

Dynamic Analysis of Telecommunication Tower for Optimum Modal Combination and Elemental Discretization

Trishit Chandra, Sanjay Sengupta

Abstract: Over the past 35 years, the growing demand for wireless and broadcast communication has spurred a dramatic increase in steel telecommunication tower construction and maintenance. Failure of such structures due to severe earthquakes is a major concern. The Indian code suggests the detailed static and dynamic analysis provisions that are to be followed for lumped mass systems like buildings. In case of continuous structures the code only suggests the static analysis provisions in details. But, due to the lack of detailed Indian codal provisions for dynamic analysis of telecommunication tower, a comparative study using response spectrum method is being carried out with the help of suitable software for different ground level conditions in case of India. According to the theoretical approach of any structural dynamics problem, the structures without lumped mass system is considered as continuous system which is further idealized as a series of small elemental segments. Furthermore, the structural analysis of these elemental segments using the concept of Finite Element Method (FEM) is being carried out with the help of the mentioned software and the results of natural frequencies, time periods of the structure are compared to obtain the optimum number of elemental discretization along with the optimum method of modal combination.

Keywords: Elemental discretization, Modal combination, Natural frequency, Response spectrum method, Steel telecommunication tower, Time period

I. INTRODUCTION

In the present era, technology in communications has developed to a very large extent. The communication industries have seen a tremendous increase in the last few years which has resulted in the installation of a large number of towers to increase the coverage area and network consistency. In wireless communication networks, these towers play a significant role, hence the failure of such structure in a disaster is a major concern. Therefore the utmost importance should be given in considering all possible extreme conditions for designing these towers.

Lu, Ou, Xing and Mills (1988)^[1] first ever presented the analysis of steel transmission tower and presented the structural response of the lattice tower and the detailed connection i.e. bolted connection. They have analyzed the structure considering the basic loads without using any codal provision and concluded in their paper that the

numerical modeling methods are reviewed from bolted connections and tower elements to individual towers. The research findings are on static and dynamic behaviors of bolted connections have been summarized and discussed through the load-displacement curve and bolt pretension degeneration situation. The static structural behaviors and failure modes of non-reinforced and reinforced lattice transmission towers (LTTs) are reviewed.

Bhosale, Kumar and Pandey (2012)^[5] analyzed towers with different bracing system while mounted on the rooftop. They analyzed the structure under wind and seismic loading condition with different bracing patterns and concluded that the design of roof top towers cannot be based on analytical results obtained for a similar configuration situated at ground level. As seen, the axial forces in rooftop tower are increased approximately by two to three times (max.) with respect to ground tower. By increasing the stiffness of the host structure in both the directions (X and Y), the axial forces (tensile & compression) in rooftop towers were increased by minimal amount of 5%. The axial forces in leg members under the effect of seismic load attain the highest value. Nevertheless, it has been observed that the forces in diagonal members are greater as compared to the horizontal members.

Rajasekharan (2014)^[7] designed the lattice tower for three heights of 30m, 40m and 50m with different types of bracings to study the effect of wind load on 4- legged lattice tower for wind zone V and VI using gust factor method. They also studied the seismic effect on the tower structures by carrying out the modal analysis and response spectrum analysis for zone II to zone V and concluded that the member stresses in bottom leg of XX braced tower are higher as compared to other tower models. The frequency of the tower with Y bracing displayed the least natural frequency since its stiffness was found to be higher due to more weight of the structure as compared to other models. It was observed that from 30m to 40m tower height, the increase in displacement is nearly linear but as the height increases from 40m to 50m there is a steep increase in the displacement in all the zones.

Sharma, Duggal, Singh and Sachan (2015)^[8] presented a comparative analysis of steel telecommunication tower under combined seismic and wind load and analyzed their structures with different bracing patterns and concluded different considerations for different conditions. Specifically they have concluded that for all wind zones tower height between 25m to 35m with different bracing patterns do not show much difference in displacement. For wind zone I to IV, tower height between 35m to 45m having K-Bracing or W-Bracing gives maximum value of displacement and V-Bracing gives minimum value of displacement.

Revised Manuscript Received on October 15, 2019.

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For wind zone V and VI tower height between 35m to 45m having W-Bracing gives maximum value of displacement and V-Bracing or XBX -Bracing gives minimum value of displacement. There is a steep increase in the displacement in Earthquake zone V for all considered type of bracing pattern. Results show that the increase in the displacement from earthquake zone II to VI is maximum for W-Bracing and it is minimum for K-Bracing. For all earthquake zones stress at the bottom leg members of the tower is maximum for XBX-Bracing and it is minimum for W-Bracing.

