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# Stability Scrutinization of Roof Top Telecommunication Tower as Per Is 875 (Part-3): 2015

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***Abstract*: There are many towers existing in Tamil Nadu and also in other states of India. Telecommunication towers and transmission towers are the major classification of tower which are importantly in use and existence. These towers play a significant role in performing its own task such as transmitting microwave signals, power supply lines hence failure of such structure in a disaster is a major concern. All these towers were designed using IS875 (PART-3): 1987 (Second revision). But in the year 2015, IS875 (PART-3) has been revised and published as IS875 (PART-3): 2015(Third revision). This Code of practice was adopted by the Bureau of Indian Standards after the draft finalized by the Structural Safety Sectional Committee has been approved by the Civil Engineering Division Council. The major difference between the 2^nd^ and 3^rd^ revision is that there was four more new factors were added. According to the new revision those existing towers stability have to be examined. By computing and comparing loads based on second revision and third revision, in this study we focus to check the stability of an existing Roof Top Hybrid Telecommunication Tower of height 21m as per IS875 (PART-3) : 2015 which was located in Chennai. Thus the tower structure is modelled using Auto-CAD and analysed using STAAD Pro to check the stability condition.***
***Keywords: Telecommunication tower, Stability, computing and comparing, analysed.***

## I. INTRODUCTION

### *A. Evolution of IS Codes Employed in Design of Tower Structures*

Indian standard was first published in 1957 for the guidance of civil engineers, designers and architects associated with the planning and design of buildings. It included the provisions for the basic design loads (dead loads, live loads, wind loads and seismic loads) to be assumed in the design of the buildings. In its first revision in 1964, the wind pressure provisions were modified on the basis of studies of wind phenomenon and its effect on structures, undertaken by the special committee in consultation with the Indian Meteorological Department. With the increased adoption of this Code, a number of comments were received on provisions on live load values adopted for different occupancies. Simultaneously, live load surveys have been carried out in America and Canada to arrive at realistic live loads based on actual determination of loading (movable and immovable) in different occupancies. Keeping this in view and other developments in the field of wind engineering, the Structural Safety Sectional Committee decided to prepare the second revision of IS : 875 in the following five parts: Part 1 Dead loads, Part 2 Imposed loads, Part 3 Wind loads, Part 4 Snow loads, Part 5 Special loads and load combinations. These are the transitions of IS875 till the year 1987. In 1980 and 1990's the loads acting on tower, both the telecom tower and transmission tower were computed using IS 875 -1987 (part-1&3) and designed using IS 802 and IS 800-1987.This was the common design practice followed in that time period. IS 802 is specially preferred for transmission tower .So, majorly used code is IS800:1987 to design telecom tower. Later on IS 800 has been revised with two major consideration. They are IS 800:2007 LSM and IS 800:2005 WSM. The major reason for revision is the introduction of an important factor called "LRFD –LOAD RESISTANCE FACTOR DESIGN ". Thus design consideration using factor of safety was introduced in towers. E.g.: 1.5D.L + 1.5W.L/E.L. These are the transitions in IS800:2007. Now again there was a shift in IS875 (part-3) in the year 2015.Due to this transition, new cyclonic factors were introduced in wind load calculation .They are $K_a$, $K_d$, $K_c$, $K_4$. According to these new factors the loads acting on towers was increased. At present IS 800:2007 and IS 875 (part-3):2015 are commonly in use & preferred in the design of telecommunication tower. Thus in this study we are going to analyze the existing tower stability using IS875 (part -3):2015.

### *B. The Revision Of IS875 (Part-3)*

*2015 – Reason:* The explanation given by the committee of Indian Standards are,

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1) The aerodynamic roughness heights for individual terrain categories have been explicitly included, and are used to derive turbulence intensity a mean hourly wind speed profiles.
2) The previous classification of structures into B and C class has been deleted and accordingly the modification factor, $k_2$ is renamed as terrain roughness and height factor.
3) The values of $k_2$ factor corresponding to previous class A type structure only, are retained in this standard.
4) An additional modification factor termed as, importance factor has been included for cyclonic regions.
5) Simple empirical expressions have been suggested for height variations of hourly mean wind speed and also turbulence intensity in different terrains.
6) Final and most important reason for revision is that in recent years the India's Climatic conditions has been varying, especially the coastal zones experiencing severe cyclones in the winter season. Those cyclones cause severe damages to high-rise structures like towers, tall buildings, tall industrial plants. So, thus to increase the stability and serviceability of the structures, a new factor has been introduced and it acts as a major factor behind revision of IS Code.

