can significantly increase the static loading, especially in tall towers of greater than 40 meters in height.

Three dimensional finite element analysis (FEA) based on space frame elements has become the accepted tool in many studies for representing the combined behaviour of axial, bending, and twisting in lattice towers. Many research works have confirmed that 3D modeling yields much more accurate results compared to the classical 2D planar-trusses, and the presence of member eccentricity at joints, which is inherent in angle-section lattice towers, must be considered.

While the seismic behavior of power line towers has received relatively less consideration compared to wind, a number of studies have shown that there can be significant contributions from earthquakes to the loadings on such towers. The results of modal and response spectrum analysis show that global mode is the one contributing most of the seismic base shears and displacements, whereas local modes contribute local member forces and torsion. Response spectrum analysis is considered a reliable technique for seismic evaluation of towers.

Work that has been done with regard to 220 kV tower structures in India has found that IS 875 (Part 3) wind pressures along with IS 1893 response spectrum always result in wind being the controlling load case in Zones I through III, whereas seismic forces may equal or even exceed wind forces in Zone IV and V locations such as northeast India and Uttarakhand.

From the existing literature review, the following research issues stand out as critical and therefore motivate the present research work: (i) The majority of research works consider wind loads only or wind and seismic loads separately and not in the integrated form of load combination; (ii) Numerical research on 220 kV single circuit lattice towers with all the Indian Standard codes incorporated is very few; and (iii) There exists an urgent need for numerical studies including full model parameters along with structural performance measures.

III. TOWER DESCRIPTION AND STRUCTURAL MODEL

3.1 Tower Configuration

The tower under investigation is a self-supported double circuit tower for a 220 kV power line. The tower tapers from an 8.5 m by 8.5 m square at the bottom to a 2.5 m by 2.5 m prismatic shape at the top, and its full height is 29 m. It should be noted that the dimensions meet the requirements of the IS 802 (Part 1/Sec 1) standard for such voltage towers. The height of each panel from the bottom five panels of the tower (each 4.0 m high) and the upper three (each 3.0 m high).

In this arrangement, the cross-arm structure supports three phase conductors, ACSR Zebra (weight 1.62 kg/m), with the design span of 350 m, along with one ground conductor. This arrangement guarantees the necessary statutory clearances from the ground and the distance between the phases as per CEA regulations and IS 5613.

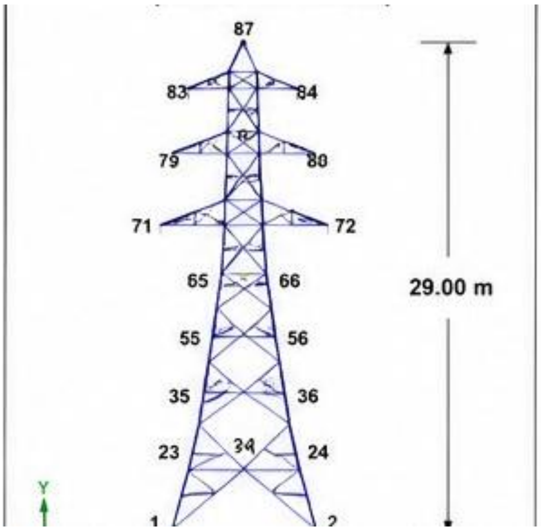


Fig. 1: Three-dimensional STAAD.Pro model of the 220 kV single-circuit lattice tower.

Table 1: Tower Geometry and Structural Parameters

Parameter	**Value**
Tower type	Self-supporting lattice tower
Voltage level	220 kV
Circuit configuration	Single circuit
Total height	29 m
Base width (square)	8.5 m × 8.5 m
Top width (shaft)	2.5 m × 2.5 m
Number of panels	8
Panel heights	5 × 4.0 m (lower) + 3 × 3.0 m (upper)
Steel grade	Fe 410 (IS 2062)
Yield strength (fy)	250 MPa
Ultimate strength (fu)	410 MPa
Elastic modulus (E)	2×10^5 MPa
Density	7850 kg/m³
Poisson's ratio	0.30

3.2 Member Section Properties

Members used have been Indian standard angle sections (ISA) as per IS 808. Sizes of the sections used have been decided by taking into account the values of the resultant forces and also adhering to the slenderness ratio requirements stipulated in IS 802.

