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Dynamic response of layered hyperbolic cooling tower considering the effects of support inclinations

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Abstract. Cooling tower is analyzed as an assembly of layered nonlinear shell elements. Geometric representation of the shell is enabled through layered nonlinear shell elements to define the different layers of reinforcements and concrete by considering the material nonlinearity of each layer for the cooling tower shell. Modal analysis using Ritz vector analysis and nonlinear time history analysis by direct integration method have been carried out to study the effects of the inclination of the supporting columns of the cooling tower shell on its dynamic characteristics. The cooling tower is supported by I-type columns and ʌ-type columns supports having the different inclination angles. Relevant comparisons of the dynamic response of the structural system at the base level (at the junction of the column and shell), throat level and at the top of the tower have been made. Dynamic response of the cooling tower is found to be significantly sensitive to the change of the inclination of the supporting columns. It is also found that the stiffness of the structure system increases with increase in inclination angle of the supporting columns, resulting in decrease of the period of the structural system. The participation of the stiffness of the tower in structural response of the cooling tower is fund to be dependent of the change in the inclination angle and even in the types of the supporting columns.

Keywords: cooling tower; hyperbolic shell; finite element analysis; dynamic response; layered shell; modal analysis; nonlinear time history analysis; support inclination

1. Introduction

Hyperbolic free standing cooling towers constitute an important component of systems dealing with thermal and nuclear power generations. Keeping in view modern requirements these structures constitute high rise structural systems. They are frequently required to be designed for severe seismic loading, exceeding the loading due to wind pressure (Lee and Gould 1985).

A lot of work is available in the literature (Albasiny and Martin 1967, Gould 1968, Lee and Gould 1967, Martin and Scriver 1961, Nagesh *et al*. 1990, Viladkara *et al*. 2006, Noorzaei *et al*. 2006) on analysis of fixed base hyperbolic cooling towers. Work had also been reported in the literature (Sen and Gould 1976, Abu-Sitta 1970, Chan 1978, Gould and Lee 1969, Kye and Wen

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1987, Karisiddappa *et al*. 1998, Hara and Gould 2002, Hyuk 2006, Vaziri *et al*. 2006) on the analysis of column supported cooling towers.

In most of the Civil Engineering analyses, structure is assumed to be fixed at the base. As cooling towers are supported by alternative supporting systems like as vertical, inclined, *V* type and *X* types of the supports, it is of interest to know that how the change of the angle of these supporting systems will influence the response of the whole structure due to the earthquake loading. Bhimaraddi *et al*. (1991) had studied the free vibration response of the column supported, ring stiffened cooling tower and concluded that under seismic excitation of the cooling tower the stiffening rings may not help increase the resistance of the structure because these rings have no influence on the modal characteristics of structures under such excitations, however, the stiffening rings help increase the load-carrying capacity of towers under wind excitations. Various attempts had been made to model the discrete columns and to incorporate their effects into the cooling tower analysis using different approaches of varying accuracy. Wolf and Skrikerud (1980) had studied the Influence of geometry and of the constitutive law of the supporting columns on the seismic response of a hyperbolic cooling tower. To determine the optimum seismic design, the influence of the geometry of the columns was investigated parametrically. Hara (2002) had studied dynamic response of cooling tower considering two alternative types of the supporting column systems namely *I*-column supports and *V*-column supports. It had been concluded that the total structural responses of the shell with supporting columns strongly depend on the column supporting systems and were different from the conventional pin-supported ideal shell. Sabouri-Ghomi *et al*. (2006) had carried out a numerical study of the nonlinear dynamic behavior of reinforced concrete cooling towers supported by columns under earthquake excitation and its influence on the integrity and stability of cooling towers and the locations of plastic hinges within the supporting columns were identified and assessed the ramifications of the plastic hinges on the stability of the cooling tower. Sabouri-Ghomi *et al*. (2006) investigated the effect of stiffening rings on the buckling stability of R.C. cooling towers supported on *X*-shaped columns by considering the important design parameters such as the number, dimension, and location of stiffening rings and concluded that added stiffening rings increase the buckling resistance of the R.C. shell. The stiffening rings behaved flexibly or rigidly, depending on their dimensions. The number of flexible stiffening rings required to maximize the buckling safety factor was found to be higher than the number of rigid stiffening rings required to maximize it. Gopinath *et al*. (2012) had presented integrated methodologies based on multilevel modeling concepts for finite element analysis of reinforced concrete hyperbolic cooling tower shell structure, with specific reference to account for nonlinear response behavior at ultimate capacity of cracked concrete. Geometric representation of the shell was enabled through multiple concrete layers. Composite characteristic of concrete was accounted by assigning different material properties to the layers. Asadzadeh *et al*. (2012) had studied the structural response of the hyperbolic cooling towers under static wind and pseudo static seismic forces. In this study two types of supporting systems namely *I* type and *I* type of column supports at the base of the towers have been considered and the finite element analysis employing higher order Mindlin formulation for the shell elements have been undertaken. They have observed that the *I* type of supports create higher flexibility at the base of the tower as compared to the *V* type of supports which behaves in a manner similar to fixed support at the base. The structural response of the towers under wind and earthquake is found to be completely different for the towers supported on different columns.

