Comparative Study of Indian and ASCE Codes Provision for Design of Transmission Tower

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Abstract—Transmission towers play an important role in the operation of a reliable electrical power system that's why considered as a lifeline system for power supply. In this study we made a comparison between Indian code IS: 802(Part l/Sec 1):1995 and ASCE 10-97(2000) code. For this comparative study has been carried on the two different codes with respect to different types of base width, height and bracings. From the study of these codes we conclude that the Indian IS: 802(Part l/Sec 1):1995 code available for the design of tower requires certain modification as to made design more structural and economical as compare to the ASCE 10-97(2000) code. Through these studies certain recommendation has been made to Indian Code so it make updated with time as the ASCE is updating itself.

***Keywords*— Axial Force, ASCE code, Base width, Indian code, Lateral Load, Transmission Tower.**

I. INTRODUCTION

A tower or mast is a tall skeleton structure with a relatively small cross section, which has a large ratio between height and maximum width. A tower is a freely standing self supporting structure fixed to the base or foundation while a mast is a tall structure, pinned to the base or foundation and braced with guys etc. Towers or masts are structures that are built in order to fulfill the need for placing objects or persons at a certain level above the ground.

II. PAST RESEARCH WORK

Shakeel Ahmad and Md. Ehtesham Ansari [1] studied about the tower failures due to High Intensity Winds (HIW) associated with tornadoes and micro bursts. They conclude that the towers are highly affected by the tornadoes and the study shows that the maximum displacement occurs at the top of the tower in both X and Y direction. They also conclude that the maximum displacement at nodal increases rapidly from 12m height to the top of the tower in both X and Y directions. All the analysis of tower is carried out using STAAD-PRO.

F. Al-Mashary, et.al. [2] Presented a case history of the failure of six tangent transmission towers, on 132 kV transmission line. A post-failure full scale load test conducted on a similar design attributed the failure to unusual gust wind above the design speed. The study shows that the first and the last towers failed by bending of the top cross-arms, three of the intermediate ones at their bases and the fourth above its first body.

G. Ghodrati Amiri and S. R. Massah [3] investigated the effects of wind and earthquake-induced loads on 4-legged telecommunication towers. For this, wind load is treated as a static load and the earthquake effects is consider according to the standard design spectrum based on the Iranian seismic code of practice. From the analysis it is concluded that in the case of towers with rectangular cross section, the effect of simultaneous earthquake loading in two orthogonal directions is important.

P Valan Arasu, et.al. [4] investigated seismic response of steel wind turbine towers, generally the wind turbine steel tower is 76 meters in height with variable tubular cross section axially therefore it is necessary to find out the risk assessed with steel wind turbine towers in the critical seismic environment. As wind turbines are installed in seismic areas and dynamic analysis was carried out to evaluate its behavior under seismic excitation, to define several damage states, and to develop a framework for determining its probability of damage. In their study they utilize RADIOSS to investigate the probability of reaching predetermined damage states under seismic loading.

Richa Bhatt, et.al. [5] Studied about the Lattice towers, Lattice towers are 3D space frames that for design are conventionally analyzed as 2D trusses. In this study the modeling of Lattice towers is done in 3 types i.e., 3D frame, 3D truss and as a hybrid of the two. In this study the wind is taken as the primary force and analysis is carried out using Gust factor method. And then the joint displacements, member forces and maximum stresses have been compared. It is seen that the results from 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 and the 3D frame and hybrid model is used to check the design. The study shows that, it is necessary to redesign the base members for the combined stresses.

M.T. Chay, et.al. [6] Presented a comparison between several existing design codes of transmission tower. In their study they presented clear differences between design standards in which wind is presented and the loading scenarios produced by the simulated downburst. From the results it is clear that loads on lattice towers in downbursts underestimated by a significant amount due to variations in the distribution in wind speed with height and also due to a vertical component to the wind that is not a feature of boundary layer winds.

M. B. P. Chhetri and A. Shakya [7] describe a comparison of wind load calculation on lattice tower with different wind load standards in Asia pacific region. For this analysis American standard ASCE-7-05, British standard BSI: CP3: chapter V: part2 and Indian standard IS 875 and IS 802(part1/sec1):1995 were used to compare wind load. Comparison of wind load is carried out by keeping similar condition of topography, return period, basic wind speed.

