

Seismic Performance Evaluation of Steel Transmission Towers using Pushover Analysis

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Abstract- Nowadays electricity is the basic need of human life, almost every person uses electricity, especially in urban areas, bringing up electricity from its source is a big task, for civil and electrical engineers, therefore constructions of electrical transmission towers comes into pictures, in other hand science and technology has advanced rapidly and brought up telecommunications in our life, therefore construction of telecommunication towers comes into pictures, for having communication from one place to other place in the form of mobiles, internet, radios etc. Therefore keeping in view the importance of transmission and telecommunication towers, their structural analysis and design plays a vital role especially when they are situated in seismic prone zone. Present study mainly focuses on nonlinear seismic response of electrical and telecommunication towers, made up of equal angle sections (ISA), hallow pipe sections, and square hallow box sections. 3D pin-jointed space mathematical FE (finite element) model have been generated in SAP2000, by considering each element as frame element having 6 DOF @ each nodal point. Basically there are 3-different configurations of steel towers, and every configuration is made up of angles, pipe and box section separately, Hence totally 9-different models are of the main interest. To evaluate the nonlinear seismic response, pushover analysis approach has been adopted; plastic hinges have been assigned to every frame element at each nodal point, based on FEMA 356 and ATC-40. Following parameters have been considered for evaluating the response such as Mode shapes and their time period, Hinge status, ADRS (Acceleration demand response spectrum) curves for evaluating performance points.

1. INTRODUCTION

The high-voltage transmission system consisting of transmission lines and their supporting towers is an important lifeline engineering structure. A transmission tower-line framework during its administration life, particularly those situated in a high seismicity area, May expose to seismic stacking. A transmission tower-line framework during its administration life.

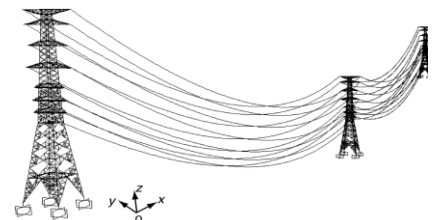


Fig. 1: Example of transmission tower.

2. OBJECTIVES OF THE STUDY

- To study the behaviors different types of transmission towers with varied structural configuration.
- To record the inelastic response of each different structural configuration by having different structural steel cross section like **Equal Angle, Hall Box or Square Tube, Hall Circular Tube or Pipe**.
- To developed the acceleration demand response spectrum (ADRS) of each different tower model To work the performance level of towers by establishing performance points.

3. DESCRIPTION OF THE STUDY

Model 1 (T1-ANGLE) tower consists of frame element having pinned end condition, structural elements are of equal angle cross section. Tower configuration 1

Model 2 (T1-BOX)–Same as model 1 having structural elements are of hallow square box/tube cross section. Tower configuration 1

Model 3 (T1-PIPE)– Same as model 1 having structural elements are of hallow circular /pipe section. Tower configuration 1

Model 4 (T2-ANGLE)– Same as model 1 having structural elements are of equal angle cross section. Tower configuration 2

Model 5 (T2-BOX)– Same as model 1 having structural elements are of hallow square box/tube cross section. Tower configuration 2

Model 6 (T2-PIPE)– Same as model 1 having structural elements are of hallow circular /pipe section. Tower configuration 2

Model 7 (T3-ANGLE)– Same as model 1 having structural elements are of angle cross section. Tower configuration 3
Model 8 (T3-BOX)– Same as model 1 having structural elements are of hallow square box/tube cross section. Tower configuration 3
Model 9 (T3-PIPE)– Same as model 1 having structural elements are of hallow circular /pipe section. Tower configuration 3

4. METHODS OF ANALYSIS

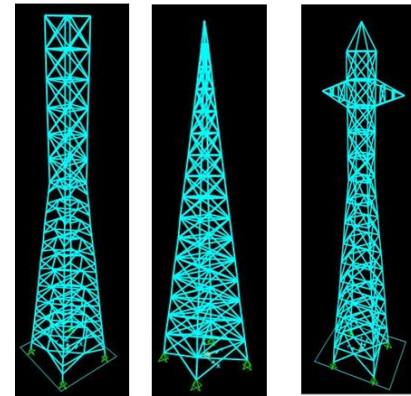
In this paper all the results are extracted from SAP2000, the parameters for non-linear response of structure are as follows.

1.Modal analysis, 2.Non-linear hinges, 3.Acceleration demand response spectrum (ADRS) and performance point.

5. DESIGN DATA:

Following are the design data consider for the analysis and design of the transmission tower. Design of all the Structural elements have been done using **IS-800-2007**

1. Frame type: pin jointed 3D space frame
2. Height : **48m**
3. Follows the parametric definition for variation of tower long the height.
4. Fe250 is has been used the primary material for design.
5. Boundary condition at base assumed as pin jointed.
6. Angle section, **40x40x5, 6. 50x50x5, 6. 60x60x5,6**
7. Hallow box, **50x50x5, 6. 60x60x5,6**
8. Hallow circular pipe, **50x50x5, 6. 60x60x5,6**
9. The tower is considered in very high seismic prone zone.



T1 T2 T3

Towers configurations

6. RESULTS AND DISCUSSIONS

The results of Modal analysis, Non-linear hinges, Acceleration demand response spectrum (ADRS) and overall performance for the different tower models are presented and compared.