Knowledge of mode shapes and frequencies are helpful in designing the structures against wind and earthquake loadings. As the wall thickness of the tower should be chosen in such a way that the requirements of buckling strength and a minimum natural frequency are fulfilled (Bhimaraddi *et al*. 1991) and since the natural frequency of the tower with supporting columns strongly depend on the supporting columns it is of considerable interest from practical point of view to study the effect of supporting columns inclinations on the modal characteristics and earthquake resistance of the tower to provide an efficient, realistic design basis for such structures without increasing the thickness of the tower shell which results in increase of the dead load makes the structure uneconomical.

However, a more economic structure would have an optimum angle of inclination of the supporting columns of a size which results in the maximum load carrying capacity of the total structure and minimum displacement.

2. Geometrical and material description of the tower

In the nonlinear finite element analysis one of the most difficult challenges is material behavior modeling of reinforced concrete shell structures. Deformation response and ultimate strength of RC shell structures are governed predominantly by material response of concrete and reinforcing steel, tensile cracking of concrete, bond between concrete and steel (Wang and Teng 2007, Rabczuk *et al*. 2008). Softening response of concrete due to quasi-brittle cracking in tension also significantly influences the nonlinear response by inducing loss of strength and stiffness (Oliver *et al*. 2008). Due to all these, nonlinear analysis of cooling towers is a complex problem and requires attention for realistic modeling of the layer of shell concrete confined between the reinforcement layers.

In this investigation, the cooling tower analyzed by Hara (2002) is considered for analysis.

The geometric configuration of reinforced concrete (R/C) shell is defined as

$$r = \Delta r + a.\sqrt{\frac{(z-125)^2}{b^2}} \tag{1}$$

Where, r is the radius of the shell at height $z(m)$. Parameters a, b and Δr are shown in Table 1. Also, the radius and the thickness of R/C shell are presented in Table 2.

In shell elements, reinforcements are provided each way that is circumferential and meridian on each face. The bars of #8 (area of bars =509 mm^2 and diameter =25.4 mm) at 0.25 m spacing

Table 1 Configuration parameters

Height (z)	9.17m-125m	125m-176m
a	51.9644	0.2578
b	113.9896	8.0293
Δr	−15.3644	36.3422

Table 2 Radius and thickness of the shell

	Lintel	Throat	Top
Height(z)	9.17m	125m	176m
Radius(m)	58.72m	36.6m	38m
Thickness(m)	1.05m	0.24m	0.2m

Table 3 Material properties

Concrete		Reinforcement	
Elastic Modulus	34GPa	Elastic Modulus	206GPa
Poisson's Ratio	0.167	Yield Stress	500MPa
Density	0.0023kg/cm^3	Tensile stress	750MPa
Compressive Strength	36MPa	Poisson's Ratio	0.21
Tensile Strength	2.7MPa		

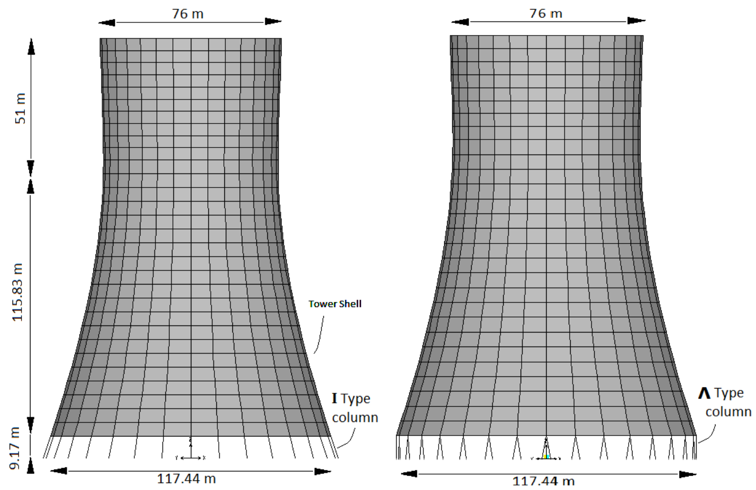

Fig. 1 Geometrical representation and analytical model of tower

in both directions having 0.03 m cover for hoop steel and 0.554 m cover for meridian steel from inner and outer surfaces are placed in the cooling towers shell. For **I-**type, Supporting columns have the area of 0.87m^2 with 0.47% reinforcements. Columns are of circular cross sections and the diameter of columns is 1.05m with #4 bars (area of bars =129 mm^2 and diameter =25.4 mm) as the spiral confinement with 0.03 m cover.

