RESPONSE OF TRANSMISSION LINE-TOWER COUPLED SYSTEM SUBJECTED TO MULTI-SUPPORT EXCITATION

A Thesis

Submitted in partial fulfillment of the requirements for the award of the degree of

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IN

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With specialization in

STRUCTURAL ENGINEERING

Under the supervision of

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May- 2018

CERTIFICATE

This is to certify that the work which is being presented in the project title **"RESPONSE OF TRANSMISSION LINE-TOWER COUPLED SYSTEM SUBJECTED TO MULTI-SUPPORT EXCITATION"** in partial fulfillment of the requirements for the award of the degree of Master of technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Rohit Sharma** during a period from Aug 2017 to May 2018 under the supervision of **Mr. Bibhas Paul** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of my knowledge.

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ABSTRACT

The transmission line-tower coupled system is analyzed using SAP2000. When transmission line tower coupled system is subjected to spatially varying ground motion, it may face higher forces and stresses than they were designed to withstand. It is sensible to expect that the transmission line-tower systems are unsafe from earthquake motion so a proper analysis is required. There are a few cases in past of harm to electrical cables during the earthquakes. A few troubles may emerge during the dynamic investigation of transmission line-tower framework as the framework as a whole is non-linear in nature. Like bridges this system is also a type of extended structure and soil conditions may change with the distance covered. A ground motion with phase difference or spatially correlated ground motion may be used to excite the system. Transmission towers will be modeled as truss elements and cables will be modeled as tension members in SAP 2000. After exciting the structure using spatially varying earthquake motion, the response of the system will be studied using Response Spectrum method and Time History analysis method.

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CHAPTER-1

INTRODUCTION

1.1 BACKGROUND

In the recent times it was found that earthquake ground motions can change significantly over distances for same order of magnitude. Some engineering structures are extended over very high distances like bridges, transmission line-tower system etc. This variation is termed as spatially varying earthquake motions (SVEGM). Quake estimated at various areas inside the measurements of a designed structure or expanded structure like bridges, transmission line-tower is typically different.

Three factors are responsible for the spatially varying earthquake ground motion waves:-

1. Different timings of earthquake waves arriving at different stations.

2. Incoherency of motion due to reflection and refraction of waves in different kind of mediums.

3. Difference in local soil condition at different stations.

This is universally accepted theory to represent spatial fluctuation as the joined impact of three causes: Incoherency in motion of the wave with distance due to its continuous change of medium caused by reflections and refractions, phase lag, and the difference of the surface motion due to upper soil layers having different kind of index properties. Due to these factors, there is variation in ground motion. Seismic analysis of transmission line-tower system is very much necessary as they might be used in areas of high seismicity where earthquake had already hit several times. . Likewise a noteworthy issue is that, earthquake motion may produce large displacements in cables and these cables may touch each other resulting in power failure. Generally cables are designed to take traverse load but due to SVEGM these cables can also be subjected to other kind of load conditions. The transmission line-tower system may not be able to wear this load and hence can collapse. It is not realistic to assume that transmission line-tower coupled system is safe from SVEGM. Earthquake ground motions can produce a noticeable effect on the response of extended structures. The spatial variation of seismic ground movements considerably affects the reaction of extended designed structures, for example, bridges, pipelines, transmission frameworks, and so forth.

Since these structures have large dimensions parallel to the ground, their supports are subjected to various movements during an earthquake. Presently, it has been explored that the variations with respect to distance in seismic ground movements, can dramatically affect the base reactions of expanded structures.

This spatially fluctuating movement may increase the reaction of these extended structures more than the reaction which they were designed to wear. The results of previous research papers in the literature shows that the effect of the spatial variation of earthquake ground movements on extended structures are very interesting and we cannot neglect these variations. This study is important in itself as there was a lot of damage which was done to transmission lines and towers due to earthquake. The investigation and outline of past more established structures to face earthquake ground movements depends on the presumption that the ground movements over the whole established expanded structure are basically the same. This supposition may not be reasonable for the differing ground movement. Cases: - Northridge tremor (1994), Landers Earthquake (1992). In case of expanded built structures, both the shifting vibration properties of neighboring ranges and the non-uniform spatial ground excitation at the continuous support can prompt differential vibrations. Taking the examples from the history such as Loma Prieta (1989) and Kobe (1995), the collapses of bridge decks were observed due to varying movements between adjacent bridge spans. It will be very useful to consider all type of complexities and all the probable kind of failures in our design so that we can create an efficient model that may relate to all kind of natural possibilities. A few endeavors in the past have been made to outline a reaction range technique in which we can think about difficulties. It is exceptionally important to locate an improved strategy to research the impact of various excitation on the nonlinear reaction of transmission lines tower coupled framework. So in this study it is tried to create a technique in which we can analyze the transmission-line tower system properly and accurately using all the factors.

1.2 OBJECTIVE OF STUDY

- To study the behavior of transmission lines and towers due to multi-support excitation.

- To develop a simple technique to study effect of spatially varying motion on transmission line-tower coupled system.

1.4 SCOPE OF STUDY

- Safety
- Reduction in maintenance cost
- No power cuts after earthquake
- Improved electricity transmission system
- Improved service quality
- Lower risk factor
- Lesser chances of short circuits during earthquake

CHAPTER-2

LITERATURE REVIEW

Ghobarah, A., Aziz, T. S., & El-Attar, M., “Response of transmission lines to multi-support excitation” [1]

They have analyzed the impact of different excitation on multi support structures on the reaction of transmission Line system. Because of ground movement transmission cables may face higher loads than the normal loads which they face due to wind and ice . In this paper towers are modeled using truss elements and cables are modeled using tension members which represent the non-linearity of the system. The outcomes demonstrate that considering uniform ground movements at all supports of a transmission line don't give the case which is most severe for the reaction computations. It was discovered that the wave spread speed considerably affects the reaction of the framework. So as to get a satisfactory investigation of transmission lines, a precise estimation of wave speed is required. These following conclusions were drawn from the study: considering uniform ground movement on the two supports may not be the most dangerous case for the reaction of transmission lines. Multi support excitation, which is really a genuine condition, can bring about bigger stresses and forces. The extra tension in transmission line because of horizontal ground movement was observed to be moderately little. [1]

Kiureghian, A. D., & Neuenhofer, A., “Response spectrum method for multi-support excitations” [2]

It is observed that earthquake ground movements can fluctuate and this variety can be huge over separations for some request of extent for broadened structures like extended structures and transmission line-tower framework. Three factors are in charge of the variety which are phase lag, loss of coherency because of reflection and refraction of waves, diverse neighborhood soil condition. In this paper dynamic investigation is done using time history approach or technique for irregular vibration. This paper depends on prior reports and built up another reaction range technique for seismic similarity for broadened structures with multi support excitation. Technique utilized by them depends on standards of irregular vibration and hypothesis and records for cross co-relations between help movement and methods of vibration. [2]

Allam, S. M., & Datta, T. K., "Seismic behavior of cable stayed bridges under multi component random ground motion" [3]

A frequency domain spectral analysis is presented in this paper. Cable stayed bridges were analyzed and they were excited using stationary random ground motion with a number of variations which was inclined at an angle with respect to the x-axis. Spatial variation of ground motion between the supports is considered. Various parameters are considered such as spatial correlation of ground motion, angle of incidence of the waves, nature of the earthquake etc. So they found that response of bridge is influenced by tower-deck ratio. Ratio between vertical and horizontal components of ground motion influences the response. Dynamic response of bridge could be significantly higher for power spectral density function of ground motion for loose soil than firm soil. [3]

Zanardo, G., Hao, H., & Modena, C., "Seismic response of multi-span simply supported bridges to spatially varying earthquake ground motion" [4]

They completed an investigation using different parameters on the pounding phenomenon which is related to the earthquake motion of multi-traverse basically bridges spans with base isolation gadgets. They have shown the relation between pounding phenomenon and different characteristics of a spatially varying earthquake ground motion. So as to incorporate the impact of the torsional part of pounding forces on the seismic reaction of the entire structure, a three-dimensional (3D) limited component demonstrate has been exhibited and three dimensional time-history analysis have been performed. So when analysis was completed it was realized that spatially varying earthquake ground motion produces forces three to four times bigger than those were produced by the uniform forces. The analysis performed in the present investigation demonstrated that spatially varying earthquake ground motion can really change and increase the reaction forces and may result in huge destruction which may cause loss of life. [4]

Zerva A., Zervas V., “Spatial variation of seismic ground motion :An Overview” [5]

This study gives the information about the spatially changing earthquake ground motion as evaluated from data recorded from the densely placed instrument arrays. It concentrates on the stochastic description of the spatial variation, and mainly concentrated on spatial coherency. The variation (w.r.t space) of earthquake ground motions has an considerable effect on the response of extended structures such as nuclear power plants, tunnels, communication transmission systems, transmission-line tower system etc. Because these structures extend over long distances and may have different soil conditions at different supports, so differently spaced supports are subjected to different amount of reactions. The spatial variability of seismic ground motions, as understood from analysis of data recorded at dense instrument arrays, was described in this paper. An alternative approach for the investigation of the spatial variability of seismic ground motions that views spatial variability as deviations of amplitudes and phases at individual stations around a coherent approximation of the seismic motions was described. [5]

Lupoi, A., Franchin, P., Pinto, P. E., & Monti, G., “Seismic design of bridges accounting for spatial variability of ground motion. [6]

In this paper the effect of spatial inconsistency of the quake ground movement on the reaction of expanded structures like bridges are explored. Following an entrenched tradition, this may be spoken to as the joined impact of three causes: the loss of soundness of the wave movement with distance because of reflection and refraction, the wave-passage effect, and the neighborhood soil conditions accessible at the support. Spatial fluctuations of seismic wave movement is a complex phenomenon, cannot be displayed using simple modeling. The method followed in the present study is quite efficient and accurate. It depends on numerical simulation brought out through non-linear dynamic analysis on number of bridges subjected to purposely chosen combinations of the components including spatial changeability. They analyzed spatial fluctuation is because of the consolidated impact of (a) change of coherence, (b) wave travelling in different sorts of mediums and (c) distinction in nearby site conditions having diverse soil properties. The outcomes demonstrate that for all kind the sort of extensions, flexibility demands at the base of the docks due to spatial variety of movement increment in the dominant part of cases. [6]

Vanmarcke, E. H., Heredia-Zavoni, E., & Fenton, G. A., "Conditional simulation of spatially correlated earthquake ground motion" [7]

Strategy displayed for simulating legitimately connected spatially fluctuating quake ground movements at a discretionary arrangement of closely spaced stations, compatible with known or recommended movements at different areas. Direct expectation estimators are utilized to produce an arrangement of measurably autonomous, recurrence particular, spatial arbitrary procedures, in view of which ground movements are analyzed by method of quick Fourier change calculation. This technique has favorable position over other prior direct estimation strategies, known as kriging, is that it effectively imitates the predefined space-time relationship structure of the earthquake wave movement. The simulation strategy created here depends on the hypothesis of multivariate straight expectation, and guarantees the fair generation of the co-variance structure of the ground movement; in this regard, it is a critical change over existing technique. For a given bridge, the likelihood of failing depends on various known and unknown parameters. [7]

Albermani F. G. A., and Kitipornchai S., "Numerical simulation of structural behavior of transmission towers" [8]

They have adopted nonlinear analytic technique for analyzing the transmission towers or lattice towers. Generally it is not easy to analyze these three dimensional structures accurately as they are very complex in nature. They have used a mathematical approach for analyzing the forces and stresses in these structures. They have calculated the load at which tower may fall and what would be the mode of failure. They have worked on strengthening the tower's structure so that their life span can be increased. [8]

ZERVA A., "Response of multi-span beams to spatially incoherent seismic ground motion" [9]

They have checked the behavior of two and three span beams which are having different lengths, under the spatially varying earthquake ground motion. They have checked the accuracy of the assumptions which are being followed from many years about spatially varying earthquake ground motion. Shear force, bending moments and displacements are the final results of the analysis and all these outputs depend on the dynamic behavior of the structure. [9]

Kahan, M., GIBERT, R. J., & Bard, P. Y., "Influence of seismic waves spatially variability on bridges: A sensitivity analysis" [10]