Table 2: Member Section Properties

Member Category	**Section**	**Designation per IS 808**
Leg members	ISA 130 × 130 × 12 mm	Double angle back-to-back
Main bracing	ISA 100 × 100 × 10 mm	Single angle
Secondary bracing	ISA 75 × 75 × 8 mm	Single angle
Cross-arm members	ISA 90 × 90 × 8 mm	Single angle

3.3 STAAD.Pro Modelling Procedure

Modeling in STAAD.Pro was done with the SPACE frame formulation. Following is a brief summary of the six major steps carried out:

Step 1 - Joint coordinate generation: Thirty-three joints were defined in order to describe the tapered shaft profile with eight elevations (at 0, 4, 8, 12, 16, 20, 23, 26, and 29 m). Joints for the legs taper inward symmetrically to the ±4.25 m at base level to meet a single joint for the apex position.

Step 2 - Member definition: Beams were defined for the members including leg, bracing, cross-arm, and apex sections. Corresponding IS section profiles were assigned to each of them from the IS sections database of STAAD.Pro software.

Step 3 - Material data: An isotropic material data was specified with E = 2 x 108 kN/m², $\nu = 0.30$, and $\gamma = 76.98$ kN/m³.

Step 4 - Support specification: Fully fixed supports were specified for all four base joints.

Step 5 - Loading specification and factoring: Ten different load cases and five different factored load combinations were defined according to Section 4.

Step 6 - Analysis: Static, modal, and response spectrum analysis was carried out successively.

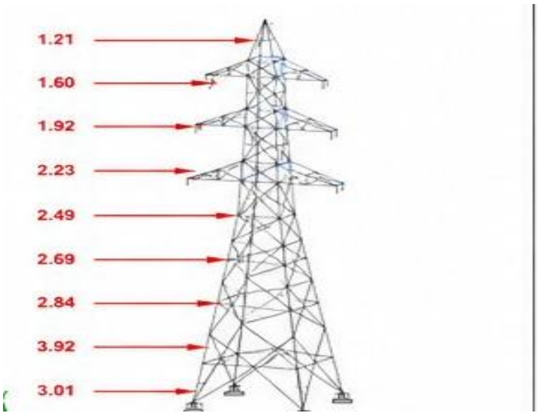


Fig-2: Node numbering and member incidence diagram in STAAD.Pro.

IV. LOAD CALCULATION AND LOAD CASES

4.1 Dead Load and Conductor Load

The tower dead load of 25 tons (245.25 kN) will be loaded using the SELFWEIGHT load case with a multiplication factor of –1 in the Y direction. The conductor loads shall be added as vertical concentrated loads to the nodes where the cross arm is located, considering the ACSR Zebra conductor load and span of 350 m; hence, three-phase conductors will add up to 16.68 kN, and ground wires to 3.50 kN.

4.2 Wind Load Calculation (IS 875 – Part 3)

The forces acting on the tower are the wind loads that have been calculated using basic principles in accordance with the provisions of IS 875 (Part 3). It can be assumed that the site lies within Wind Zone III, which includes most of peninsular India, including Hyderabad.

Table 3: Wind Speed and Pressure Computation

Parameter	**Symbol**	**Value**
Basic wind speed (Zone III)	Vb	44 m/s
Risk factor (50-year return)	k_1	1.00
Terrain & height factor (Cat. 2, H ≈ 30 m)	k_2	1.08
Topography factor (flat terrain)	k_3	1.00

Design wind speed	$Vz = Vb\ k_1k_2k_3$	47.52 m/s
Design wind pressure	$pz = 0.6\ Vz^2$	1355 N/m^2 (1.355 kN/m^2)
Drag coefficient (lattice tower)	Cd	3.2
Solidity ratio	φ	0.25 (assumed)
Effective frontal area (tower body)	Ae	62 m^2
Total wind force on tower	Fw = pz × Ae	84.01 kN
Wind force on 3 phase conductors	Fc	39.84 kN
Wind force on ground wire	Fg	6.50 kN
Total lateral wind force	FW,total	130.35 kN

4.3 Seismic Load Calculation (IS 1893 – Part 1)

Seismic design has been done as per IS 1893 (Part 1): 2016 by the response spectrum method. The structure is assumed to be situated in Zone II of the seismic zone, and it applies mainly to Hyderabad and the Deccan Plateau region.