N. R. Cuevas and R.M. Hernández, [8] analyzed that in the study of support towers, the support towers are considered as isolated structures, without any consideration of their interaction with isolators chains, hardware and cables. In their study, they analyzed dynamic characteristics of wind loads acting on support towers, considering their interaction with isolator's chains, hardware and cables. The main aim of their study is to analyze the dynamic effects produced by the accessories on the support towers. For their study in order to obtain the cable breakage, they developed a method. The analysis is carried out on two support tower, with different foundation conditions and the dynamic effects were evaluated by using time dependant function.

Efthymiou E, et.al. [9] Investigated the structural response of steel lattice masts subjected to the influence of wind loading, as well as the combination of wind loading and ice. . For their study they analyzed 6 types of steel masts, namely 4 masts located on the ground and 2 masts located on buildings. The analysis showed that the wind pressure itself produces significant forces and results to high capacity ratios on the members, but combined with the ice loading, it causes the maximum displacements and many members exceed their structural capacity. The study was carried out by means of innovative software in order to introduce the wind actions as thoroughly as possible.

J.M. Eidinger and L. Kempner Jr. [10] describe a reliability model that is used to forecast failures of transmission towers. They studied for three high voltage transmission circuits (230 kV, 345 kV and 500 kV) that have had collapsed towers under high wind and ice loading events. The fragility of existing towers to withstand extreme wind and ice loading is described in terms of nominal design wind speeds, and glaze ice thicknesses, and actual distances between towers. The model can be used to reasonably forecast end-to-end circuit reliability. These types of results can often be suitable for emergency planning purposes. The model can provide the transmission engineer with rational quantified information from which decisions can be made about selecting cost effective design wind (ice) loads.

G.V. Rao [11] presented optimized tower design for extra high voltage transmission tower. The optimization is carried out with respect to tower weight and geometry. In this study he developed a derivative free method for the configuration, analysis and design of transmission tower. The optimization is achieved by chosen a set of design parameters, fuzziness in the definition of these design parameters is also include in the design process. The study presented both crisp and fuzzy optimization, relevant to design of a double circuit transmission line tower under multiple loading.

L.Grcev,et.al. [12] analyzed the dynamic behavior of power transmission line and telecommunication towers. . Usually, time-domain surge impedance is used to characterize tower dynamic behavior. The main drawback in the definition of such surge impedance is that it is dependent on the excitation wave shape and there is no consensus on the current wave shape to be used. They explored the possibilities for a systematized approach to the analysis and uniquely defined quantities that characterize transient response of towers. They emphasized, limitations associated with simplified approaches by examining examples of direct comparison between computations based on transmission-line approach and antenna theory for a 100-m tall tower. It is pointed out that problems in the definition of voltages might occur above 100 kHz, especially near resonant frequencies, while differences in current distribution exist already at the lowest frequencies.

Siddesha.H [13] studied about the open Lattice towers, he observed that generally angle sections are commonly used in microwave antenna towers but he analyzed open Lattice towers using angle and square hollow section. In his research he analyzed open Lattice towers with Static and Gust Factor Method (GFM) and he made a comparative analysis between these two methods.

Sriram Kalaga, [14] proposed a simple method for designing structures using probabilistic principles. The procedure involves in deriving design strengths of system members for a specified level of reliability. He suggested an approach for evaluating the system's overall reliability using the concept of 'utilization ratios and the application of the proposed Reliability Based Design (RBD) method is illustrated through the analysis and design of a small planar transmission tower structure composed of axially-loaded angle sections. With the help of the results in order to obtain a polynomial expression for empirical failure probabilities a regression analysis is made. And the failure of the selected tower is verified through a Monte Carlo simulation of its collapse modes.

Srikanth L and Neelima Satyam D, [15] studied about the dynamic analysis of tower. The dynamic analysis of tower is carried out using numerical time stepping finite difference method. The important parameters in designing the tower include acceleration, frequency, and velocity. From the study of tower it is clear that breaking load is the critical combination among the forces developed in the structure. It is also clear that the maximum axial force in the leg members is 1600kN including the breaking load combination and it is reduced to 522.382kN without considering breaking load.

