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RESEARCH ARTICLE

SEISMIC RESPONSE OF TRANSMISSION TOWER USING SAP2000

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Abstract

The basic needs of every human being for survival are considered as food, cloth and shelter. This phenomenon existed for several hundreds of years, till recently. But now the trend has changed as electric power also become one of the basic needs. The electric power plays a very vital role not only in the survival of human being but also for the development of nation. National economy and growth largely depended on the availability of electric power supply. Many of these failures are due to earthquakes. These towers are of a variety of designs and were constructed to different specifications. In India, development of electric power over the years has been phenomenal. The installed capacity has risen from a 2301 MW in 1950-51 to 167,278.36 MW as on 31st October 2010.The analysis and results are conducted using FEM package SAP 2000 Vs 14. The results are predicted for time period, lateral displacement and critical member.

INTRODUCTION

Human being having all comfort abilities in his finger tips. This is because of Machines. These machines are dead when they are without power. This power is contributed by electricity. According to Indian Grid Corporation Census 2010-2011,the demand of electricity of the only Karnataka state is about 7634 MW and it is showing that the growth of population and increasing the standard of living. Power cannot be produced in our houses, it requires a source of fossil fuels, water, wind and the temperature or luminous to produce electricity. Produced electricity should be distributed. This needs a systematic network of distribution, in this distribution major role is played by one of a structure known as “Transmission tower”. A Nation has got good road means it is better in saving fuel and at the same time if it has got good electricity network means better workability. The bulk electricity Transmission lines are assets of a Nation and where it is subjected to different environmental conditions such as wind, temperature, landslide and earthquake.

Due to the ignorance of the seismic behavior of tower, many wrong practices remain continued. There are numerous examples enlisted in the damage reports of past earthquakes. Collapse of transmission line towers is reported from many corners of the nation interrupting the power and affecting the national growth as well as common man life. The restoration of collapsed transmission line tower will incur huge expenditure in terms of time, man power, materials and human life. Collapses of transmission towers were often observed in previous earthquakes. Five migrant labourers from West Bengal were crushed to death after a power transmission tower fell on them in Haryana. Eleven people were also injured in the accident. Around 125 labourers were at work on tower when it was collapsed. These collapses were partially caused by the pulling forces from the transmission lines generated from out-of-phase responses of the adjacent towers owing to spatially varying earthquake ground motions. Therefore there is a need to understand the seismic response of such tower and to retrofit the existing towers so that they can withstand further probable earthquake generated forces.

Transmission tower history: Structures which support electrical transmission lines are of two kinds namely, poles and towers. Towers are used when lines are to be supported at longer distances. Long spans of conductors require tall tower to maintain proper clearance from ground at the location of greatest cable sag. Tall structures with relatively small cross-section, with a large ratio between height and maximum base width are known as towers.The first transmission of electrical impulses over an extended distance was demonstrated on July 14, 1729 by the physicist Stephen Grey. Electric power transmission was accomplished in 1882 with the first high-voltage transmission between Munich and Miesbach.

In 1950, the company Kamani Engineering Corporation (KEC) founded by Ramjibhai Kamani received an order from the Indian Government to supply transmission towers for the prestigious Bhakra Nangal Dam project and a steel tower fabrication plant was established in Bombay. The second unit was started in Jaipur (Rajasthan) and by 1967, KEC was supplying three-fifths of India's demand for transmission towers.

SEISMIC ANALYSIS

The structure resting on the ground starts vibrating when an earthquake occurs. Because it induces inertia forces on the structure. So in order to find out these forces and behavior of the structure during the activity, several researches have been conducted all over the world. This research involves the various analysis techniques to determine the lateral forces ranging from purely linear to nonlinear inelastic analysis. In India standardized method of analysis is followed by using a code-IS 1893 (Part 1):2002-"Criteria for Earthquake resistant design of structures".

Methods of seismic evaluation

Once the structural model has been selected, it is possible to perform analysis to determine the seismically induced forces in the structure. There are different methods of analysis provides different degrees of accuracy. Currently seismic evaluation of structures can be divided into two categories.

i. Qualitative method
ii. Analytical method

The qualitative method are based on the available background information of the structures, which involves the visual inspection report, some non-destructive test results etc. Whereas analytical methods involve the estimation of forces and behavior of structures during the earthquakes depending on the available data. The methods in these categories are as shown in the below fig 1.