The Λ **-**Type, Supporting columns which are pair columns, total cross sectional area and steel reinforcements in each pair column are the same as that in one **I-**type column. Covers to reinforcements are also the same.

The material properties of the concrete and the reinforcements are shown in Table 3.

The geometrical representation of the towers supported on the **I-**type columns and the **Λ-**Type columns are given in the Fig. 1.

3. Finite element modeling

Modeling and analysis of the tower is done by SAP2000 package Ver. 14. The supporting columns have been modeled as 2-noded line elements having 6 degrees of freedom at each node. The cooling tower shell has been modeled as the layered nonlinear shell elements, which is the key that allows us to define the different layers of reinforcements and concrete by considering the material nonlinearity of each layer for the cooling tower shell (SAP2000 Analysis Reference

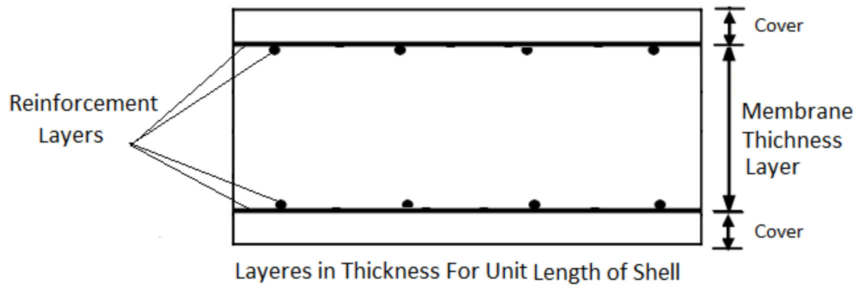

Fig. 2 Layered shell element to model the different layers of concrete and reinforcement

2009). This element is the 4 nodded shell elements which permits the full shell behavior of the tower which is a combination of membrane and plate behavior and supports all forces and moments except the drilling moment. This element has six degrees of freedom (with three translational and rotational degrees of freedom) at each node (Ibrahimbegovic and Wilson 1991). The use of the full shell behavior (membrane plus plate) is recommended for all three- dimensional structures (SAP2000 Analysis Reference 2009). The three dimensional analysis has been carried out.

3.1 Four-node quadrilateral shell element (layered shell element) in SAP2000

The layered shell (shown in Fig. 2) in SAP2000 allows any number of layers to be defined in the thickness direction, each with an independent location, thickness, behavior, and material (reinforcement). The shell element is having 2 reinforcement layers at inner face, 2 reinforcement layers at outer face and one concrete level having the thickness of 0.3 m (i.e., totally 4 layers of reinforcement and 3 layers of concrete are used).

Membrane deformation within each layer uses a strain projection method (Hughes 2000, Page. 232). In-plane displacements are quadratic. For bending, a Mindlin/Reissner formulation is used which always includes transverse shear deformations. Out-of-plane displacements are quadratic and are consistent with the in-plane displacements.

Fig. 1 shows the numerical models of the cooling towers. Models are divided into 32 elements in hoop direction and into 30 elements in meridian direction. The height is 176 m. The thickness of the shell changes from 105cm at the lintel through 20cm at the top (see Table 2). Total numbers of 32 **I-**Type columns are circumferentially supporting the shell structure. Area of the each column is 0.87 m^2. And total numbers of columns for Λ**-**Type are 64 having the 0.43 m^2 each or 0.87 m^2 for a pair.

Seven models are created for various angles of inclination of the columns. 1 model for the vertical **I-**type columns which are without inclination and 3 models for the **I-**type columns having the inclination toward the centre of the tower (Radial inclination) and 3 models for the Λ**-**type columns having circumferential inclination and without radial inclination. The column perpendicular to the ground surface or **I-**90^o is considered as zero inclination. The angles have been changed by 5^o therefore the columns have the angles of 90^o, 85^o, 80^o and 75^o with the horizontal ground surface toward the center of the tower for **I-**type of columns. For Λ**-**type columns, each column of the pair having the inclination of 5^o (i.e., Λ**-**85) in circumference direction of the tower thus having the angle of 10^o between the columns of a pair and therefore this