V.Lakshmi. et.al. [16] Observed the performance of 21M high 132kV tower with medium wind intensity. They evaluate the performance of the tower and the member forces in all the vertical, horizontal and diagonal members are evaluated. The performance of tower under abnormal conditions such as localized failures is also evaluated. The wind intensity converted into point loads and loads are applied at panel joints. By using IS 875-1987, Basic wind speeds, Influence of height above ground and terrain, Design wind speed, Design wind pressure, Design wind force is explained.

V.Lakshmi, et.al. [17] Had made an attempt to develop software for load calculations on transmission line towers of capacity 220 kV, only as per new code i.e., IS – 802 (Part- I / Sec-1):1995 by considering reliability, security, safety and anti-cascading conditions. They suggest that in MS-office the EXCEL is used to develop program for wind load calculations on the tower and other loads like wind load on conductor, conductor weights are calculated manually. They applied these loads on the model done in the STADD Pro for analysis. They modeled the different types of failure conditions and their effects on the performance of the tower are studied.

Ying-Hui Lei et.al. [18] Investigated about the dynamic behavior of a group of transmission towers linked together through electrical wires and subjected to a strong ground motion. In performing the seismic analysis, the wires and the towers concerned are modeled, respectively, by using the efficient cable elements and the 3-D beam elements considering both geometric and material nonlinearities. In addition, the strength capacities and the fracture occurrences for the main members of the transmission tower will be examined with the employment of the appropriate strength interaction equations. It is expected that by aid of this investigation, those who are engaged in code constitution or practical designing of transmission towers may gain a better insight into the roles played by the interaction force between towers and wires and by the configuration of the transmission towers under strong earthquake.

Jonathan Z. Liang et.al. [19] had studied the performance of transmission tower under damage-limitation earthquake and rare earthquake derived from the seismic hazard analysis of Perth Metropolitan Area (PMA). He conclude that many studies and post earthquake investigations have revealed that the material and geometric non-linearity have a major effect on the ultimate strength of towers and the tower collapse is due to either spread of plasticity or premature buckling. Hence, transmission towers designed by equivalent static analysis method should be examined to check their performance under dynamic loading conditions. The results are compared with code provisions and recommendations for the design of transmission towers.

Marcel I. R. et.al. [20] had proposed an alternative structural analysis modeling strategy for guyed steel towers design, considering all the actual structural forces and moments, by using three-dimensional beam and truss finite elements because in the usual structural analysis, models assumed as simple truss where all the steel connections are considered hinged. Initially the comparisons are based on static and dynamic structural behavior of the towers. Later the linear buckling analysis is carried out to determine the influence of the various modeling strategies on the tower stability behavior.

Mr. T. Raghavendra [21] studied that the Transmission-line tower is highly indeterminate structure. In their study optimization of a typical 132-KV double circuit transmission-line tower is carried out. The optimization of the structure is carried out with respect to configuration and materials are considered as variable parameters. The tower is modeled and analyzed using STAAD-PRO and ANSYS software's. . The basic model of the tower is analyzed in STAAD-PRO and validation of the results with respect to the member axial forces are carried out using ANSYS.

Sudipta Sen, et.al. [22] had designed 132/33 KV EHV sub-station. In their design they provide incoming power at 132 KV and through the combination of isolator-circuit breaker-isolator the power was transferred to main bus. The entire substation was design considering various factors like socio-economic factor of the surrounding locality, political developments, union of workers and contractors. Economic factors become chief aspect in this design. The entire substation was design to fulfill the most basic requirements of a proper substation including the civil and domestic requirements.

L. Skarbek and A. Żak, [23] analyzed about various types of structural damage such as line breakage, permissible sag, bolt loosening, fatigue cracking or insulator contamination.
In their study, they introduced concept of an electric power line monitoring system based on fiber sensors. In order to measure strains induced by icing, temperature or current loads, they suggested the application of distributed sensing networks with Fibre Bragg Grating (FBG) sensors or Brillouin sensors.