In this study, the main consideration that has been carried out is the comparative analysis among the time periods and natural frequencies of self-supporting telecommunication tower under seismic loading in different ground level conditions. The scope of this thesis is primarily focused on the comparative study of the peak-response quantities (natural frequency, time period) of the structure under seismic loads to obtain the optimum modal combination along with the optimum elemental discretization.

II. STRUCTURAL DESIGN

From the review of literature, it has been observed that the various methods of earthquake analysis are considered while analyzing a self-supporting tower including dead loads and live loads. As the structural configuration depends upon the loading intensities and variations, it also depends upon the zone and soil conditions along the loads. So, the design has to satisfy both the maximum allowable criteria for the zone and soil type considerations together.

The self-supporting towers^[2] are mainly considered when the base area is limited and the required height of the tower is much higher. The guyed towers cannot be used for the limited base area as they required a large area to guy the cables and also the monopoles cannot be considered as the required height is much higher for which the wind intensity will be pretty higher. In case of the self-supporting tower it can be designed as three-legged as well as four-legged whichever is required; but in most of the cases when the height of the tower is much larger like 40m-50m or higher than that, four-legged structure is generally considered as it consists of more number of members than three-legged structure, to transmit the loads from the superstructure to the soil beneath it; so it is a basic intuition that when the applied load is same for two structure but the base areas of the structures which are subjected to the external loads varies, then in case of the larger area the developed stresses will be lower and in case of the smaller area the developed stresses will be higher. Therefore, following this basic convention, the bracing systems are used to distribute the loads among the members and to resist any kind of failure of the main legs including the bracings themselves. The towers are generally considered as a cantilever structure as a whole and as we know the bending moment of a cantilever member is highest at the fixed end and for that reason the whole structure of the tower is designed as a tapered structure whose largest area is on the fixed end and smallest area is on the free end. For designing a tapered section the legs are designed with a little angle deviation with respect to the vertical for which the legs are more effective to carry both components of force viz. horizontal and vertical.

Depending upon the loading types and the distribution of loads among the members, the sections are chosen. Generally for consideration of member of a trussed tower the angle sections are used because the truss structures have only the axial forces to be considered (no shear force or bending moment in the members) and for that reason among all the sections which are to be considered only for axial forces, the angle section has the least specific area compared to the other which is advantageous from the economic and self-weight point of view.

In a trussed tower the bracing systems^{[6],[10]} and the connections among the members are very important aspects. The bracing systems consist of horizontal bracing as well as diagonal bracings to resist the failure of the members. The horizontal bracing members are provided to resist the buckling of the legs hence they are considered as tension members and to resist the torsional effect as well as to distribute the loads, the diagonal bracing members are used. From the structural analysis point of view of a trussed structure, as it is being known that the triangular structures have the highest stability and strength to resist the external loads the bracings are designed in such a way that the elementary structures of the bracings consist of triangular formation. The connections are to be designed very cautiously as it is the main part of the structure that holds all the members together; so the failure of connections will lead to the failure of members which will cause the total structural failure. In case of trussed towers, as there is no bending moment developed in the members, the moment connections are not needed to be considered; only the shear connections are enough to maintain the stability and strength among the members.

III. ANALYSIS METHODS

This topic reveals the basic considerations of earthquake analysis of a self-supporting trussed tower. The seismic loads are considered to act as a result of horizontal relative acceleration among the different panels which causes drifts in the structures. Earthquake is an unpredictable phenomenon for which the analysis of a structure under seismic load condition has to be done with the previous collected statistics of earthquakes for a particular zone and soil. The particular collected statistical data has been taken into account with a probabilistic approach to obtain the optimum possible safety. Therefore, basically the structures are analyzed by the previous seismic loads with the highest intensities for the certain design criteria. IS 1893:2005 (Part-4)^[12] gives the provisions for static analysis of seismic load for communication towers with consideration of different zones and soil structures. IS 1893:2002 (Part-1)^[11] provides the basic adaptation of different methods of analysis of building structures subjected to seismic loads. There are three basic methods of analysis for seismic loads which are as follows:

- Equivalent Static Load Method (ESL)
- **Response Spectrum Method (RSM)**
- Time History Method (THM)

