https://doi.org/10.15623/ijret.2018.0709001 Received: 13-06-2018, Accepted: 25-07-2018, Published: 15-08-2018

CAPACITY UTILIZATION STUDY OF 60m HIGH THREE-LEGGED HYBRID SELF-SUPPORTING TELECOMMUNICATION TOWER (INDIAN AND AMERICAN STANDARDS)

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Abstract

There are different tower configurations like four legged angular, three legged angular and three-legged hybrid towers are popularly adopted in telecom tower constructions. Hybrid tower means the tower with main legs are pipes and other members like bracings, horizontals and redundant members are angular members. Selection of hybrid tower is mainly due to three-legged configuration and pipe sections have less drag coefficients compared to four legged angular towers which resulting less wind load on tower for structure weight optimization. Lattice towers are light weight structures and governed by wind loads due to tower body, antennas and other equipment. Current industry practice for design of telecom structures from the available general structures building loading and design standards like IS : 875 (Part 3) – 1987 for load calculations and IS 800-1984 for design of pipe sections and IS 802 (Part 1/sec 2) – 1992 for angular sections design.

Recently the loading standard and design standard has been revised IS 875 (Part-3)-2015 and IS 800-2007. So, it is important to understand the effects of new standards on already designed and installed towers using old codes and understanding the provisions of revised standard in line with the international design standards like ANSI/TIA-222G.

60m high 3-Legged Hybrid self-supporting telecommunication tower with X pattern bracing has been selected forthe study. This paper explains the codal provisions for calculating wind loads [IS 875 (Part-3)-1987 &IS 875 (Part-3)-2015] and members capacities [IS 802 (Part 1/Sec 2)-1992, IS 800-1984 & IS 800-2007] using old and new Indian codes along with American Code (ANSI/TIA-222-G).Comparison statements and conclusions have been presented at the end of the paper.

KeyWords: *Towers and masts, Telecom tower, Angular tower, Tower body, GBT tower, RTT tower, GSM antenna, Microwave antenna, Appurtenances, IS 875, IS 802, IS 800, ANSI/TIA-222G, Tower optimization, Tower sharing, Tower strengthening etc.*

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1. INTRODUCTION

With the recent demands for communication due to the upgrading of technology from 2G to 3G,4Gand 5G/LTE has robust the requirement for better wireless and broadcast communication. Telecom sector has become the core infrastructure required for growth and modernization in various sectors of the economy.

The rise in the telecommunication sector has also increased the demand for the steel production to setup the network infrastructure. The spike in telecommunication industry and the ever-competent market amongst the service providers has increased the demand for installing towers capable of attracting more subscribers to fulfill their need.

Thus, to fulfill ongoing demand and accounting the expense, optimized tower design and less utilization of steel in telecom construction driving the cost of infrastructure. It will not only ensure cost but also less use of steel lead to sustainable development by emitting less carbon.

Wind load is governing criteria apart from earthquake and snow load thus a tower of different configuration can be selected to suit the location and terrain conditions to ensure a safe design. It is important for the structural engineer, to understand the various loading and design code provision for effective utilization of the steel ensuring economy in tower construction.

Considering the economy and safety;with a view to optimize the geometrya Three-legged self-supporting hybrid telecommunication tower has been selected for the study. The tower has a tubular pipe member for main legs and angle for bracings which are aligned in an X pattern.The tubular member is taken for main legs since it provides less wind load and higher buckling capacity due to more radius of gyration as compared to angular tower.

In the current study, capacities utilization of the 3-legged hybrid tower has been compared using Indian and American code are presented in the paper.

Following are the aspects attempted to study in this project:

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1. Calculation of tower body, ladders, cables, antennae and other appurtenances wind loads as per IS 875(Part3)-1987, IS 875(Part3)-2015, ANSI/TIA – 222G
2. Calculation of main leg capacities, bracing capacities as per IS800:1984, IS 800:2007, IS 802 (Part 1/Sec 2)-1992 and ANSI/TIA – 222G
3. Comparative study between the Indian Standards and American Standards for wind loads, member forces, member capacities and earthquake base shears has been presented.
4. Even though knowing the fact that the towers are not governed by earthquake loads compared to wind loads, an independent modal analysis carried out to validate the overall base shear due to earthquake is less than wind loads.

2.TOWER AND APPURTENANCES DETAILS

2.1 Tower Geometry

The Tower chosen for the study is a 60m high, self-supporting, 3-legged Hybrid tower with top 10m height as a straight portion and remaining 50m height as a tapered portion having a base width of 6.5 m and top width of 1.5 m. The tower has a tubular pipe member for main legs and angular members for bracings.

Table 1. Details of Tower

Height of Tower	60m
Base width	6.5m
Width at the top of Tapered Portion	1.5m
Top width	1.5m
Height of Top Straight Portion	10m
Height of Tapered Portion	50m

2.2 Modelling

The lattice tower model has been analyzed in STAAD.Pro.V 8i. The model was created using the coordinate data for the points and suitable cross-sectional properties were assigned to the members created. The boundary condition was assigned in the model by fixing the three lowermost nodes of the modelled structure as a pin joint. For the present study, a self-supporting 3-legged hybrid tower has been modelled with Main legs as a Tubular member, bracings and horizontals as Indian Standard Angles. Warren system (X type) bracings pattern are considered; with horizontals in the top straight portion. Whole tower has been divided into 24 panels (12 sections) as represented in Fig-1. Tower sections have been divided in such way that the main leg member fabrication cut lengths to the best of utilization to avoid wastage from the standard bar sizes available in the market. A section (panel) is considered for every 5m and each section contains 2 main legs, 4 diagonals in tapered portion and additional 1 horizontal in a top straight portion in each frame. Tower Member will carry only axial force, it will not carry moment due to which member releases are done for main legs and member truss is done for bracings in truss modelling.Beta angles are used to specify the orientation of angular members in the tower model as per the actual tower geometry. The wind loads calculated are applied at appropriate nodes and the member forces, utilizations and deformation of the structure under the effect of the applied load is studied.