III. ANALYTICAL APPROACH

Type of Loads according to ASCE 10-97(2000):

The loads acting on a transmission tower are (ASCE 10-97(2000) :

- Dead load of tower (self-weight).
- Dead load from conductors and other equipment.
- Load from ice, rime or wet snow on conductors and equipment.
- Ice load on the tower itself.
- Erection and maintenance loads.
- Wind load on the tower.
- Wind load on conductors and equipment.
- Loads from conductor tensile forces.
- Damage forces.
- Earthquake forces.

Wind Load

Wind possesses kinetic energy by virtue of its velocity and mass, which is transformed into potential energy of pressure when a structure obstructs the path of wind. Natural wind itself is neither steady nor uniform; it varies along the dimensions of the structures as well as with time.

When the complete assembly of the lattice structures is considered, wind forces on different members of the structure are only partially correlated and time varying.

Wind Characteristics

Due to variation of wind speeds with height, terrain and averaging time, wind load codes describe a reference wind speed. The mean wind speed is usually represented by power law as:

$$U_Z = U_{ref} \quad (Z_a, Z_{ref}) \qquad \ldots. \qquad (1)$$

Where:
U_Z : velocity of wind that varies with height;
U_{ref}: mean velocity of wind;
Za: height above ground in terrain;
Z_{ref} : reference height =10 m;
: Power law exponent = 0.16 for exposed and windy areas.

Wind loads on each element can be determined according to Eq. (2). If U_Z is assumed to be much larger than along wind fluctuation *u (z, t)*, the second-order term involving *u (z, t)* can be ignored. Thus the magnitude of drag force, F_z, acting on the element along a specific direction is:

$$F=0.5\rho C_d A \{U_{(Z)} + U_{(Z,t)}\}^2 \qquad (2)$$

By ignoring the second order fluctuation component:

$$F=0.5\rho C_d A\, U_{(Z)}{}^2 + \{2U_{(Z,t)}\} \qquad (3)$$

Where:
F_Z : drag force;
$_{z,t}$: along wind fluctuation velocity;
: air density;
C_d: drag coefficient which is empirically calculated and depends on various factors such as solidity ratio;
A: surface area of member.

When the fluctuation velocity is much less than the mean velocity, the magnitude of drag force acting normal or in the across wind direction to a surface of area, is defined as: $F=0.5\rho C_d A\, U_{(Z)}{}^2$ (4)

Eq. (4) is used to determine the wind loads on the mast. For all cables, drag coefficient is assumed to be equal to 0.6 [19]. In this study, the direction of wind is taken in two directions.

(1) Maximum wind velocity at 90° to line direction

The maximum design wind velocity considered in this study is 40 (m/s) acting at an elevation of 10 m above the ground level. The pressure on members of tower structure is calculated with the following formula:

$$P_t=0.5\rho C_d A\, U_{(t)}{}^2 \qquad (5)$$

where:
P_t: pressure at tower elevations;
U_t: maximum mean velocity at tower elevations.

(2)Maximum wind velocity at 45° to line direction

When the wind direction is at 45° to tower line, this case can be stated as:

For longitudinal face of tower:

$$P= U^2 (1+0.2\sin^2 2ø) \cos ø \qquad (6)$$

For transverse face of tower

$$P= U^2 (1+0.2\sin^2 2ø) \sin ø \qquad (7)$$

In calculating wind loads the effects of the terrain, structure height, and wind gust and structure shape are included. The effective height of conductor and shield wires can be explained as follows:

The effective height of the conductors is calculate

$$H_e = h_c - l_s - 1/3\ S_c \tag{8}$$

Where

H_e : effective height of the conductor;
h_c :average height of the conductor (height above the ground of wire attachment points);
l_S : length of insulator;
S_c : sag of conductor
The effective height of the shield wires is calculated as:

$$h_{esh} = h_{sh} - 1/3\ S_{sh} \tag{9}$$

Where,
h_{esh}: effective height of the shield wire;
h_{sh} : average height of the shield wire;
S_{sh}: sag of shield.

Swinging of Isolator

Swinging of isolator is dimensioning the head of tower; i.e., vertical distance between cross-arms and their length. In modeling of a tower with isolator, the isolator swinging with one of three angle cases depends on the wind loading case. These cases are explained in Table (3). In this study, isolator swinging is considered to be 60^o for maximum speed.