### *C. Telecommunication Towe*

Telecommunication towers are structures that house the antennae, dishes and receivers required for wireless communication and data transfer. It is also defined as tall structures designed to support antennas for communication and broadcasting. The different types of communication towers are based upon their structural action, their cross-section, the type of sections used and on the placement of tower. Based on structural action: Towers are classified into three major groups based on the structural action. They are,

1) Self-Supporting towers,
2) Guyed Tower,
3) Monopole.

## II. MODEL OF TOWER

The tower which is taken for analysis has height of 21m which is located at Chennai. It has been modelled in Auto CAD and analysed in STAAD.Pro V8i. Details of the tower are shown in the Table I and sectional details are shown in Table II. Fig.1 and Fig.2 shows the model made in both Auto CAD and STAAD.Pro V8i.

Table I Details of tower

| Panel number | Height |
|---|---|
| 1 | 0-6 m |
| 2 | 6-12 m |
| 3 | 12-18 m |
| 4 | 18-21 m |

Table II Section Details of tower

| Panel number | Main Leg | Horizontal member | Bracing members |
|---|---|---|---|
| 1 | PIP1397M | ISA75X75X6 | ISA65X65X6 |
| 2 | PIP1143M | ISA65X65X6 | ISA50X50X6 |
| 3 | PIP761M | ISA50X50X6 | ISA45X45X6 |
| 4 | PIP603M | ISA45X45X6 | ISA45X45X6 |

## III. DESIGN LOAD CALCULATION

Computaion of loads at different panel heights was done from panel 1 to 4 and is described below for both second and third revision of code.

### *A. Load Calculation As Per IS 875(Part-3): 1987*

1) *Basic Wind Speed $V_b$* : Basic wind speed map of India as applicable at 10 m height above mean ground level for the six wind zones of the country. Basic wind speed $V_b$ is based on peak gust velocity averaged over a short time interval of about 3 seconds,

corresponds to mean heights above ground level in an open Terrain (category 2) and have been worked out for a 50 years return period [refer IS875 (part 3 )].

2) *Design Wind Speed $V_z$*: Reference wind speed obtained shall be modified to include the following effects to get the design wind speed:

Risk level (**$k_1$**),
Terrain, roughness and height of structure (**$k_2$**), and
Local topography factor (**$k_3$**).
It may be mathematically expressed as follows:

$$\mathbf{V_z = V_b * k_1 * k_2 * k_3}$$

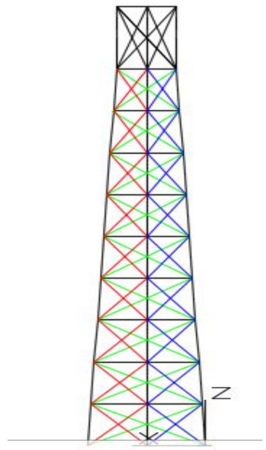
Fig.1 Autocad model

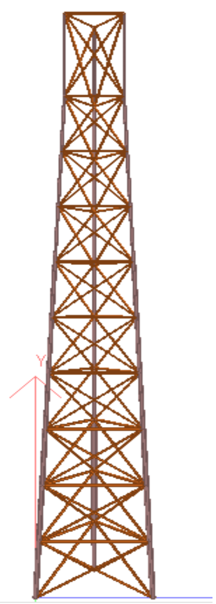
Fig.2 STAAD.Pro V8i model

3) *Design Wind Pressure $p_z$* : The design wind pressure on towers, conductors and insulators shall be obtained by the following relationship:

$$p_z = 0.6 * (V_z)^2$$

Where, $p_z$ = design wind pressure in N/m², and $V_z$ = design wind speed in m/s.