MODAL ANALYSIS

This parameter is extremely essential to evaluate the dynamic or free vibration responses of any type of structural systems, there to understand the modal nature of the above mentioned towers, 12 modes of vibrations have been considered, and for each mode modal mass participation has been evaluated to understand the nature of mode of vibrations of every single mode. And an attempt is made to understand the natural and circular frequencies of the structures with Eigen values for each single mode. due to space restriction only 3 fundamental modes of vibrations are only added here

Model 1							
Mode No	T, Period in Sec	freq cycl/sec	Cir freq rad/sec	Eigen value rad2/sec2	Modal Mass participation		
					UX	UY	RZ
1	1.481	0.675	4.241	17.987	0.101	0.121	73.091
2	1.423	0.703	4.417	19.509	40.465	47.074	0.001
3	1.413	0.708	4.445	19.762	47.402	40.730	0.175
Model 2							
1	0.875	1.142	7.177	51.509	87.457	0.013	0.000
2	0.875	1.142	7.177	51.509	0.013	87.457	0.000
3	0.460	2.173	13.652	186.382	2.531	0.000	0.000
Model 3							
1	0.961	1.041	6.540	42.768	87.901	0.222	0.000
2	0.961	1.041	6.540	42.768	0.222	87.901	0.000
3	0.508	1.969	12.369	152.981	0.334	0.975	0.000
Model 4							
1	0.450	2.222	13.962	194.928	36.420	9.929	0.000
2	0.450	2.222	13.962	194.944	9.929	36.421	0.000
3	0.135	7.392	46.448	2157.419	19.378	4.086	0.000
Model 5							
1	0.465	2.150	13.506	182.411	15.957	32.378	0.000
2	0.465	2.150	13.506	182.411	32.378	15.957	0.000
3	0.135	7.407	46.540	2165.954	8.612	14.087	0.000
Model 6							
1	0.444	2.253	14.157	200.434	0.135	46.467	0.000
2	0.444	2.253	14.157	200.434	46.467	0.135	0.000
3	0.132	7.568	47.550	2260.962	0.039	23.222	0.000
Model 7							
1	0.757	1.321	8.299	68.879	0.000	0.000	0.000
2	0.728	1.373	8.626	74.399	0.004	0.004	0.000
3	0.613	1.631	10.251	105.080	0.003	0.003	0.000

Model 8							
1	0.525	1.906	11.975	143.394	49.761	0.000	0.014
2	0.517	1.935	12.160	147.872	0.000	50.002	0.000
3	0.238	4.204	26.415	697.754	0.000	0.000	0.000
Model 9							
1	0.538	1.859	11.681	136.444	49.592	0.000	0.011
2	0.530	1.887	11.858	140.606	0.000	49.771	0.000
3	0.272	3.681	23.126	534.805	0.000	0.000	0.000

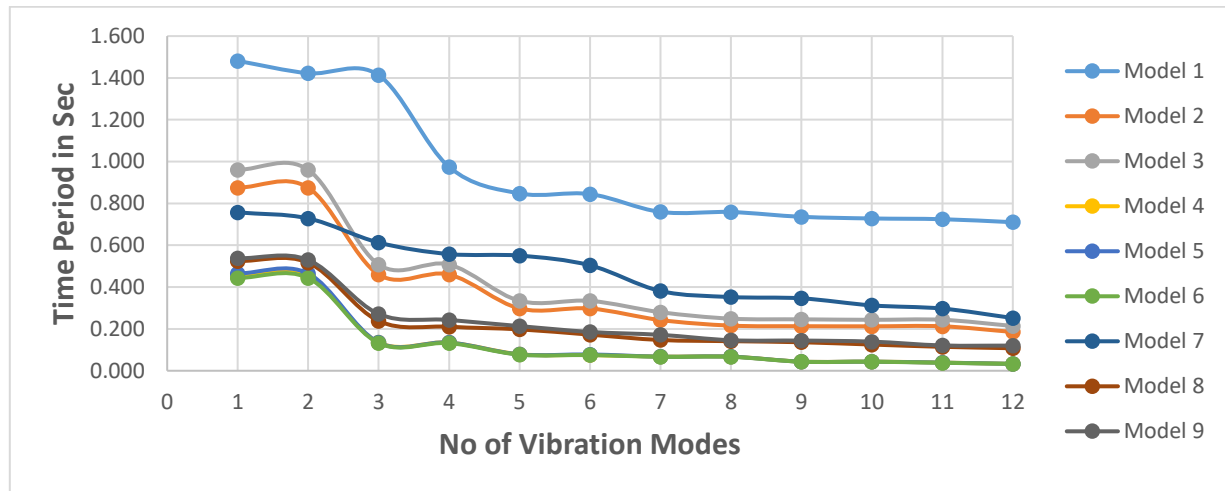


Chart: Shows the Fundamental natural time periods of different tower models

Model 1: this particular model records the highest amount of fundamental natural time periods among all other models and first mode is in maximum torsion, which is not at all a good structural combination for seismic responses. And a consistent increase in Eigen value after 3rd fundamental mode and frequency of different modes are considerably less when compare with other models. May angle section be the reason for not having proper flow of inertial forces, because of not having same moment inertia in both the directions.