Table 4: Seismic Design Parameters

Parameter	**Symbol**	**Value**
Seismic zone	—	Zone II
Zone factor	Z	0.10
Importance factor (power transmission infrastructure)	I	1.5
Response reduction factor (steel lattice)	R	4
Soil type	—	Medium (Type II)
Fundamental time period (from modal analysis)	T_1	0.20 s
Normalised spectral acceleration (IS 1893 Fig. 2)	Sa/g	2.50
Design seismic coefficient	Ah = (Z/2)×(I/R)×(Sa/g)	0.0469
Total seismic weight	W	265 kN
Design base shear	Vb = Ah × W	12.43 kN

4.4 Load Cases and Combinations

Ten independent load cases were defined to cover all directional variants of lateral loading:

Load Case	**Description**
LC 1	Dead load (self-weight + conductor + ground wire)
LC 2	Vertical conductor load
LC 3	Wind load in +X direction
LC 4	Wind load in −X direction
LC 5	Wind load in +Z direction
LC 6	Wind load in −Z direction
LC 7	Earthquake in +X direction
LC 8	Earthquake in −X direction

LC 9	Earthquake in +Z direction
LC 10	Earthquake in −Z direction

Five factored load combinations were defined per IS 800 and IS 1893:

Combination	**Expression**
Comb. 1	1.5 (DL + WL)
Comb. 2	1.5 (DL + EQ)
Comb. 3	1.2 (DL + WL + EQ)
Comb. 4	0.9 DL + 1.5 WL
Comb. 5	0.9 DL + 1.5 EQ

Table 5: Nodal Displacement Variation with Height

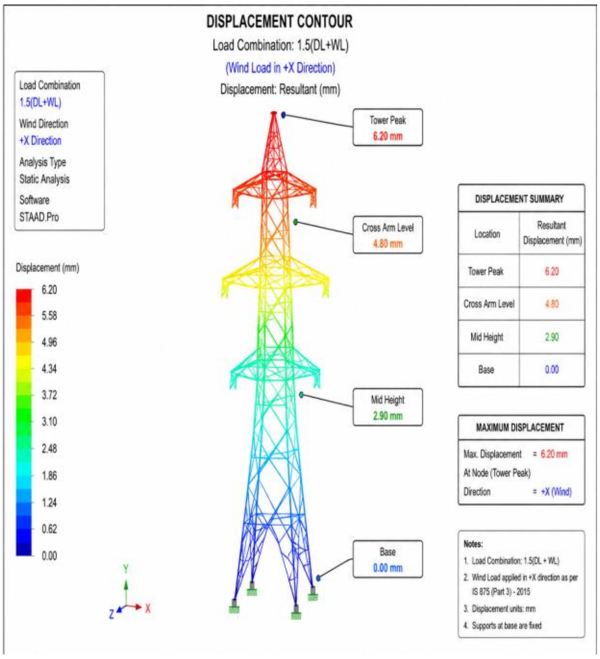
Elevation (m)	Displacement — X (mm)	Displacement — Y (mm)	Displacement — Z (mm)
0 (Base)	0.00	0.00	0.00
4	0.75	—	—
8	1.40	—	—
12	2.15	—	—
16	3.05	—	—
20	4.15	—	—
23	5.00	—	—
26	5.75	—	—
29 (Apex)	6.20	1.40	0.80

Horizontal displacement at the top point is 6.20 mm. Allowable displacement based on IS 802 is H/100=29,000/100=290 mm. Displacement is 46.8 times less than the allowable displacement, which signifies an outstanding structural performance from serviceability considerations. Vertical displacement of 1.40 mm is insignificant and shows that the legs are axially rigid.

V.RESULTS AND DISCUSSION

5.1 Nodal Displacement Response

The node displacements due to the prescribed combination of wind loads (Combination 1: 1.5(DL + WL)) have been tabulated in Table 5 below. The tower is treated as a fixed-base vertical cantilever and thus displays displacement behavior that rises continuously from the base (zero point) to the topmost point.