4) *Wind Load on Tower*: In order to determine the wind load on tower, the tower is divided into different panels having a height h. These panels should normally be taken between the intersections of the legs and bracings. For a lattice tower of square cross-section, the resultant wind load, **F** in (N), for wind normal to the longitudinal face of tower on a panel height, h applied at the centre of gravity of this panel is

$$F = p_z*C_z*A_e$$

Where, $p_z$ = design wind pressure, in N/m², $C_d$ = drag coefficient for panel under consideration against which the wind is blowing. Values of $C_d$ for different solidity ratios, Solidity ratio is equal to the effective area (projected area of all the individual elements) of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction, $A_e$ = total net surface area of the legs, bracings, cross arms and secondary members of the panel projected normal to the face in m². (The projections of the bracing elements of the adjacent faces and of the plan-and-hip bracing bars may be neglected while determining the projected surface of a face).

5) *Wind Load Calculation*: Table III shows the values obtained by calculation using the above formulas given in the code book.

Table III Values of wind load at different panels

| Panel | $V_b$ | $k_1$ | $k_2$ | $k_3$ | $V_z$ | $p_d$ | $C_f$ | F |
|---|---|---|---|---|---|---|---|---|
| Panel 1 | 50m/s | 1.08 | 0.940 | 1 | 50.78m/S | 1542.290N/m$^2$ | 1.730 | 12.83kN |
| Panel 2 | 50m/s | 1.08 | 0.98 | 1 | 52.92m/S | 1680.3 N/m$^2$ | 1.714 | 10.50kN |
| Panel 3 | 50m/s | 1.08 | 1.015 | 1 | 54.81m/S | 1802.48 N/m$^2$ | 2.06 | 9.23kN |
| Panel 4 | 50m/s | 1.08 | 1.03 | 1 | 55.62m/S | 1856.15 N/m$^2$ | 2.08 | 4.03kN |

### B. *Load Calculation As Per IS 875(Part-3): 2015*

1) *Design Wind Speed $V_z$*: The basic wind speed ($V_b$) for any site shall be obtained from code book and shall be modified to include the following effects to get design wind speed, $V_z$ at any height, for the chosen structure:

Risk level ($k_1$),
Terrain, roughness and height of structure ($\mathbf{k_2}$),
Local topography factor ($\mathbf{k_3}$), and
Importance factor for the cyclonic region ($\mathbf{k_4}$).
It may be mathematically expressed as follows:

$$\mathbf{V_z = V_b * k_1 * k_2 * k_3 * k_4}$$

2) *Design Wind Pressure*: The wind pressure at any height above mean ground level shall be obtained by the following relationship between wind pressure and wind speed

$$\mathbf{p_z = 0.6 * (V_z)^2}$$

Where, $p_z$ = design wind pressure in N/m², and $V_z$ = design wind speed in m/s.
The design wind pressure $p_d$ can be obtained as,

$$\mathbf{p_d = k_d * k_a * k_c * p_z}$$

Where, $k_d$ =wind directionality factor, $k_a$ = area averaging factor, and $k_c$ =combination factor.

3) *Wind Load on Tower:* The value of force can be calculated when multiplying the force coefficient ($C_f$), effective frontal area $A_e$ and design wind pressure of the structure.

$$\mathbf{F = p_d * C_z * A_e}$$

Where F is the force acting on the tower in N and $C_f$ is the force coefficient for the structure.

4) *Wind Load Calculation:* Table IV (a) & (b) shows the values obtained by calculation using the above formulas given in the code book.

Table IV (a) Value of wind load at different panels as per IS 875(Part-3): 2015

| Panel | $V_b$ | $k_1$ | $k_2$ | $k_3$ | $k_4$ | $V_z$ | $p_z$ |
|---|---|---|---|---|---|---|---|
| Panel 1 | 50 m/s | 1.08 | 0.97 | 1 | 1.3 | 68.1m/s | 2782.56 N/m$^2$ |
| Panel 2 | 50 m/s | 1.08 | 1.015 | 1 | 1.3 | 71.25m/s | 3046 N/m$^2$ |
| Panel 3 | 50 m/s | 1.08 | 1.045 | 1 | 1.3 | 73.36m/s | 3229 N/m$^2$ |
| Panel 4 | 50 m/s | 1.08 | 1.06 | 1 | 1.3 | 74.4m/s | 3321.22 N/m$^2$ |