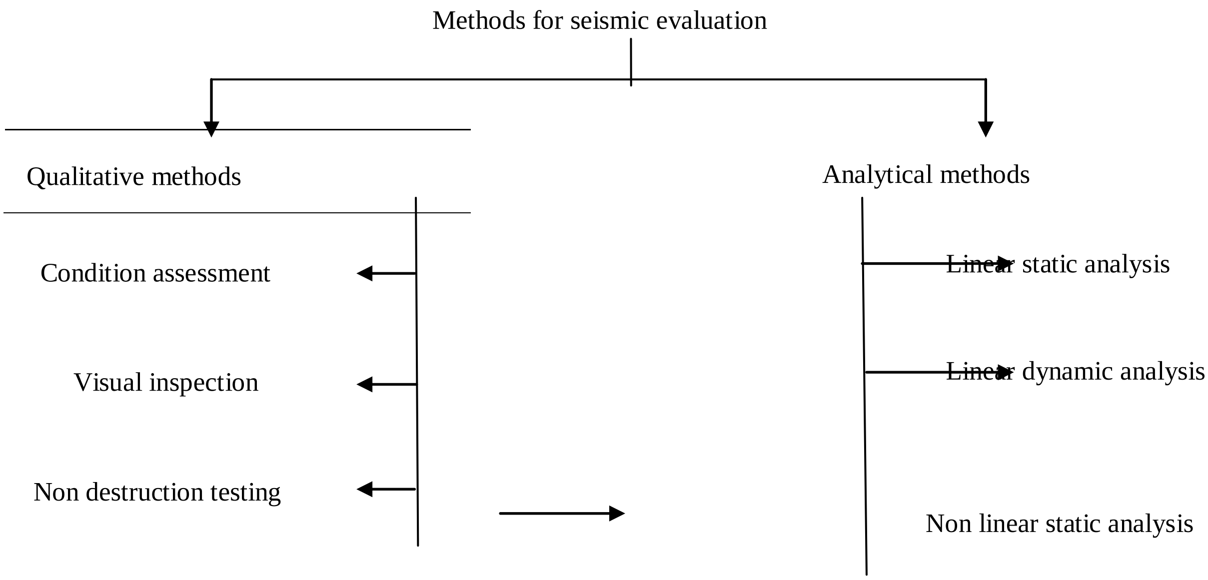


Fig:1 Different methods of seismic evaluation

Qualitative methods

Here the Rapid Screening Procedure (RSP) is aimed for identifying potentially hazardous towers in the study area, without going into detailed analysis. RSP utilizes a methodology based on visual inspection of a tower and noting the structural configuration. The methodology begins with identifying the primary structural lateral load resisting system and materials of the tower. The method generates a Structural Score 'S', which consists of a series of 'scores' and modifiers based on tower attributes that can be seen during tower survey. The structural score 'S' is related to probability of the tower sustaining life threatening damage should a severe earthquake in the region. A low 'S' score indicates that the tower is probably safe for defined earthquake loads. Thus, the expression for structural score is:

(Structural score) = (Basic Structural Hazard) + (Performance Modification Factor)
S BSH PMFs

Analytical methods

Analytical methods are broadly classified as linear static, linear dynamic, nonlinear static and nonlinear dynamic methods. In these the first two methods are suitable when the structural load are small and no point the load will reach to collapse load are differs in obtaining the level of forces and their distribution along the height of the structure. Whereas the nonlinear static and dynamic analysis are the improved methods over linear approach. During earthquake load the structural loading will reach to collapse load and the material stresses will be above yield stress. So in that case material nonlinearity and geometrical nonlinearity should be incorporated into the analysis to get better results. These methods also provide information on the strength, deformation and ductility of the structures as well as distribution of demands.

Linear static analysis or equivalent static method

Here the total design lateral force or design base shear 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 (T_a) of the structure

$T_a = 0.085h^{0.75}$ Moment resisting steel tower.

Where,

h- Height of the tower in m.

ii. Determination of base shear (V_B) of the tower.

$$V_B = A_h \times W$$

Where

A_h =Design horizontal seismic coefficient for a structure

W = seismic weight of tower

A_h shall be determined by the following expression:

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

Provided that for any structure with $T \leq 0.1$s, the value of A_h will not be taken less than Z/2 whatever be the value of (I/R).

Where

Z = Zone factor given in table 2, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the maximum considered earthquake zone factor for Design Basic earthquake.

I = Importance factor, depending upon the functional use of the structures, characterized by hazardous consequences of its failure, Post earthquake functional needs, Historical value, or economics importance (Table -6 of IS 1893 (Part 1): 2002).

R =Response reduction factor, Depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However the ratio of (I/R) shall not be greater than 1.0 .The values of R for buildings are given in Table 7 of IS 1893 (Part 1): 2002.