A. Response Spectrum Analysis

The dynamic analysis is defined as the analysis of structure considering the motion of the structure which depicts that all the parameters are taken as time-dependant. In case of the seismic analysis, the ground vibration is considered as random vibration which can be evaluated using probabilistic approach. The random vibration is idealized as the summation of different series or “modes” of elementary harmonic vibrations. The theoretical analysis of any stack-like structure is done by idealizing the structure as continuous system and with the help of the dynamic matrix method the elemental segments are analyzed to get the response of the whole structure for each modes of vibrations. From the first mode of vibration, the natural time period is being calculated. In case of the assignment of the seismic loads in the software viz. STAAD.Pro V8i, the calculations of the parameters that have to be made for the RS analysis are according to the provisions of IS 1893: 2002 (Part-1) and IS 1893: 2005 (Part-4). The analysis has been done using STAAD.Pro V8i and the parameters which have to be put in the software for the analysis are as follows:

- Zone Factor (Z) is a factor that deals with the ratio of probable average intensity of earthquakes in case of particular zones. [Cl. 6.4.2, Table 2, Pg-16 of IS 1893: 2002 (Part-1)]
- Response Reduction Factor (R) is the factor by which the actual base shear force should be reduced to obtain the design lateral force. [Cl. 16, Table 9, Pg-17 of IS 1893: 2005 (Part-4)]
- Importance Factor (I) is a factor used to obtain the design seismic force depending on the functional use of the structure. [Cl. 16, Table 8, Pg-17 of IS 1893: 2005 (Part-4)]
- Damping Ratio (MCE) is the ratio between actual damping of the system to the critical damping (free vibration) of the system. [Cl. 15, Table 7, Pg-16 of IS 1893: 2005 (Part-4)]
- Cross-sectional Area (A) at the base of the tower
- Shear Force Co-efficient (C_v) [Cl. 17.1, Table 6, Pg-16 of IS 1893: 2005 (Part-4)]
- Slenderness Co-efficient (C_T) [Cl. 14.1, Table 6, Pg-16 of IS 1893: 2005 (Part-4)]
- Slenderness Ratio (k) = h/r_0 ; where h = Effective height of the structure, r_0 = Radius of Gyration= $\sqrt{\frac{I}{A}}$
- Time Period (T) [Cl. 14.1, Pg-15 of IS 1893: 2005 (Part-4)]
- Modal Combination [Cl. 10.2.5.2, Pg-12 of IS 1893: 2005 (Part-4)]
- Base Shear Multiplication Factor (V'_B/V_B) [Cl. 7.8.2, Pg-25 of IS 1893: 2002 (Part-1)]

In the dynamic analysis, the design base shear (V_B) shall be compared with a base shear (V'_B) calculated using the fundamental period T (corresponding to first mode of vibration). All the response quantities such as member forces, displacements, storey forces, storey shears and base reactions shall be multiplied by V'_B/V_B . This base shear multiplication factor is mentioned in the codal provision but the value that has to be given in the software is evaluated as the ratio of the calculated time period to the time period corresponding to the first mode of vibration.

IV. ANALYSIS OF THE TOWER USING RESPONSE SPECTRUM METHOD

Initially, the dead loads and the seismic loads are being considered and the combination of both of them is then taken according to the clauses of IS 1893: 2002 (Part 1) for the dynamic analysis. Then the whole structure is analyzed using STAAD.Pro V8i with the following values of the parameters:

Table I: Detailed parameters used for RSM

Zone Factor (Z)	0.10	0.16	0.24	0.36
Response Reduction Factor (R)	4			
Importance Factor (I)	1.5			
Cross-sectional Area at the base (A)	5m x 5m			
Shear Force Co-efficient (C_v)	1.387			
Slenderness Co-efficient (C_T)	64.352			
Slenderness Ratio (k)	34.64			
Damping Ratio (MCE)	2%			
Time Period (T)	0.734 s			
Modal Combinations	CQC	SRSS	ABS	
Base Shear Multiplication Factor	0.025875			

The above mentioned parameters have been used for the analysis in STAAD.Pro and the number of cut-off mode shapes are taken as 50. The eigen analysis of the tower as a continuous structure has been done by STAAD.Pro and the natural frequencies and time periods are tabulated for first ten modes. The tower structure is analysed by elemental discretization of the members to get the mentioned results. The study of optimum method of modal combination along with optimum number of elemental discretization is being performed. From Fig. 1 to Fig. 8, the mode shapes of the tower are shown for the first eight natural frequencies.