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Table IV (b) Value of wind load at different panels as per IS 875(Part-3): 2015

| Panel | $P_d$ | $k_d$ | $k_a$ | $k_c$ | $C_f$ | $A_e$ | F |
|---|---|---|---|---|---|---|---|
| Panel 1 | 2504.3 N/m$^2$ | 1 | 1 | 0.9 | 1.73 | 4.8 m$^2$ | 20.79kN |
| Panel 2 | 2741.4 N/m$^2$ | 1 | 1 | 0.9 | 1.714 | 3.648 m$^2$ | 17.10kN |
| Panel 3 | 2906.1 N/m$^2$ | 1 | 1 | 0.9 | 2.06 | 2.48 m$^2$ | 14.94kN |
| Panel 4 | 2989.09 N/m$^2$ | 1 | 1 | 0.9 | 2.08 | 1.045 m$^2$ | 6.5kN |

## IV. RESULTS OF ANALYSIS

The Analytical study of Hybrid Rooftop Telecommunication tower as per IS875:1987 and IS875:2015 was illustrated in this paper.

*A. Analytical Behaviour of tower as per IS 875(Part-3): 1987:*

1) Deflection due to load calculated is shown in Fig.3 (a)
2) Bending moment of tower along Z&Y-axis is shown in Fig.4 (a)
3) Torsional behaviour of Tower is shown in Fig.5 (a)
4) Shear Force on Tower along Y & Z-axis is shown in Fig.6 (a)
5) Stress Patterns of Tower is shown in Fig.7 (a)

*B. Analytical Behaviour of Tower as Per IS 875(Part-3): 2015:*

1) Deflection due to load calculated is shown in Fig.3 (b)
2) Bending moment of tower along Z&Y-axis is shown in Fig.4 (b)
3) Torsional behaviour of Tower is shown in Fig.5 (b)
4) Shear Force on Tower along Y & Z-axis is shown in Fig.6 (b)
5) Stress Patterns of Tower is shown in Fig.7 (b)