Sa/g = Average response acceleration coefficient for rock or soil sites as given in fig 2 and table 2 .of IS 1893 (Part 1): 2002 (See fig 6.5).

The included reduction factor "R" in base shear formula is an attempt to consider the structures inelastic characteristics in linear analysis method, since it is undesirable as well as uneconomical that a structure will be designed on the basis that it will remain in elastic range for all major earthquakes. The response reduction factor R is also called Response modification factor or behavior factor.

Seismic weight of tower (w) is the sum of the seismic weight of angle sections, the seismic weight at any level would be equal to dead weight of the tower system plus weight of conductor and weight of insulator plus appropriate amount of imposed load as specifies.

iii. Lateral distribution of design base shear

The design base shear V_B thus obtained is then distributed along the height of the tower using a parabolic distribution expression

$$Q_i = V_B \frac{W_i \cdot h_i^2}{\sum_{i=1}^{n} W_i \cdot h_i^2}$$

Where

Q_i = Design lateral force at i th level

W = Seismic weight of i,th level

h_i = Height of tower measured from base

n = Number of levels at which masses are located.

Linear dynamic analysis by response spectrum method

The response spectrum represents an interaction between ground acceleration and the structural system, by an envelope of several different ground motion records. For the purpose of the seismic analysis the design spectrum given in Fig.2 of IS 1893 (Part 1): 2002 is used. This spectrum is based on strong motion records of eight Indian earthquakes.

Following procedure is generally used for the spectrum analysis which involves undamped free vibration of the entire tower using established methods of mechanics.

i. Select the design spectrum.

ii. Determine the Eigen values (ω^2), Eigen vectors (ϕ) and periods of vibration (T) using the basic equation of motion.

$$MX+CX+KX=0$$

iii. Read the level of response from the spectrum for the period of each modes considered.

iv. Determination of modal participation factor Pk for mode k is given below.

$$Pk = \frac{\sum_{i=1}^{n} W_i \phi ik}{\sum_{i=1}^{n} W_i (\phi ik)^2}$$

v. Determination of modal mass Mk of mode k is given as follows.

$$Mk = \frac{\left[\sum_{i=1}^{n} W_i \phi ik\right]^2}{g\left[\sum_{i=1}^{n} W_i (\phi ik)^2\right]}$$

vi.

vii. Select number of modes k such that the sum total of modal masses of all modes considered is at least 90% of the total seismic mass.

viii. Design lateral force Qik for floor I and mode k is given as follows.

$$Q_{ik} = A_k \phi_{ik} P_k W_i$$

Where Ak is the design horizontal acceleration spectrum and W is the weight of the building.

ix. Peak value of design lateral force for floor I and storey shear forces in each mode can be obtained by one of the Modal combination rules

SRSS (Square Root of Sum of Squares) rule,

$$Qi = \sqrt{\sum_{k=1}^{r} Q_{ik}}$$

CQC (Complete Quadratic Combination) rule

$$Qi = \sum_{k=1}^{r} Q_{ik}$$

By this way convert the combined maximum response into shears and moments for use in design of the structure. According to the code, dynamic analysis may be performed using either response spectrum method or the time history method. In either method, the design base shear (V_B) is compared with a base shear $\bar{V}_B$ calculated using the fundamental period T_a. It suggests that when V_B is less than, $\bar{V}_B$ all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) must be suitably scaled by multiplying with $\frac{\bar{V}_B}{V_B}$.

The code IS: 1893-2002(Part 1) suggests that the number of modes to be used in the analysis be such that the total of modal masses of all modes considered is at least 90% of the total seismic mass. The modes are considered as closely spaced if the natural frequencies differ from each other by 10% or less of the lower frequency. The peak response quantities are combined using Complete Quadratic Combination (CQC) method. Alternatively, it accepts Square Root of Sum of Squares (SRSS) method be used for modes which are not closely spaced. If there were few closely spaced modes, then it suggests the use of Sum of Absolute Values (ABSSUM) method and rest of the modes could be combined using CQC method.

In equivalent static force procedure, the magnitude of lateral forces is based on only the fundamental period of vibration of the tower calculated using empirical formula given in the seismic code. Tower has multiple degrees of freedom, therefore many possible patterns of deformations are possible. The actual distribution of base shear over the height of building is obtained as the superposition of all the modes of vibration of the multi degree of freedom system. Response Spectrum Method takes into account the effects of various modes of vibration of the tower to calculate the peak response. In this way the, response spectrum method is closer to the dynamic behavior of the system. The difficulty however is the selection of the design spectrum, which is constructed for chosen set of strong motion earthquake records. This may be either conservative or under safe for the design life of a particular tower depending on the seismicity of the site where the tower is located. For the case of critical structures time history analysis is carried out by applying the actual recorded earthquake accelerations with help of computer program. But the selection of earthquake record for the site requires expert knowledge.