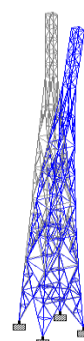


Fig. 1. Mode Shape 1 for the tower in plain ground



Fig. 2. Mode Shape 2 for the tower in plain ground

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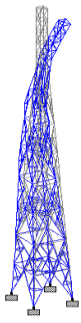


Fig. 3. Mode Shape 3 for the tower in plain ground

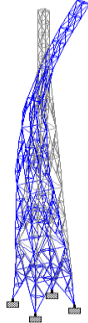


Fig. 4. Mode Shape 4 for the tower in plain ground



Fig. 5. Mode Shape 5 for the tower in plain ground

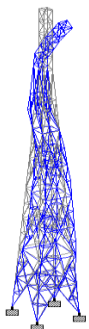


Fig. 6. Mode Shape 6 for the tower in plain ground

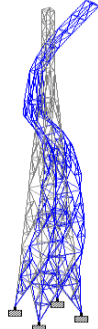


Fig. 7. Mode Shape 7 for the tower in plain ground

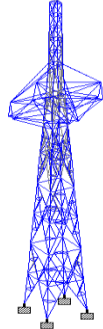


Fig. 8. Mode Shape 8 for the tower in plain ground

The natural frequencies of the tower denote the response frequencies against the action of horizontal movement of the ground. The ground motions in different magnitude and directions produces different ground frequencies which are transferred to the tower while an earthquake occurs and as a result of that external action the tower gives some response frequencies. The corresponding time periods are calculated from the response frequencies. The natural frequencies and time periods of the tower for different ground level conditions are tabulated from Table II to Table IV and shown from Fig. 9 to Fig. 14. The comparisons among them are listed in Table V and shown in Fig. 15 and Fig. 16.

Table II: Natural frequencies and Time periods of the tower in plain ground

Modes	Frequency (cycles/s)	Time period (s)
1	1.643	0.60873
2	1.643	0.60867
3	5.445	0.18364
4	5.453	0.18337
5	7.235	0.13822
6	8.765	0.11409
7	8.777	0.11394
8	11.116	0.08996
9	11.728	0.08527
10	12.044	0.08303

Table III: Natural frequencies and Time periods of the tower in 1m sloped ground

Modes	Frequency (cycles/s)	Time period (s)
1	1.656	0.60371
2	1.657	0.60357
3	5.481	0.18243
4	5.489	0.1822
5	7.274	0.13748
6	8.846	0.11305
7	8.856	0.11292
8	11.116	0.08996
9	11.936	0.08378
10	12.05	0.08299

Table IV: Natural frequencies and Time periods of the tower in 3m sloped ground

Modes	Frequency (cycles/s)	Time period (s)
1	1.677	0.59632
2	1.68	0.5953
3	5.533	0.18073
4	5.539	0.18055
5	7.328	0.13646
6	8.962	0.11158
7	8.973	0.11145
8	11.116	0.08996
9	12.054	0.08296
10	12.234	0.08174