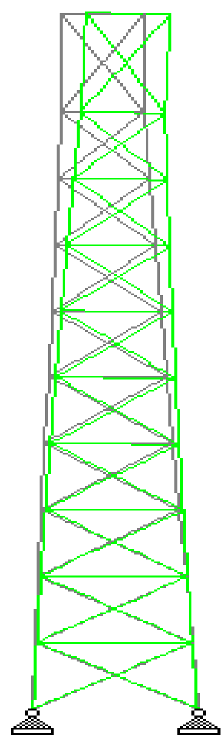

Fig.3 (a) Deflection due to load calculated from IS 875(Part-3): 1987

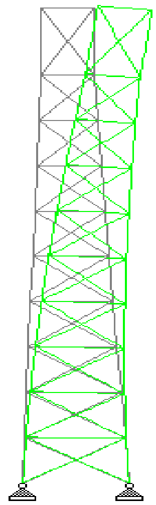

Fig.3 (b) Deflection due to load calculated from IS 875(Part-3): 2015

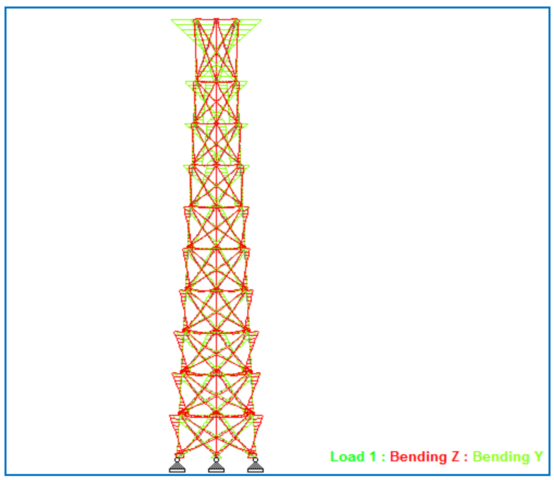


Fig.4 (a) Bending moment of tower along Z&Y-axis

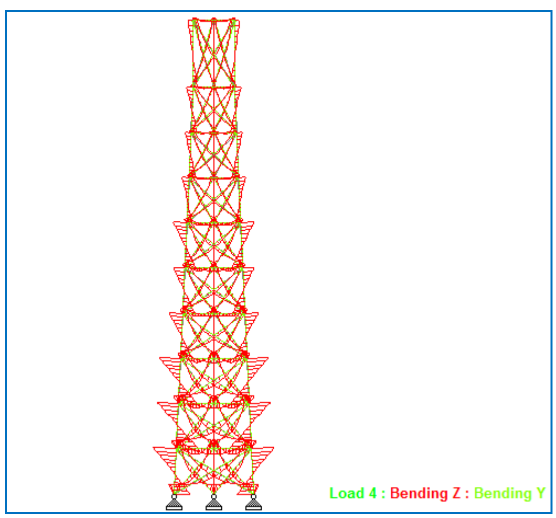


Fig.4 (b) Bending moment of tower along Z&Y-axis

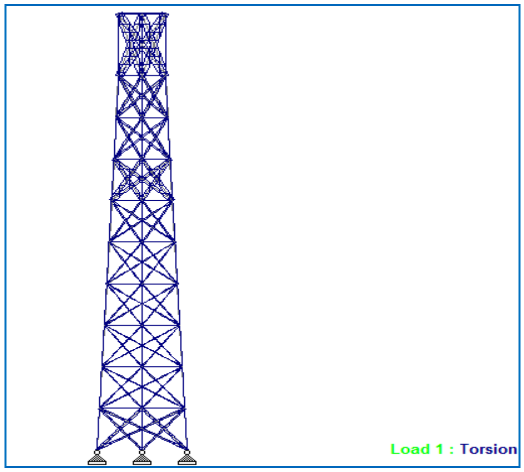


Fig.5 (a) Torsional behaviour of Tower

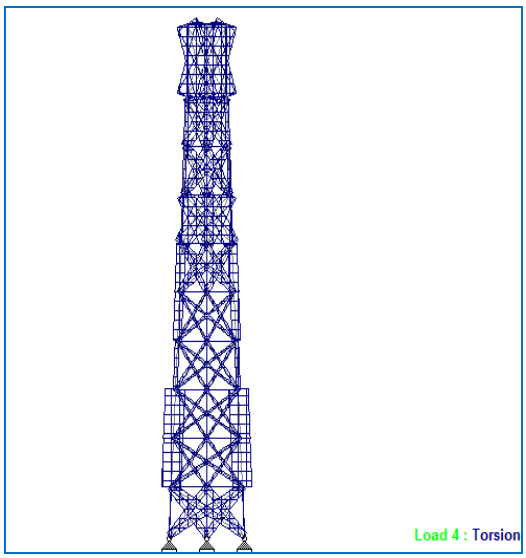


Fig.5 (b) Torsional behaviour of Tower

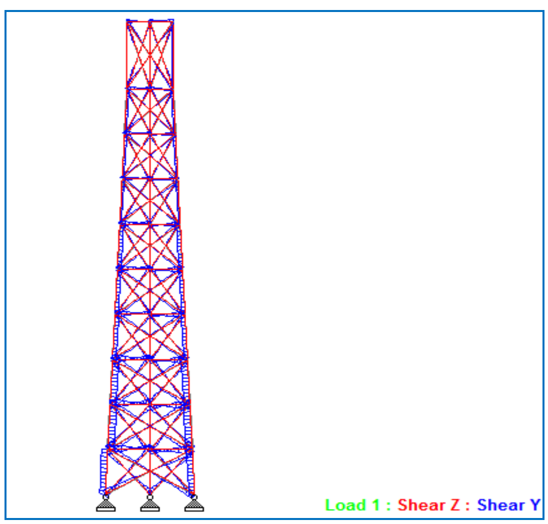