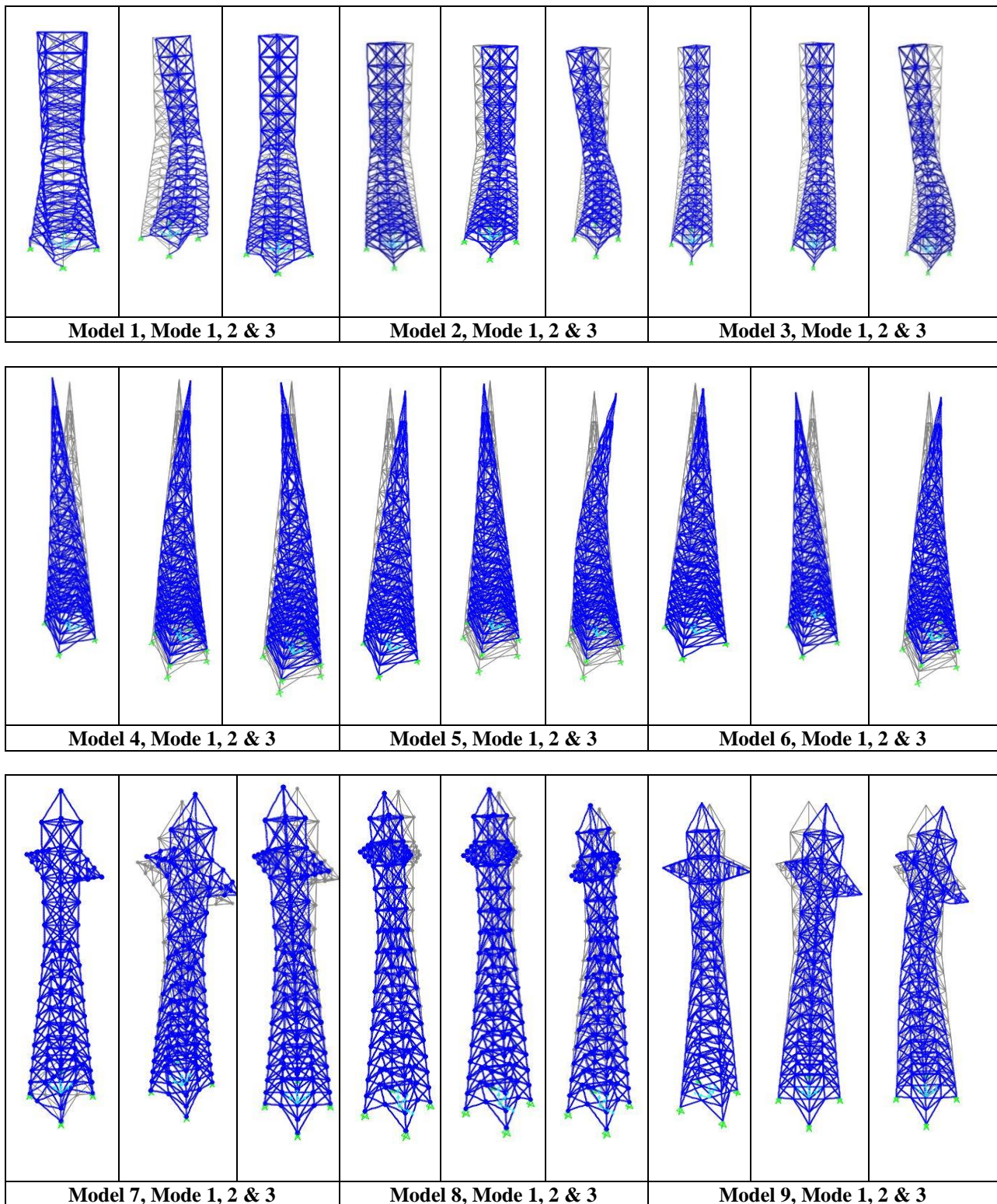
Model 2: fundamental natural time periods of first 3 modes are considerably less than the model 1, and have got huge mass participation along x direction and y direction in mode 1, and mode 2, and torsion is appearing in mode 8, and the Eigen values are considerably increasing in nature after every mode. Circular frequency drastically increasing after first 3 modes of vibrations. Therefore this configuration seems to good enough against seismic threatening.

Model 3: fundamental natural time periods of first 2 modes are quite same, and have got huge mass participation along x direction and y direction in mode 1, and mode 2, and torsion is appearing in mode 8, and the Eigen values are considerably increasing in nature after every mode. Circular frequency drastically increasing after first 3 modes of vibrations. Therefore this configuration seems to good enough against seismic threatening. Model 2 and model 3 are seems to be analogous to each other.

Model 4, Model 5 and model 6: these models have got the least fundamental natural time period of the structure, when compare with model 1, time period of model 4, 5 and 6 have decreased by, 69.6%, 68.6%, 70.0%, there it can be state that, this configuration of model 4, 5, and 6 has got the better dynamic response among all the other towers. These models have got huge amount of mass participation in the first fundamental modes in either x or y and with combination of x and y direction and torsion mode appears to be in 5th and 6th mode. Circular frequencies appeared to be drastically increased in nature after 3rd mode of vibrations. These models have got highest positive Eigen values among all the other different towers. Therefore, rightly can states that, these models have got the better dynamic response among all models, hence these may feasible for seismic prone zone provide proper seismic design have been done.