Non linear static analysis

Pushover method is one of the methods available to understand the behavior of structures subjected to earthquake forces. As the name implies, it is the process of pushing horizontally with a prescribed loading pattern incrementally until the structure reaches a limit state [ATC-401996]. The static approximation consists of applying a vertical distribution of lateral loads to a model which captures the material nonlinearity of an existing or previously designed structure, and monotonically increasing those loads until the peak response of the structure is obtained on a base shear Vs level displacement plot.

The selection of Lateral load pattern for a performance evaluation is likely to be more critical than the accurate determination of the target displacement. It plays an important role due to the fact that it is supposed to deform the structure in the similar manner experienced in earthquake occurrence. Conventionally as shown in a fig 2 an inverted triangular or uniform shape is used consistent with the codified static lateral force distribution but use of adaptive load shape is on the increase. The importance of the loading shape increases when the response is not dominated by the single mode.

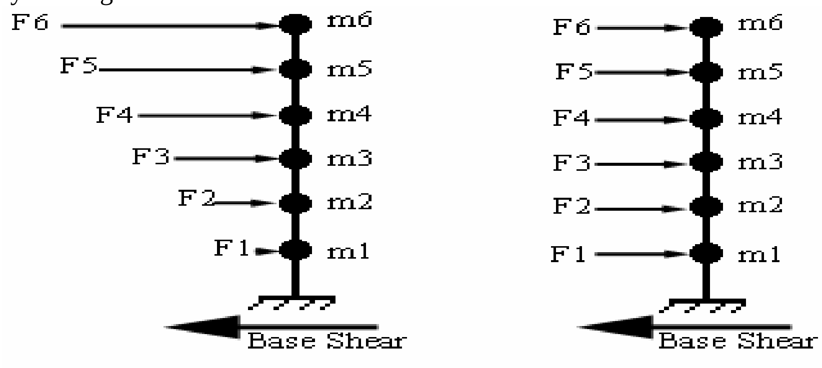


Fig: 2 Conventional lateral load distributions

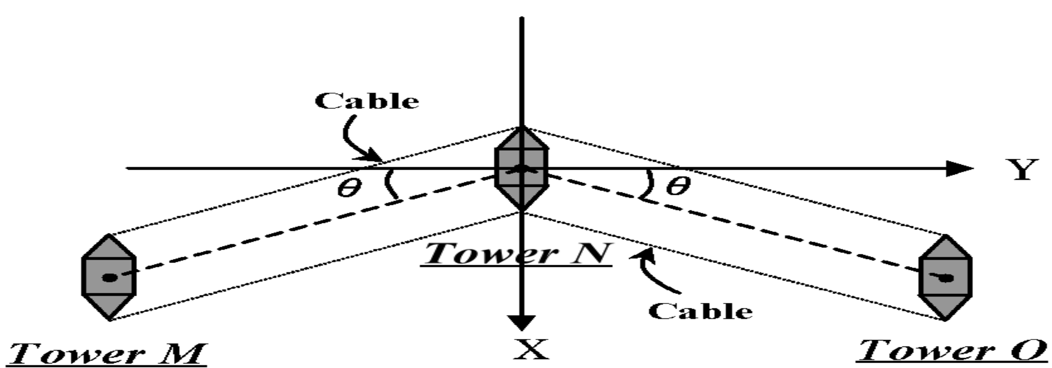
MODELLING AND ANALYSIS

Most of the existing towers cannot resist major or moderate earthquakes as these are primarily designed for gravity loads only, without considering the lateral forces, which makes these towers vulnerable during the event of an earthquake. It is therefore essential to consider the lateral force while designing the towers to mitigate the effects of major earthquakes.

The conventional seismic analysis of transmission towers is usually undertaken by taking each of towers as an isolated structure without taking the strong traction given by high-voltage electrical wires lining up in various directions in the air into account. Furthermore, many of structural engineers were used to simply ignore all wire mass or to take the wire mass as the lumped mass affiliated with the tower in seismic analysis. The results obtained from such analytical schemes would not be able to reflect effectively the actual forced conditions of the tower structure itself as well as base foundation beneath it. In earthquake, wave propagation will take place in the cable which has characteristics of small original stress, large deflection and strong geometry nonlinearity. Large scale movements of cable in vertical direction will cause discharge or short circuit account for too small space between cables, even tensile failure may be caused by large tension stress. So it is not sufficient only taking uniform excitations.