DISPLACEMENT SUMMARY

Location	Resultant Displacement (mm)
Tower Peak	6.20
Cross Arm Level	4.80
Mid Height	2.90
Base	0.00

MAXIMUM DISPLACEMENT

Max. Displacement = 6.20 mm
At Node (Tower Peak)
Direction = +X (Wind)

Notes:

1. Load Combination: 1.5(DL + WL)
2. Wind Load applied in +X direction as per IS 875 (Part 3) - 2015
3. Displacement units: mm
4. Supports at base are fixed

Fig. 3: Displacement contour under governing wind load combination (1.5(DL+WL)).

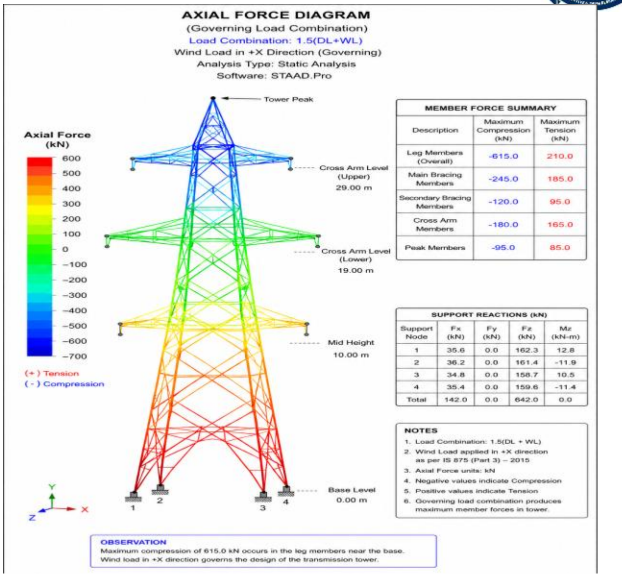
5.2 Member Force Distribution

Axial member forces were extracted for all load combinations. Table 6 summarises the peak compressive and tensile forces by member category.

Table 6: Maximum Member Axial Forces

Member Category	Max. Compression (kN)	Max. Tension (kN)	Critical Member
Leg members	615	105	M-12 (lower panel leg)
Main bracing	248	165	M-35
Secondary bracing	132	92	M-48
Cross-arm members	96	64	M-72

The forces in the leg members have the highest compressive loading (615 kN) as they provide the most direct load path for transferring the loads to the foundation. The main bracing members have substantial axial loads, including compression (248 kN) and tension (165 kN). This is due to their ability to transfer the wind shear between the legs through axial actions. The cross-arm members have considerably lower forces.



MEMBER FORCE SUMMARY

Description	Maximum Compression (kN)	Maximum Tension (kN)
Leg Members (Overall)	-615.0	210.0
Main Bracing Members	-245.0	185.0
Secondary Bracing Members	-120.0	95.0
Cross Arm Members	-180.0	165.0
Peak Members	-95.0	85.0

SUPPORT REACTIONS (kN)

Support Node	Fx (kN)	Fy (kN)	Fz (kN)	Mz (kN-m)
1	35.6	0.0	162.3	12.8
2	36.2	0.0	161.4	-11.9
3	34.8	0.0	158.7	10.5
4	35.4	0.0	159.6	-11.4
Total	142.0	0.0	642.0	0.0

NOTES

1. Load Combination: 1.5(DL + WL)
2. Wind Load applied in +X direction as per IS 875 (Part 3) – 2015
3. Axial Force units: kN
4. Negative values indicate Compression
5. Positive values indicate Tension
6. Governing load combination produces maximum member forces in tower.

OBSERVATION

Maximum compression of 615.0 kN occurs in the leg members near the base.
Wind load in +X direction governs the design of the transmission tower.

Fig. 4: Axial force diagram under governing load combination.

5.3 Member Utilization Ratios

The design check was performed using STAAD.Pro's built-in IS 800 steel design module. The utilization ratio (UR) defined as the ratio of the computed stress to the allowable stress must satisfy UR ≤ 1.0 for structural adequacy.