In this paper they have checked the sensitivity of the bridges when they are subjected to earthquake ground motion which varies spatially. The method they have used in their analysis was developed by Der Kiureghia and Neuenhofer. They have found that each mode shape contribute independently in the dynamic behavior of the structure which ultimately affects the sensitivity of structure. They have categorized the spatial variations in two parts: - small and large. Sensitivity of structure is checked for small and large variations differently. [10]

Monti, G., Nuti, C., & Pinto P. E., "Nonlinear response of bridges under multisport excitation" [11]

They have presented a study on bridges which are having different stiffness and ductility values. All the bridge models were subjected to multisport excitation which is generated due to earthquake ground motion. A numerical analysis was carried out to study the response of bridge. The peak response the bridge had been correlated with ductility of the structure in this structure. Ductility and force reduction factor (q), both play an important role in deciding the dynamic behavior of the structure. Bridges are checked for both uniform and non-uniform motions and stresses in all the components of a bridge are compared. [11]

Kocer, F. Y., & Arora, J. S., "Optimal design of latticed towers subjected to earthquake loading" [12]

They have presented a study on design of towers. As we know tower is a very important structure in our daily life so it should function well at the time of earthquake as well as after the earthquake. In this study tower is designed for normal loads which occurs generally and earthquake load also. Two methods were used in this study: - Adaptive discrete assignment method and Genetic algorithm. First method is easy but requires more time as compared to second one. But second method is difficult to perform comparatively. Allover both the methods are easy to use when compared to conventional methods and give significant results. [12]

Martire, G., Faggiano, B., Esposto, M., Mazzolani, F. M., Zollo, A., & Stabile, T. A., "The seismic response of submerged floating tunnel under multisport excitation" [13]

In this study submerged floating tunnel is subjected to multisport excitations. This tunnel lies inside the water at a particular depth and the tunnel is stabilized using the cables inside the water. Even it is very difficult to design such a complex model in any software but once it got completed correctly it can be very useful in in our practical life. Response of the designed tunnel due to multisport excitation is studied and presented in this paper. [13]

Pasticier, L., Amadio, C., & Fragiacomo, M. "Non-linear seismic analysis and vulnerability evaluation of a masonry building by means of the SAP2000 V. 10 code" [14]

The aim of this paper is to check and compare the results of a pre-existing structure in a software named SAP2000. A pre-existing structure is analyzed and analysis was compared to that of pre-existing numerical methods, strength of the columns of structure was checked. Main aim of the paper was to show that we can rely on this software for the analysis. Seismic analysis of the structure was also done in the software. Response spectra curves were drawn using software itself and residue strength of the structure was checked after the earthquake. [14]

Price, T. E., & Eberhard, M. O., "Effects of spatially varying ground motions on short bridges" [15]

The study presented in this paper is based on the response of the short bridge when subjected to the spatially varying ground motions. Seven different types of motions were generated and response from each support was noted. The excitations due to coherency effect and wave passage effect were compared to those excitations which were supposed to not altering due to any effect. So all the information gathered was shown in a table and the variations were studied. [15]

Rampure, A. B., & Mangulkar, M. N. "Comparison between response spectrum and time history method of dynamic analysis of concrete gravity dam" [16]

A study is presented on the dam structure on the basis of response spectrum method and time history analysis method. A comparison is basically shown between the results of the two methods. It is very important to design and analyze dam structure accurately because dam stores huge amount of water and serves the mankind through various ways. During the analysis dam was subjected to all the normal forces which is due to its own dead load and forces due to water, moreover that dam model was subjected to earthquake forces and analysis from both the methods was completed. Dam structure should not fall during or after the earthquake as it can lead to a huge destruction, so from the study they have tried to improve the design method of the structure so that we can reduce the vibrations which are induced due to earthquake motion and we can save our structure. [16]

Sabetta, F., & Pugliese, A., "Estimation of response spectra and simulation of non-stationary earthquake ground motions" [17]

They have used Italian strong motion data for their study to analyze the effect of various earthquakes. They have considered 65 numbers of earthquakes which have magnitude lying between 4.6 to 6.8. Artificial accelerograms were used to measure the effect of earthquake wave motion. Response was recorded at various distances from the main station and factors affecting the wave motion were also searched out. All the responses from the accelerograms were noted down and peak response was searched from all the results. The wave motion is altered by source distance, condition of the soil strata at the particular station and magnitude of the earthquake. [17]

CHAPTER 3

3.1 MODELLING OF SYSTEM

Transmission line-tower coupled system is modeled in SAP2000 where transmission towers are designed as Truss elements and transmission lines are designed using cable tension members. Fixed support is provided to each support of towers. Tower here used is of Elevation 50m. Width of tower at highest level is 7.65m. Tower is provided with four fixed supports at ground level. Distance between two towers is 500m. Material used in tower modeling is steel. Now, deformed length of cable is 500.579m. Maximum vertical sag in each cable is 10.4139m. Cable has a horizontal tension component of 148.9325 KN. Material used for cable modeling is A992fy50. Diameter of cable is 0.0287m and cross – sectional area is 0.0006452 sq. meter.

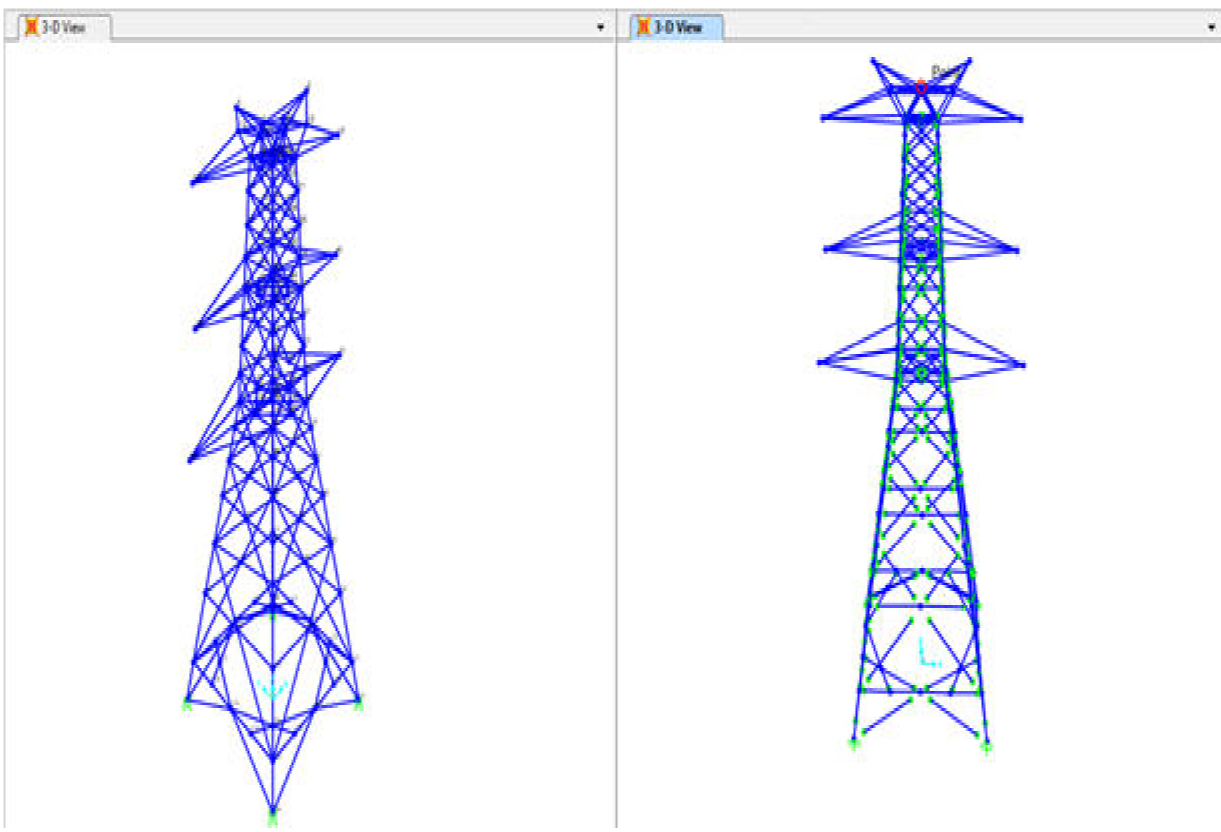

Figure3.1 3D model of transmission line tower

3.2 MODAL ANALYSIS

Modal analysis or the mode-superposition strategy is a straight unique reaction method which assesses and superimposes free-vibration mode shapes to ascertain relocation designs. Mode shapes discloses to us the quantity of number of routes into which a structure can normally distort or dislodge. Mainly, lateral movements are of essential concern. Mode states of low-arrange numerical expressions have a tendency to give the best contribution to structural response. As order increment, mode shapes contribute less, and are anticipated less dependably. It is sensible to stop investigation when the quantity of mode shapes is adequate.

A structure having X number of degrees of modes will have X relating mode shapes. Each mode shape is an autonomous and standardized displacement design which might be intensified and superimposed to get a resultant displacement design.
Here few mode shapes with their time and frequency are shown, total mode shapes obtained were 130.

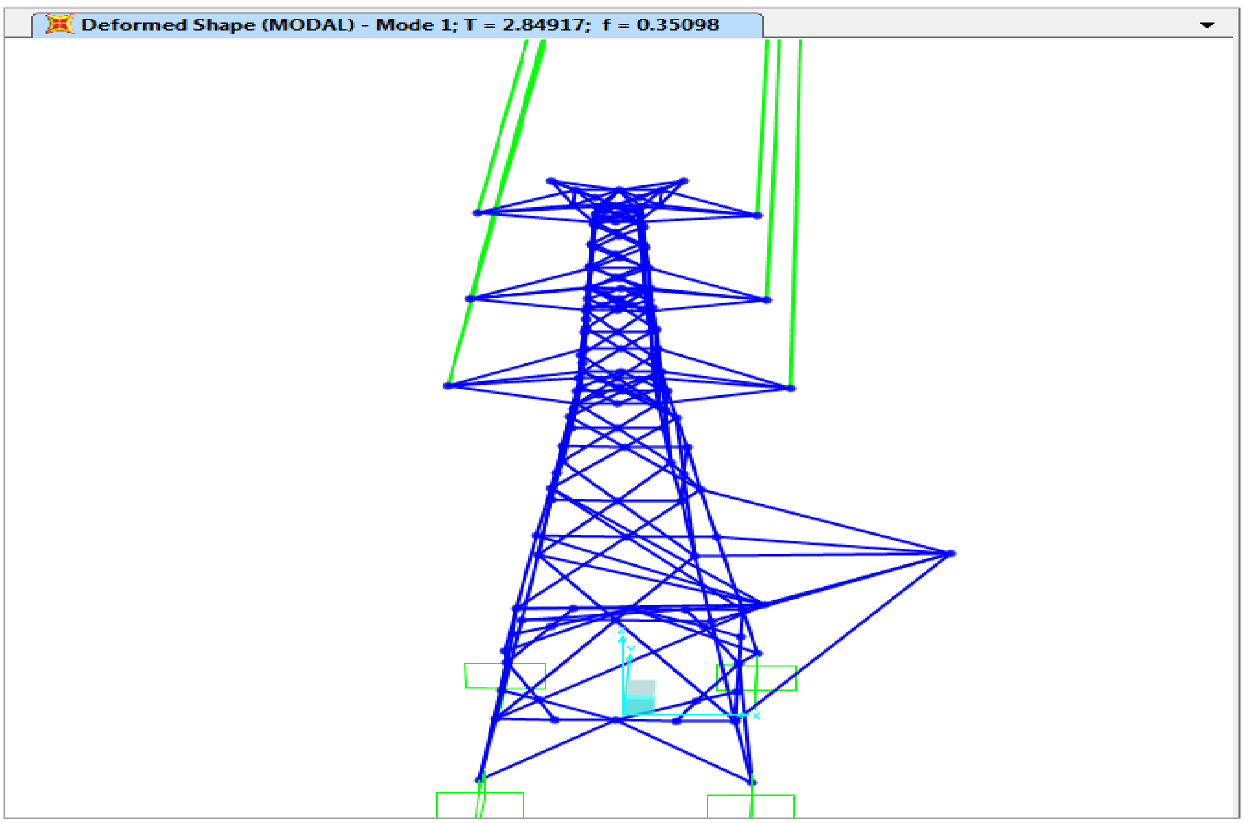

Figure3.2 First mode shapes of the model in which deformation at the base can be seen with frequency value of 0.35098 and time period is 2.84917 seconds