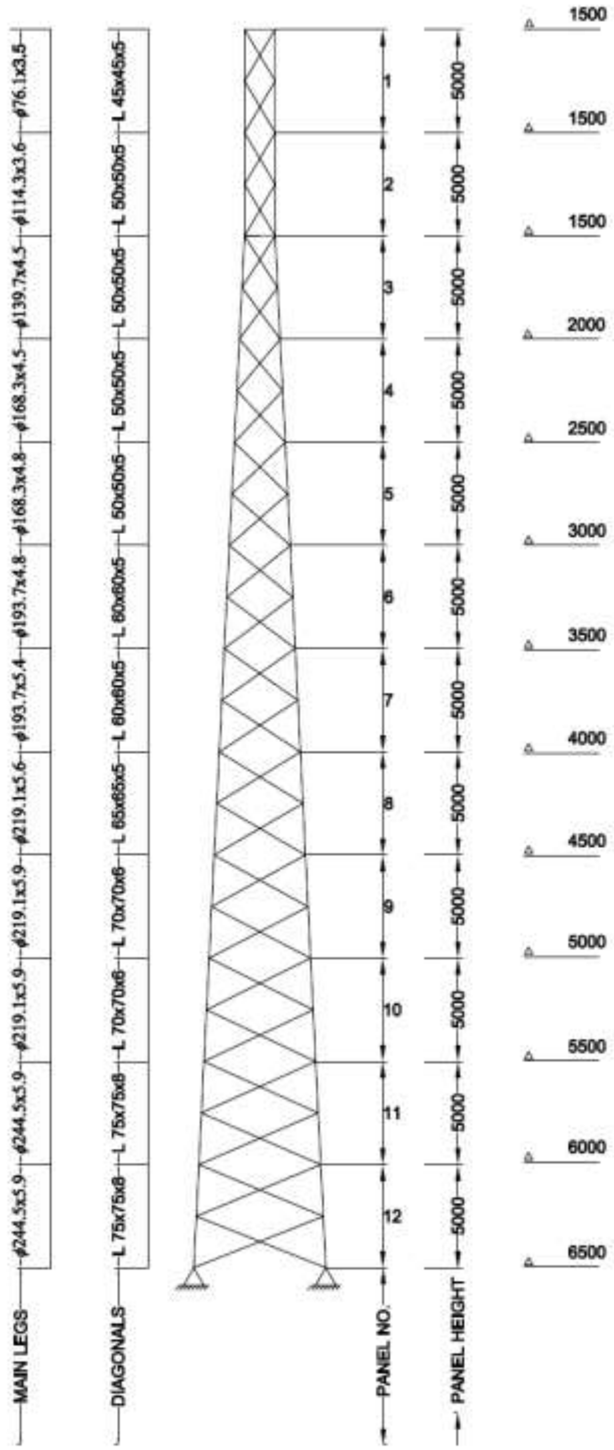

Fig -1: Three-Legged Hybrid Tower Geometry

Table 2. List of Sections used in the Design

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PANEL NO #	MAIN LEGS	BRACINGS	HORIZONTALS
1	PIP761M	ISA 45X45X5	ISA 45X45X5
2	PIP1143L	ISA 50X50X5	-
3	PIP1397L	ISA 50X50X5	ISA 45X45X5
4	PIP1683L	ISA 50X50X5	-
5	PIP1683M	ISA 50X50X5	-
6	PIP1937L	ISA 60X60X5	-
7	PIP1937M	ISA 60X60X5	-
8	PIP2191M	ISA 65X65X5	-
9	PIP2191H	ISA 70X70X6	-
10	PIP2191H	ISA 70X70X6	-
11	PIP2445H	ISA 75X75X8	-
12	PIP2445H	ISA 75X75X8	-

Panel from top to bottom

- For Pipe Sections Yield Stress = 310MPa as per IS 1161-2014
- For Angular Sections Yield Stress = 250Mpa as per IS 2062-2006

2.3 Appurtenances Specifications

Tower is equipped with Appurtenances which include discrete ancillaries like GSM antennas, Microwave (MW) antennas, platforms and linear ancillaries like access & cable ladder and cables. For the present study,the tower antenna loading comprises of 4-operator antenna loading equal to 9 nos. of GSM (size: 2.562mx0.262mx0.2m) + 6 nos. of GSM (size: 1.936m x 0.262m x 0.2m) and 9 nos. of 0.6m dia standard MW antennae with radome with 20mm diameter cables running along the height of tower. Access and cable ladder consists of 400mm width ,16 mm diameter solid rod for climbing with two vertical angles of L45x45x4 supported and safety guard by horizontal hoop and vertical flats of 30x4mm.

Following are the equivalent effective exposed area of appurtenances considered in the present study.

- Antenna area = 1.5m^2/m (applied for top 10m vertical part equally)
- Access and cable ladder = 0.3m^2/m (Overall height of tower
- Cables for antenna = 0.2m^2/m (Overall height of tower)

2.4 Twist & Sway at operational wind speed

Service loads are the loading combination used to calculate serviceability limit state deformations. The deformations like twist & sway under service loads at any point on a structure shall not exceed 1.0 degree at 75% of the basic wind speed. This limit defined from radio frequency and microwave signal line of sight (LOS) requirements.

Calculated limits of Sway in local vertical angular rotation and twist of horizontal angular rotation of the antenna beam path from the unfactored no-wind load position.

3.WIND LOAD

As per clause 7.1 of IS 875 (Part3) :1987 and cl. 9.1 of IS 875 (Part3) :2015, wind induced dynamic effects need to be considered for flexible slender structures whose natural frequency less than 1.0 Hz. Natural frequency has been calculated using eigen value method in STAAD pro and cross verified with empirical formulas provided in ANSI/TIA-222G and noted that the first mode frequency is greater than 1.0 Hz, which directing the wind loads calculations to be performed for static wind loads as defined in clause 6.3 of IS 875 (Part 3)-1987 and clause 7.4 of IS 875 (Part 3) – 2015.

The tower is designed for wind loads with a basic wind speed of 47 m/s. Mean probable design life of tower is taken as 50 years with terrain category 2 and flat terrain. Basic wind speed is adopted for design purposes, there is always a probability (however small) that it may be exceeded in a storm of exceptional violence. Mean probable design life of 50 years is normally considered sufficient for design of towers with risk level probability level of 0.63 against wind effects. This is the acceptable probability that the wind speed of 47m/s will be exceeded in any one year during the 50 years life of the structure for the accepted risk. Recommended mean probable design life of all building and structures is 50 years as per IS 875 (Part 3) wind loading standard.

In modern telecom tower design standards ANSI/TIA-222G, the tower loads calculated based on classification of structure considering the reliability requirement of the structure, on the land use surrounding the structure and performance requirements of the services provided. Structures used for services that may be provided for cellular and microwave communications selected as class II structures for calculating wind loads on tower.

3.1IS 875 Part 3-1987

The design wind speed is given as follows by incorporating all the factors:

$$V_z = V_b\, k_1\, k_2\, k_3$$

where,

V_z = Design wind speed in m/s
V_b = Basic wind speed in m/s
k_1 =Probability factor (risk coefficient)
k_2=Terrain roughness and height factor
k_3 =Topography factor
z = a height in meters or distance above the ground.

Wind pressure is determined as follows:

$$p_z = 0.6V_z^2$$

where,

p_z = Wind pressure at any height z in N/m^2

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V_z = Design wind speed in m/s

Finally, the wind load acting on the tower body is computed as follows:

$$F_z = p_d \times C_f \times A$$

where,

p_d = Design wind pressure, p_z
A = Exposed area
C_f = Force coefficient of the component.

Input Parameter

- Basic wind speed: 47m/s
- Terrain category: 2 (Obstructionsbetween 1.5m to 10m in height)
- Class C (60m in height Dimension)
- Mean Probable Design life of structure: 50 years
- Risk Coefficient k_1 – 1.0
- Terrain, height factor k_2 – varies with height
- Topography factor k_3 – 1.0 (assumed flat terrain)

3.2 IS 875 Part 3-2015:

The design wind speed is given as follows by incorporating all the factors:

$$V_z = V_b\, k_1\, k_2\, k_3\, k_4$$

where,

V_z = Design wind speed in m/s
V_b = Basic wind speed in m/s
k_1 =Probability factor (risk coefficient)
k_2 =Terrain roughness and height factor
k_3 =Topography factor
k_4 =Importance factor for cyclonic region
z = a height in meters or distance above ground

$$p_z = 0.6\, V_z^2$$

where,

p_z= wind pressure at any height z in N/m2
Vz = design wind speed in m/s

Design Wind pressure is determined as follows:

$$P_d = K_d\, K_a\, K_c\, p_z$$

where,

K_d = Wind directionality factor
K_a = Area averaging factor
K_c = Combination factor
p_z= Wind pressure at any height z

Finally, the wind load acting on the tower body is computed as follows:

$$F_z = p_d \times C_f \times A$$

where,

p_d = Design wind pressure
A = Exposed area
C_f = Force coefficient of the component.

Input Parameter

- Basic wind speed: 47m/s
- Terrain category: 2 (Obstructions between 1.5m to 10m 10m in height)
- Mean Probable Design life of structure: 50 years
- Risk Coefficient k_1 – 1.0
- Terrain, height factor k_2 – varies with height
- Topography factor k3 – 1.0 (assumed flat terrain)
- Wind directionality factor K_d : 0.9 (Lattice Tower)
- Area averaging factor K_a (To be calculated based on the tower Tributary Area for the selected configuration)
- Combination factor K_c :1.0

3.3 ANSI/TIA-222G (AMERICAN CODE)

The velocity pressure, q_z, evaluated at height z determined by the following formula:

$$q_z = 0.613\, K_z\, K_{zt} K_d\, V^2\, I$$

where,

z = Height above ground level at the base of the structure
K_z= velocity pressure coefficient
$= 2.01(z/z_g)^{2/\alpha}$
K_{zt}= topographic factor
K_d= wind direction probability factor
V = the basic wind speed for the loading condition under investigation,m/s
I = importance factor