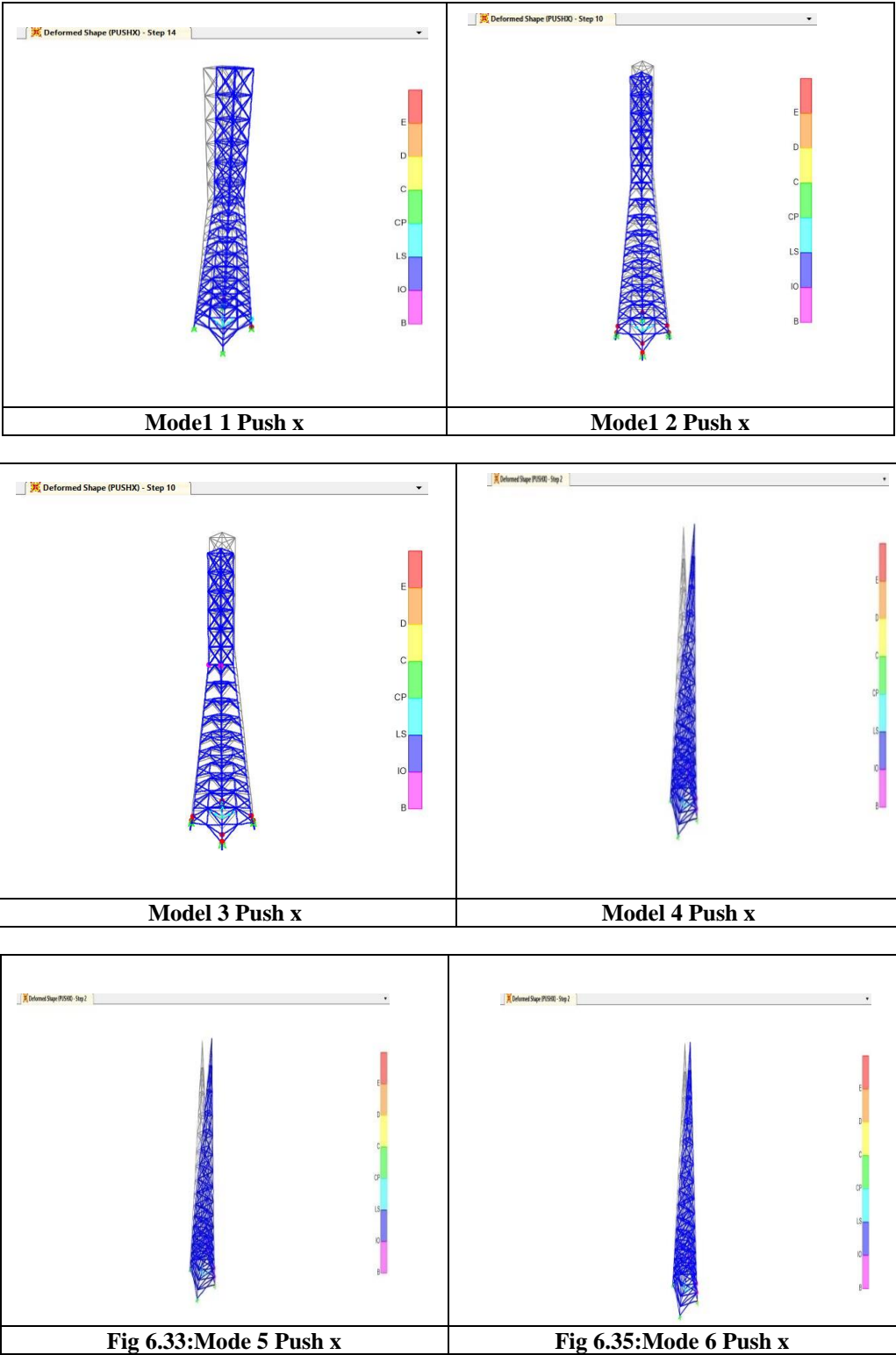
Model 7: initial three modes are not at all participating in seismic modal mass, the tower is participating in mode 4 and 5 in x and y translation. Fundamental natural time period of the structure considerably less than the model 1, 2, and 3 but the overall seismic mass participation is not in a good agreement, hence this model is showing least dynamic response among all the other different towers. Need to consider even higher modes. Study shows, model should go for Ritz modes instead of Eigen modes.