Fig.6 (a) Shear Force on Tower along Y & Z-axis

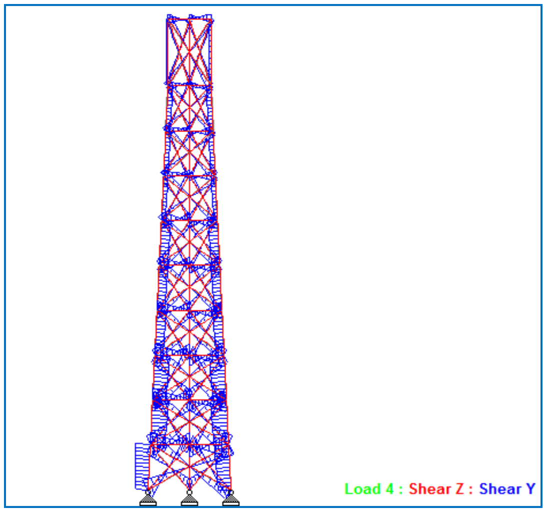


Fig.6 (b) Shear Force on Tower along Y & Z-axis

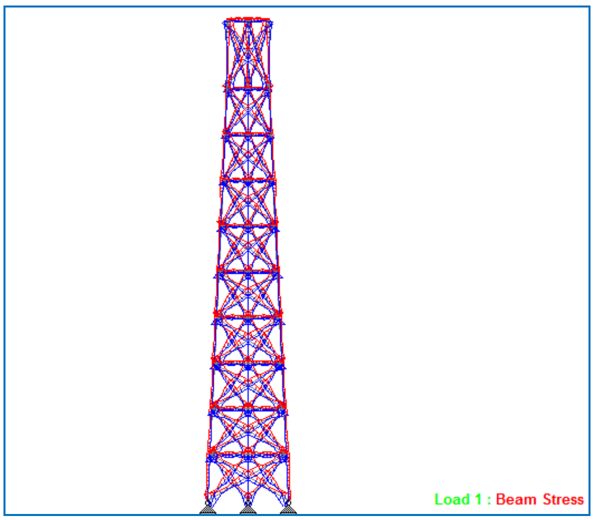


Fig.7 (a) Stress Patterns of Tower

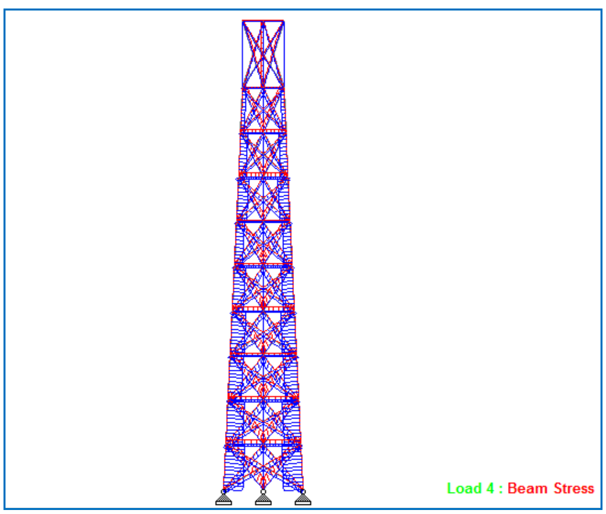


Fig.7 (b) Stress Patterns of Tower

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Fig.8 shows the comparison of wind pressure between the second revision and the third revision at different height. Similarly Fig.9 shows the comparison of wind load on tower at different panels.

Fig.10 shows the comparison of increase in percentage at each panel from second revision to third revision.