The design wind load on the tower computed as a sum of load on structure and appurtenances is given as below

$$F_w = F_{ST} + F_A$$

The design wind force, applied to each section of a structure shall be determined in accordance with the following:

$$F_{ST} = q_z\, G_h\, (EPA_s)$$

where,q_z = velocity pressure

G_h = gust effect factor
= 0.85 + 0.15 ((h/45.7)-3.0), h in meters
$0.85 \le G_h \le 1.0$
h = Height of structure in meters
$(EPA)_s$ = effective projected area of the structure

$$EPA_s = C_f\, [D_f \sum A_f + D_r \sum (A_r\, R_r)]$$

where,

C_f $= 3.4\varepsilon^2 - 4.7\varepsilon + 3.4$(for triangular cross sections)
ε = solidity ratio = $(A_f + A_r)/A_g$
D_f = 1.0 (For normal wind direction)
= 0.85(± 90 degrees wind direction)
D_r = Wind direction factor for round components
= 1.0 (For normal wind direction and ± 90 degrees wind direction)
R_r = Reduction factor for round components
$= 0.57 - 0.14\,\varepsilon + 0.86\,\varepsilon^2 - 0.24\,\varepsilon^3 \le 1$ when C<4.4 (subcritical flow)
$= 0.36 + 0.26\,\varepsilon + 0.97\,\varepsilon^2$ when C>8.7] (supercritical flow)

where:

$$C = [I\, K_Z\, K_{ZT}]^{1/2}\, V\, D$$

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I = importance factor
K_Z = velocity pressure coefficient
K_{ZT} = topographic factor
V = the basic wind speed for the loading condition under investigation,m/s
D = outside diameter of the structural component without ice, m

The design wind force on appurtenances shall be determined from the equation:

$$F_A = q_z G_h (EPA)_A$$

where,
q_z = velocity pressure at the centerline height of the appurtenance
G_h = gust effect factor
$(EPA)_A$ = effective projected area of the appurtenance

Input Parameter

- Basic wind speed considered: 47m/s
- Wind direction probability factor, K_d : 0.85
- Exposure category: C (Obstructions<9.1m in height)
- For Class – II, Importance factor, I = 1.0
- Z_g = 274m, α = 9.5, $K_{z\ min}$ =0.85
- Velocity pressure coefficient, $K_z=2.01(z/z_g)^{2/\alpha}$
- For Topographic category – 1, K_{zt}= 1.0 (flat terrain)

3.4 Wind directions

For Lattice triangular structures the loading of wind should be checked with following possible directions.

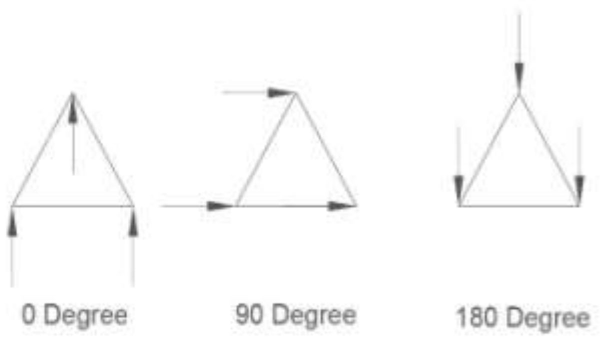

Fig -2: Wind Load Directions

3.5 Wind Load Combinations

Limit state of strength: A limit state of collapse or loss of structural integrity.

Limit state of serviceability: A limit state of deflection limit beyond which the service criteria are no longer met.

Following load combination has been used for analyzing the tower from various codes.

DL = Dead Load, WL = Wind Load.

Table 3. Load combinations

Code	Limit State of Strength	Limit State of Serviceability
IS 875 (Part-3) + IS 800 & IS 802	0.9DL+1.5WL 1.5DL+1.5WL	1.0DL+1.0WL
ANSI/ TIA-222G	0.9DL+1.6WL 1.2DL+1.6WL	1.0DL+1.0WL

4. MEMBER DESIGN

Tower members are designed for both tension and compression force. However, optimized profile size governed by member compression force 0with critical slenderness ratio than tension force with net effective cross section of the member. Thus the theory, calculations and comparisons presented in this paper corresponding to member compression case only.

4.1 Slenderness limits

f_y = Yield stress of steel of Pipe members = 310 MPa
f_y = Yield stress of steel of angular members = 250 MPa
E = Modulus of elasticity of steel= $2x10^5$ MPa
Kl/r = Effective length to appropriate radius of gyration

Table 4. Slenderness Limits

MEMBER	IS 802:1992	ANSI/TIA-222G
	Applicable on KL/r	Applicable on L/r
Main Legs	120	150
Bracings	200	200
Horizontals	200	200
Redundants	250	250

Based on type of members and value of L/r, the following conditions of KL/r have been used in calculations as applicable.

Table 5. Effective slenderness ratios

Slenderness ratio <120, Eccentricity Governs		
Formula Number	**Equation**	**Conditions at Ends of the Buckling Length Under Consideration**
1	KL/r = L/r	Concentric at both ends
2	KL/r = 30+0.75 L/r	Concentric at one end and normal framing eccentricity at the other end
3	KL/r = 60+0.5L/r	Normal framing eccentricity at both ends

https://doi.org/10.15623/ijret.2018.0709001 Received: 13-06-2018, Accepted: 25-07-2018, Published: 15-08-2018

Slenderness ratio ≥ 120, Restraint Governs		
Formula Number	**Equation**	**Conditions at Ends of the Buckling Length Under Consideration**
4	KL/r = L/r	Unrestrained against rotation at both ends
5	KL/r = 28.6+0.762 L/r	Partially restrained at one end and unrestrained at other end
6	KL/r = 46.2+0.615 L/r	Partially restrained against rotation at both ends

4.2 IS 800: 1984

The permissible stress of the compression member is determined as follows:

$$\sigma_{ac} = 0.6 \frac{f_{cc} f_y}{[f_{cc}^n + f_y^n]^{1/n}}$$

where,

f_y = yield stress of steel, in MPa
f_{cc} = elastic critical stress in compression

$$= \frac{\pi^2 E}{(KL/r)^2}$$

E = modulus of elasticity of steel :2 x 10^5 MPa
n = constant,1.4

4.3 IS 800-2007

It is based on principles of Limit State Design.
The design compressive strength of the member is calculated as follows
The following equation to find design stress in axial compression (f_{cd}).

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}}$$

where,
ϕ =0.5[1+ α (λ – 0.2) + λ^2]
λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r}\right)^2 / \pi^2 E}$$

f_{cc} = Euler buckling stress = $\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

γ_{m0} = partial safety for material strength
For Pipe sections α value is 0.21 and Angle sections α value is taken as 0.34 (i.e., class-b) as updated in NBC: 2015 (typo mistake in current IS 800-2007)

$$P_d = A_e f_{cd}$$

where,
A_e = effective sectional area
f_{cd} = design compressive stress

Effective slenderness ratio (KL/r)

Taking IS 800: 2007 compression formula and applying IS 802 (part1/section2): 1992 effective lengths to tower member design leads to lesser compression capacities. For present study effective lengths considered same as industry adopted IS 802(part1/section2): 1992.

4.4 IS 802 (PART-1/ SECTION-2): 1992

It is based on principles of Ultimate load concept.
Permissible Axial Stress in compression can be computed as follows:

a) F_a={1-0.5*$\left(\frac{KL/r}{C_c}\right)^2$}x F_y ; when KL/r ≤ C_c,

b) $F_a = \frac{\pi^2 E}{(KL/r)^2}$;when KL/r > C_c,

The above formula is applicable provided the largest width thickness *b/t* is not more than the limiting value and if exceed the limiting value then the formula given above shall be modify by substituting for F_y the value of F_{cr} which is given by:

a) F_{cr}={1.667-0.667*$\left(\frac{(b/t)}{(b/t)_{lim}}\right)$ }*F_y ;
when $(b/t)_{lim} \le (b/t) \le 378/\sqrt{F_y}$

b) $F_{cr} = \frac{65550}{(b/t)^2}$;when $(b/t) > 378/\sqrt{F_y}$

where,

$$(b/t)_{lim} = 210/\sqrt{F_y}$$

b = distance from the edge of the fillet to the extreme fibre in mm, and
t = thickness of flange in mm,

$$C_c = \pi \sqrt{\frac{2E}{f_y}}$$

F_y= minimum guaranteed yield stress of material, in MPa
E = modulus of elasticity of steel: 2 x 10^5 Mpa
KL/r =largest effective slenderness ratio of any unbraced segment of the member

4.5ANSI/TIA-222G

It is based on principles of Load and resistance factor design.