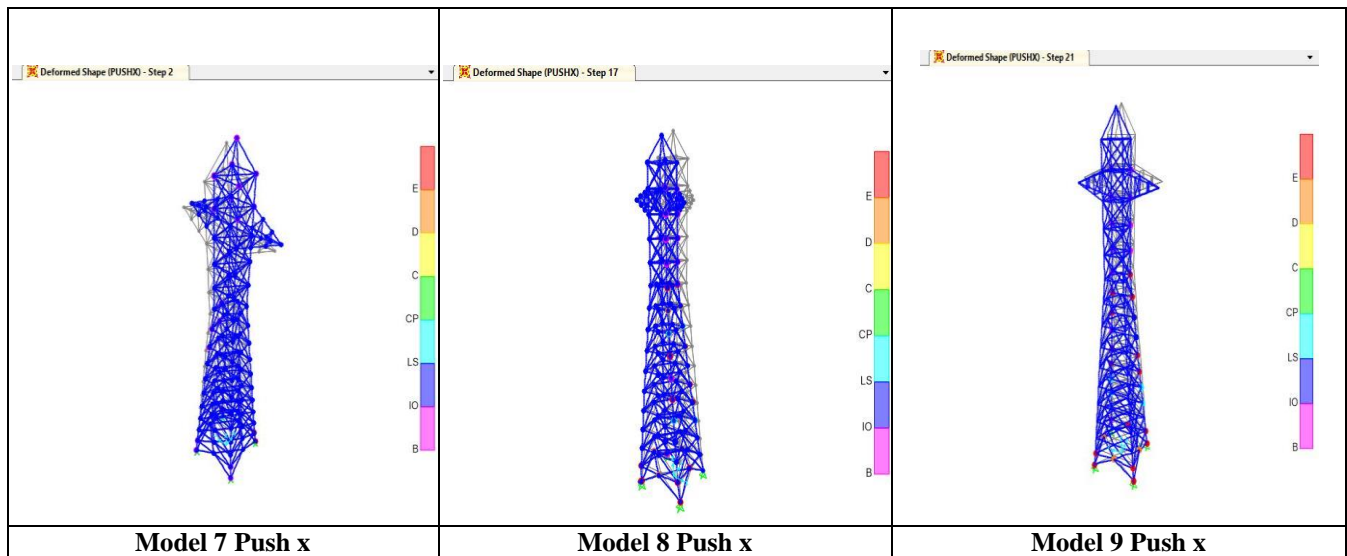
Model 8, Model 9: fundamental natural time periods of first 2 modes are nearly same, and have got huge mass participation along x direction and y direction in mode 1, and mode 2, and there is no seismic mass participation after 3rd mode, in x and y direction but mass participation start once in 8th mode and in 9th mode respectively.



Non-linear Hinges

This is one of the important parameter, for recognizing the non-linear damages in the structure after any earthquake, in this particular study P-hinge or the hinges for only axial load have been considered, because all the structural members in pin-jointed space frames are either in tension length of 0.2m. Pushover analysis have been done SAP2000, in both the longitudinal and transverse direction. An automated acceleration load pattern in SAP2000 has been considered, pushover or non-linear static analysis is iterative procedure (SAP2000 has used Newton Rapson method). The no of hinges generation with their acceptance criteria for each different tower model is given as fallows.or compression. The axial hinges are assigned at the ends of every structural element for a hinge





Model 1: Push over analysis has been performed along both orthogonal direction i.e. x and y directions. This model is performing up to the nonlinear range of life safety (LS) to collapse prevention (CP), the plastic deformation in the nonlinear hinges are going beyond the limit specified by FEMA356 and ATC40 and max no od iteration have been done. Results for only 7H2 hinge only provided for the both pushover cases (PUSHX and PUSHY).

Model 2: Results are shown for Hinge4H1 for both the cases (PUSHX and PUSHY).13 iterations have been done in x direction and 7 iteration have been done in y direction by SAP2000, the hinges are going up to a range of E or more than E range, in initial steps the nonlinear range of plastic hinges are between immediate occupancy (IO) to collapse prevention (CP). Therefore here it can be state that, the cross section or the structural elements are not performing well or up to the immediate occupancy (IO) level.

Model 3: Results are shown for Hinge9H2 for both the cases (PUSHX and PUSHY).10 iterations have been done in x direction and y direction by SAP2000, the hinges are going up to a range of IO (immediate occupancy) level in both cases not a single hinge is going beyond IO levels, All the hinges are in this particular range only. Therefore this model shows great performance towards seismic forces.

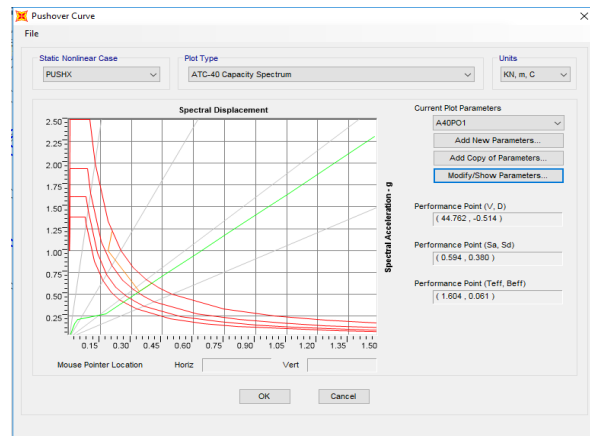
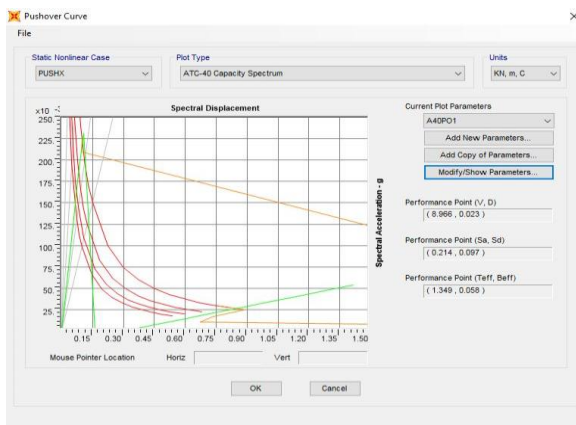
Here it's clear that, type of cross section for the structural elements plays a vital rule for the nonlinear performance as from the above results.