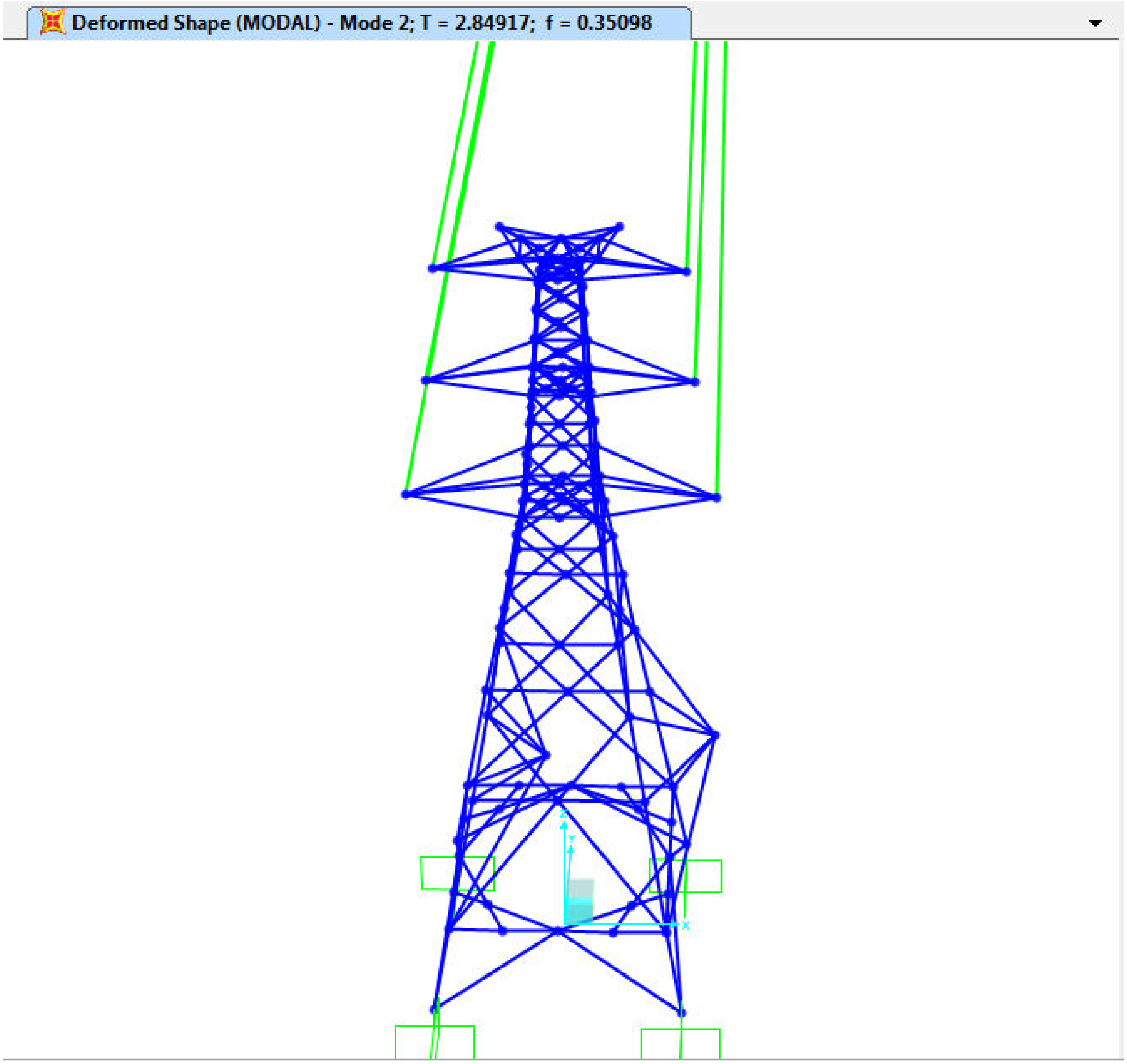

Figure3.3 This is second mode shape in which deformation can be seen at all the four legs, time and frequency values are nearly similar to first mode shape but deformation is different. Various elements are deformed and shape at base level is changed due to deformation.

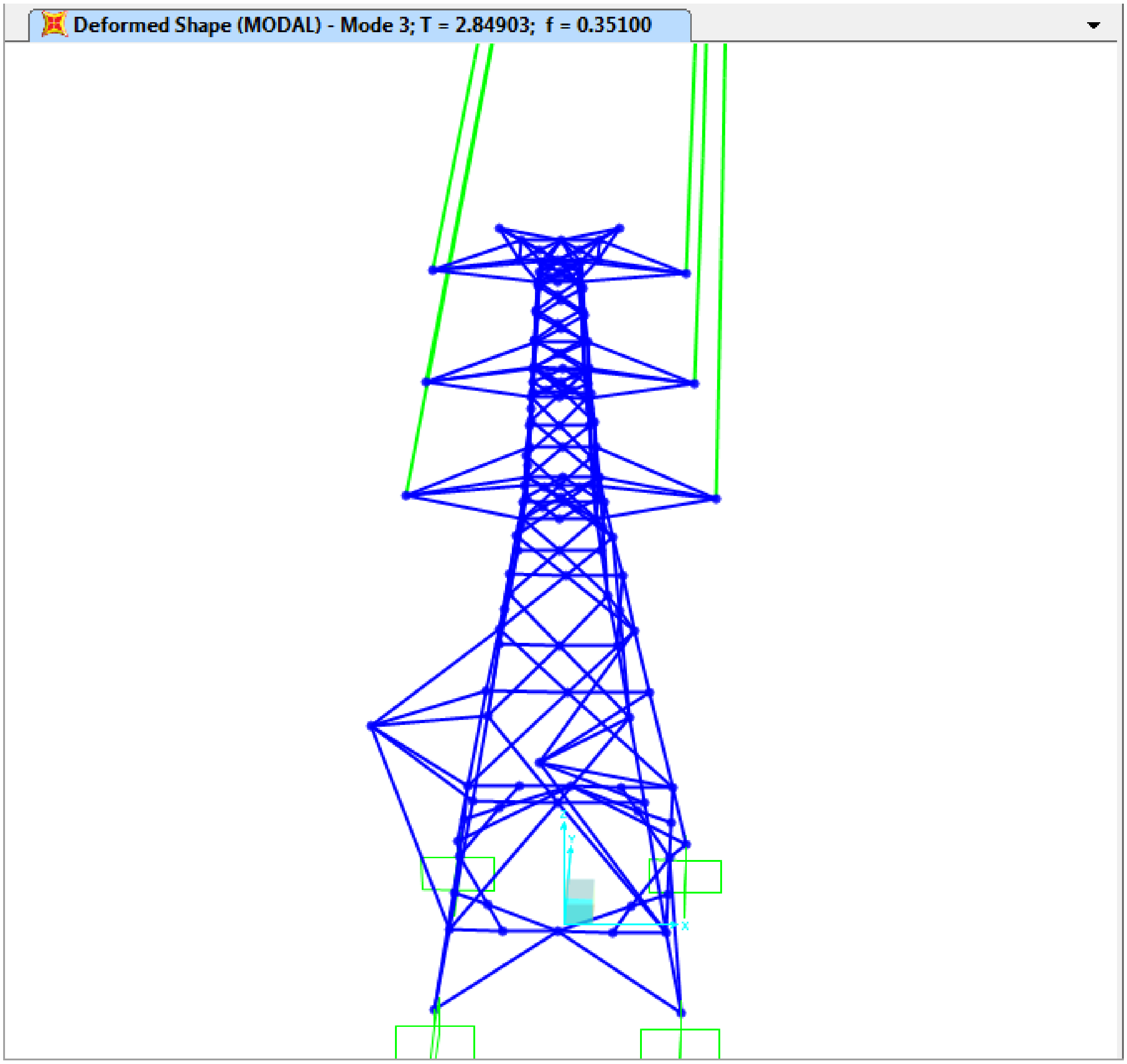

Figure3.4 This is mode shape 3 on which deformation at various parts of tower can be seen. Value of frequency is 0.3510 and tine period is 2.84903 seconds. In this mode shape deformed sections can be seen in left side of tower. Inner parts of the tower are also deformed .

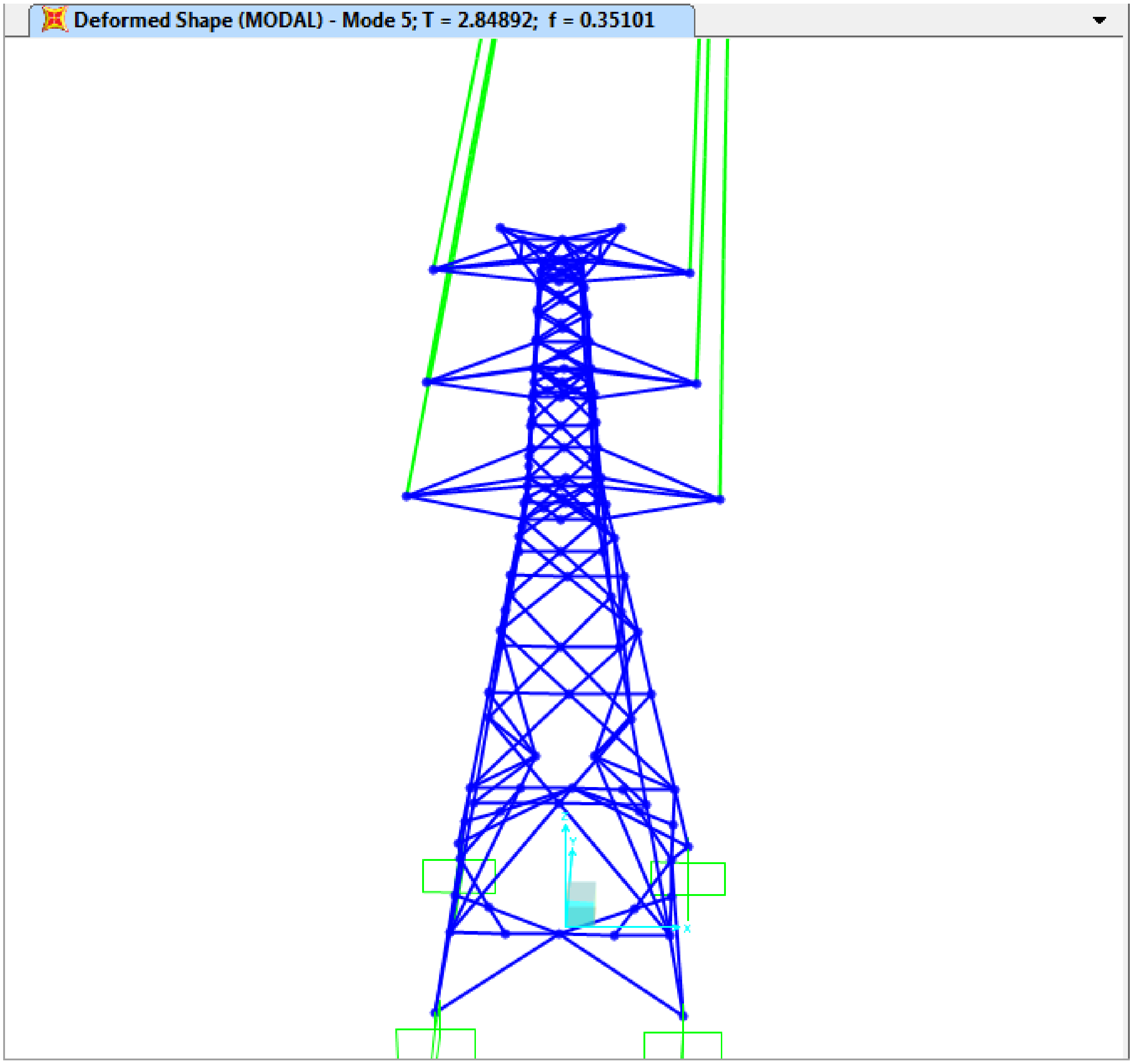

Figure3.5 Deformed shapes from mode number 5 and value of frequency is 0.35101 and time period is 2.84892 seconds. In this mode shape deformations are mainly inside the structure. The design and symmetry of the tower is changed due to deformation and it can result in failure of structure.