The design axial strength of the compression member is determined as follows:

a) $F_{cr} = (0.658^{\lambda_c^2})$ for $\lambda_c \le 1.5$
b) $F_{cr} = [0.887/\lambda^2_c]* F'_y$ for $\lambda_c > 1.5$
where,

$$\lambda_c = \frac{KL}{r\pi} \sqrt{\frac{F'_y}{E}}$$

A_g = gross area of member, [mm2]
F'_y = effective yield stress, [MPa]
E = modulus of elasticity, [MPa]
K = effective length factor
L = laterally unbraced length of member, [mm]
r = governing radius of gyration about the axis of buckling, [mm]

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The design axial strength of compression members shall be taken as $\phi_c P_n$:
$P_n = A_g * F_{cr}$
$\phi_c = 0.9$

For 60° and 90° angle members, the effective yield stress for axial compression, F'_y, shall be determined as follows:

$$\frac{w}{t} \leq 0.47\sqrt{\frac{E}{F_y}} \qquad F'_y = F_y$$

$$0.47*\sqrt{\frac{E}{F_y}} < \frac{w}{t} \leq 0.85\sqrt{\frac{E}{F_y}} \qquad F'_y = \left[1.677 - 0.677\left[\frac{\frac{w}{t}}{0.47\sqrt{\frac{E}{F_y}}}\right]\right]*F_y$$

$$0.85*\sqrt{\frac{E}{F_y}} < \frac{w}{t} \leq 25 \qquad F'_y = [0.0332 * \pi^2 * E/(\frac{w}{t})^2]$$

The width to thickness ratio (w/t) shall not exceed 25 for angle members

For solid round members, the effective yield stress, F'_y, shall be equal to F_y

For tubular round members, the diameter to thickness ratio (D/t) shall not exceed 400. The effective yield stress, F'_y, shall be determined as follows:

$$\frac{D}{t} \leq \frac{0.114E}{F_y} \qquad F'_y = F_y$$

$$0.114\frac{E}{F_y} < \frac{D}{t} \leq 0.448\frac{E}{F_y} \quad F'_y = \left(\frac{0.0379E}{\frac{D}{t}F_y} + \frac{2}{3}\right)F_y$$

$$0.448\frac{E}{F_y} < \frac{D}{t} \leq 400 \qquad F'_y = 0.337E/(\frac{D}{t})$$

where ,
D = outer diameter of tubular member
t = wall thickness of tubular member
E = Elastic modulus of the tubular members

5. EARTHQUAKE LOADS

Total design lateral force or design base shear along any principal direction is given in terms of design horizontal seismic coefficient and seismic weight of the structure.

Design horizontal seismic coefficient depends on the zone factor of the site, importance of the structure, response reduction factor of the lateral load resisting elements and the fundamental period of the structure.

The procedure generally used for the equivalent static analysis is explained below:
i) Determination of fundamental natural period of structure
ii) Determination of Seismic base shear

$$V_b = A_h . W$$

A_h = Design horizontal seismic coefficient,

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$$

- Zone factor = 0.36 (Zone-5)
- Importance factor = 1.5 (Important structures)
- Response reduction Factor = 4.0
- For medium stiff soil site and for Time Period 0.55<T<4 ; Sa/g = 1.36/T = 1.36/0.714 = 1.905

W = total weight of structure including all appurtenances on the tower

6. OBSERVATIONS

1. The Current Industry practice of designing approach for the telecommunication towers is as per IS 875 part3:1987 for the calculation of wind loads and IS 802(Part1/Section2):1992 for the design of angular members of bracings, horizontals and IS 800-1984 for design of pipe members of main legs.
2. IS 875 (Part 3)- 1987 represents classification of structures depending upon their sizes into A (<20m), B (20m-50m) and C (>50m) to choose terrain roughness coefficient and height factor. Whereas in new IS 875 (Part 3) – 2015, B and C classes have been deleted and Class A values retained same as 1987 version of the standard to the 2015 version. This parameter resulted structures of B class (20m-50m) to 6% and C Class (>50m) to 11%-17% increase in wind loads.
3. A new "Cyclonic importance factor, K_4" based on the structures post cyclone importance introduced in IS 875 (Part 3)- 2015. This factor applies only for cyclone regions of 60m wide on the east coast as well as on the Gujarat coast. Current tower design study of 47m/s is not cyclonic region. Hence factor of 1.0 considered.
4. A new "wind directionality factor, K_d" of 0.9 for lattice towers introduced in IS 875 (Part 3)- 2015. This is higher compared to ANSI/TIA-222 G "wind direction probability factor of 0.85.
5. A new "Area averaging factor, K_a" based on the tributary area introduced in IS 875 (Part 3)- 2015. As the area becomes larger, the correlation of measured values decreases and vice-versa. The decrease in pressures due to larger areas may be taken into account by Ka factor.
6. In the current study, tower face tributary area is ≥100m2 for the tower configuration, Ka factor = 0.8 as per Table 4 of IS 875 (Part 3) :2015. This is in line with Gust effect factor Gh =0.85 as given in ANSI/TIA-222G for structures of less than 137m height. However, Ka factor impact on overall tower design studied in this paper using Ka factor of both 0.8 and 1.0.
7. IS 800:2007, the angular members buckling class for strength design mentioned as "class c". It is a typographical error of buckling "class b" and updated in latest amendment of National Building Code. Current study done with buckling "class b".
8. IS 800:1984 having 0.6xFy to calculate permissible compressive stress. As the towers being designed using

https://doi.org/10.15623/ijret.2018.0709001 Received: 13-06-2018, Accepted: 25-07-2018, Published: 15-08-2018

factor of safety of 1.5, the permissible stress been modified to suit to overall factor of safety of 1.5.

9. IS 800 :2007 having 1.5 for partial safety factor for loads and 1.1 for partial safety factor for material. This leads to overall safety factor of 1.65 as against current Industry practice of maintaining 1.5 overall tower factor of safety. Hence in the current study comparisons done for both conditions of material partial safety factors 1.0 and 1.1.

7. TABLES

The following table provides the comparison of Wind Load, Forces obtained from Staad-Pro, Capacities from codes finally the Utilization. Thereby, indicating the Tower rating

Table -6: Wind Load-3-Legged Hybrid
(For all wind directions 0deg., 90 deg.& 180 deg.)

Panel No.#	**Total Wind Load (kN)**			
	IS 875 Part 3: 1987	**IS 875 Part 3:2015**		**ANSI/TIA-222G**
		Ka =0.8	**Ka =1**	
1	21.03	20.52	21.49	18.07
2	20.60	20.16	21.08	18.28
3	9.19	8.31	9.37	8.52
4	9.48	8.59	9.75	9.00
5	9.43	8.56	9.74	9.20
6	10.50	9.51	10.95	10.22
7	10.56	9.64	10.92	10.33
8	11.22	9.97	11.59	10.98
9	11.54	10.26	11.99	11.16
10	11.17	9.94	11.64	10.84
11	11.83	10.45	12.31	10.72
12	12.30	10.84	12.80	8.83

Panel from top to bottom

Table -7: Main Legs Capacity-Pipes members, in kN

Panel No.#	**IS:800-1984**	**IS:800-2007, γ=1.0**	**IS:800-2007, γ=1.1**	**ANSI/TIA-222G**
1	123.46	131.42	119.47	122.60
2	285.44	308.48	280.44	266.92
3	488.90	513.54	466.86	444.98
4	638.86	654.87	595.34	572.28
5	679.88	697.02	633.66	609.07
6	814.38	824.58	749.62	724.91
7	911.15	922.68	838.80	811.09
8	1099.08	1105.63	1005.12	975.77
9	1155.07	1161.99	1056.35	1025.49
10	1155.07	1161.99	1056.35	1025.49
11	1312.85	1316.33	1196.66	1164.60
12	1312.85	1316.33	1196.66	1164.60