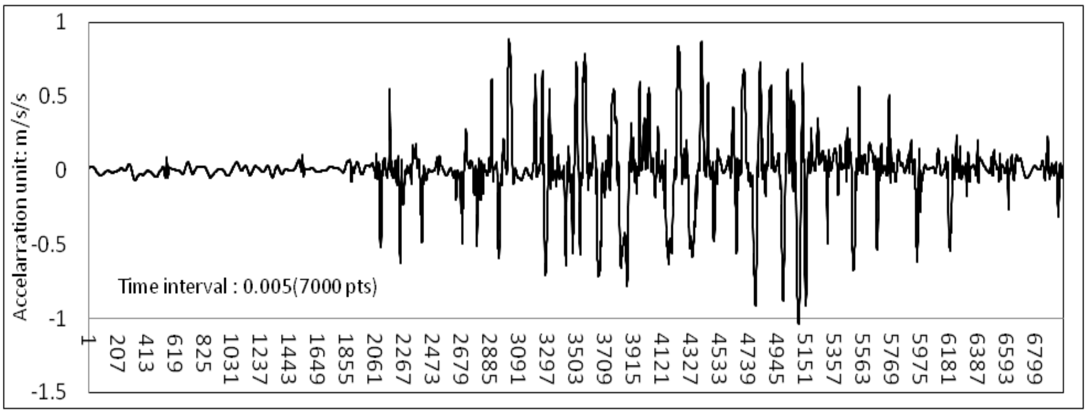

Fig. 3 Earthquake records (accelerations in m/sec^2) of Bhuj city

angle will be 20°and 30° degrees for Λ-80 and Λ-75 types, respectively.

The dynamic analyses are performed as modal analysis of the tower by Ritz Vectors and nonlinear time history analysis by direct integration method. The structure system is analyzed for the strong ground motion data recorded at Ahmedabad in longitudinal direction for the recent earthquake on 26 January 2001 in the Bhuj city in the Gujarat state of the India. Due to symmetric configuration of structure the accelerations have been applied only in X direction. The accelerogram of Bhuj (2001) earthquake is given in Fig. 3. The nonlinear analysis is including elastic and inelastic material nonlinearity.

4. Numerical results

Once the analytical models have been created for all the seven models having the same geometry, same material properties, same height and same thickness but different support inclination and the finite element analysis of the cooling tower has been carried out; the parametric study will be for four models having the 0°, 5°, 10° and 15° different radial inclinations of the supporting columns with respect to the meridian axis of the tower. Theses towers are named I-90, I-85, I-80 and I-75 respectively. The other study will be for the three models having the circumferential inclination of 5°, 10° and 15°. Theses towers are named Λ-85, Λ-80 and Λ-75 respectively. It must be noted that the angle between pair columns of Λ-85 Type is 10°, Λ-80 Type is 20° and Λ-75 Type is 30°. The results are discussed for free vibration analysis, and nonlinear direct integration time history analysis of the tower.

5. Modal analysis results

Table 4 shows the results for the periods of the tower system by Ritz vectors for seven cases. These modes are circumferential, lateral, (Nasir *et al*. 2002) or transverse, ovalling and rotational in nature.

For I-Types of supports; the ovalling mode happens for I-90 at mode 3 and I-85 type at mode 2 however with increase of the inclination angle of the supporting columns (or with increase of

Table 4 Circumferential, lateral and rotational periods of vibration modes of the tower

MODE	period (sec)						
	I-90	I-85	I-80	I-75	Λ-85	Λ-80	Λ-75
1	2.700892	2.4024	2.035592	1.711292	2.837733	2.490166	2.200007
2	2.618125	2.079197	1.704929	1.651623	2.837502	2.490138	2.199943
3	2.49688	1.905389	1.671995	1.502802	2.477162	1.860488	1.814771
4	1.928146	1.813151	1.624396	1.501001	1.89873	1.828527	1.533172
5	1.927675	1.764883	1.622183	1.444427	1.496784	1.295242	1.282161
6	1.866444	1.682306	1.257516	1.083597	1.493826	0.986028	0.979573
7	1.605017	1.537111	1.227277	0.98516	1.305532	0.976854	0.847247
8	1.289199	1.281409	0.937768	0.862773	0.997075	0.850519	0.82326
9	0.919861	0.919355	0.63693	0.574789	0.859205	0.68529	0.663829
10	0.434931	0.436673	0.445001	0.464163	0.429583	0.390904	0.343892
11	0.237468	0.15646	0.144979	0.145751	0.388637	0.16025	0.154125
12	0.105123	0.100255	0.081817	0.077896	0.095383	0.102292	0.112786

stiffness of the total structure) the ovalling mode does not occur for I-80 and I-75 types. For Λ-Types of supports; the ovalling mode occurs at mode 3 for Λ-85, at mode 4 for Λ-80 and Λ-75 types therefore with increase of the inclination angle of the supporting columns it happens at higher modes.