Span Length Design

In transmission line calculations, there are different span lengths that can be considered. These are weight span and wind span, as well as the angle of tower line deviation which is related with span, where a decreased angle can be accommodated with an increased span or *vice versa*. In this study, a light angle of (2^o) is used.

Table 1. Span Length Design

Loading Case	Swinging angle of Isolator
Without Wind	$\pm 0^{\circ}$
Reduced Wind Speed	$\pm 15^{\circ}$
Maximum Wind Speed	$\pm 30^{\circ}$

Earthquake Tower Design
Earthquakes are natural phenomena which cause the ground to shake violently; thereby triggering landslides, creating floods, causing the ground to heave and crack and causing large-scale destruction to life and property. In particular, the effect of earthquakes on structures and the design of structures to withstand earthquakes with no or minimum damage form the subject of earthquake resistant structural design. The important factors which influence earthquake resistant design are: the geographical location of the structure, the site's soil and foundation conditions, the importance of the structure as well as the dynamic characteristics of the structure such as the natural periods and the properties of the structure, like: strength, stiffness, ductility and energy dissipation capacity.
Seismic Coefficient Method

This is the simplest of the available methods and is applicable to structures which are simple, symmetric and regular**.** In this method, the seismic load is idealized as a system of equivalent static loads, which is applied to the structure, and an elastic analysis is performed to ensure that the stresses are within allowable limits. The sum of the equivalent static loads is proportional to the total weight of the structure and the constant of proportionality, known as the seismic coefficient, is taken as the product of various factors which influence the design and are specified in the codes.

Base Shear-International Building Code (IBC)

The IBC addresses the probability of significant seismic ground motion by using maps of spectral response accelerations (S_s and S_1) for various geographic locations. These mapped spectral response accelerations are combined with soil conditions and building occupancy classifications to determine Seismic Design Categories A through F for various structures. Seismic Design Category A indicates a structure that is expected to experience very minor (if any) seismic activity. Seismic Design Category F indicates a structure with very high probability of experiencing significant seismic activity. The equivalent static force procedure in the International Building Code (IBC) specifies the following formula for calculating base shear (*V*):

$$V = C_s W \tag{10}$$

Where the seismic response coefficient, C_s, is defined as:

$$C_s = (2/3)\, F_v\, S_1 I_E / (R_s\, T_f) \tag{11}$$

The IBC specifies the following upper and lower bounds for C_s:

Upper bound:

$$C_s < (2/3)\, F_a\, S_s\, I_E / R_s \tag{12}$$

Lower bound:

$$C_s > (0.044)\, (2/3)\, F_a\, S_s\, I_E \ldots \tag{13}$$

For structures located where, $S_1 > 0.6g$, *Cs* shall not be less than:

$$C_s > 0.5\, S_1\, I_E / R_s \tag{14}$$

W = effective seismic weight of the structure
(Dead loads plus applicable portions of some storage loads and snow loads).
I_E = seismic importance factor.
The IBC provides the following simplified method for estimating
T_f based on the height of the structure (h_n):

$$T_f = C_t\, (h_n)^{3/4} \tag{15}$$

Where:
T_f = fundamental (natural) period of vibration for a structure.
C_t=0.0853 for steel frames; C_t=0.0731 for other structures;
h_n = height of the top level of a structure (ft).

For structures with flat roofs,

- h_n is the distance from the ground to the roof/ceiling system. For structures with sloped (pitched) roofs, h_n may be taken as either the height of the ceiling system above the ground or the mean roof height

- R_s= structural response modification factor.
- S_s and S_1 are maximum spectral response accelerations for short (0.2 second) periods of vibration and for longer (1.0 second) periods of vibration, respectively. Values for S_s and S_1 are provided as contour lines superimposed on maps, in units of percent acceleration due to gravity (%g).

F_v and F_a are seismic coefficients associated with structural sensitivity to the velocity and acceleration (respectively) of seismic ground motion. F_v and F_a are based on the spectral response accelerations (S_s and S_1) associated with the geographic location of the structure and soil conditions at the site. Values for F_v and F_a are specified in IBC. For this study, it is assumed that S_1=0.39 and S_s=0.98.