Panel from top to bottom

Table -8: Bracings Capacity- Angular members, in kN

Panel No.#	**IS 802-1992**	**IS:800-2007, γ=1.0**	**IS:800-2007, γ=1.1**	**ANSI/TIA-222G**
1	35.65	28.80	26.19	28.14
2	47.50	37.34	33.95	37.49
3	38.26	31.10	28.27	30.20
4	33.24	27.52	25.02	26.24
5	28.55	24.04	21.86	22.53
6	39.90	33.02	30.02	31.49
7	34.39	28.95	26.32	27.14
8	37.18	31.32	28.47	29.35
9	47.30	39.90	36.27	37.62
10	41.54	35.49	32.27	33.04
11	57.97	49.60	45.09	46.58
12	51.55	44.56	40.51	41.42

Panel from top to bottom

Table -9: Main Legs Forces-Pipes members, in kN

Panel No.#	**IS:875-1987**	**IS:875-2015, Ka =0.8**	**IS:875-2015, Ka =1.0**	**ANSI/TIA-222G**
1	38.18	37.22	38.91	34.01
2	200.37	195.55	204.48	181.09
3	347.77	339.19	354.92	315.56
4	475.14	460.51	485.08	433.76
5	586.92	565.76	599.50	538.82
6	691.7	663.68	707.10	638.46
7	793.69	758.30	812.05	736.16
8	894.94	851.61	916.30	833.65
9	996.80	944.94	1020.00	931.83
10	1100.00	1040.00	1130.00	1030.00
11	1200.00	1130.00	1230.00	1130.00
12	1310.00	1230.00	1340.00	1230.00

Panel from top to bottom

Table -10: Bracings Forces- Angular members, in kN

Panel No.#	**IS:875-1987**	**IS:875-2015, Ka =0.8**	**IS:875-2015, Ka =1.0**	**ANSI/TIA-222G**
1	13.80	13.44	14.08	12.54
2	30.60	29.87	31.24	28.28
3	29.81	28.36	30.54	27.85
4	24.47	23.25	25.01	23.38
5	23.97	22.51	24.56	23.34
6	24.83	23.14	25.52	24.58
7	26.52	24.52	27.31	26.56
8	29.56	27.08	30.48	28.94
9	32.96	30.00	34.00	32.05
10	36.25	32.85	37.46	35.22
11	39.66	35.81	41.03	38.26
12	43.33	38.99	44.89	40.66

Panel from top to bottom

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Table -11: Utilization Percentage for Main Legs – Pipes members,%
For Loading and Design Standard Combinations (Indian And American Standards)

Panel No.#	IS:875-1987 & IS:800-1984	IS:875-1987 & IS:800-2007, γ=1.0	IS:875-1987 & IS:800-2007, γ=1.1	IS:875-2015, Ka =0.8 & IS:800-1984	IS:875-2015, Ka =0.8 & IS:800-2007, γ=1.0	IS:875-2015, Ka =0.8 & IS:800-2007, γ=1.1	IS:875-2015, Ka =1.0 & IS:800-1984	IS:875-2015, Ka =1.0 & IS:800-2007, γ=1.0	IS:875-2015, Ka =1.0 & IS:800-2007, γ=1.1	ANSI/TIA-222G
1	31%	29%	32%	30%	28%	31%	32%	30%	33%	28%
2	70%	65%	71%	69%	63%	70%	72%	66%	73%	68%
3	71%	68%	74%	69%	66%	73%	73%	69%	76%	71%
4	74%	73%	80%	72%	70%	77%	76%	74%	81%	76%
5	86%	84%	93%	83%	81%	89%	88%	86%	95%	88%
6	85%	84%	92%	81%	80%	89%	87%	86%	94%	88%
7	87%	86%	95%	83%	82%	90%	89%	88%	97%	91%
8	81%	81%	89%	77%	77%	85%	83%	83%	91%	85%
9	86%	86%	94%	82%	81%	89%	88%	88%	97%	91%
10	95%	95%	104%	90%	90%	98%	98%	97%	107%	100%
11	91%	91%	100%	86%	86%	94%	94%	93%	103%	97%
12	100%	100%	109%	94%	93%	103%	102%	102%	112%	106%

Panel from top to bottom

Table -12: Utilization Percentage for Bracing- Angular members,%
For Loading and Design Standard Combinations (Indian And American Standards)

Panel No.#	IS:875-1987 & IS:802-1992	IS:875-1987 & IS:800-2007, γ=1.0	IS:875-1987 & IS:800-2007, γ=1.1	IS:875-2015, Ka =0.8 & IS:802-1992	IS:875-2015, Ka =0.8 & IS:800-2007, γ=1.0	IS:875-2015, Ka =0.8 & IS:800-2007, γ=1.1	IS:875-2015, Ka =1.0 & IS:802-1992	IS:875-2015, Ka =1.0 & IS:800-2007, γ=1.0	IS:875-2015, Ka =1.0 & IS:800-2007, γ=1.1	ANSI/TIA-222G
1	39%	48%	53%	38%	47%	51%	39%	49%	54%	45%
2	64%	82%	90%	63%	80%	88%	66%	84%	92%	75%
3	78%	96%	105%	74%	91%	100%	80%	98%	108%	92%
4	74%	89%	98%	70%	84%	93%	75%	91%	100%	89%
5	84%	100%	110%	79%	94%	103%	86%	102%	112%	104%
6	62%	75%	83%	58%	70%	77%	64%	77%	85%	78%
7	77%	92%	101%	71%	85%	93%	79%	94%	104%	98%
8	80%	94%	104%	73%	86%	95%	82%	97%	107%	99%
9	70%	83%	91%	63%	75%	83%	72%	85%	94%	85%
10	87%	102%	112%	79%	93%	102%	90%	106%	116%	107%
11	68%	80%	88%	62%	72%	79%	71%	83%	91%	82%
12	84%	97%	107%	76%	88%	96%	87%	101%	111%	98%