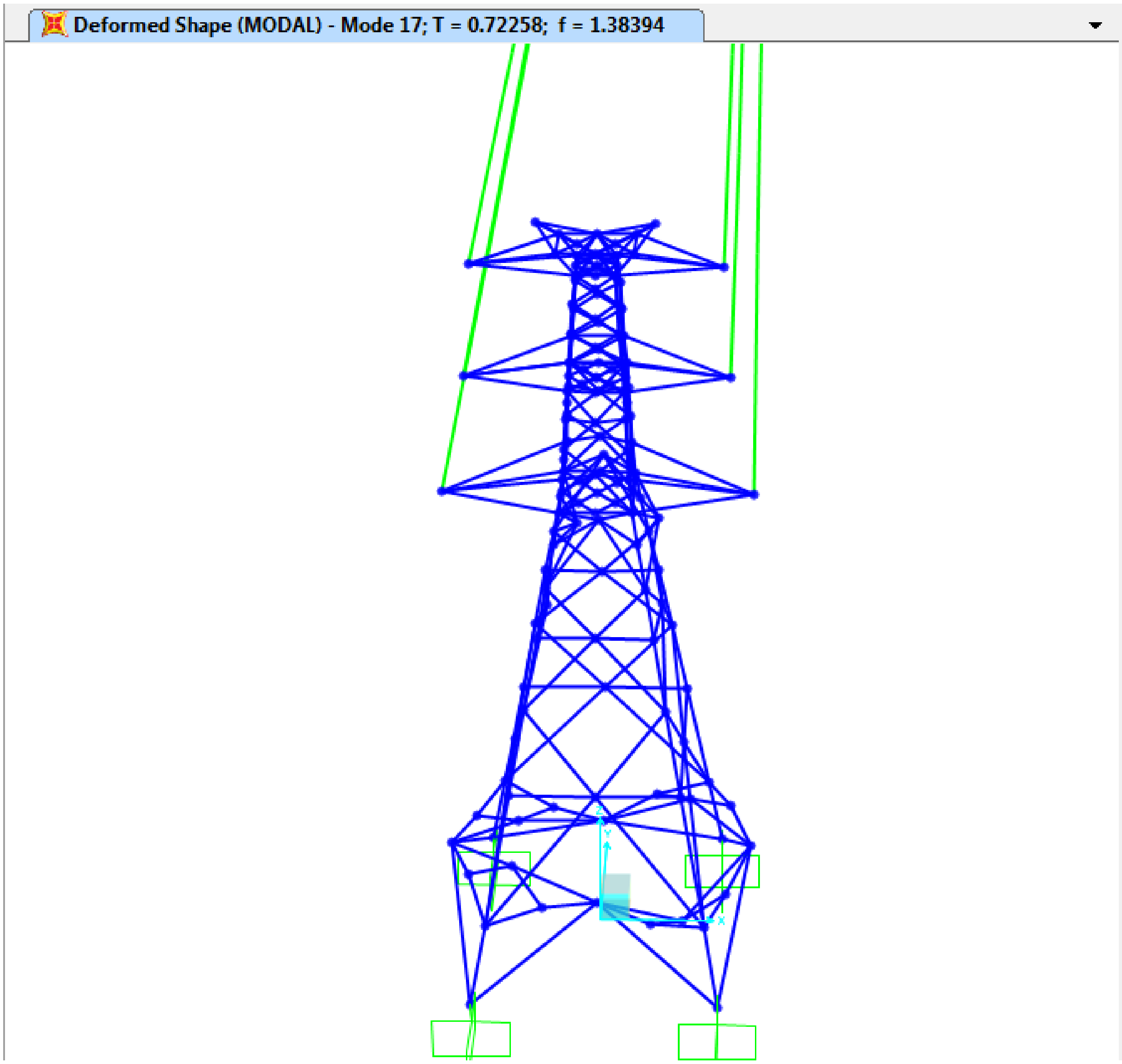

Figure3.6 Deformed shape from the mode 17 and the value of frequency is 1.38394 and time period is 0.72258 seconds. Here all the four legs are deformed near the supports and structural stability may be distorted due to this. Some deformation can also be seen in middle part of the structure where geometry of the tower is changed.

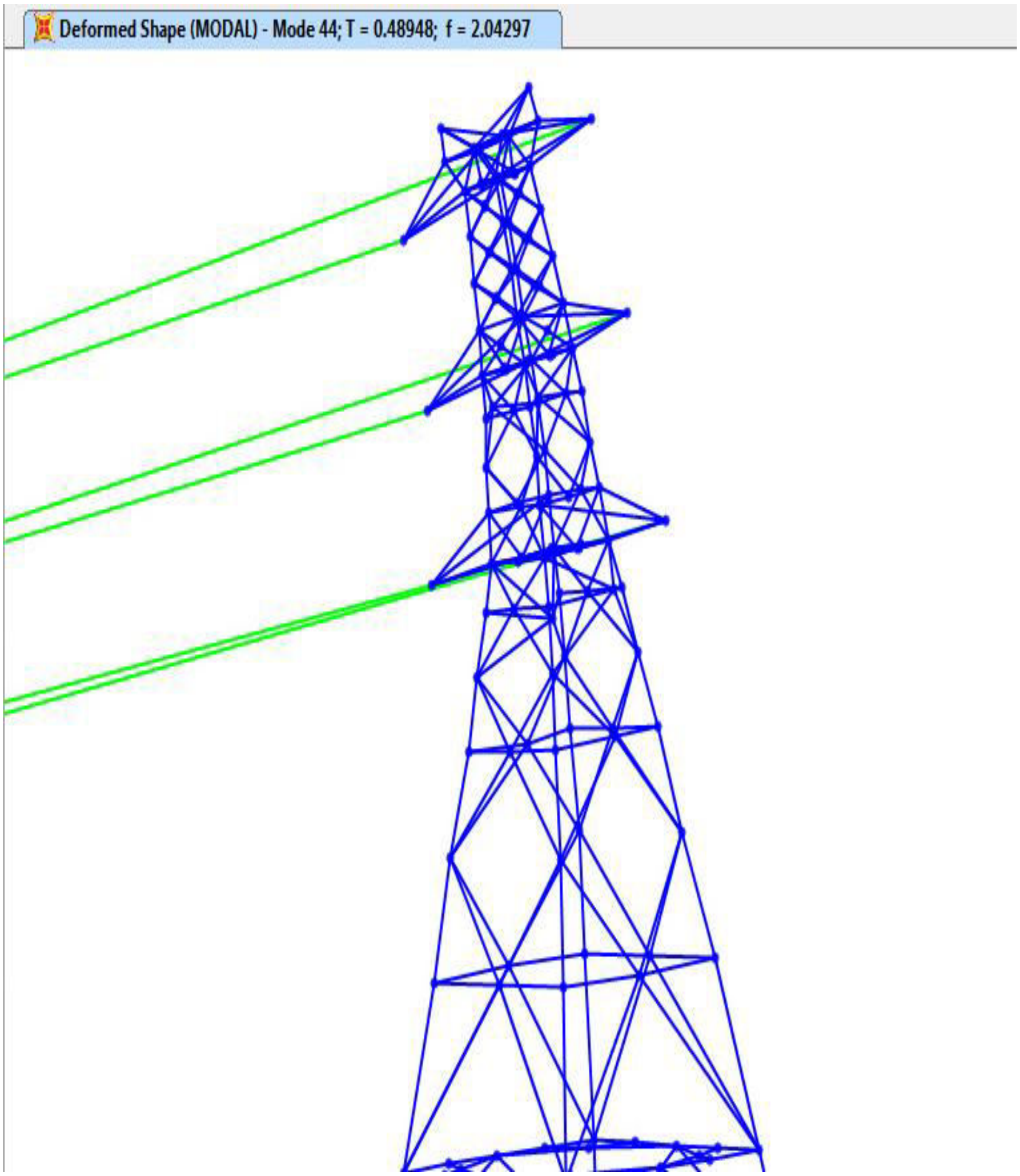

Figure3.7 Deformed shape and frequency of mode number 44 having value of frequency 2.04297 and time period is 0.48948 seconds. In this mode shape there is no deformation in inner parts of the tower but shape of the tower has changed it is no more straight or erected. Tower has bend over on side.

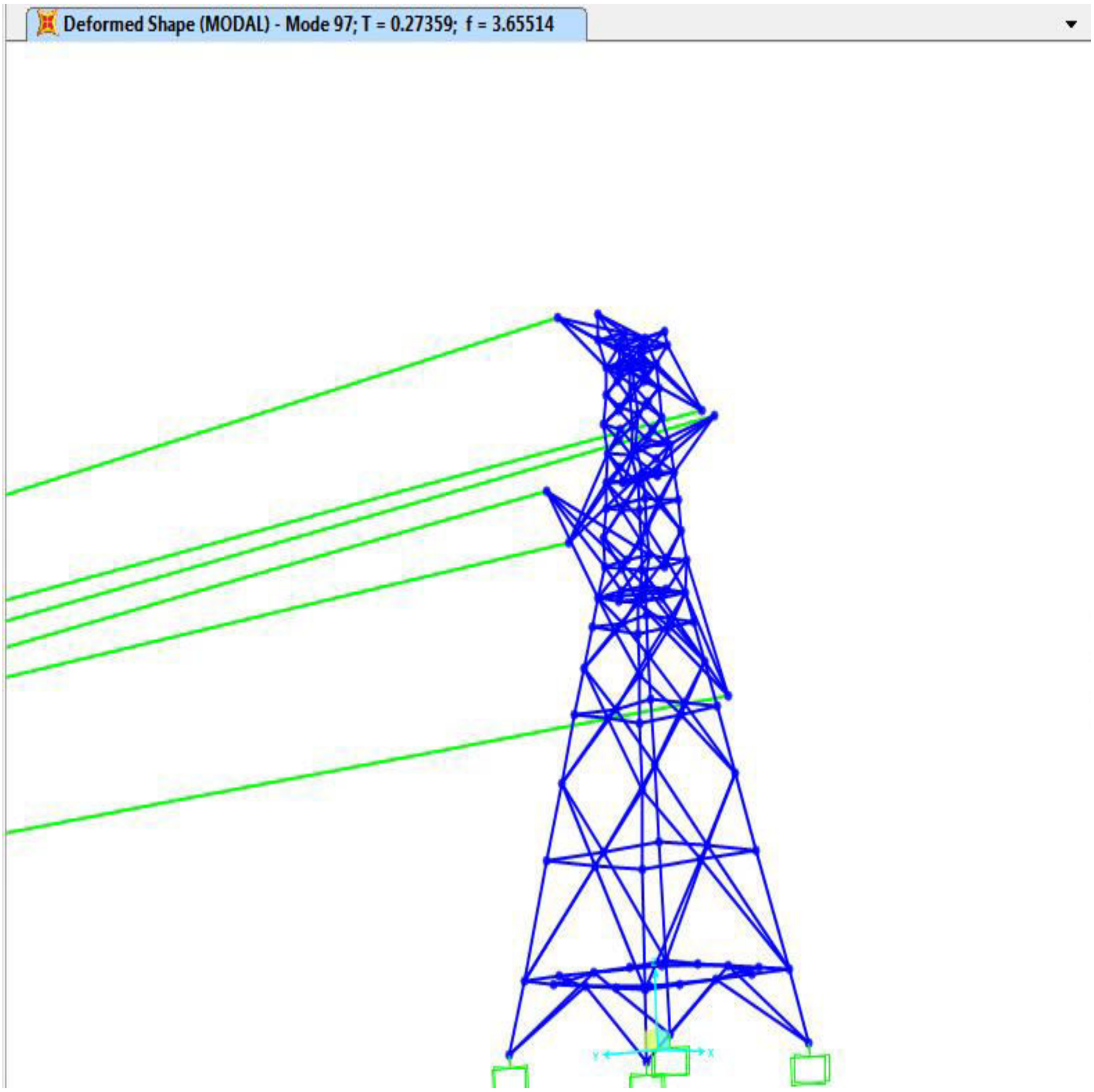

Figure3.8 Deformed shape and frequency of mode number 97 having frequency value 3.65514 and time period is 0.27359 seconds. In this mode shape arms of the tower has been distorted One side of 2nd arm has gone upside which may lead to short circuit and may cause serious problem.