Table 7: Member Utilization Ratios

Member Category	Maximum Utilization Ratio (UR)	Status
Leg members	0.82	Safe (UR < 1.0)
Main bracing	0.75	Safe (UR < 1.0)
Secondary bracing	0.68	Safe (UR < 1.0)
Cross-arm members	0.54	Safe (UR < 1.0)

All utilization ratios are less than one, which means that none of the members is overstressed in view of the load combinations being applied. The legs are the most heavily utilized members (UR = 0.82). There is efficiency in using the materials while at the same time ensuring sufficient safety factor. The smaller utilization ratios for the secondary bracing and cross-arms are a consequence of redundancy.

5.4 Support Reactions

The reactions due to the controlling load case are shown in Table 8. The fairly equal distribution of vertical loads on all four legs shows the symmetrical nature of the loading condition. This implies that there is no differential uplift created due to asymmetrical loading conditions.

Table 8: Support Reactions at Tower Base Nodes

Support / Leg	**Fx (kN)**	**Fy (kN)**	**Fz (kN)**	**Remarks**
Leg A (Node 1)	156	420	18	Windward leg
Leg B (Node 2)	148	411	20	
Leg C (Node 3)	152	425	19	
Leg D (Node 4)	149	418	17	Leeward leg

Leg C experiences the highest vertical reaction value of 425 kN due to its position as the windward leg, taking on the maximum effect from both gravity and overturning loads. There is no presence of tensile (lifting) reaction under this combination of loading, indicating that there will be no net uplift force on the foundation.