Panel from top to bottom

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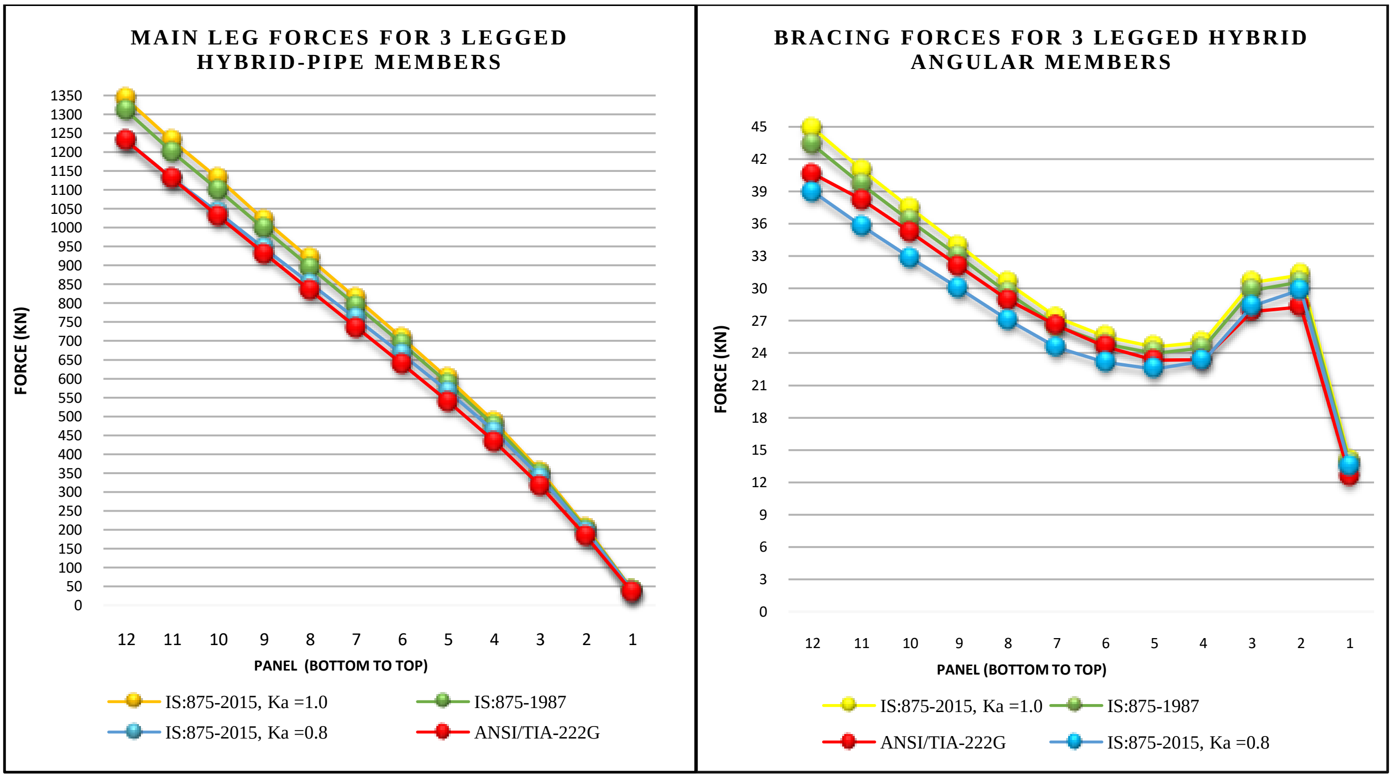
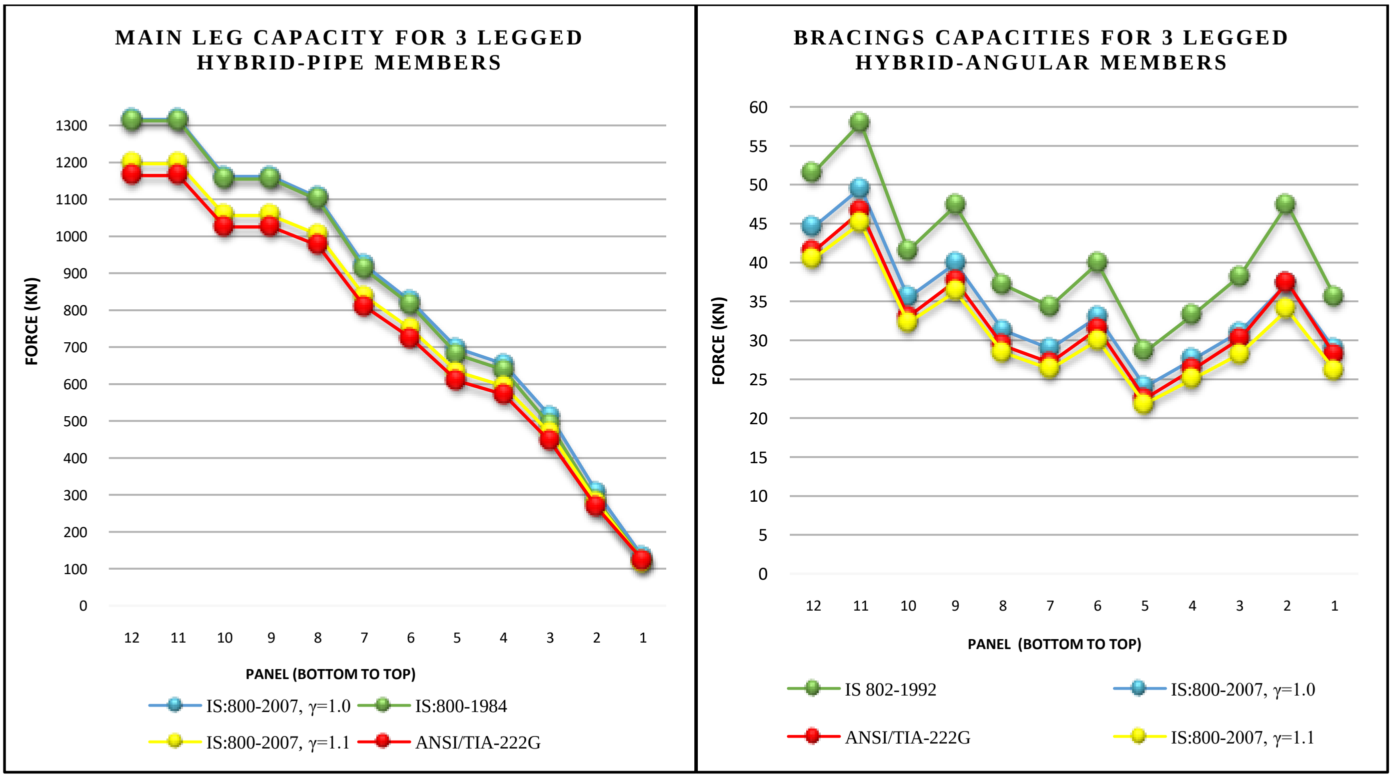
8. GRAPHS



(a) (b)

Fig -3: Variation of Internal Forces of Members



(a) (b)