3.2.1 Modal Analysis Result

Table3.1 Modal periods and frequencies of first 30 modes is shown in this table

TABLE: Modal Periods And Frequencies						
OutputCase	**StepType**	**StepNum**	**Period**	**Frequency**	**CircFreq**	**Eigenvalue**
Text	Text	Unitless	Sec	Cyc/sec	rad/sec	rad2/sec2
MODAL	Mode	1	2.849169	0.35098	2.2053	4.8632
MODAL	Mode	2	2.849169	0.35098	2.2053	4.8632
MODAL	Mode	3	2.849169	0.35098	2.2053	4.8632
MODAL	Mode	4	2.848917	0.35101	2.2055	4.8641
MODAL	Mode	5	2.848917	0.35101	2.2055	4.8641
MODAL	Mode	6	2.848917	0.35101	2.2055	4.8641
MODAL	Mode	7	2.643104	0.37834	2.3772	5.6511
MODAL	Mode	8	2.643103	0.37834	2.3772	5.6511
MODAL	Mode	9	2.643102	0.37834	2.3772	5.6511
MODAL	Mode	10	2.642941	0.37837	2.3773	5.6518
MODAL	Mode	11	2.642941	0.37837	2.3773	5.6518
MODAL	Mode	12	2.642941	0.37837	2.3773	5.6518
MODAL	Mode	13	0.887523	1.1267	7.0795	50.119
MODAL	Mode	14	0.887514	1.1267	7.0795	50.12
MODAL	Mode	15	0.887501	1.1268	7.0796	50.121
MODAL	Mode	16	0.887247	1.1271	7.0817	50.15
MODAL	Mode	17	0.887247	1.1271	7.0817	50.15
MODAL	Mode	18	0.887247	1.1271	7.0817	50.15
MODAL	Mode	19	0.830957	1.2034	7.5614	57.175
MODAL	Mode	20	0.830951	1.2034	7.5614	57.175
MODAL	Mode	21	0.83095	1.2034	7.5614	57.175
MODAL	Mode	22	0.830725	1.2038	7.5635	57.207
MODAL	Mode	23	0.830725	1.2038	7.5635	57.207
MODAL	Mode	24	0.830724	1.2038	7.5635	57.207
MODAL	Mode	25	0.77511	1.2901	8.1062	65.71
MODAL	Mode	26	0.77511	1.2901	8.1062	65.71
MODAL	Mode	27	0.77511	1.2901	8.1062	65.71
MODAL	Mode	28	0.729931	1.37	8.6079	74.096
MODAL	Mode	29	0.729878	1.3701	8.6085	74.107
MODAL	Mode	30	0.729838	1.3702	8.609	74.115

3.3 RESPONSE SPECTRUM ANALYSIS

Response spectra are bends which are plotted between most extreme reaction of single degree of freedom subjected to a specific quake ground movement and its time for which it happens. Response spectrum may be called as the locus of maximum response of single degree of freedom system for specified damping ratio. So we can say that response spectrum is a kind of analysis which takes contribution from all mode shapes and after combining them it gives the maximum response which can occur to the structure. Generally time domain or frequency domain analysis may be used for the calculation of the maximum response. The analysis can be used for all ranges of mode shapes and time periods for a single degree of freedom. To plot the final graph of response spectrum analysis, time period is marked on X-axis and response quantity on Y-axis with a specified damping ratio. Now to get a overall maximum response graph is plotted with different damping ratios.

This analysis is very much helpful in designing earthquake resistant structures and also helps in making decision that what type of structure should be used as it shows us dynamic performance of the structure under earthquake ground motion. Structures having shorter time-period experience higher acceleration and vice versa. The performance of the structure should be taken in consideration during primary design and response-spectrum analysis.

3.3.1 Damping and Response Spectrum Analysis

It gives understanding how extraordinary damping proportions changes the reaction of a structure. A group of reaction curves might be produced with variable levels of damping. As we increment the damping proportion somewhat, reaction spectra move descending.

The International Building Code (IBC) depends on 5% damping. This records for coincidental damping from hysteretic conduct, which isn't expressly demonstrated during RSA.

Viscous damper for the most part does not influence solidness of the structure, so they are not demonstrated during RSA, and are not recommended in the IBC arrangement for 5% damping

3.3.2 Factor Influencing Response Spectra

The response spectral values depends upon the following parameters,

I. Mechanism through which energy is released

II. Distance of station under consideration from epicenter

III. Depth of epicenter from ground level

IV. Properties of soil

V. Magnitude of the earthquake

VI. Damping Ratio

VII. Time for which mechanism occurs.

3.3.3 Modal Combination Rules

These are the most commonly used methods which are used to calculate the maximum response from the mode shapes from single degree of freedom system:

1. Absolute Sum (ABSSUM) Method,
2. Square root of sum of squares SRSS method, and
3. Complete quadratic combination (CQC) method

In ABSSUM method, all the modes are added algebraically to find the maximum response, supposing that all modal peaks occur at one instant of time. The formula for the maximum response is expressed by

$$r_{max} = \sum_{i=1}^{n} |r_i|$$

In the SRSS method, to obtain the maximum response, square root of sum of square of responses in every single mode of vibration is calculated and formula is given by

$$r_{max} = \sqrt{\sum_{i=1}^{n} r_i^2}$$

To combine maximum modal responses the SRSS method is supposed to be fundamentally sound as all the modal frequencies are well separated. However, this method does not give good results because frequencies of modes which contribute majorly are very closely spaced.

Another procedure is the Complete Quadratic Combination (CQC) method. To calculate the maximum response from all the modes, formula is given by

$$r_{max} = \sqrt{\sum_{i=1}^{N} \sum_{j=1}^{N} r_i \, \alpha_{ij} \, r_j}$$

where ri and rj are maximum responses in the ith and jth modes, respectively and aij is correlation coefficient

where,

$$\alpha_{ij} = \frac{8 \left(\xi_i \, \xi_j\right)^{1/2} \left(\xi_i + \beta \xi_j\right) \beta^{3/2}}{\left(1 - \beta^2\right)^2 + 4 \, \xi_i \xi_j \beta \left(1 + \beta^2\right) + 4 \left(\xi_i^2 + \xi_j^2\right) \beta^2}$$

where εi and εj are damping ratio in i^{th} and j^{th} modes of vibration, respectively and,

$$\beta = \frac{\omega_i}{\omega_j} \qquad (\omega_j > \omega_i)$$

The value of this coefficient lies between, α_{ij} is $0 < \alpha_{ij} < 1$ and $\alpha_{ii} = \alpha_{jj} = 1$.

If two modes are having same value of damping ratio i.e. $\xi_i = \xi_j = \xi$, then

$$\alpha_{ij} = \frac{8 \, \xi^2 \left(1 + \beta\right) \beta^{3/2}}{\left(1 - \beta^2\right)^2 + 4 \, \xi^2 \beta \left(1 + \beta\right)^2}$$

3.3.4 Defining Response Spectrum :-

Three response spectrums are defined for this model. Each response spectrum has different scale factor multiplied by the g value. Different scale factors are used here for different response spectrums because it is clear from previous research papers that when earthquake wave passes through the ground then there is variation in magnitude of the wave due to incoherence effect, wave passage effect, reflection and refraction through the soil media. When earthquake motion starts near to Tower 1 and when it reaches to Tower 3 magnitude of wave changes and we get different joint reactions and axial forces in the members. This change of magnitude also depends upon soil conditions and type of loadings. So earthquake motion around tower 1 is defined by Response Spectrum 1 and when it reaches to foundation of tower 3 it is defined by Response spectrum 3 in the model. Formula used for finding scale factor is

$$\gamma_{ki} = e^{-(\alpha w d/Vs)\text{square}}$$

where α is incoherence factor having value 0.5.
w is the frequency of the earthquake wave while in motion which is 12∏ radian/second
Vs is velocity of wave in the medium is distance between the towers which is 500 meter in the model. The value of γ_{ki} lies between -1 and 1 and computing here it comes out to be between 0.9 and 0.8.

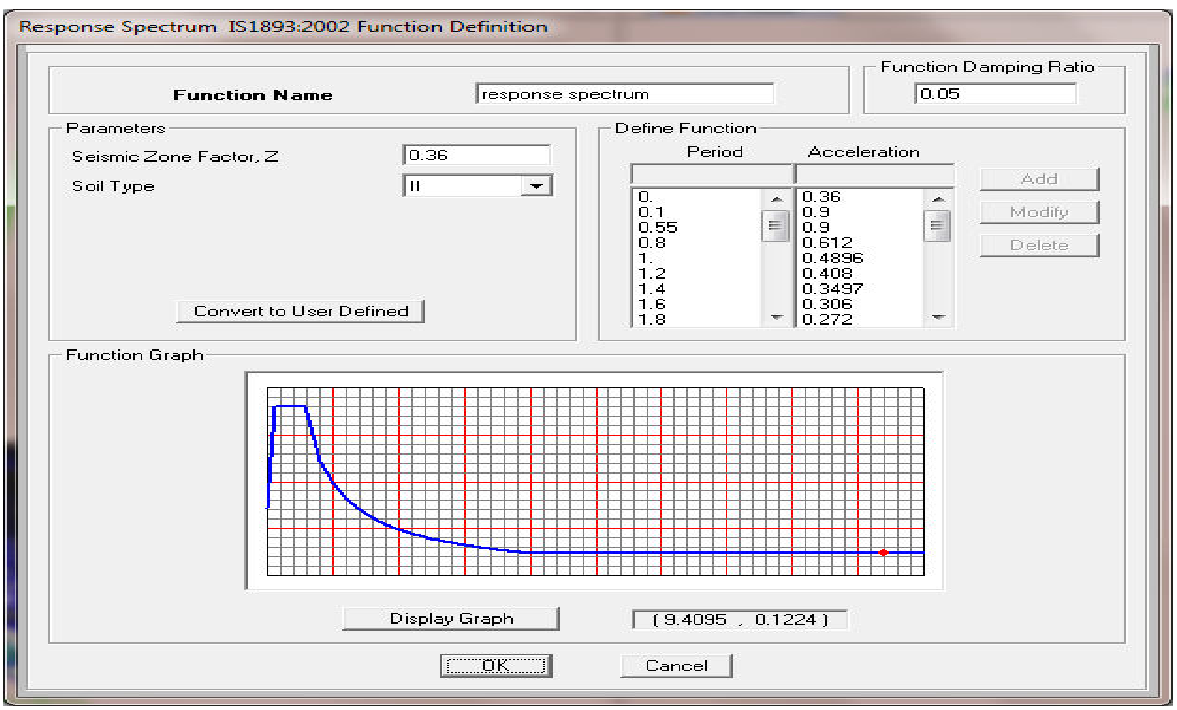

Figure3.9 function defining response spectrum is shown in this figure.

3.3.5 Response Spectrum Analysis Results : Here four legs are selected of Tower 1 and Tower 3. Then axial forces and support reactions are compared and it is seen that there is difference between joint reactions and axial forces for tower 1 and 3. For tower 1 the value of joint reactions and axial forces is slightly higher than that of the tower 3 forces.