Seismic codes prescribe different methods to carry out lateral load analysis. In the present study the gravity load analysis and seismic load analysis as per the seismic code IS 1893 (Part 1):2002 are carried out for towers.

Models of the transmission tower are done based on the horizontal angle of tower as per IS 802-1984, using finite element software SAP 2000 Vs 14. Horizontal angle of the tower is also called as angle of line deviation, which is defined as, the angle between the axis of the cross arm and the power line which is connected through end of cross arm. The illustration of this parameter is shown in Fig1. In this fig, the towers are named as Tower M, Tower N and Tower O, for distinguish them.



We prepared four types of analysis model. The models have been made according to clause 4.3, PN-13, IS 802-1984. Model 1 is three tower models consisting of one modeled tower in the centre and two towers on each side for considering the effect of conductors. The distance between the two towers is 200m. These three towers are in XY plane with 0^0 line deviation. Model 2, model 3, and model 4 also having three towers as in the model 1, with 15^0, 30^0, and 60^0 line deviation respectively. The foundation of the steel tower is assumed as partially constraint. Main members of the steel tower are modeled by three dimensional truss elements. The power lines are not modeled. The damping ratio for steel tower in a moderate earthquake is 2%.

The tower used here is 220kv double circuit, of height 33.5m and the base width is 5.09m. Three dimensional view of single tower is as shown in the below figures.

RESULTS

The results obtained for different transmission tower models considered for different types of analysis carried out namely gravity load analysis, equivalent static analysis, response spectrum analysis are presented. An effort is made to study the response of tower with different angle of deviations.

Here in the present study, the behavior of each models are studied and the results are tabulated in the form of Time period, and Lateral displacements.