Model 4, Model 5, Model 6: Results are shown for Hinge9H2 for **Model 4**, Hinge100H1 for **model 5** and **model 6** for both the cases (PUSHX and PUSHY).only 2 iterations have been done in x direction and y direction by SAP2000, the hinges are going up to a range of IO (immediate occupancy) level in both cases not a single hinge is going beyond IO levels, All the hinges are in this particular range only. SAP2000 unable to find the solutions for more steps, therefore these model shows great performance towards seismic forces. From the above results it's clear that not shape of the structural cross play a vital rule but overall geometry also play very important rule for nonlinear performance of the structures.

Model 7: this model shows the same performance as of model4, 5 and 6. All the nonlinear hinges are forming at the base level of the tower.

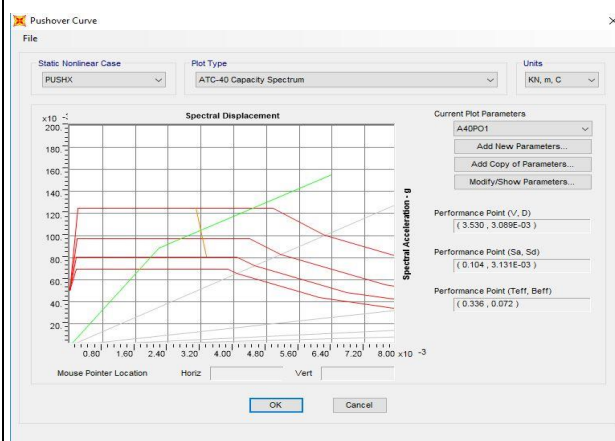
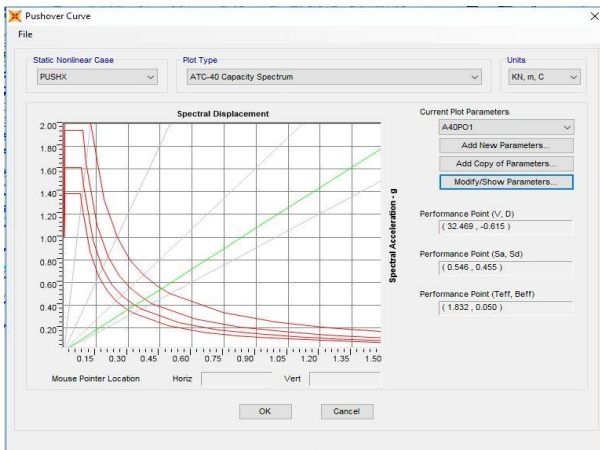
Model 8, Model 9: Results are shown for Hinge293H1 for **Model 8**, and **model 9** for both the cases (PUSHX and PUSHY). SAP2000 has done several iteration along both the direction and recorded many steps, the hinges are going up to a range of IO (immediate occupancy) level in initial steps but for all cases the hinges are reaching their max plastic rotations, reaching beyond CP(collapse prevention), even range is going beyond E. therefore this combination and configurations are not showing much nonlinear performance against seismic loading in both the directions.

Acceleration Demand Response Spectrum (ADRS) And Performance Point:-



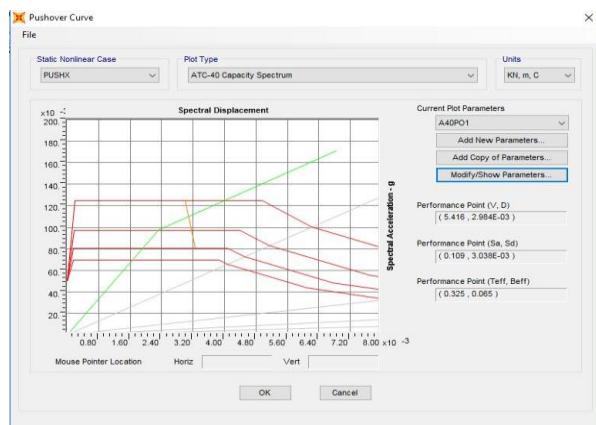
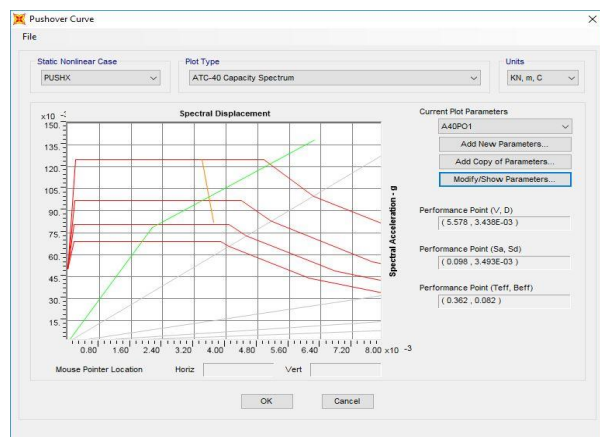
ADRS in longitudinal direction for Model 1