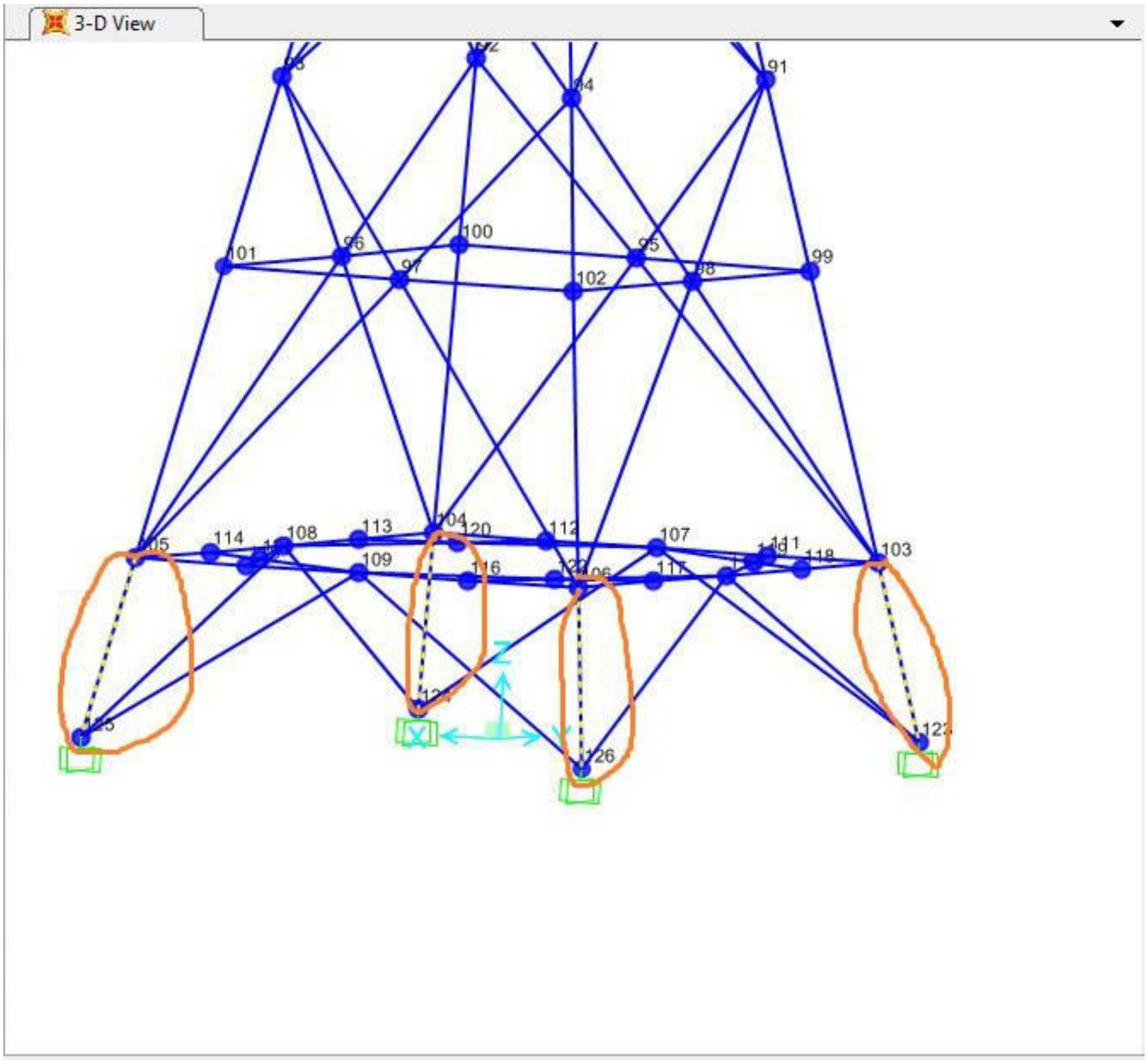

Figure3.10 four legs of tower selected for analysis of axial force is shown in this figure.

Support Reactions (Kilo Newton)

Table3.2 In this table, support reactions for TOWER 1 using Response Spectrum Method is shown. Load case defined for tower1 is named as RS_1 and we get following results:-

Support	U1 (KN)	U2 (KN)	U3 (KN)
123	23.06	26.16	128.104
124	22.59	25.87	125.77
125	22.87	26.82	127.97
126	22.76	25.77	125.92

Table3.3 In this table, support reactions for TOWER 3 are shown:-

Support	U1 (KN)	U2 (KN)	U3 (KN)
594	18.11	20.53	100.53
595	17.73	20.31	98.71
596	17.95	20.62	100.43
597	17.86	20.23	98.82

Axial Force (Kilo Newton)

Table3.4 In this table axial forces for four members which lie near the supports of TOWER 1 are shown:-

Leg Joints	Force (KN)
103 – 123	129.92
104 – 124	127.43
105 – 125	129.85
106 – 126	127.59

Table3.5 In this table axial forces for four members which lie near the supports of TOWER 3 are shown:-

Leg Joints	Force (KN)
574 - 594	102.01
575 – 595	100.03
576 – 596	101.91
577 – 597	100.14

Joint Displacements: - Here top four points are considered in tower 1 and tower 3 and displacements are noted down at these points and presented in a tabular form and comparing these values of displacement at Tower1 and Tower3 we can clearly see that there is decrease in these value which shows us that as the earthquake motion wave travels from its epicenter to other points its magnitude decreases.

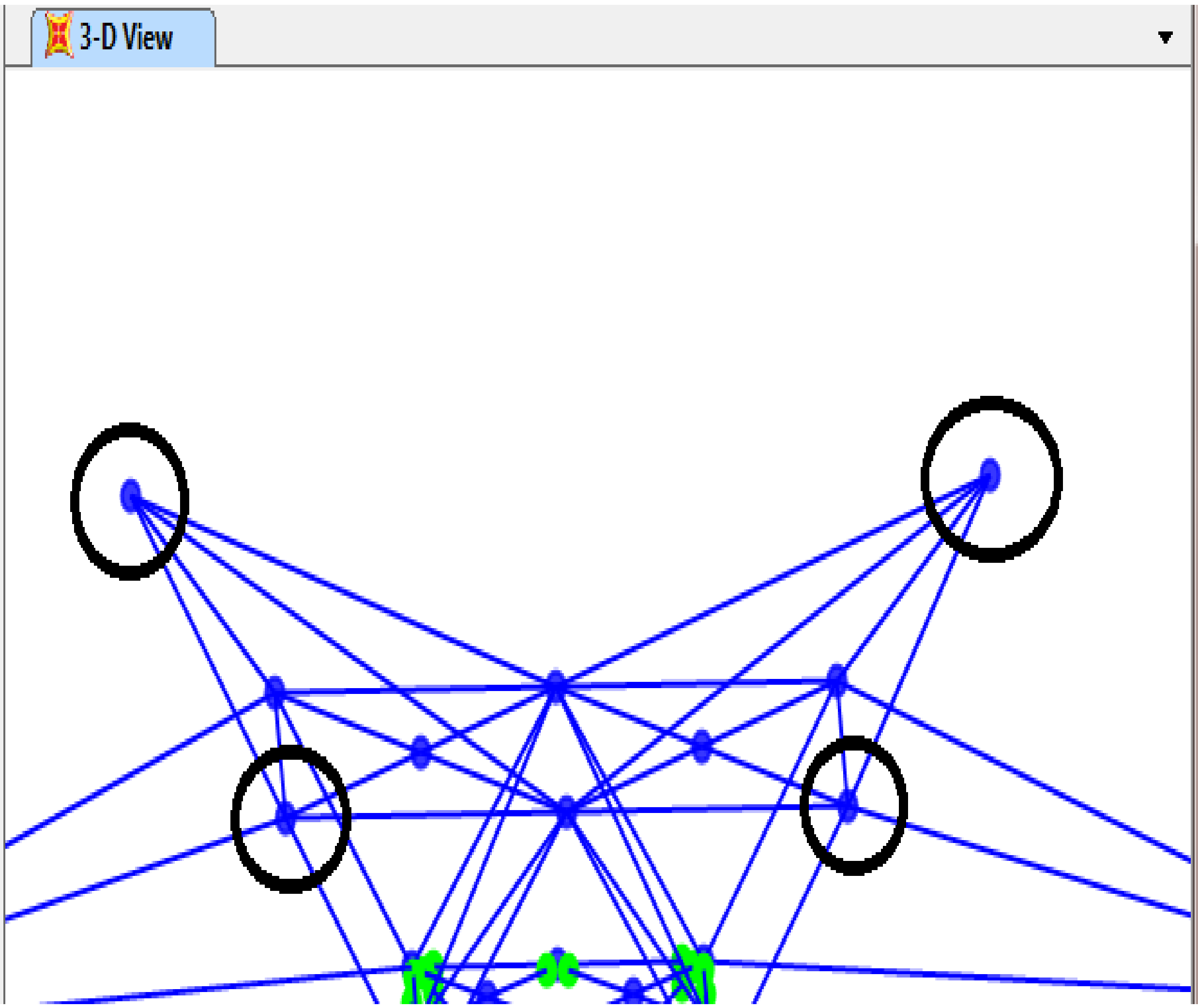

Figure3.11 In this figure four points at tower are shown which are selected for checking the displacement.

Displacements (Meter)

Table3.6 In this table, displacements at the joints which lie at top position of TOWER 1 are shown:-

Joint	**X direction**	**Y Direction**	**Zdirection**
1	0.00273	0.07913	0.000532
2	0.00273	0.07240	0.00057
15	0.00218	0.06815	0.00365
16	0.00254	0.06783	0.00337

Table3.7 In this table, displacements at the joints which lie at top position TOWER 3 are shown:-

Joint	**X direction**	**Y Direction**	**Z direction**
472	0.00215	0.05645	0.000417
473	0.00214	0.05682	0.000491
486	0.00171	0.05348	0.00286
487	0.00199	0.05323	0.00265

3.4 TIME HISTORY ANALYSIS

The time-history analysis is the most reliable method to predict the force and displacements at different sections of the structure. This method is still not very much popular because the dynamic behavior is very sensitive to the modeling and spatially varying earthquake ground motion characteristics. The structure should be properly modeled and cyclic load-deformation characteristics should be properly defined, and properties of all the important components should be considered carefully. The time required in computation, input preparation, and interpreting the output, makes the use of this analysis difficult for seismic performance evaluation. In the present study, SAP2000 was used in performing the nonlinear dynamic time-history analysis on the three-dimensional model of the bridge.

Defining Time History Function:- While defining time history function, a file defining El Centro earthquake array 6 with 3909 points of acceleration data equally spaced at 0.01 second was chosen. The file name for chosen file was given as Array_1. Header lines to skip in this file were 2 and numbers of points were 8. Now while assigning load cases three types of loads were assigned for three towers respectively having different scale factors so that effects due to wave passage, incoherence and phase lag can be taken into consideration. Number of output time steps taken were 4000 and size of each time step output was 0.01 second. Load cases name given were TH_Array01, TH_Array02, TH_Array03 for tower1, tower2 and tower3 respectively.

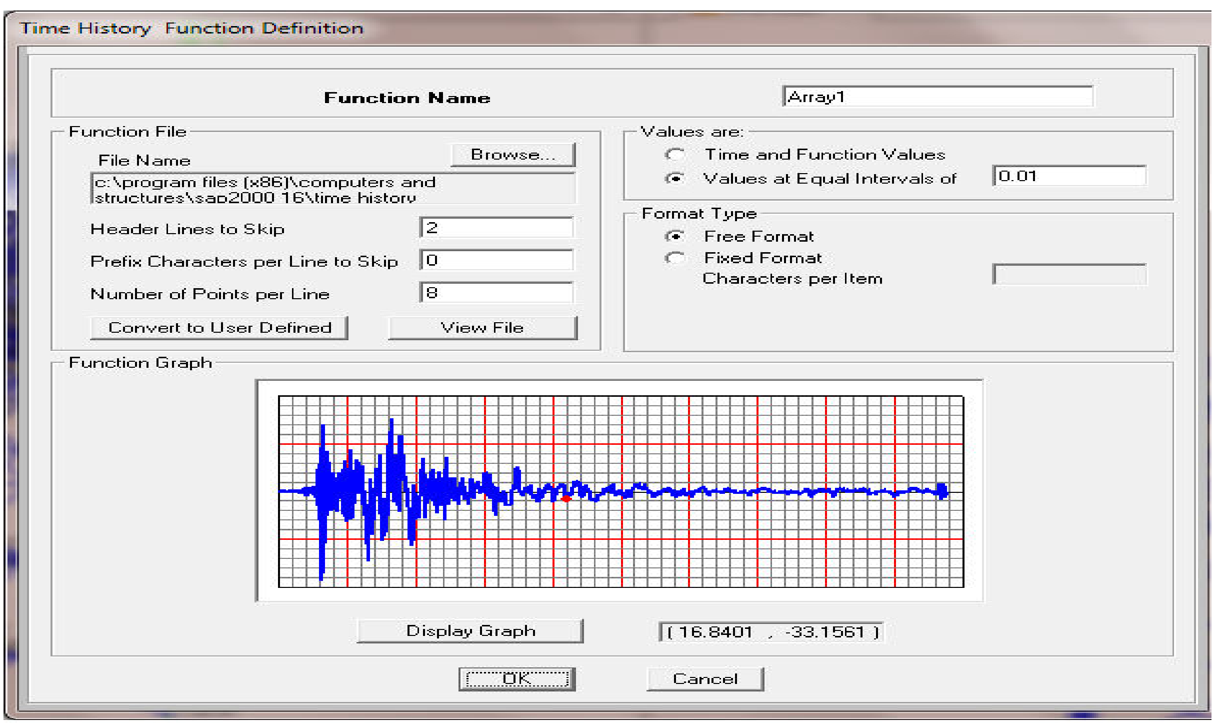

Figure3.12 Defining of time history function is shown above picture.