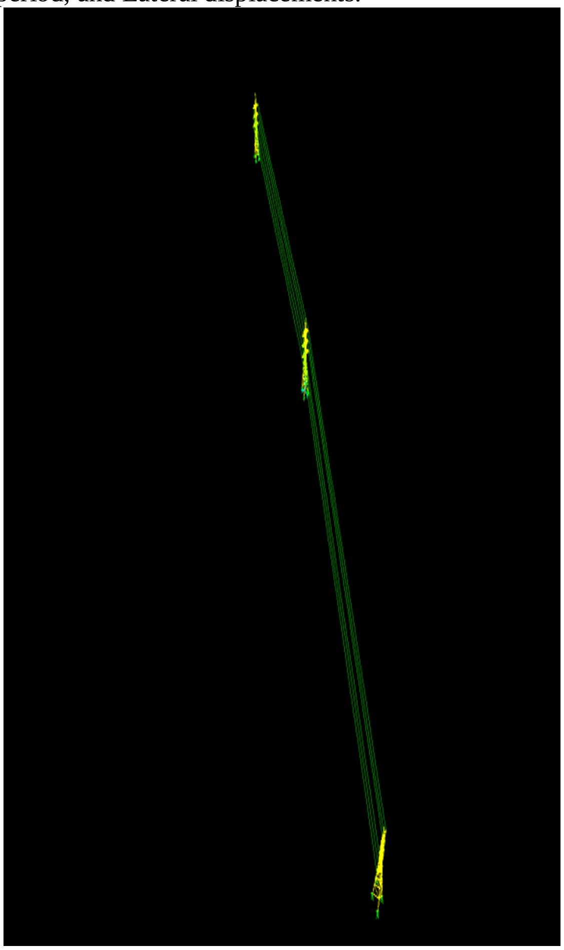


Fig:4 Three dimensional view of Model 1

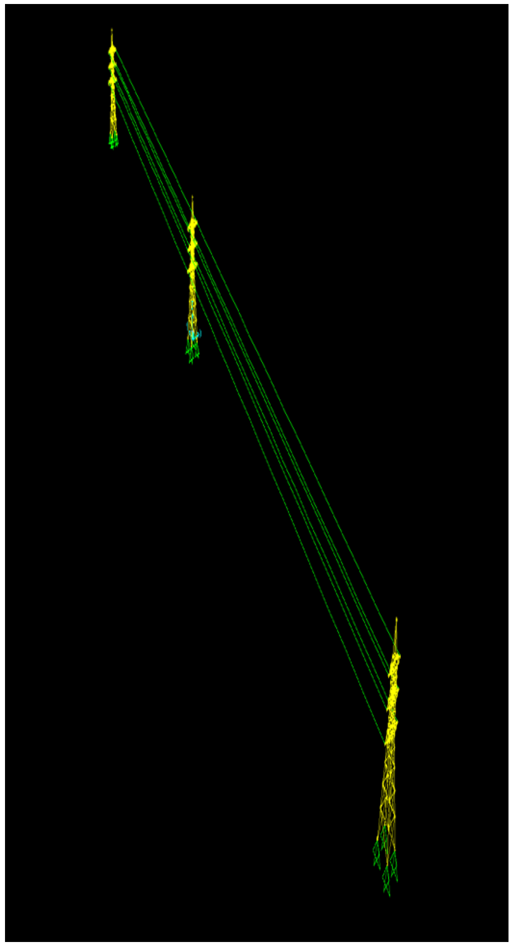


Fig:5 Three dimensional view of Model 2

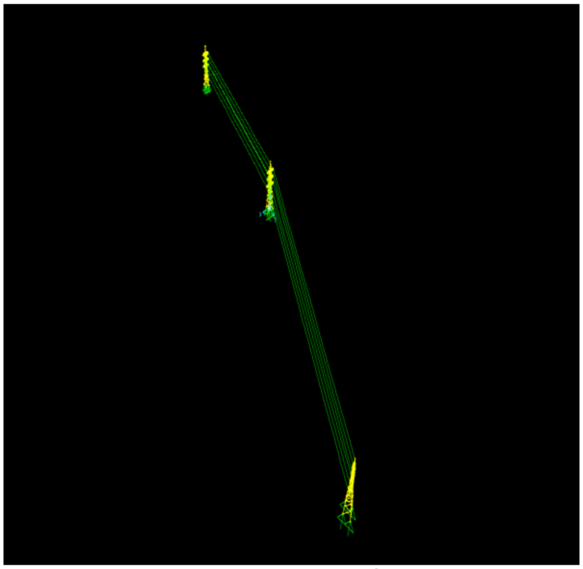


Fig: 6 Three dimensional view of Model 3

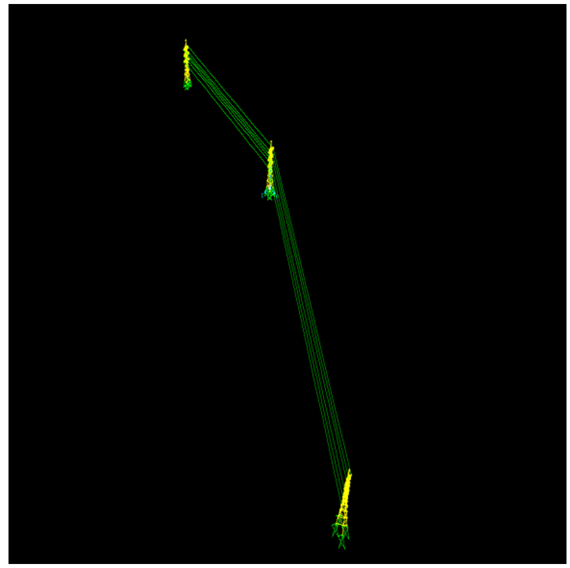


Fig: 7 Three dimensional view of Model 4

Natural periods

The natural time periods obtained from seismic code IS1893 (Part 1) 2002 and analytical (SAP 2000 v14) are given in table. Codal and analytical values do not tally with each other. It can be observed that models with line deviation (angle towers) significantly affects fundamental natural period, which is a function of mass, stiffness, and damping characteristics of the tower.

The comparison of natural period presented in the table shows that, the code IS 1893 (Part 1) 2002 uses empirical formula to calculate natural period which directly depends on the height of the structure. The analytical procedure calculates the natural period on the basis of mass and stiffness of the structure (Eigen value and Eigen vectors). The code doesn't consider the effects of certain irregularities.

Codal and Analytical fundamental Natural periods for Transmission tower models in seconds.

Transmission tower	Gravity analysis Time period in Sec		Natural frequency Cyc/sec
	Code	Analytical	
Model 1	1.1782	0.5743	1.7411
Model 2	1.1782	0.5536	1.8060
Model 3	1.1782	0.5335	1.8742
Model 4	1.1782	0.5584	1.7907

Lateral deformation

According to code IS1893 (Part 1) 2002 lateral displacements of all models along transverse and longitudinal direction for different load combinations are performed. Sufficient data are not available with regard to the permissible limits of displacement of towers. However, one practice given below is followed.