The rotational modes only observed for the cooling towers supported by I-Types of supports and experienced at mode 7 for I-90 type and at mode 1 for I-75 types.

5.1 Lateral modes and their influence on the tower system

For earthquake loading, the frequency corresponding to the first lateral mode of vibration is of primary interest, even if it is not the lowest value (Nasir *et al*. 2002). For I-Types of supports; first lateral mode in *X* direction occurs at mode 4 for I-90, at mode 5 for I-85 and I-80, and at mode 3 for I-75, with the periods of 1.928146, 1.764883, 1.622183, and 1.502802 second, respectively. The maximum period is for I-90 and the minimum is for I-75. It can be realized that the period of the first lateral mode of the tower from maximum to minimum is decreasing by 22% as the inclination of the supports of tower increases. It is of interest to mention that the cooling tower supported by the vertical columns is flexible than the tower supported by inclined supports and as much as the inclination degree increases the whole structure becomes stiffer. It can be realized that the stiffness of the structure system increases with increase in inclination of the supporting columns, resulting in decrease in the period (T).

For Λ-Types of supports; first lateral mode in *X* direction occurs at mode 6 for Λ-85, at mode 6 for Λ-80 and at mode 8 for Λ-75 with the periods of 1.493826, 0.986028 and 0.82326 second, respectively. The maximum period is for Λ-85 and the minimum is for Λ-75. The period decreases by 45% as the inclination increases from 5 to 15 degree. By comparing the periods of vibrations it can be realized that the flexibility of the structure system increases with decrease in inclination of the supporting columns, resulting in increase in the natural period (T) of the vibration of the system.

The variation of period of vibration and associated mode shape due to the change of the angle of the inclination of the supporting columns can significantly influence the dynamic response of

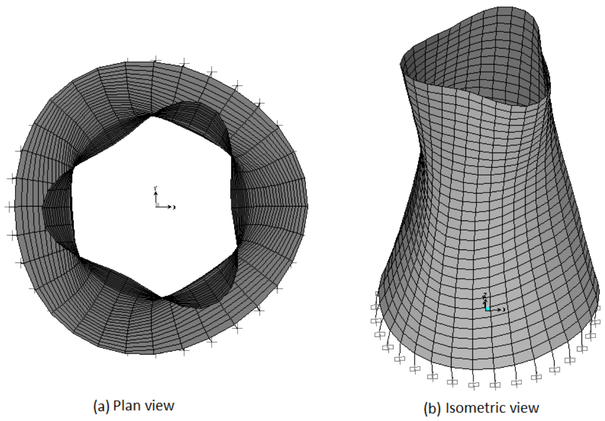

Fig. 4 Circumferential mode having three waves in the highest period of vibration

the entire structure and therefore the earthquake resistance of the cooling tower seems to be an interesting phenomenon. To study these influences the numerical results of modal analysis are represented and discussed in details for the highest periods of the vibrations.

5.2 Circumferential modes and their influence on the tower system

Results for the periods of vibration for the circumferential modes are also included in table 4. Circumferential modes provide meridian deformation and the circumferential deformation (waves) in the tower. Such modes are of prime importance in studying the cooling tower response under wind disturbances and the lowest frequency (highest period) of modes is of primary interest, even if it occurs for a high circumferential wave number (Bhimaraddi *et al*.1991, Nasir *et al*. 2002).

For I-type column supports the lowest frequency is for I-90 and the highest is for I-75. These modes occur at mode 1 for I-90, I-85, I-80 and at mode 2 for I-75. The period of the vibration decreases by 39%. Further observation reveals that, there are three circumferential waves in the fundamental mode (also referred to as mode having highest period) shape, as shown in Fig. 4. It is also interesting to note that the number circumferential waves can change for the different supporting system and even for the same supporting system at different modes of the vibration which is in agreement with the discussions made in references (Shupeng *et al*. 2013, Nasir *et al*. 2002).

For Λ-type column supports in the mode1 the lowest frequency is for Λ-85 and the highest is for Λ-75. The period of the vibration decreases by 22%. For Λ-type also there are three circumferential waves in the fundamental mode shape, as shown in Fig. 4.