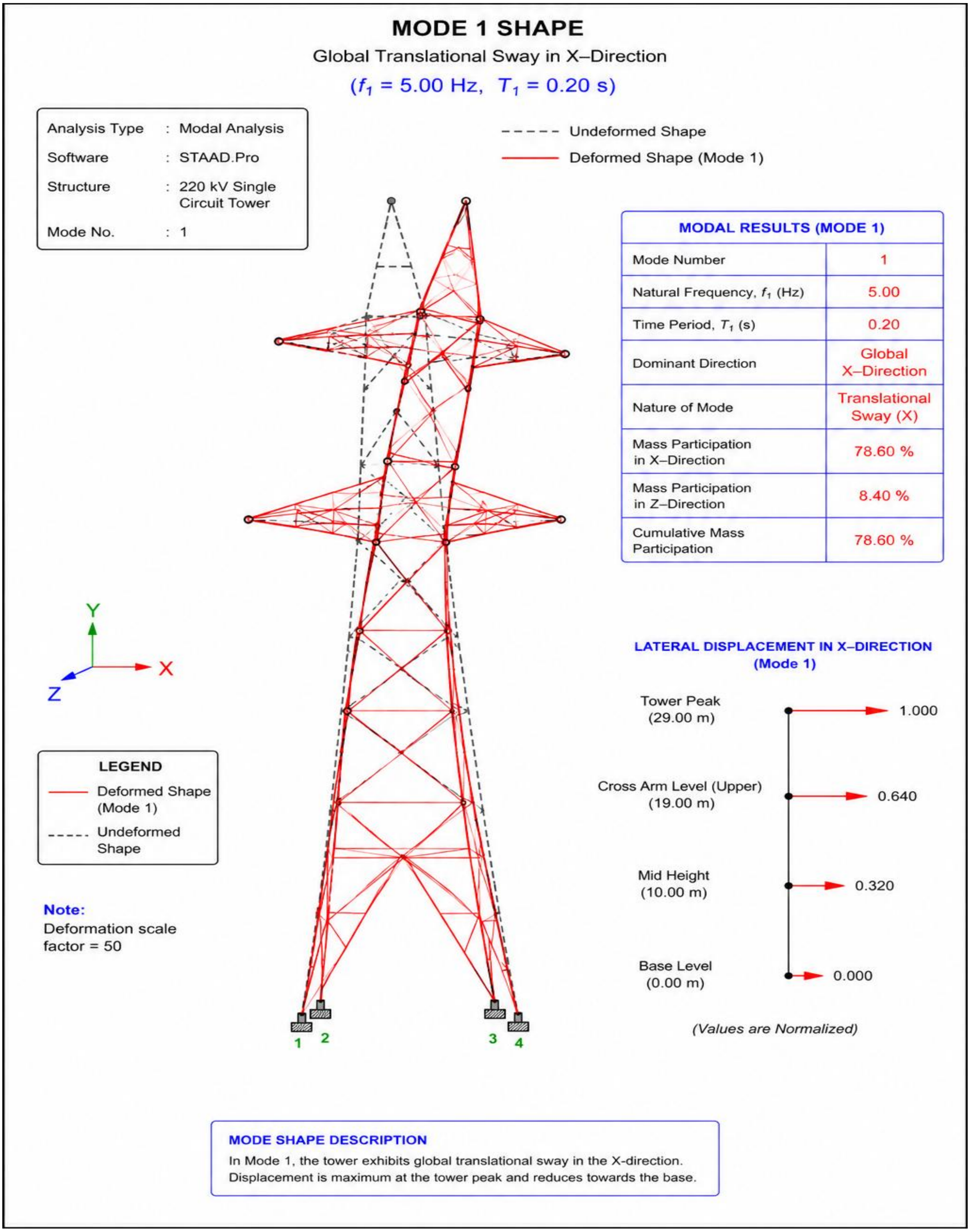
5.5 Modal Analysis Results

Modal analysis was performed for the ten lowest natural modes. The first six modes are summarised in Table 9.

Table 9: Natural Frequencies and Mode Shapes

Mode	**Time Period T (s)**	**Natural Frequency f (Hz)**	**Mode Shape Description**
1	0.200	5.00	Global translational sway — X direction
2	0.190	5.26	Global translational sway — Z direction
3	0.110	9.09	Torsional mode about vertical axis
4	0.090	11.11	Combined sway and torsion
5	0.070	14.28	Second-order sway — X direction
6	0.060	16.67	Second-order sway — Z direction

It is clear that the natural frequency at 5.00 Hz ($T_1 = 0.20$ s) is very much higher than the frequency range associated with wind buffeting (usually 0.1-1.0 Hz), and is not even in the seismic ground motion frequency range (0.5-5.0 Hz for medium to stiff soil types). Thus, it is confirmed that there will be no problem of resonance for this tower either by wind or by seismic loads. The similarity between the first and second modes' frequencies is due to the symmetric shape of the square tower.



Analysis Type : Modal Analysis
Software : STAAD.Pro
Structure : 220 kV Single Circuit Tower
Mode No. : 1

MODAL RESULTS (MODE 1)	
Mode Number	1
Natural Frequency, f_1 (Hz)	5.00
Time Period, T_1 (s)	0.20
Dominant Direction	Global X–Direction
Nature of Mode	Translational Sway (X)
Mass Participation in X–Direction	78.60 %
Mass Participation in Z–Direction	8.40 %
Cumulative Mass Participation	78.60 %

Note:
Deformation scale factor = 50

MODE SHAPE DESCRIPTION
In Mode 1, the tower exhibits global translational sway in the X-direction. Displacement is maximum at the tower peak and reduces towards the base.

Fig-5: Mode 1 shape: global translational sway in X-direction (f_1 = 5.00 Hz, T_1 = 0.20 s).

5.6 Response Spectrum Analysis Seismic Response

Table 10: Seismic Analysis Results Summary

Parameter	Wind Load	Seismic (Response Spectrum)
Maximum lateral displacement (mm)	6.20	4.10
Maximum base shear (kN)	130.35	12.43
Maximum member compressive force (kN)	615	432
Governs design?	YES	No

The wind loading generates lateral displacement, base shear, and critical members' loadings significantly higher than those generated by the seismic load on the particular site. The base shear due to wind (130.35 kN) is roughly 10.5 times higher than the base shear generated due to seismic action (12.43 kN), indicating clearly that wind is the design lateral loading on this particular site-tower combination. The conclusion drawn is in harmony with the existing knowledge that transmission towers located in peninsular India are governed by wind loadings and not by seismic action.

5.7 Comparative Summary and Structural Safety Assessment

The structural performance indicators are collectively summarised in Table 11 to provide a holistic picture of the tower's safety status.