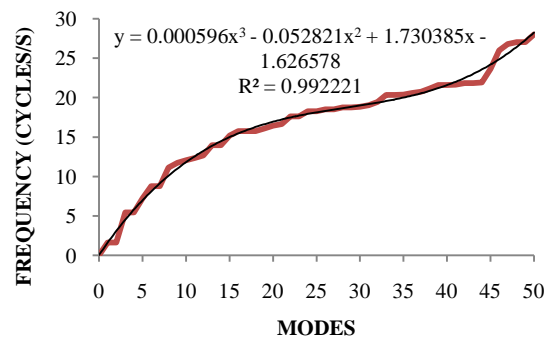


Fig. 9. Natural frequencies of the tower in plain ground

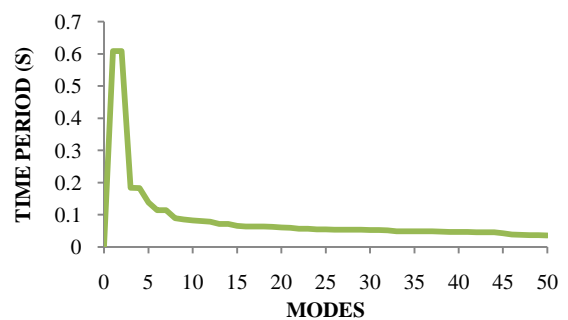


Fig. 10. Time periods of the tower in plain ground

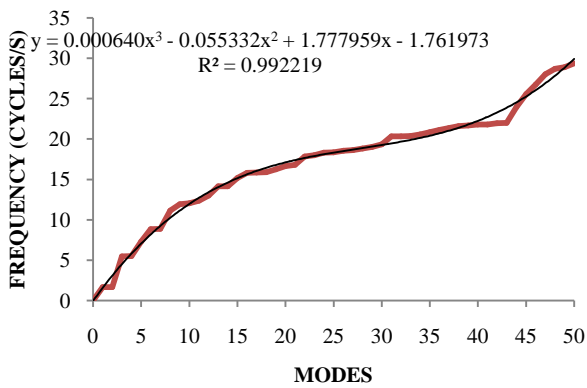


Fig. 11. Natural frequencies of the tower in 1m sloped ground

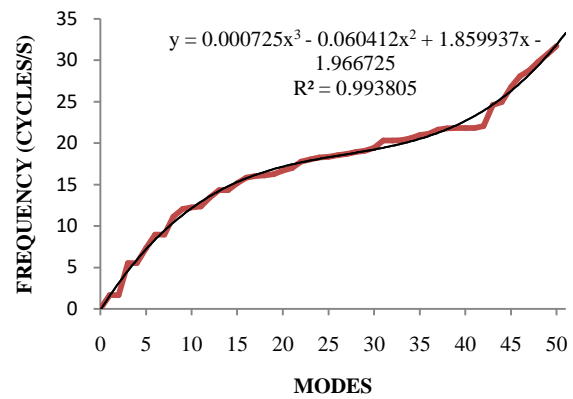


Fig. 13. Natural frequencies of the tower in 3m sloped ground

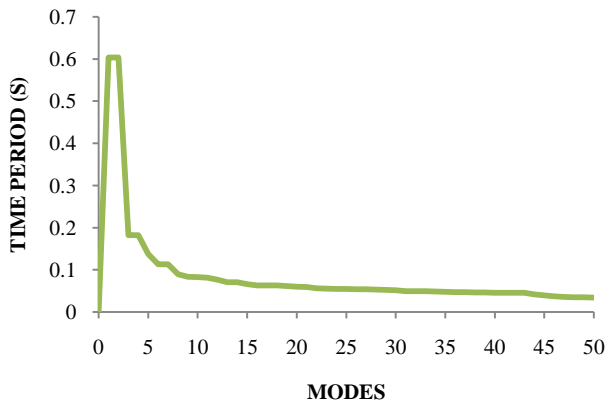


Fig. 12. Time periods of the tower in 1m sloped ground

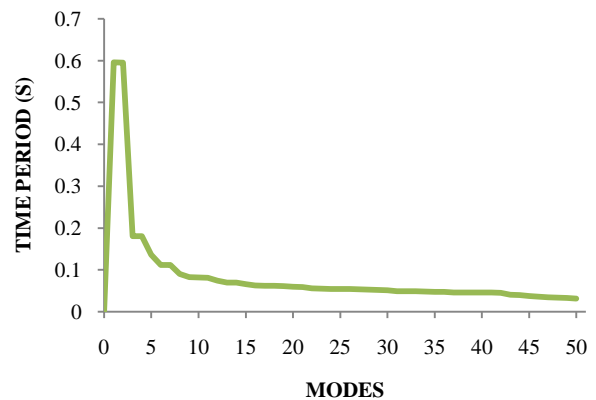


Fig. 14. Time periods of the tower in 3m sloped ground

Table V: Comparison among natural frequencies and time periods of the tower

Modes	Plain ground		1m sloped ground		3m sloped ground	
	Frequency (cycles/s)	Time Period (s)	Frequency (cycles/s)	Time Period (s)	Frequency (cycles/s)	Time Period (s)
1	1.643	0.60873	1.656	0.60371	1.677	0.59632
2	1.643	0.60867	1.657	0.60357	1.68	0.5953
3	5.445	0.18364	5.481	0.18243	5.533	0.18073
4	5.453	0.18337	5.489	0.1822	5.539	0.18055
5	7.235	0.13822	7.274	0.13748	7.328	0.13646
6	8.765	0.11409	8.846	0.11305	8.962	0.11158
7	8.777	0.11394	8.856	0.11292	8.973	0.11145
8	11.116	0.08996	11.116	0.08996	11.116	0.08996
9	11.728	0.08527	11.936	0.08378	12.054	0.08296
10	12.044	0.08303	12.05	0.08299	12.234	0.08174