Time History Analysis Results :- Firstly joint reactions of tower 1 and tower 3 are compared for time history analysis functions which are defined separately for the first and third tower respectively. There is decrease in joint reaction forces as we are moving from first tower to the third tower.

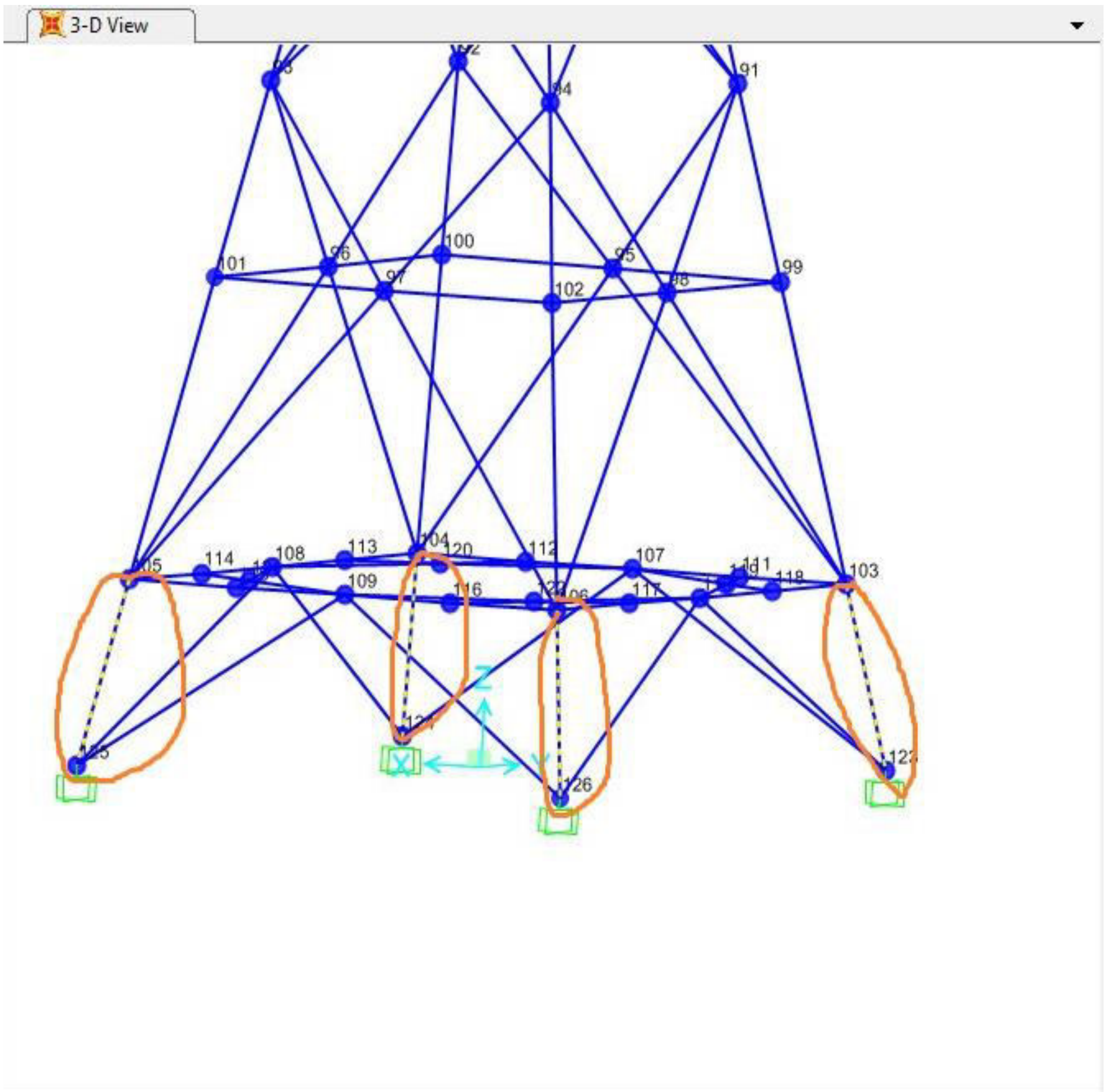

Figure3.13 in this figure four marked legs of tower for which axial force is to be checked in tower1 and tower3 using Time History Analysis, support reaction is also checked for the supports shown in figure.

Table3.8 Support reactions (in Kilo-Newton) for Tower1 using Time History Analysis are shown:-

Support	U1 (KN)	U2 (KN)	U3 (KN)
123	-15.782	20.364	-87.378
124	15.406	20.152	-85.663
125	-15.638	20.460	87.258
126	15.548	20.054	85.759

Table3.9 Support reactions for Tower3 using Time History Analysis are shown:-

Support	U1 (KN)	U2 (KN)	U3 (KN)
594	-12.626	16.291	-69.902
595	12.335	16.121	-68.530
596	-12.510	16.368	69.807
597	12.438	16.044	68.607

Axial Force (Kilo-Newton):-

Table3.10 In this table axial forces of four legs which are shown in figure 3.11 for Tower 1 are shown:-

Leg Joints	**Force (KN)**
103 – 123	87.103
104 – 124	85.236
105 – 125	-85.992
106 – 126	-85.324

Table3.11 In this table axial forces of four legs which are shown in figure 3.11 for Tower 3 are shown:-

Leg Joints	**Force (KN)**
574 - 594	68.692
575 – 595	68.189
576 – 596	-69.590
577 – 597	-68.259

Joint Displacements :- Here top four points are considered in tower 1 and tower 3 and displacements are noted down.

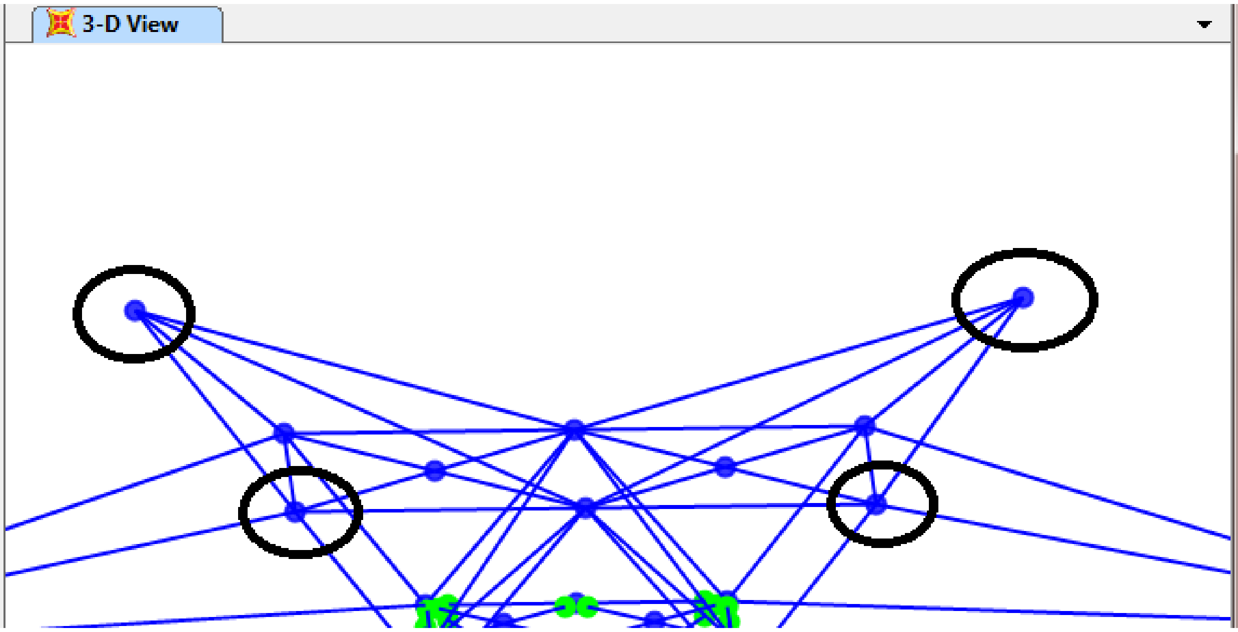

Figure3.14 In this figure four points are selected which are rounded with red circles which are seleced for checking displacements

Table3.12 Displacements (Meter) of above selected points for Tower 1 are shown:-

Joint	X direction	Y Direction	Z direction
1	-0.00122	-0.04565	-0.000249
2	-0.00122	-0.04598	0.000245
15	-0.00104	-0.04335	-0.00224
16	-0.00110	-0.04313	-0.00206

Table3.13 Displacements (Meter) of similar points for Tower 3 are shown:-

Joint	X direction	Y Direction	Z direction
472	-0.000979	-0.03652	-0.000199
473	-0.000979	-0.03678	0.000196
486	-0.000834	-0.03468	-0.00179
487	-0.000882	-0.03450	-0.00165

Joint Displacements Using plot function: Here top 4 four joints of Tower 1 and 3 are considered and their displacements are plotted with respect to time using time history function. For tower 1, function defined is named as TH_array1 and for tower 3, it is named as TH_array3.
.

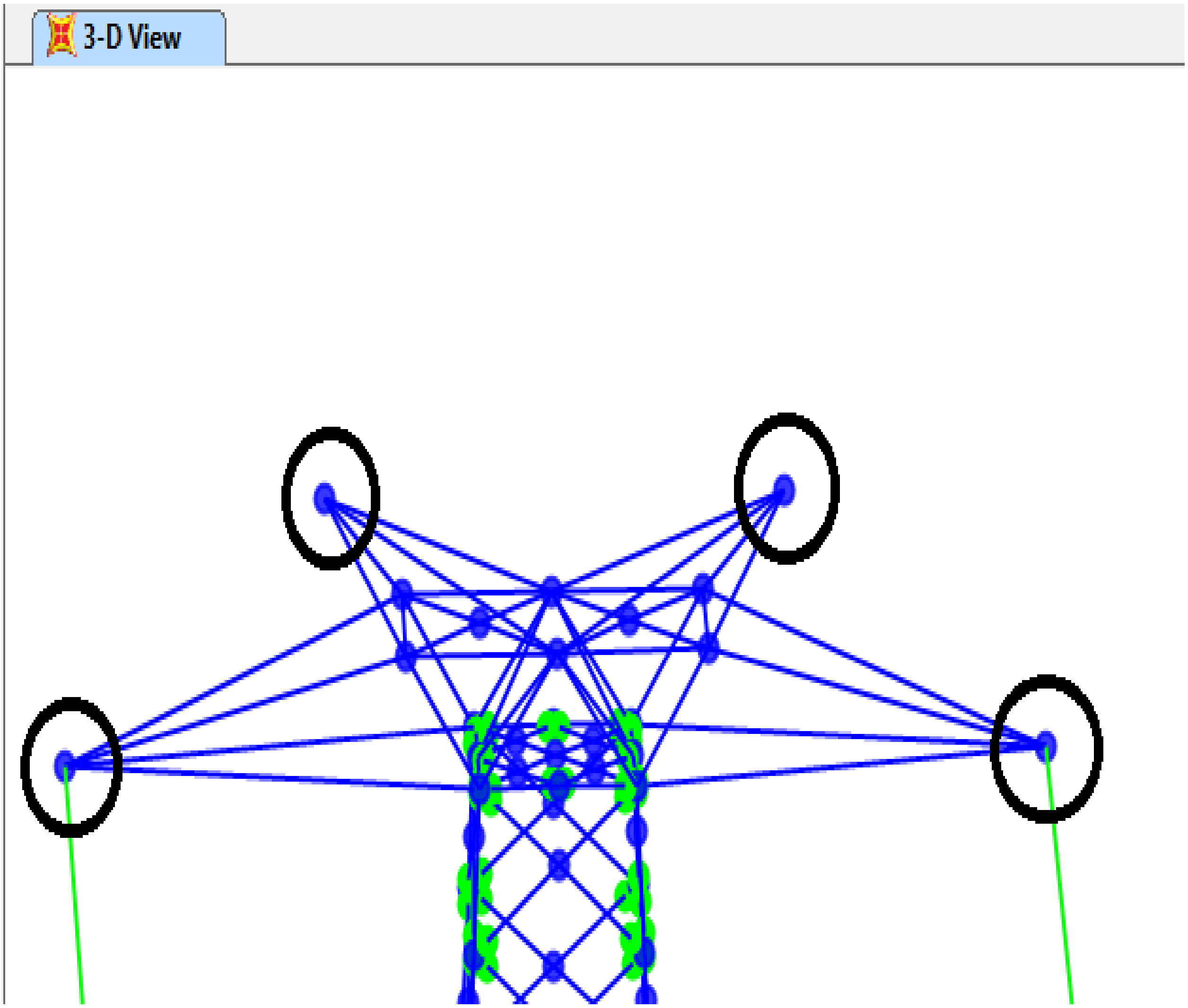

Figure3.15 In this figure four points which are rounded with black circles are selected which are checked for displacements using Time History plot functions.