Fig -4: Variation of Capacities of Members

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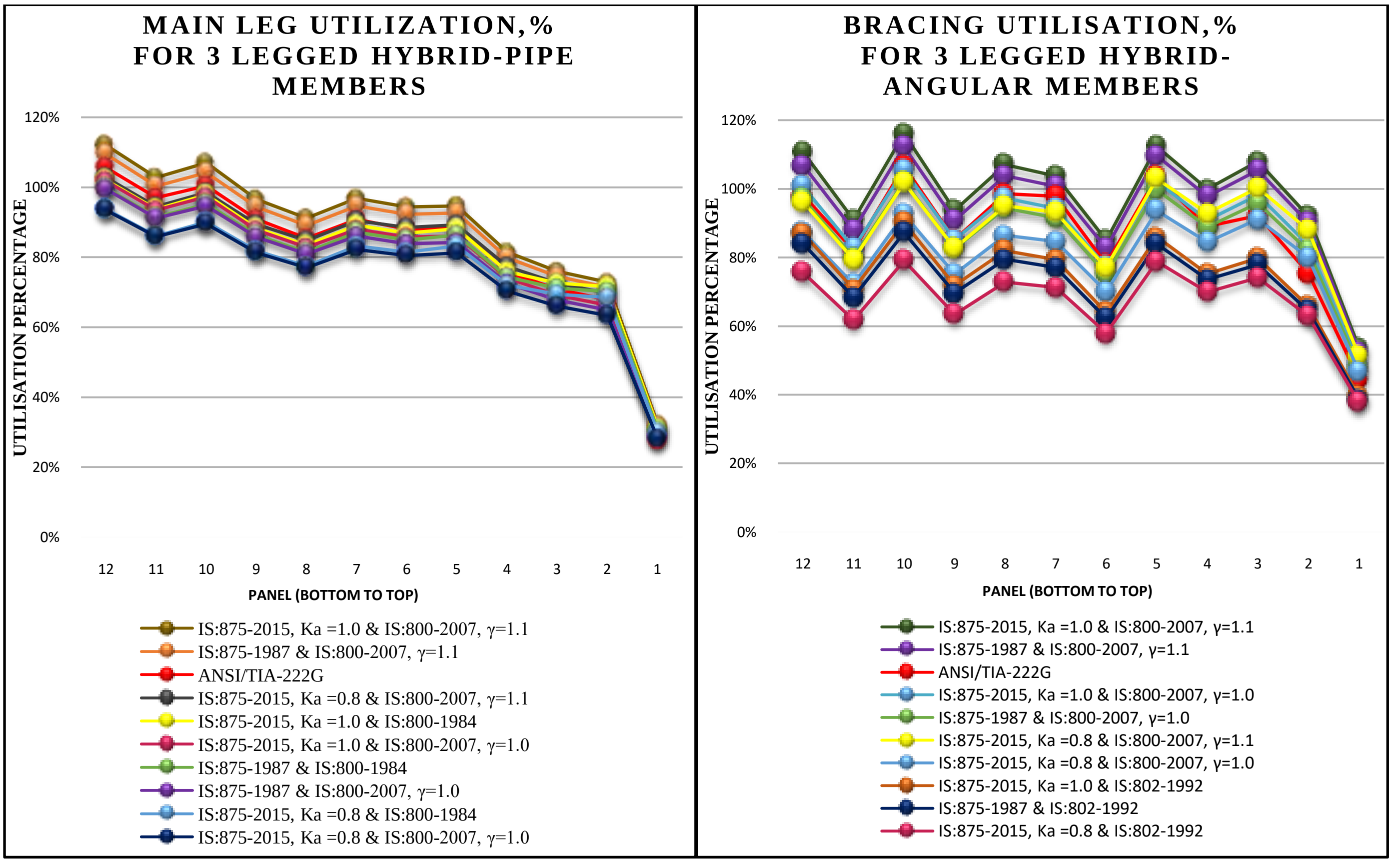
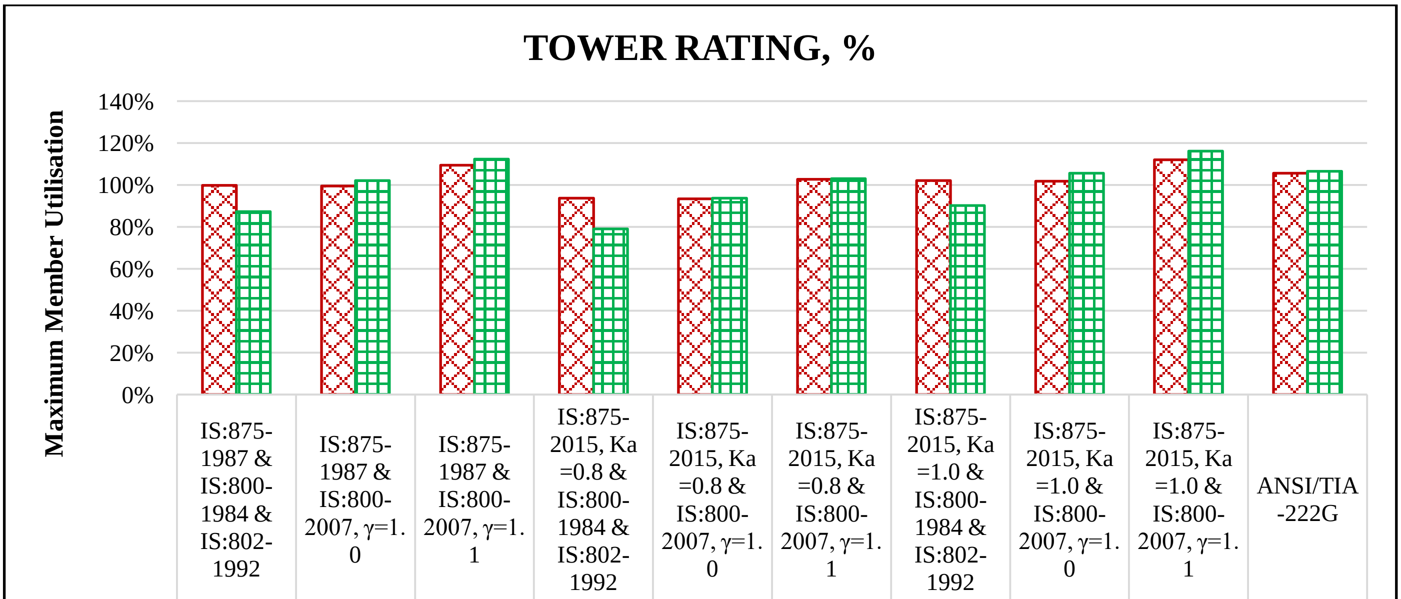


Fig -5: Utilization percentage of Members (Main legs and bracings)



	IS:875-1987 & IS:800-1984 & IS:802-1992	IS:875-1987 & IS:800-2007, γ=1.0	IS:875-1987 & IS:800-2007, γ=1.1	IS:875-2015, Ka =0.8 & IS:800-1984 & IS:802-1992	IS:875-2015, Ka =0.8 & IS:800-2007, γ=1.0	IS:875-2015, Ka =0.8 & IS:800-2007, γ=1.1	IS:875-2015, Ka =1.0 & IS:800-1984 & IS:802-1992	IS:875-2015, Ka =1.0 & IS:800-2007, γ=1.0	IS:875-2015, Ka =1.0 & IS:800-2007, γ=1.1	ANSI/TIA-222G
Main legs	100%	100%	109%	94%	93%	103%	102%	102%	112%	106%
Bracings	87%	102%	112%	79%	94%	103%	90%	106%	116%	107%

Fig -6: Tower Rating

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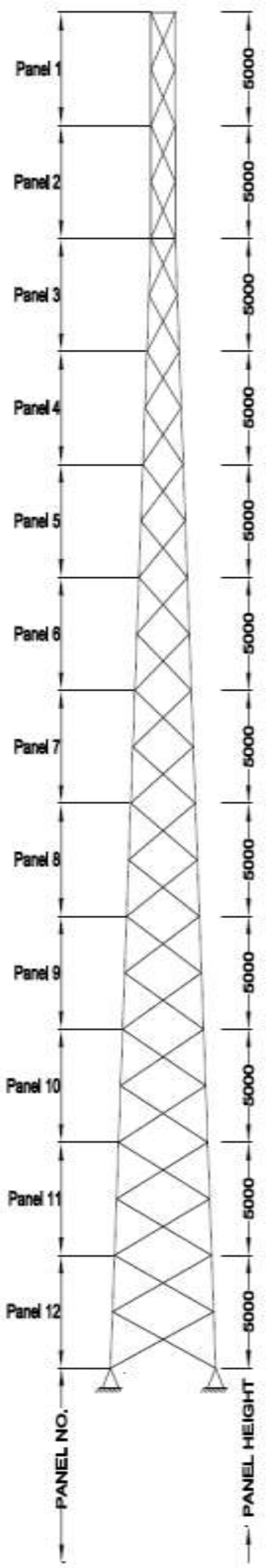

Table-13: Deflection (Sway) of Tower

Deflection (Sway) of tower (maximum of 0^0, 90^0, 180^0 wind directions)								
	IS 875-1987		IS 875-2015 (Ka=0.8)		IS 875-2015 (Ka=1)		222-G	
Level (m)	1.0(DL+WL), mm	Angle in degree	1.0(DL+WL), mm	Angle in degree	1.0(DL+WL), mm	Angle in degree	1.0(DL+WL), mm	Angle in degree
60	331.1	0.654	315.7	0.628	339.1	0.669	288.7	0.565
55	274.1	0.628	260.8	0.603	280.7	0.643	239.4	0.544
50	219.2	0.527	208.2	0.505	224.6	0.539	191.9	0.458
45	173.2	0.449	164.1	0.429	177.5	0.460	152.0	0.391
40	134.0	0.378	126.6	0.360	137.4	0.387	117.8	0.331
35	101.0	0.311	95.2	0.295	103.6	0.318	89.0	0.273
30	73.9	0.255	69.5	0.240	75.8	0.261	65.2	0.224
25	51.6	0.205	48.5	0.193	53.0	0.211	45.7	0.181
20	33.8	0.159	31.6	0.150	34.7	0.162	29.9	0.140
15	19.9	0.117	18.6	0.110	20.5	0.121	17.7	0.104
10	9.7	0.071	9.0	0.066	10.0	0.073	8.6	0.063
5	3.5	0.040	3.3	0.037	3.5	0.041	3.1	0.035
0	0	0.000	0	0.000	0	0.000	0	0.000