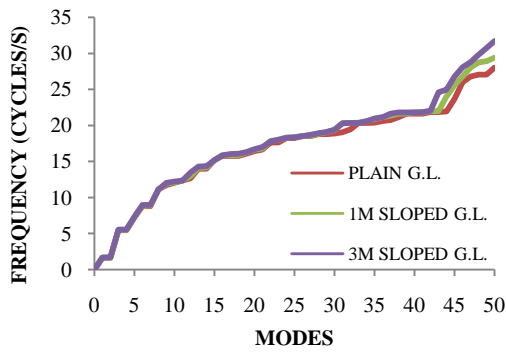


Fig. 15. Comparison among natural frequencies of the tower

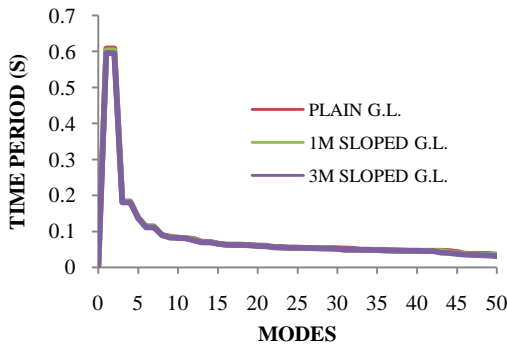


Fig. 16. Comparison among the time periods of the tower

In the above comparison among the natural frequencies and time periods of the tower in different ground level conditions, it can be seen that there is no huge difference in the change of the frequencies or the time periods. Now the study proceeds to the method of discretization. The eigen analysis of the tower structure is done using the concept of discretization for continuous structure (i.e. without lumped mass). The analysis using discretization is just a finite element method where all the members are discretized in elements. The mechanics of those elements are studied using FBD of a spring-damper-mass system to come up with the resulting response of the total structure as a whole. The accuracy of the result depends on the number of discretized elements; higher the elements greater the accuracy but along with the accuracy the structure should also be economical. For this reason, the study of optimum number of discretized elements have been carried out and listed in Table VI and Table VII with graphical representations in Fig. 17 and Fig. 18.

Table VI: Comparison among different elemental discretizations of members

Modes	Member with one element discretization		Member with three elements discretization		Member with eight elements discretization	
	Freq.	Time Period	Freq.	Time Period	Freq.	Time Period
1	1.643	0.6087 3	1.646	0.6075 4	1.646	0.6075 2
2	1.643	0.6086 7	1.646	0.6075 2	1.646	0.6075
3	5.445	0.1836 4	5.532	0.1807 5	5.535	0.1806 8
4	5.453	0.1833 7	5.539	0.1805 2	5.542	0.1804 5
5	7.235	0.1382 2	7.356	0.1359 4	7.357	0.1359 2
6	8.765	0.1140 9	8.59	0.1164 1	8.585	0.1164 8

7	8.777 4	0.1139 4	8.602	0.1162 5	8.597	0.1163 2
8	11.11 6	0.0899 6	11.97 5	0.0835 1	11.97 7	0.0835
9	11.72 8	0.0852 7	11.99 4	0.0833 8	11.99 8	0.0833 5
10	12.04 4	0.0830 3	12.72 6	0.0785 8	12.73 9	0.0785

Table VII: Comparison among percentage changes in frequencies of different elemental discretization

Modes	% Change in three elemental member with respect to one elemental member (a)	% Change in eight elemental member with respect to one elemental member (b)	% Change in eight elemental member with respect to three elemental member (c)
1	0.182592818	0.182592818	0
2	0.182592818	0.182592818	0
3	1.597796143	1.652892562	0.054229935
4	1.577113515	1.632129103	0.054161401
5	1.672425708	1.686247408	0.013594345
6	-1.996577296	-2.053622362	-0.058207218
7	-1.993847556	-2.050814629	-0.058126017
8	7.727599856	7.74559194	0.016701461
9	2.268076398	2.30218281	0.033350008
10	5.662570575	5.770508137	0.102153072
Total % change	16.88034	17.0503	0.157857
Average % change	1.688034	1.70503	0.015786