From the above analysis we have got the following results:-

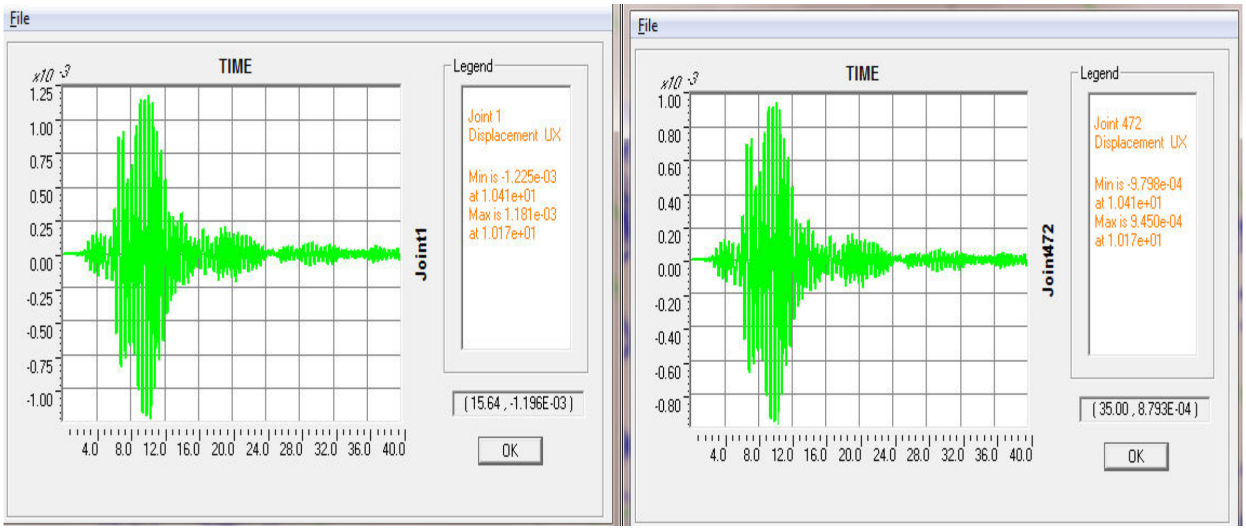

Figure3.16 Here maximum displacement for joint no.1 of Tower1 which is $1.181*10^{-3}$ meter and for joint no.472 of Tower3 is $9.45*10^{-4}$ meter is shown in graphical form w.r.t time.

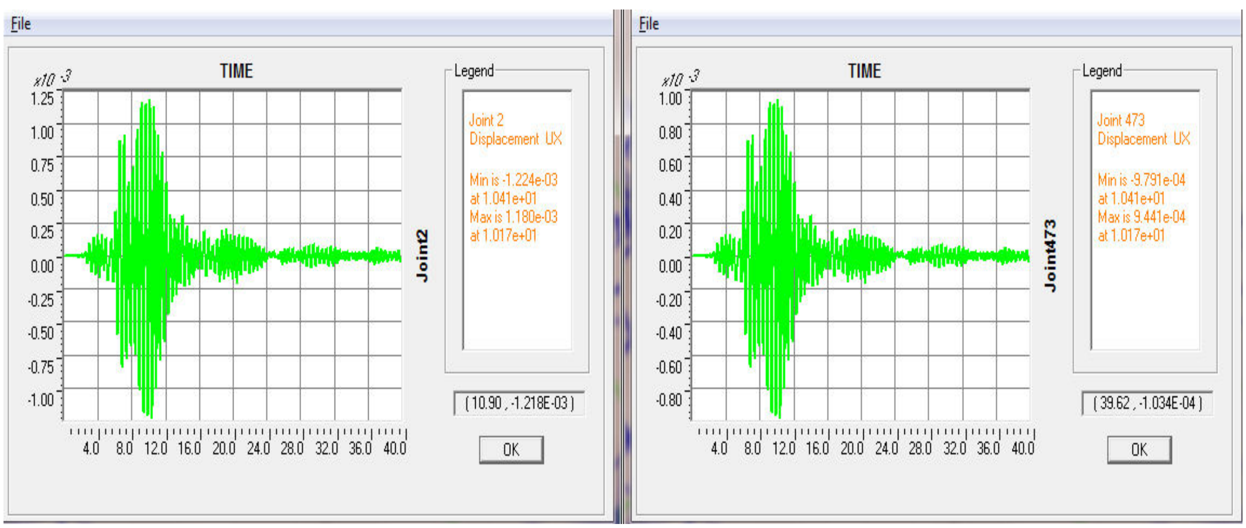

Figure3.17 Here maximum displacement for joint no.2 of Tower1 which is $1.80*10^{-3}$ meter and for joint no.473 of Tower3 is $9.441*10^{-4}$ meter is shown in graphical form w.r.t time.

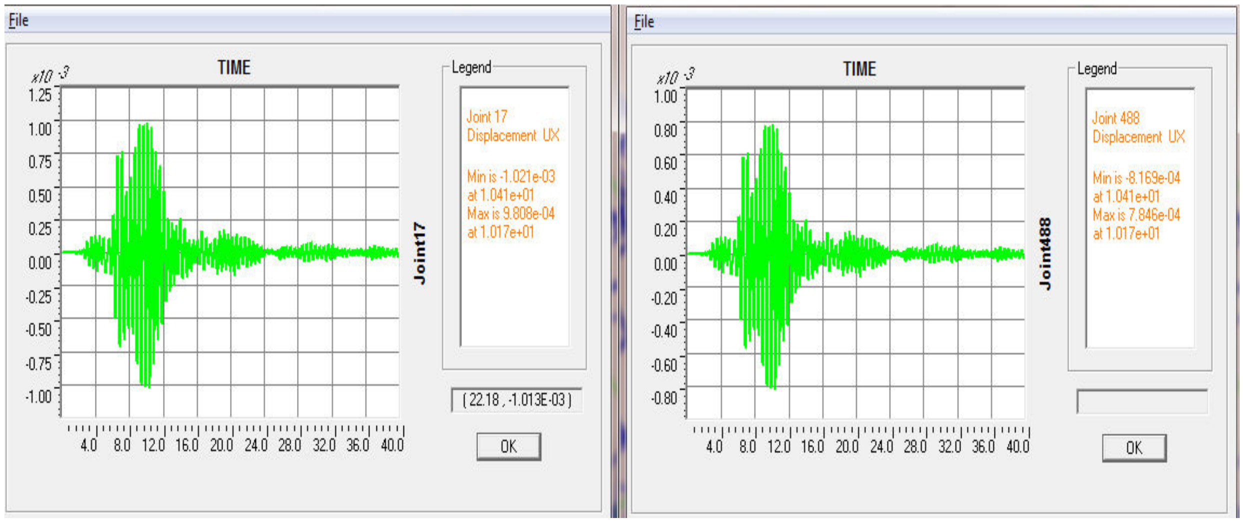

Figure3.18 Here maximum displacement for joint no.17 of Tower1 which is $9.808*10^{-4}$ meter and for joint no.488 of Tower3 is $7.846*10^{-4}$ meter is shown in graphical form w.r.t time.

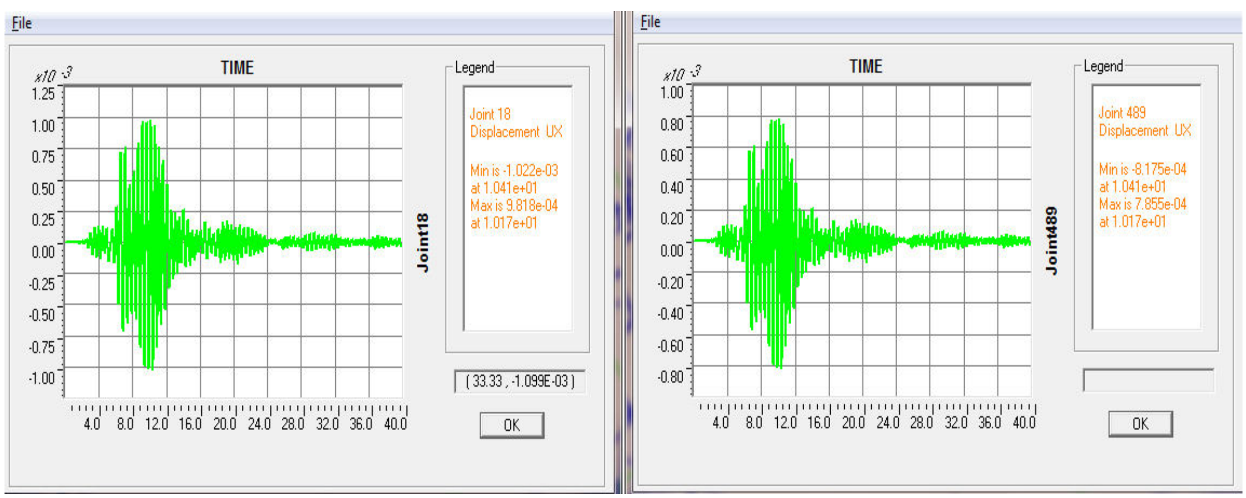

Figure3.19 Here maximum displacement for joint no.18 of Tower1 which is $9.818*10^{-4}$ meter and for joint no.489 of Tower3 is $7.855*10^{-4}$ meter is shown in graphical form w.r.t time.

CHAPTER 4

WORK PLAN

Month	Activity
July – October	Literature Review
October – December	Modeling of transmission line-tower system
January - March	Response spectrum Analysis (RSA)
April	Time history Analysis (THA)
May	Final Submission

CHAPTER 5

CONCLUSION: - It can be concluded that there is a slight change in the earthquake ground wave magnitude as it travels through soil media and to show that we performed the above analysis in which we have seen there is a difference between the base reactions, axial forces and joint displacements of Tower1 and Tower3.

1. First of all modal analysis was completed to see the natural mode shapes of the structure.

2. Second step was to perform Response Spectrum which took the contribution from each natural mode and gave us the maximum response in terms of base reaction or axial forces. It is clear from the previous papers that seismic waves undergo a change while travelling through a medium and it depends upon three factors:- wave passage effect, time lag and coherency effect.

3. The base reactions, axial forces and joint displacements are found out using Time History method which was our third step and results from both the methods are presented in tabular form in this study.

4. Joint displacements of top four points in Tower1 and Tower3 are also shown in the graphical form using Time History method which shows us that at what time instant what was the value of displacement and at what instant of time the joint displacement is maximum.

5. Each value has decreased as we go from tower1 to tower3 which makes it clear that when the earthquake wave starts travelling from one point to another its magnitude keep decreasing which is shown in the result values.

Future Scope:- There is a lot of scope in future for this study. The present study was done on transmission tower system which can be used for the better design of the system so that these towers can withstand all the forces at the time of earthquake and even after the earthquake and can perform well so that we don't need to face electric failures after the earthquake. This can be really helpful for the mankind. Same analysis can be done on the various structures which are very important for human beings like bridges, dams or nuclear power plants. This study can help us in building earthquake safe or earthquake resistant structures.

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