IV. OPTIMIZATION OF TOWER GEOMETRY

There is a need for reducing the number of independent design variables. Accordingly, the optimization problem is completely stated by only three independent design variables; these are:

1. B: base width of tower.
2. NP: number of panels.

3. R: Panel height ratio.

R can be defined as:

$$H(i+1)=R*H(i) \quad i=1,2,3,\ldots (NP-1)$$

Where:
H (i): height of the i_{th} panel (m);
NP: no. of panels;
R: panel height ratio.
The objective function is also expressed in terms of these three independent design variables as:

$$W=f (B, NP, R).$$

The method of Hooke and Jeeves consists of sequence of exploration steps about a base point, which if successful, are followed by pattern moves.

Design variable for 1S2 tower with angle section and X-bracing under Anti cascading loading condition. (Refer Table No.1).

Table No.1

Iteration no. / Design variable	1	2	3	4	5	6	7	8
B(m)	6.1	5.8	5.3	5.0	4.8	4.3	4.0	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
WEIGHT(KN)	43.79	40.55	38.47	38.56	39.18	39.1	40.75	50.2

Design variable for 1S2 tower with angle section and K-bracing under Anti cascading loading condition. (Refer Table No.2)

Table No.2

Iteration no. / Design variable	1	2	3	4	5	6	7	8
B(m)	6.1	5.8	5.3	5.0	4.8	4.3	4.0	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
WEIGHT(KN)	46.92	44.82	43.37	41.8	41.46	41.45	43.86	43.86

V. ANALYTICAL MODEL DEVELOPED

1. To study the effect of change of base width and height.
2. To study the effect of change of weight and height.

Design of Tower by Indian Code

Data
Voltage of transmission tower 132KV
Line deviation 0-2^0
Take 1.5^0
Normal span 325 m
Wind pressure
Weight of conductor 974KG/KM
0.00974KN/M
Permissible axial tension
Minimum 42.2 KN
Everyday temperature 32^0 22.9 KN
(No wind)
Everyday temperature 32^0 46.4 KN
(Full wind)
Young's modulus 0.815
= 81500 KN/mm
Coefficient of expansion 17.8
= 0.0000178 Per^0 C
Shape factor 0.667
No. of wire 30mm (0.03m)
Diameter 0.003mm
Ground wire
Overall diameter 0.01mm
Effective diameter 9.45mm
Weight 428 KG
Rated UTS 5710 KG
Axial tension (min. temp.) 22.5 KN
Tension at Everyday temperature
32^0(No wind) 11.9 KN
Tension at Everyday temperature
32^0(Full wind) 26.05 KN
Clearance requirement:
Vertical wt. of conductor
Above ground (minimum) 6.1 m
Vertical spacing of power
Conductor (minimum) 4.0 m
Horizontal clearance (mini) 8.2 m
Length of insulator 1.63 m
Variation of temperature
Conductor (5-75 ^{0} C)
70^0 C
Ground wire (5-55 ^{0} C)
50 ^{0} C
Wind pressure

Conductor	1.35 KN/M^2
(consider 0.75)	
Ground wire	0.1 75 KN/M^2
Tower	0.715 KN/M^2
Snow (not expected)	
Every day temperature	32^0C
Horizontal base	1.4 m
Geometry of tower	
Minimum	3.0 m
Power conductor	4.1 m
6.1 m	
4.0 m	
14.2 m	
Maximum sag for	
Power conductor	5.60 m
Tension in transmission cable	
W1	22.5 KN
Max. Temp. No wind W2	11.9 KN
Max temp full wind W3	26.05 KN
Area	0.00021195
$(W2)^2L^2EA/24$	10765654.37
Wind load	2.02601E-05
KN/m	
W1	10765654.38
W2	11.1003293
T2	13.4 KN
d	9.596 m
Step 3:	
Height of tower	23.79
Height of tower	
(may be fixed as)	23.8 m
Step 4:	
Width of base of tower	5.84 m
Take base width	5.8 m
Wind percentage	0.75
Constant	2.975
Step 5:	
Length of members	
DD’	2.154
CC’	3.196
BB’	4.379
AA’	5.8
EE’	1.4
FF’	1.4
GG’	1.4
ED’=ED’	3.48 M
DC’=CD’	4.52 M
CB’=BC’	5.13 M
AB’=BA’	6.17 M
AB=A’B’	7.02 M
BC=B’C’	5. 13M
CD=D’C’	4.52 M
DE=E’D’	3.69 M
Length of EF = E’F’	2 m