It can be observed from the Fig. 4 that there is a reversal change in the nature of the waves along the height of the tower. The waves those are toward the centre at the bottom of the tower are outward at the top of the tower. To study the effect of the inclination of the supporting columns on the nature of the circumferential vibration modes; the modes for highest period and their relative displacement along the height of the tower with reference to the key diagram given in Fig. 5 are shown in Fig. 6 to Fig. 7. Circumferential waves are symmetric about each axis having 60^o with m1 shown in Fig. 5. There are three axes of symmetry at $\theta=0^o$, $\theta=120^o$ and $\theta=240^o$ for the tower having three circumferential waves.

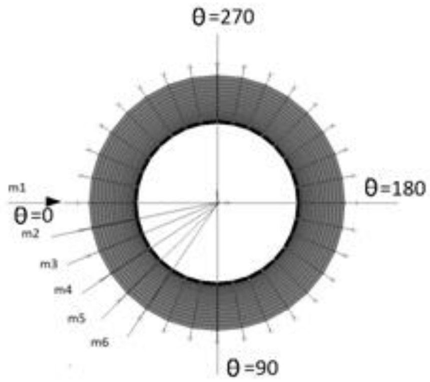

Fig. 5 Key diagrams to plot the axial modes varying by 11.25^{o} from m1 to m2

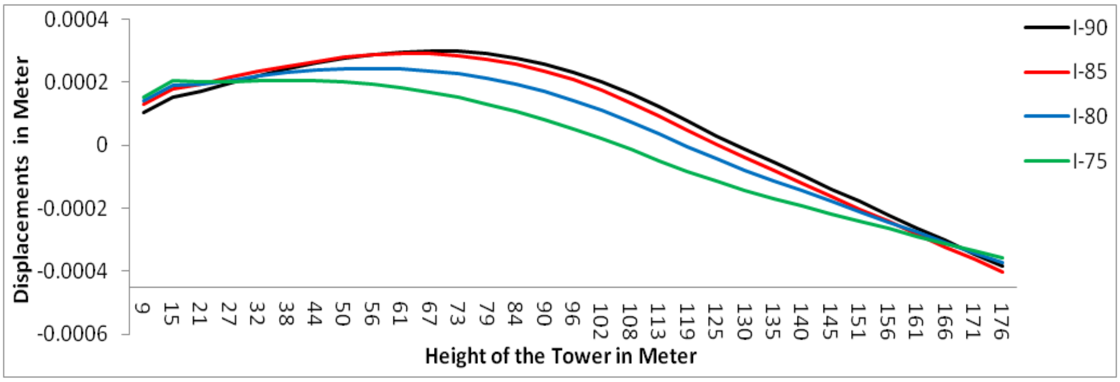

Fig. 6 Vibration modes for the highest period of the I-type column supports at m1

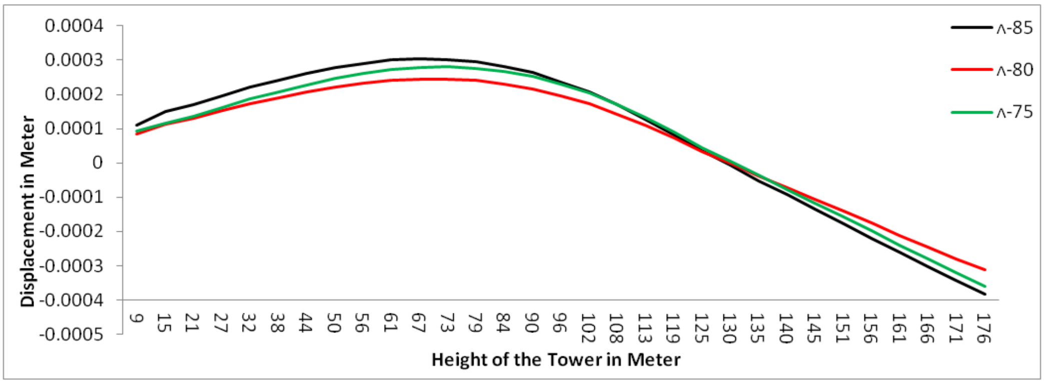

Fig. 7 Vibration modes for the highest period of the Λ-type column supports at m1

Mode shapes (of normal deflection) for the highest period in the meridian direction of the tower with the I-type column supports are shown in Fig. 6 (for m1 shown in Fig. 5). One may observe from the figure that I-90, I-85 and I-80 types of supports display almost identical mode shapes, however increase of the inclination angle of the supporting columns results towards flattening trend of the mode and reversal of the mode becomes more conspicuous for I-75 type of supports.

For the tower supported by I-types of the supports the location of the reversal point is shifting from almost throat level (125 m) toward the base at 104 m.