Table 11: Structural Safety Verification Summary

Performance Check	Computed Value	Permissible / Limit	Status
Max. lateral displacement (apex)	6.20 mm	290 mm (H/100)	✓ Safe
Max. member utilization ratio	0.82 (leg)	1.00	✓ Safe
Max. support reaction (vertical)	425 kN	Within foundation capacity	✓ Safe
Fundamental frequency	5.00 Hz	Away from resonance zone	✓ Safe
Wind base shear vs. capacity	130.35 kN	Within design resistance	✓ Safe
Seismic base shear vs. capacity	12.43 kN	Within design resistance	✓ Safe
Overall structural stability	Confirmed stable	—	✓ Safe

All checks are satisfied with adequate margins, validating the overall structural integrity of the proposed 220 kV tower under the complete set of prescribed loading conditions.

VI. CONCLUSION

In this research work, an intensive FEA design has been carried out for a 220 kV single circuit steel lattice self-supporting transmission tower by employing the software STAAD.Pro, with wind and seismic forces accurately determined based on the relevant Indian Standards. Conclusions from the above analysis results are given as follows:

1. Wind loading governs the structural design. Wind-induced lateral force, which totals to 130.35 kN, is about 10.5 times the seismic base shear (12.43 kN) at the Zone II location. All key design factors, namely apex displacement, peak member force, and maximum support reactions, are governed by the wind load combination 1.5(DL + WL).

2. Excellent displacement serviceability. The maximum lateral displacement of 6.20 mm at the apex represents only 2.14% of the permissible limit of 290 mm (H/100), demonstrating that the adopted section sizes provide substantially greater stiffness than required.

3. All members are structurally adequate. Utilization ratios range from 0.54 (cross-arm members) to 0.82 (leg members), all satisfying the $UR \leq 1.0$ limit of IS 800. Leg members are the most critically loaded and should be the focus of any future weight-optimisation study.

4. Dynamic performance is satisfactory. The fundamental natural frequency of 5.00 Hz ($T_1 = 0.20$ s) lies well outside the resonance-prone frequency ranges of both wind buffeting and earthquake ground motion, confirming that dynamic amplification does not pose a design concern under normal service conditions.

5. Support reactions are well-distributed and safe. The four base reactions are nearly uniform, and no uplift was observed under any load combination, thereby simplifying foundation design and confirming the structural symmetry of the adopted geometry.

6. STAAD.Pro is an efficient and reliable platform. The software enabled seamless integration of static, modal, and response-spectrum analysis within a single model, and its IS 800 design module provided direct member verification without post-processing. It is recommended as the analytical tool of choice for similar transmission-tower design projects.

To conclude, it can be seen that the design of a single circuit 220kV lattice tower is in conformance with all the strength and stability considerations set forth by the standards of IS 802, IS 875(Part 3), IS 1893, and IS 800, and it will be structurally sound to install such towers in India.

VII. RECOMMENDATIONS AND FUTURE SCOPE

Based on the findings and limitations of the present study, the following directions are recommended for future investigation:

(i) Topology and size optimisation using genetic algorithms or particle swarm methods to minimise structural steel weight while satisfying all IS 802 constraints.

(ii) Geometric and material nonlinear analysis (GMNA) to capture second-order P-Δ effects and the post-elastic behaviour of angle-section members under combined compression and bending.

(iii) Soil–structure interaction studies to evaluate the influence of foundation flexibility on the dynamic response and load distribution among the four legs.

(iv) Wind tunnel experimentation to calibrate the force coefficients and gust response factors used in the equivalent static wind load model.

(v) Extension of the analysis framework to 400 kV, 765 kV, and Ultra High Voltage (UHV) tower configurations, and to guyed mast alternatives.

(vi) Progressive collapse analysis to assess residual structural capacity following the failure of a single bracing or leg member.

ACKNOWLEDGMENT

I express my sincere gratitude to Dr. T. Siva Sankar Reddy, Department of Civil Engineering, Mahatma Gandhi Institute of Technology, for his guidance, critical review, and encouragement throughout this work. I also acknowledge the facilities and computational resources provided by Mahatma Gandhi Institute of Technology.

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