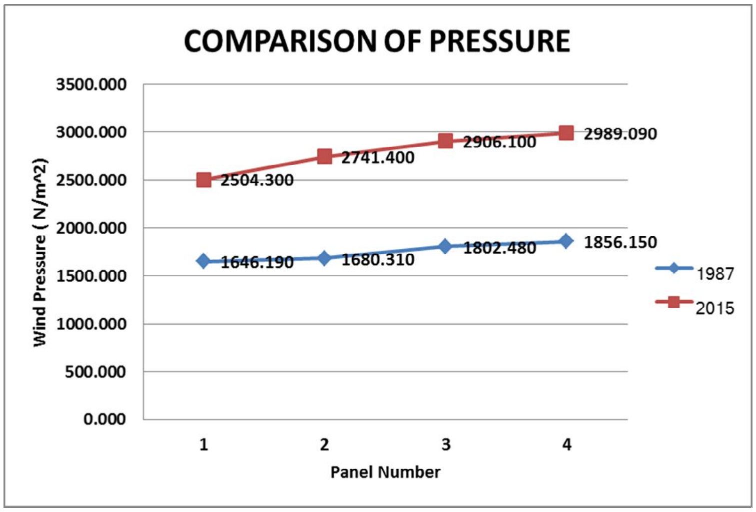


Fig.8 Comparison of wind pressure in $N/m^2$

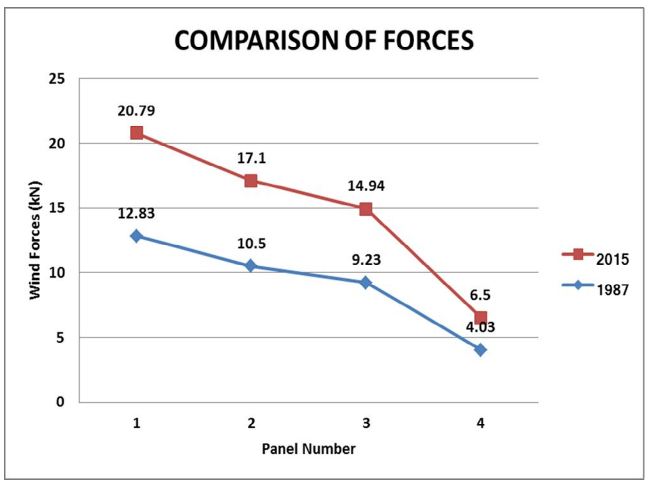


Fig.9 Comparison of Force

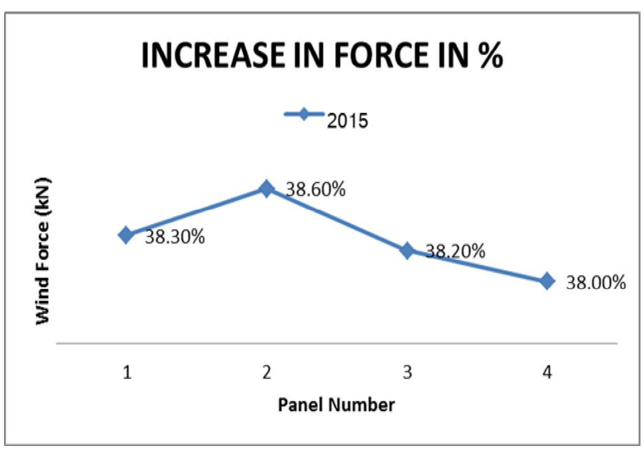


Fig.10 Increase in Force

## V. CONCLUSION

From the analysis, the tower begins to fail in accordance with IS875:2015 part-3. The recommended explication to strengthen the tower structure are:

*A.* When tower is subjected to failure, it's also due to overload of antennas and it is not recommended or not possible or a good idea to remove the antennas from a tower, since the ultimate purpose of tower installation would be obsolete. In order to balance loads from antennae, the antennae can be placed as scattered & spread load. According to this, the antennae's can be spread along all the phases and in all top most panels of tower & placed. It should not be placed only over a single member of tower like pointing whole antennae's load towards it in a clumsy manner. By distributing antennae load to all members the additional forces from antennas can be resisted without disturbing the structure.

*B.* In towers, in order to subdue failures, a leg member is added or connected adjacent to old leg in such way that the additional force from the original member exceeds its capacity to transfer load completely the new member will share the excess load & transmit it to the foundation. This measure will prevent failure and enhance the capacity of the tower but needs periodic maintenance check for deformation or plastic hinges formed in the original leg member

*C.* Commonly prevailing method of strengthening bracing member is to connect a reinforcing member to the already working or existing bracing member at intervals by clamps or bolts

*D.* The replacement of all the failed members with new member with higher strength & design property is also a simplest solution. But it involves more time & cost if the quantity of members to be replaced is more.

*E.* In this study, most of the horizontal & bracing members begins to fail. So we can also change the bracing type such a

*F.* D, XB bracing to increase the stability. By changing the bracing members with bracing type of increased steel area the structure can be made stable to resist & transmit excess loads.Other major reason behind tower failure is uneven load distribution of cables running all over the tower. So, a separate cable ladder should be provide to carry only the cable load & another ladder to human movement.

## VI. ACKNOWLEDGMENT

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