It is of interest to mention that more is the stiffness of the supporting columns less is the height of the reversal point (where the nature of the waves change) from the supports and the shell becomes less deformable. It can also be noted from this comparison that the tower shell with inclined supports will definitely produce a higher frequency.

Mode shapes (of normal deflection) for the highest period with the Λ-type column supports in the meridian direction of the tower are shown in Fig. 7 (for m1 shown in Fig. 5). For the towers supported by Λ-type columns, like the towers supported by I-type column supports, one may observe from the figure that all types of supports display almost identical mode shapes however with this Λ-type supporting column the reversing is conspicuous with Λ-80 type, not with Λ-75. For a tower supported by **Λ**-types of the supports the location of the reversal point is almost same for all the inclinations and it is at the height of about 130 meter which is above the throat level (125 m).

Studying the modal characteristics of the tower seems to be interesting and the results are encouraging as it may be realized that the dynamic response of the tower is highly sensitive to the change of the inclination angle of the supporting columns and significantly alters the modal characteristics of the total structure. It is very difficult to draw any definite conclusions from the comparison of mode shapes, periods and frequencies of the tower. But it may mention that the amount of increase or decrease in the frequency and periods of vibration on the bases of types of the supporting systems of the tower and even the angle of the supports. For higher inclination angles, period increases and the frequency decreases and vice versa. As it has been concluded that increase in the frequency and flatten the peaks in the mode shapes result in higher load-carrying capacity of the tower against the wind load (Bhimaraddi *et al*. 1991) implying that, the towers having I-75 and Λ-75 will be more efficient for wind loading.

As the inclination of the supporting columns are influencing the both circumferential and lateral modes of the structure they will alter the load carrying capacity of the tower against the wind load and therefore will affect the earthquake resistance of the tower. Both Wind and earthquake are dynamic loads in nature, however in the present work nonlinear time history analysis is carried out to study the dynamic response of the tower under seismic loading.

6. Nonlinear time history analysis results

The nonlinear time history analysis has been performed in SAP2000 by using the direct integration method to study the dynamic response of the structure for seismic loads by applying the ground accelerations at the base level of the structure. The recorded ground accelerations data given in Fig. 3 for 2001 earthquake of Bhuj city in India is used. Due to symmetric configuration of structure the accelerations have been applied only in X direction. The analysis is set to start from the unstressed state or zero initial condition and the time history motion type is set to be transient and the analysis is done for 15 seconds duration of the excitation for the time intervals of 0.005 seconds starting on 13 (sec) and ending on 28 (sec) from accelerogram shown in Fig. 3.

6.1 Radial displacements

Due to the symmetric configuration of the tower variation of the radial displacements along the

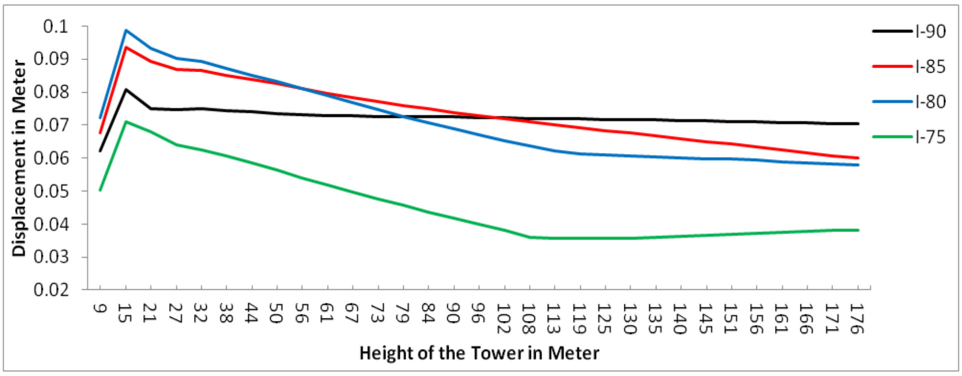

Fig. 8 Maximum radial displacements along the tower height at θ=0^o at m1 for I-type column supports

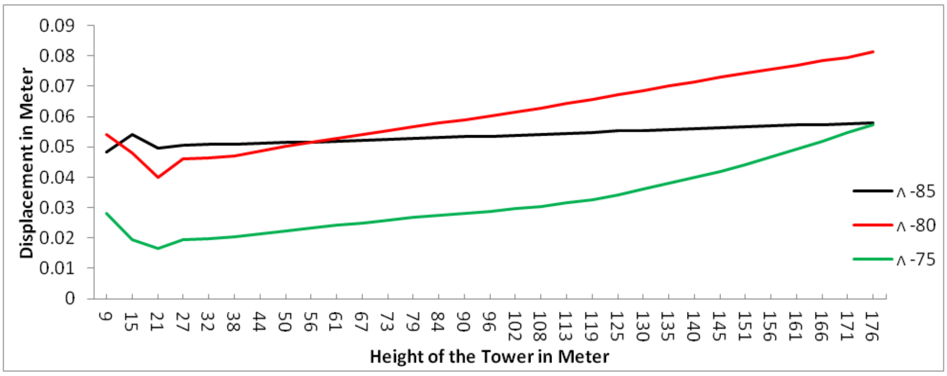

Fig. 9 Maximum radial displacements along the tower height at θ=0^o at m1 for Λ-type column supports