Lateral load due to wind	
Panel joint B	
Column	3.40
Horizontal	0.36
Diagonal	0.87
Secondary’s	0.31
Lateral load due to wind	6.70
Panel joint C	
Column	2.70
Horizontal	0.24
Diagonal	0.74
Secondary’s	0.25
Lateral load due to wind	5.33
Panel joint D	
Column	2.30
Diagonal	0.63
Secondary’s	0.21
Lateral load due to wind	4.25
Panel joint E	
Column	0.56
Horizontal	0.10
Diagonal	0.33
Cross arm	5.15
Lateral load due to wind	8.30
Lateral load (wind on conductor)	7.53
Lateral load due deviation	46.4 KN
2T Sinϴ	2.42 K N
Total	18.26
Panel joint F	
Column	1.08
Horizontal	0.21
Diagonal	0.18
Cross arm	1.81
Lateral load due to wind	4.43
Lateral load (wind on conductor)	10.60
Lateral load due deviation	46.6KN
2T Sinϴ	2.42 K N
Total	17.47 KN
Panel joint G	
Total lateral load	17.47
Panel joint H	
Lateral load due to wind	0.945 KN
Lateral load (wind on ground wire)	2.19 KN

Lateral load due deviation 46.6KN
2T Sinϴ 1.17KN
Total 4.31KN

Dead load acting on the tower
M 1416.90 KN
W 35.830 KN
Trial weight of the tower
Column 46.10
Horizontal 6.32
Diagonal 6.03
Secondary's 5.75 KN
Cross arm 7 KN
Total estimated weight of tower 71 KN
Weight of 3 powers Conductor 09 KN
Weight of ground wire 1.95 KN
Weight of line man and tool 1.5 KN
Total dead load 84.16 KN

Forces on the tower under top most power conductor in broken condition
Lateral load due to wind on panel joint G 4.43 KN
Lateral load (wind on conductor (60%)) 0.057 KN
Deviation on conductor 2.21 KN
Total 6.71 KN
Tensile force on tower (broken condition) 27.84 KN
Torsional force (broken condition) F1 33.05KN
Lateral load on conductor (40%) 1.26
Total dead load 82.89

Lateral load on ground wire (broken condition)
Lateral load due to wind on panel joint G 0.945 KN
Lateral load ground wire (broken condition) 60%
Lateral load on conductor (60%) 1.316 KN
Deviation on conductor 1.11 KN
Total 2.43 KN
Working Tensile (100%) 26.05 KN

Dead load on ground wire (40%)
Total dead load 83.38
Stresses in various member of the tower under normal operational condition of the conductor
Horizontal reaction @ foot column13.025 KN
Maximum B.M 708.454 KN

Spacing between
Column 5.84 m
Vertical reaction 121.11 KN
Axial force in column Point A
Column 134.71
Diagonal 132.51
Axial force in column Point C
Reaction at C 88.38
Column 76.89
Diagonal 11.49
Axial force in column Point E
Reaction at E 58.95
Column 46.57
Diagonal 12.38
Stresses in tower under top most power condition
M' 1171.45 KN M
Maximum B.M 585.72 KN
Vertical reaction 100.13 KN
Horizontal reaction 0.929 KN
Spacing between Column 5.84 m
Axial force in column Point A'
Column 129.17
Diagonal -16.60
Reaction at C' 75.08
Column 65.32
Diagonal 9.76
Reaction at E' 50.08
Column 39.56
Diagonal 10.51
Stresses in member due to longitudinal force acting on tower
Maximum B.M 317.37 KN
Vertical reaction 54.25 KN
Horizontal reaction 79.34 KN
Axial force in column Point E
Column 138.33
Diagonal -59.05
Reaction at C' 52.87
Column 46.00
Diagonal 6.87
Reaction at E' 35.26
Column 27.86
Diagonal 7.40
Stresses in member due to torsional force acting on tower
Maximum B.M 753.68 KN
Vertical reaction 128.85 KN
Horizontal reaction 16.52 KN
Axial force in column Point E
Column 105.24
Diagonal 23.81
Reaction at C' 86.08
Column 74.89
Diagonal 11.19
Reaction at E' 57.41
Column 45.36