Assuming that there is no shifting of the foundation, the deflection of the top of the support in the longitudinal direction from the vertical should not exceed the following limits:

i. For dead-end heavy-angle structure (1/120) H.
ii. For small angle and straight line structures with strain insulators (1/100) H.
Iii For supports with heights exceeding 160m and intended to be used at crossing locations (1/140) H
Where H is the height of the tower.

The above limits of deflection are applicable to supports having a ratio of base width to height less than 1/12. For suspension supports with heights up to 60m, no limit of displacement of the tower top from the vertical is specified. Below table shows the lateral displacement of transmission tower for first load combination. Similarly we have done the analysis for all load combinations given in IS 1893 (Part 1) 2002.

Lateral displacement of Transmission tower models in longitudinal direction by 1.2(DL+LL+EQX) and 1.2(DL+LL+RSX) load combinations.

(a) Longitudinal direction for seismic combination 1.2(DL+LL+EQX) and 1.2(DL+LL+RSX)

Levels	Equivalent static method Lateral displacement (m)				Response spectrum method Lateral displacement (m)			
	Model 1	Model 2	Model 3	Model 4	Model 1	Model 2	Model 3	Model 4
15	0.0117	0.0047	0.01	0.0049	0.0115	0.0091	0.0132	0.0092
14	0.0084	0.0035	0.0076	0.0037	0.0081	0.0067	0.0096	0.0068
13	0.0076	0.0031	0.0069	0.0032	0.0073	0.0061	0.0087	0.0062
12	0.007	0.003	0.0064	0.0031	0.0067	0.0055	0.008	0.0056
11	0.0062	0.0025	0.0057	0.0026	0.006	0.005	0.0071	0.0051
10	0.0057	0.0025	0.0052	0.0026	0.0054	0.0045	0.0064	0.0045
9	0.005	0.0021	0.0046	0.0021	0.0048	0.004	0.0058	0.0041
8	0.0044	0.0019	0.0041	0.002	0.0042	0.0035	0.005	0.0035
7	0.0038	0.0017	0.0035	0.0017	0.0036	0.003	0.0043	0.003
6	0.0033	0.0015	0.0031	0.0016	0.0031	0.0026	0.0037	0.0027
5	0.0029	0.0012	0.0027	0.0012	0.0028	0.0023	0.0033	0.0023
4	0.0023	0.001	0.0022	0.0011	0.0022	0.0018	0.0026	0.0018
3	0.0015	0.0007	0.0014	0.0007	0.0014	0.0011	0.0017	0.0011
2	0.0007	0.0003	0.0007	0.0003	0.0007	0.0006	0.0008	0.0006
1	0.0002	0.0001	0.0002	0.0001	0.0002	0.0002	0.0003	0.0002
0	0	0	0	0	0	0	0	0