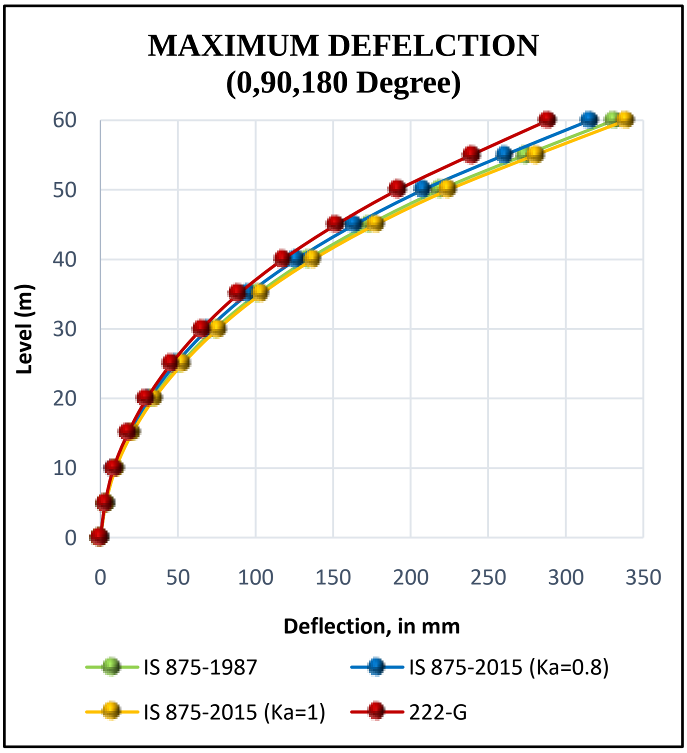

Fig -7: Deflection Graph
(For 0-degree, 90-degree and 180-degree wind directions)

https://doi.org/10.15623/ijret.2018.0709001 Received: 13-06-2018, Accepted: 25-07-2018, Published: 15-08-2018

9. SEISMIC BASE SHEAR

9.1 Calculation of Time Period:

- Height of the structure = 60m
- Face width at the base of the structure (W_o) = 6.5m
- Average face width of the structure (W_a) = 3.58m
- Total height of the structure including appurtenance (W) = 125.8kN
- Weight of the appurtenance within top 5% of the structure Height (W_2) =7.79 kN
- Ks =1500 for W_a in meters
- $W_1 = W\left[\left(\frac{W_a}{W_o}\right)^2 + 0.15\right]$ = 57.03kN
- $f_1 = \frac{K_s * W_a}{h^2}\sqrt{\frac{W_1}{W_1+W_2}}$ = **1.40 Hertz**
- Time Period = $1/f_1$ = 1/1.40 =0.714 Sec

9.2 Calculation of Horizontal Acceleration

Coefficient:

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$$

- Zone factor = 0.36 (Zone-5)
- Importance factor = 1.5 (Important structures)
- Response reduction Factor = 4.0
- For medium stiff soil site and for Time Period 0.55<T<4 ; Sa/g = 1.36/T = 1.36/0.714 = 1.905
- Horizontal Acceleration coefficient A_h =0.1285875

9.3 Calculation of Seismic base Shear

- Total weight of the structure including appurtenance (W) = 125.8 kN
- Seismic base shear V_b = A_h*W = 0.1285875*125.8 = 16.1763 kN

Base seismic shear obtained by equivalent static analysis as per IS 1893 (Part4):2016 is 16.1763 kN whereas the wind load the horizontal reactions as per IS 875 (Part3):1987 for DL+ WL (0/90/180 degree) case is 148.5 kN, IS 875 (Part3): 2015 case is 136.75kN. Therefore, it can be inferred that Earthquake is not a governing load case for design of lattice towers.

10.CONCLUSIONS AND RECOMMENDATIONS

1. While it is believed that all standards are accurate, the information should not be relied upon for a specific application without verification of its accuracy, suitability and applicability.
2. Current tower designs adopted overall tower factor of safety of 1.5 using IS 875 (Part 3)-1987 and IS 802 (Part 1/Sec 2)-1992 for angle members and IS 800-1984 for pipe members. However IS 800:2007 having partial safety factor for loads of 1.5 and partial safety factor for material of 1.1, resulting overall factor of safety of 1.65. This consideration is very conservative compared to already existing practice and compared to international standards on over all loads plus member design.
3. Recent version of IS: 875 (Part 3) - 2015 has introduced
   i) Wind directionality factor K_d of 0.9 for lattice towers, whereas ANSI/TIA/222G has 0.85.
   ii) Area averaging factor Ka to decrease the calculated wind pressure values in larger tributary area of structure. This factor in general compensating the increased wind load due to single Class A structure in terrain roughness coefficient K_2 factor calculation by removing of class B & C structures compared to previous version standard.
4. It is compared that seismic base shear reaction is far lower than shear reaction due to wind load. As it state that lattice telecom and transmission line towers are light weight structures and maximum wind pressure is the Chief criterion for the design, also the concurrence of earthquake and maximum wind pressure is unlikely to takes place. Earthquake prone areas the design of towers/foundations shall be checked for earthquake forces corresponding to nil wind minimum temperature conditions.
5. It can be observed that capacity calculated as per IS 800:2007 buckling class B was found to be on lesser side when compared to IS 802 (Part 1/Section 2):1992. Compression formula used in IS 800-2007 is referred from ENV 1993-1-1. However, ENV 1993-3-1, a specialized standard been used for towers, masts and Chimney's with a special buckling length factors provided for angle and pipes for different member slenderness ratios. Therefore, special attention needed while referring to IS 800 :2007 for tower and masts designs.
6. IS 802 (Part1/Sec 2):1992 angular member capacity was higher compared to ANSI/TIA-222G. The latest revision standard of ANSI/TIA-222H updated with similar provisions of IS 802, resulting same capacity for angular member design.

https://doi.org/10.15623/ijret.2018.0709001 Received: 13-06-2018, Accepted: 25-07-2018, Published: 15-08-2018

7. Referring to "Fig 6 - Tower rating", current design approach of IS 875 (part 3)-1987 and IS 802 (Part 1/Sec 2)& IS 800-1984 is already comparable to ANSI/TIA-222G from member design aswell as in deflection check as in Fig 7. Any further increase in loads due to IS 875 (Part 3)-2015 and material partial safety factor in IS 800:2007 resulting conservative and uneconomical tower weights.
8. Nearly 4,60,000 towers are already installed and these existing structures need not to be re-analyzed unless there are changed conditions like change in appurtenances (Antennas, cables, ladders) or change in serviceability requirements. Any conservativeness while reanalyzing with new standards may leading to huge costs for retrofitting and strengthening of towers.
9. Overall it can be seen that the optimized loading and design procedure from the current available Indian standards comparable to specific standards like ANSI/TIA-222G for masts and towers alone :

   A) Calculate loads using IS 875 (Part 3)-2015 with consideration of wind directionality factor and area averaging factor based on the total tributary area of tower.

   B) Following design standard of IS 802 (Part 1/Sec 2) – 1992* with a factor of safety of 1.5

   *Note :New IS 802 (Part 1/sec 2) – 2016 has been released with same formulas for angular members design and new annexure added for circular hollow sections design.

11. REFERENCES

[1]. IS 875 (part 3):1987 – Design Loads (other than earthquake) for buildings and structures- Code of Practice part3 Wind Loads
[2]. IS 875(part 3):2015 - Design Loads (other than earthquake) for buildings and structures- Code of Practice part3 Wind Loads
[3]. ANSI/TIA-222G – Structural Standard for Antenna Supporting Structures and Antennas
[4]. IS 800:2007- General Construction in Steel – code of practice.
[5]. IS 802 (Part1/Section2) :1992 – Use of Structural Steel in Overhead Transmission Line Towers – Code of Practice
[6]. IS 808-1989 Dimensions for Hot Rolled Steel Beam, Column, Channel and Angle Sections
[7]. IS 1893 (Part4): 2005 Criteria for Earthquake Resistant Design of Structures (Industrial Structures including Stack like Structures)
[8]. Comparative study of four legged Self-Supported Angular Telecommunication Tower on Ground and Mounted on Roof Top- C Preeti, Sankara Ganesh Dhoopam
[9] Telecom communication structures – Sankara Ganesh Dhoopam
[10] ENV 1993-3-1 : 2006 Eurocode 3 – Design of Steel structures – Towers, masts and chimneys

ACKNOWLEDGEMENT

The authors thank management of Ramboll India Private Limited – Telecom department, Hyderabad, for giving an opportunity to work on a case study of Telecom project which helped us to understand the areas of improvement technically and professionally. Any opinions, findings and conclusions or recommendations expressed in this paper are those of the Authors. The authors also acknowledged the thanks to the supporters, researchers and writers whose literatures was used here for supporting this research paper.

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