Diagonal	12.05
Stresses in tower under top most power condition	
M’	1372.00 KN M
Maximum B.M	686.00 KN
Vertical reaction	117.28 KN
Horizontal reaction	12.08 KN
Axial force in column Point A’	
Column	131.56
Diagonal	-2.61
Reaction at C’	86.00
Column	74.82
Diagonal	11.18
Reaction at E’	57.36
Column	45.31
Diagonal	12.04

Stresses in member due to longitudinal force acting on tower	
Maximum B.M	309.995 KN
Vertical reaction	52.99 KN
Horizontal reaction	6.51 KN
Axial force in column Point E	
Column	44.08
Diagonal	11.35
Reaction at C’	36.97
Column	32.97
Diagonal	4.80
Reaction at E’	24.66
Column	19.48
Diagonal	5.17

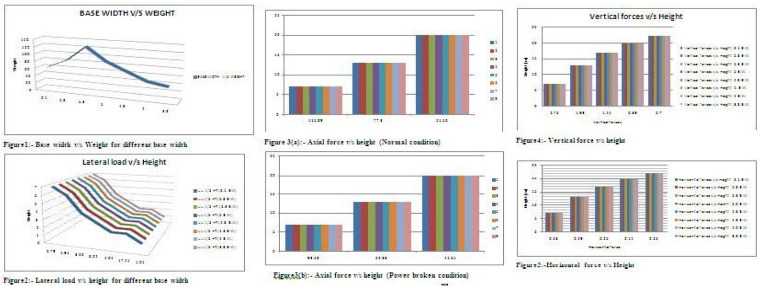

Figure 1: The effect of change of base width and height

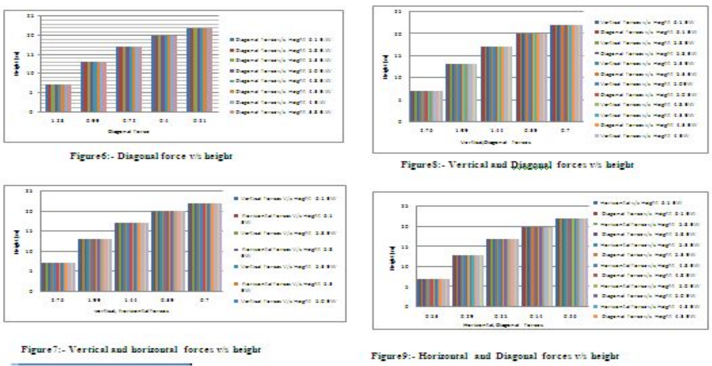

FIGURE 2. Graph represents Force V/S Height of transmission tower

VI. CONCLUSION

1. After the study of tower with different types of base width, it is clear that height of tower is directly proportional to the base width.

2. After the study of tower with different types of base width for normal condition, it is clear that base width of tower is inversely proportional to the axial forces.

3. The study of different loading conditions on structures is very important to recognize the case that will cause the larger deflection in tower model and exceed the yield stress to decide which case will be optimized.

4. The geometry parameters of the tower can efficiently be treated as design variables, and considerable weight reduction can often be achieved as a result of geometric changes.

5. The tower with different angle section decides the weight of Tower. And the Tower structure with least weight is directly associated in reduction of the foundation cost.

6. The transmission tower with X-bracing is lighter than that with K-bracing with angle sections under wind and seismic load conditions.

7. It is found that the 1S2 tower with angle section and X-bracing, under anti-cascade loading condition, has a reduction in weight of about 14% of the weight before optimization.

8. The tower with angle section and X-bracing has the greater reduction in weight after optimization (reaching 21%).

9. For seismic loading conditions, the reductions were about 24% for 1S2 type of tower.

10. Optimization of tower geometry with respect to member forces. The tower with base width 4.8 M is concluded as the optimum tower configuration with respect to geometry.

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