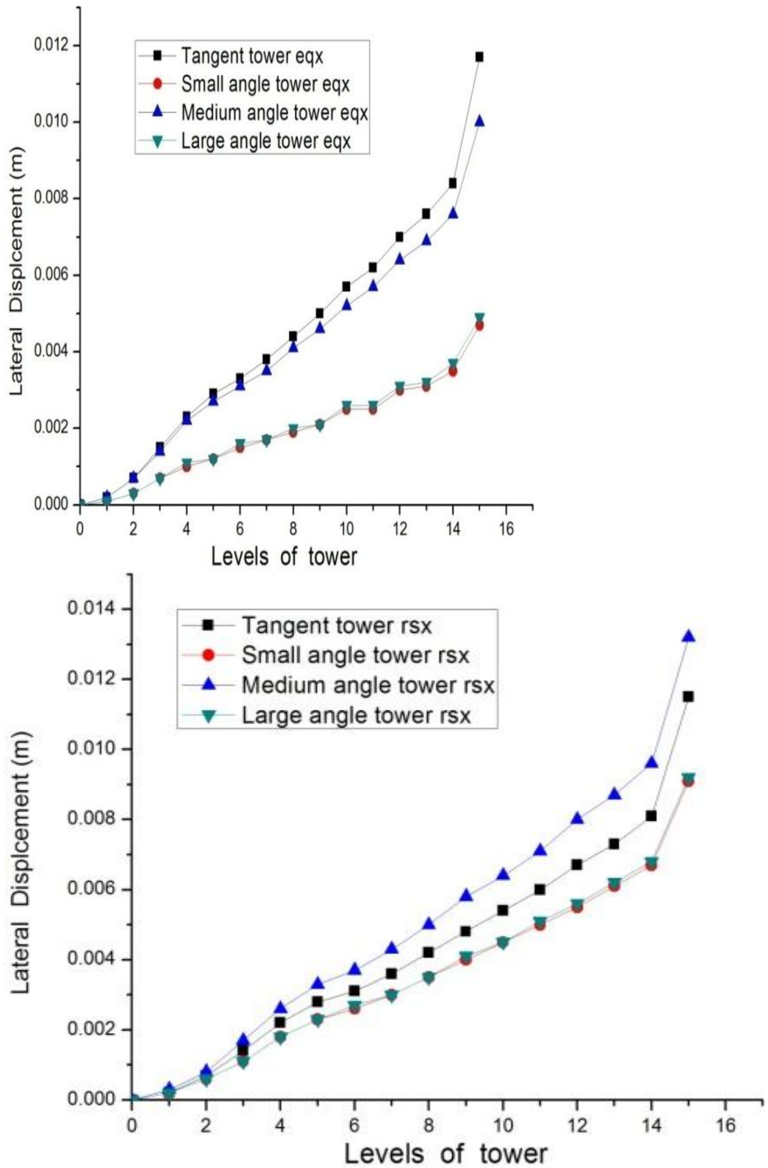


Fig:8 Lateral displacement (m) profile for tower in longitudinal direction by 1.2(DL+LL+EQX) and 1.2(DL+LL+RSX) load combinations.

Critical member

The failure of the transmission tower is triggered by the fracture of any component main-member in the structure. This is generally leg member in the transmission tower. The axial load induced in this member is maximum, is called as critical member in the structure. This member fracture is usually not caused by the combined action resulting from various kinds of internal forces including axial, flexural and shear forces.

Being slender and tall in appearance, the transmission tower is destined to be susceptible to the effect of geometric nonlinearity. On the other hand, the phenomenon of material nonlinearity is often observed on the primary leg members especially for those in the lowermost part of the tower subjected to a seismic force. The critical member in the structure is identified by performing Gravity analysis and Equivalent static analysis. The below table shows the critical member number and maximum compressive axial load induced in that member for gravity analysis, seismic analysis in transverse direction and seismic analysis in longitudinal direction.

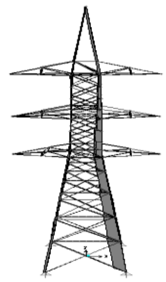


Fig:9 Maximum value of compressive Axial force of main member

Critical member and maximum compressive axial force for gravity and seismic analysis in transverse direction.

Transmission tower	Gravity Analysis		Seismic Analysis in transverse direction					
			1.2 Comb		1.5 Comb		0.9 Comb	
	Member	Force (c) KN	Member	Force (c) KN	Member	Force(c) KN	Member	Force(c) KN
Model 1	2	26.220	2	25.630	2	31.373	2	18.824
Model 2	2	25.158	2	24.864	2	31.385	2	21.963
Model 3	2	23.636	2	23.639	2	29.820	2	17.871
Model 4	2	25.404	2	30.438	2	37.762	2	22.829

Critical member and maximum compressive axial force for seismic analysis in longitudinal direction.

Transmission tower	Seismic Analysis in longitudinal direction					
	1.2 Comb		1.5 Comb		0.9 Comb	
	Member	Force (c) KN	Member	Force(c) KN	Member	Force(c) KN
Model 1	3	20.229	3	24.472	3	22.047
Model 2	3	24.951	3	30.904	3	21.866
Model 3	3	18.949	3	23.685	3	14.212
Model 4	3	25.311	3	31.285	3	18.740

CONCLUSIONS

To investigate the response of tower members under all types of configurations of transmission towers are performed. From the results of all models, the peak failure would occur at the time when the seismic force is parallel to the diagonal, which passes through the leg member considered, of the tower base.

Time period of all models in longitudinal directions are approximately agrees with each other. In the transverse direction also, all models having approximately same values. But the time period of transverse direction is shorter than that of time period of all models in longitudinal direction. The steel tower and power line shake independently in transversal direction by the difference of stiffness. However the steel tower and power lines shake together in longitudinal direction. This is mainly due to the inertia force of the power lines, affects on the tower.

The response in terms of lateral displacement is more for large angle tower and less in small angle tower. This is mainly due to difference of stiffness in the structure. Small angle tower having more stiffness compared to large angle tower.

Larger the value of ϴ, the displacement of the tower also more. Displacement for large angle tower is maximum, because it is having a maximum angle of line deviation. Small angle tower having a lesser probability to failure.

Maximum Axial compressive force induced in the large angle tower is more compared to other towers, and more damage possibility compared to other models. This is mainly due to difference of stiffness in the structure and geometrical nonlinearity of the material.

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