ADRS in longitudinal direction for Model 2



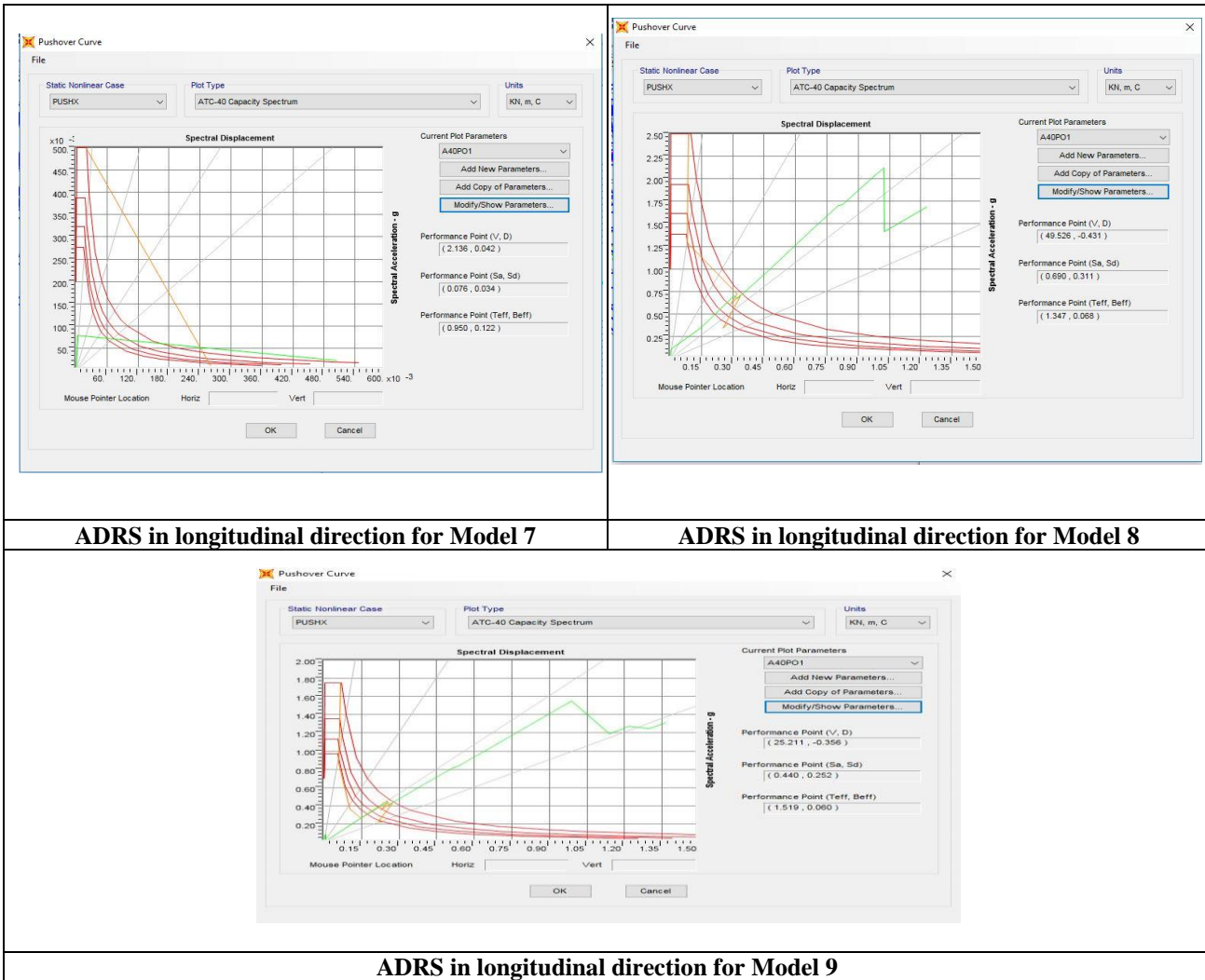
ADRS in longitudinal direction for Model 3

ADRS in longitudinal direction for Model 4



ADRS in longitudinal direction for Model 5

ADRS in longitudinal direction for Model 6



Model 1: the performance point hasn't been successfully formed for this particular model, the configuration and type of cross section used in making of transmission towers plays very important rule when they are subjected to seismic threatening.

Model 2: the performance point successfully formed for both the load cases, the base shear and the displacement at performance point are $V = 46.76 \text{ KN}$, $D = -0.514 \text{ M}$ FOR PUSHX AND $V = 94.19 \text{ KN}$, $D = 0.00006 \text{ M}$ FOR PUSHY. This type of tower will perform up to the performance point, where capacity and demand lines bisect each other.

Model 3: the performance point successfully formed for both the load cases, the base shear and the displacement at performance point are $V = 32.469 \text{ KN}$, $D = -0.615 \text{ M}$ FOR PUSHX AND $V = 25.976 \text{ KN}$, $D = -0.492 \text{ M}$ FOR PUSHY. The capacity curve seems to be linear, not showing any type of nonlinearity, but the overall performance of this tower is good enough as far as nonlinear performance is concern, this type of tower will perform up to the performance point.

Model 4: the performance point successfully formed for both the load cases, the base shear and the displacement at performance point are $V = 3.53 \text{ KN}$, $D = 0.0036 \text{ M}$ FOR PUSHX AND $V = 3.53 \text{ KN}$, $D = -0.0036 \text{ M}$ FOR PUSHY. The capacity curve seems to be nonlinear, but the overall performance of this tower is good enough as far as nonlinear performance is concern, but the performance point is at very low seismic base shear force, there it very clear that this model will not survive and will not be able to show the ductile behavior.