The percentage change in three elemental members with respect to one elemental member, percentage change in eight elemental members with respect to one elemental member and percentage change in eight elemental members with respect to three elemental members are denoted as ‘(a)’, ‘(b)’ and ‘(c)’ in the above table. The average changes of all the frequencies are calculated to conclude the comparative study among them. From the above table, it can be clearly observed that the average percentage change in (a) is much higher than the average percentage change in (c). And from the table, one can easily depict that the summation of (a) and (c) is equal to the value of (b). From this relations among the percentage changes, it can be concluded that the percentage change in three elemental member with respect to one, is much significant than the other two. Therefore, three elemental member will be the optimum number of elemental discretization in case of the economic and safe design of the tower using dynamic analysis of continuous structure.



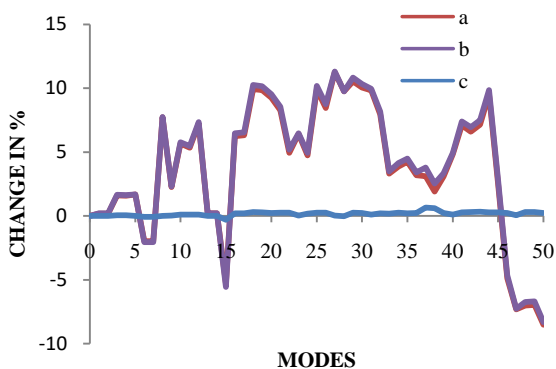


Fig. 17. Comparison among % changes in frequencies

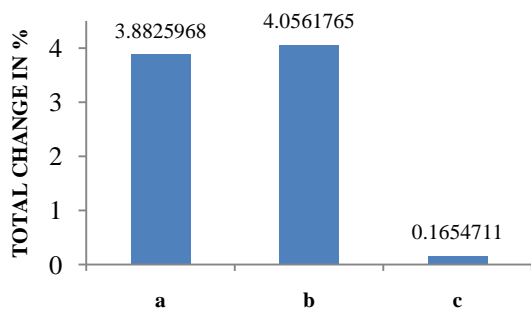


Fig. 18. Comparison among total % changes in frequencies

The peak response quantities like base shear, storey shear, displacement etc. are calculated using the various methods of modal combinations like SRSS, ABS, CSM and CQC. The individual base shears produced by individual modes are calculated for the final resulting base shear of the tower. Square-Root-of-the-Sum-of-the-Squares (SRSS) is a method where every base shear value is added after squaring and then the square root of the addition is taken as the resulting base shear. Absolute-Sum (ABS) is a method where just the normal additions of the base shears are taken into account to get the resulting base shear. Complete-Quadratic-Combination (CQC) is a method where quadratic summation of the base shears are considered to obtain the resulting base shear. The comparison among these methods have been carried out to get the optimum method which can assure a safe structure with economical. The comparison among the resulting base shear values obtained from different methods are being done and listed in Table VIII with graphical visualisation in Fig. 19 and Fig. 20.

Table VIII: Comparison among the methods of dynamic analysis

Base Shear (N)		Total SRSS Shear	Total ABS Shear	Total CQC Shear
Plain Ground	X	17265.54	29499.35	17518.94
	Z	17261.43	29499.12	17515.18
1m Sloped Ground	X	16931.22	27576.36	17081.69
	Z	16972.16	28607.06	17136.67
1.5m Sloped Ground	X	16957.29	28211.79	17088.6

2m Sloped Ground	Z	16913.27	28061.24	17049.84
	X	16791.3	26072.48	16900.49
2.5m Sloped Ground	Z	16821.41	27712.12	16945.46
	X	16718.34	25727.51	16818.4
3m Sloped Ground	X	16657.46	25398.96	16745.54
	Z	16615.67	25844.35	16713.35

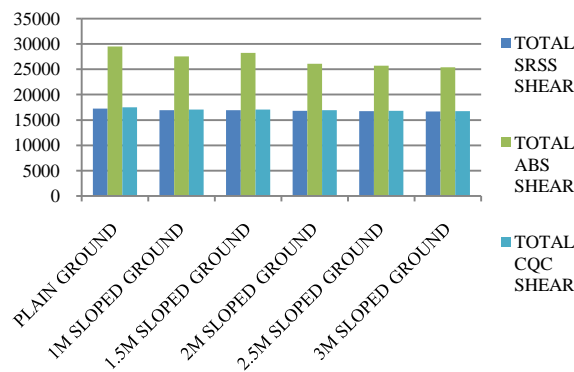


Fig. 19. Base shear variations in X direction for different ground level conditions