Model 5, Model 6: these models are analogous to model 4, but performance points are achieved at little higher load than model 4, as far as strength and nonlinearity is concern it's much better than model 4, because the capacity spectrum is showing much more nonlinear behavior than model 4.

Model 7: this particular model is showing very undesirable ADRS curves, where performance points, capacity spectrum and demand spectrum are not properly formed, capacity and demand curves are bisecting at very undesirable range.

Model 8: this model is performing better in longitudinal direction and there is undesirable behavior is seen in transverse directions, the performance point is achieved successfully for PUSHX case, the base shear and displacement are $V = 49.52 \text{ KN}$, $D = -0.431 \text{ M}$. This model for pushx case showing huge amount of ductility and lots of strain hardening and huge nonlinearity is observed for capacity spectrum.

Model 9: this model is performing well for both load cases, the performance point is achieved successfully for PUSHX and PUSH Y cases, and base shear and displacement are $V = 25.211\text{KN}$, $D = -0.356\text{M}$ and $V = 21.597\text{KN}$, $D = -0.282\text{M}$. This model is showing huge amount of ductility and lots of strain hardening. Huge nonlinearity is observed for capacity spectrum.

CONCLUSION

1. MODAL ANALYSIS

Model 1: this particular model records the highest amount of fundamental natural time periods among all other models and first mode is in maximum torsion

Model 2: fundamental natural time periods of first 3 modes are considerably less than themodel1

Model 3: fundamental natural time periods of first 2 modes are quite same

Model 4, Model 5 and model 6: these models have got the least fundamental natural time period of the structure, when compare with model1, time period of model 4, 5 and 6 have decreased by, 69.6%, 68.6%, 70.0%,

Model 7: initial three modes are not at all participating in seismic modal mass, the tower is participating in mode 4 and 5 in x and y translation.

Model 8, Model 9: fundamental natural time periods of first 2 modes are nearly same, and have got huge mass participation along x direction and y direction in mode 1, and mode 2

2. NONLINEAR HINGES

Model 1: Push over analysis has been performed along both orthogonal direction i.e. x and y directions. This model is performing up to the nonlinear range of life safety (LS) to collapse prevention (CP), the plastic deformation in the nonlinear hinges are going beyond the limit specified by FEMA356 and ATC40 and max no od iteration have been done.

Model 2: Results are shown for Hinge4H1 for both the cases (PUSHX and PUSHY).13 iterations have been done in x direction and 7 iteration have been done in y direction by SAP2000.

Model 3: Results are shown for Hinge9H2 for both the cases (PUSHX and PUSHY).10 iterations have been done in x direction and y direction by SAP2000, the hinges are going up to a range of IO (immediate occupancy) level in both cases not a single hinge is going beyond IO levels, All the hinges are in this particular range only.

Model 4, Model 5, Model 6: Results are shown for Hinge9H2 for **Model 4**, Hinge100H1 for **model 5** and **model 6** for both the cases (PUSHX and PUSHY).only 2 iterations have been done in x direction and y direction by SAP2000, the hinges are going up to a range of IO (immediate occupancy) level in both cases not a single hinge is going beyond IO levels, All the hinges are in this particular range only.

Model 7: this model shows the same performance as of model4, 5 and 6. All the nonlinear hinges are forming at the base level of the tower.

Model 8, Model 9: Results are shown for Hinge293H1 for **Model 8**, and **model 9** for both the cases (PUSHX and PUSHY).

3. PERFORMANCE POINT FROM ADRS CURVES.

Model 1: the performance point hasn't been successfully formed for this particular model, the configuration and type of cross section used in making of transmission towers plays very important rule when they are subjected to seismic threatening.

Model 2: the performance point successfully formed for both the load cases, the base shear and the displacement at performance point are $V = 46.76\text{KN}$, $D = -0.514\text{M}$ FOR PUSHX.

Model 3: the performance point successfully formed for both the load cases, the base shear and the displacement at performance point are $V = 32.469\text{KN}$, $D = -0.615\text{M}$ FOR PUSHX.

Model 4: the performance point successfully formed for both the load cases, the base shear and the displacement at performance point are $V = 3.53\text{KN}$, $D = 0.0036\text{M}$ FOR PUSHX.

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Model 8: this model is performing better in longitudinal direction and there is undesirable behavior is seen in transverse directions, the performance point is achieved successfully for PUSHX case, the base shear and displacement are $V = 49.52\text{KN}$, $D = -0.431\text{M}$. This model for pushx case showing huge amount of ductility and lots of strain hardening and huge nonlinearity is observed for capacity spectrum.

Model 9: this model is performing well for both load cases, the performance point is achieved successfully for PUSHX case, and base shear and displacement are $V = 25.211\text{KN}$, $D = -0.356\text{M}$. This model is showing huge amount of ductility and lots of strain hardening. Huge nonlinearity is observed for capacity spectrum.

REFERENCES:

- [1] Applied technological council(ATC40)
- [2] FEMA (federal emergency management agency) 356.
- [3] I.S 800-2007, criteria for design of steel structures
- [4] I.S1893-2016, criteria for earthquake resistant design of structures.
- [5] SAP2000v17, CSI manual for sap2000.
- [6] Google search.