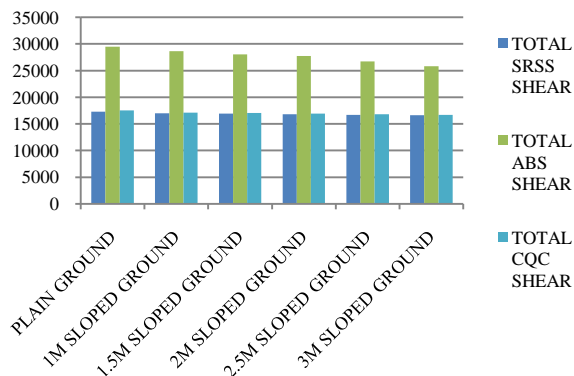


Fig. 20. Base shear variations in Z direction for different ground level condition

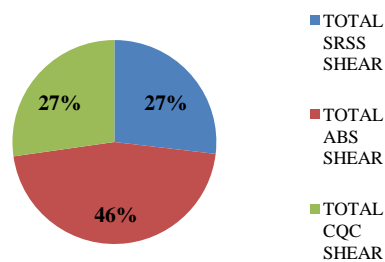


Fig. 21. Base shear variations of the tower for different ground level conditions

The comparison among the methods have been shown to obtain the optimum method. According to the IS 1893: 2002 (Par 1), the CQC method is the most reliable and approved method for the modal combination of general building structures.

In this study, it has been shown that for the analysis of telecommunication tower the peak response quantites are very much similar for SRSS and CQC method but for ABS the values are much higher. From the economic point of view the ABS method cannot be considered as optimum method. Therefore, comparing the SRSS and CQC, it has been obtained that the SRSS values are little lower than CQC values. Henceforth, from the safety point of view the CQC is considered as the optimum method of modal combination which will give an economic and safe design.

V. RESULT AND DISCUSSION

In the above analysis, the results are evaluated from the STAAD.Pro output file. The resulting graphs have been generated to draw suitable conclusions from them. The conclusions that can be made from the results and discussions are as follows:

- From Fig. 9 to Fig. 14, it can be concluded that there is no significant change among the natural frequencies or the time periods of the tower in different ground level conditions. In Fig. 15 and Fig. 16, it can be seen that there is some certain changes among the natural frequencies or the time periods in case of the higher modes of vibrations.
- In Table VII the comparisons among the percentage change of frequencies are being done along with the graphical representations in Fig. 17 and Fig. 18. The change in three elemental members with respect to one elemental member is 1.68% and the change in eight elemental members with respect to three elemental members is 0.02%. Therefore, from this result it can be concluded that the optimum number of discretization should be taken as three elements. The maximum accuracy can be achieved by taking upto three elemental discretizations.
- In Table VIII the comparison among the methods of the dynamic analysis is being listed along with the graphical visualization from Fig. 19 to Fig. 21. The ABS shear value is much higher than the other values. Therefore it cannot be an economical method relative to the others. The SRSS shear value is minimum among all of them, that is why it is not considered because of the strength criteria. The CQC value is the mediate value which can be treated as safe from strength point of view while it is also economical. This method is also suggested by the IS 1893:2002 (Part 1) for dynamic analysis of general buildings.

VI. CONCLUSION

The comprehensive dynamic analysis of the self-supporting telecommunication tower is performed using STAAD.Pro V8i. The method of Response Spectrum is opted for the analysis, where the parameters of the study are different ground elevations. Due to the lack of detailed codal provisions for dynamic analysis of telecommunication tower, this structure is exposed to the theoretical dynamic analysis in which all the members have satisfied their purpose under the external excitations for each and every

case. From the result of the analysis it can be concluded that there are no significant changes among the response natural frequencies or the time periods of the tower in different ground elevations. Telecommunication tower is considered as a continuous system where the discretization of the system must be carried out to analyze it. From the result and discussions it has been presented that the optimum number of discretization should be taken as three elements. Furthermore, the analysis for continuous system demands for the optimum method of modal combinations. From the results of this study it is observed that the CQC value is the mediate value which can be treated as safe from strength point of view as well as cost effective from economical point of view. Therefore, it is considered as the optimum method for the calculations of peak response quantities.

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