height of the tower are symmetric about the *X*- axis or direction of earthquake loading but not symmetric in *Y*- direction; therefore the displacements are plotted at 0^{o}shown in Fig. 8 and Fig. 9.

The variation of maximum radial displacements along the height of tower at θ= 0^o meridian (Earthquake direction) for towers with I-type column supports are given in Fig. 8. These are relative displacements toward the center of the tower. For the tower with I-90 type of the supports the displacement is almost constant from the height 21 meter up to its top. For all the cases the maximum displacement happens at the height of 15 meter, the largest value of which is for I-80 and smallest value for I-75, and above that the displacements are decreasing along the height of the tower. The trend demonstrates that increase of the inclination angle of the supporting columns is significantly influencing the deformed shape of the tower and as the total structure system becomes more stiff the displacements decrease in lower rates and for tower with I-75 supports which is the stiffest one, the displacements tend to increase above the height of 113 meter along the height of the tower. The tower with I-75 type supports has the minimum displacements, which is the stiffest as compared to the other cases, and it agrees with the results of the modal analysis. The numerical results reveal that for the towers with I-type column supports, if the tower shell is flexible at the bottom level it is stiffer at higher levels and with increase in the inclination angle of the supporting columns, stiffness of the tower shell at higher levels increases.

Fig. 9 shows the variation of the maximum values of the relative displacements toward the center with Λ-type column supports along the meridian direction of the tower in the direction of earthquake considered that is θ=0°. It can be seen from the figure that for all the cases the maximum displacement happens at the top of the tower shell which is the largest value for Λ-80 and smallest value for Λ-75 and the minimum displacements are at the bottom level at the height of 21 meter for all the cases. Again it can be observed that Λ-75 type which is the stiffest structure is having the minimum displacements. For the towers with Λ-type supports the displacements are increasing along the height of the tower and beyond the height of about 60 meter it becomes larger than Λ-85 for the tower having Λ-80 type supports in spite of being stiffer than Λ-85. The numerical results reveal that for the towers with Λ-type column supports, the tower shell is stiffer at the bottom level and flexible at higher levels. The stiffness at bottom level increases with increase in inclination angle of the supporting columns. It can be observed from Fig. 9 that, the rate of the increase in the displacement at the higher levels of the towers is increasing with increase in the stiffness at the bottom level.

It is difficult to conclude that what angle of the inclination will give the optimum structure as response of the whole structure system and even only the tower shell is significantly sensitive to change of the angle of the supporting columns; but one may be mentioned that as we were expecting from the modal analysis results, we have got smaller displacements for the stiffest structure which is confirmed with time history analysis results. Surprisingly there is a shift in the location of the maximum displacements for the supports considered that is, I-type column and Λ-type column supports. This is a unique observation in this study and it can be seen that for I-type column supports the maximum displacements are happening at the bottom level of the tower shell but for Λ-type column supports the minimum displacements are happening at bottom level. This states that as the supporting columns become stiffer the tower shell becomes more deformable at the higher levels. Another remarkable observation in this study is that, the change in the angle and even in the types of the supporting columns influences the nature of the deformation of the tower shell and even the participation of the stiffness of the tower in structural response of the tower; as it observed that for I-type column supports the tower is stiffer at top level and more flexible at bottom and opposite in the case of Λ-type column supports.

The other interesting observation is that the decrease in the inclination angle of the supporting columns decreases the stiffness of the structure which flattens the deformed shape of the tower shell. By comparing the Figs.8 and 9, it can be observed that the use of pair columns (i.e., Λ-type) having the same cross section area and same reinforcement percentage of a single I-type column makes the total structure more resistant against the seismic loading.

Another engaging observation can be made by comparing Fig. 8 and Fig. 9 is that for I-type and Λ-type supports the locations of the maximum displacements are significantly different; which is at the bottom for I-types and at the top for Λ-types. This implies that the cooling tower with I-type supports is flexible at bottom and stiffer at top whereas with Λ-type supports, is stiffer at bottom levels and flexible at top levels.

6.2 Membrane stresses

Membrane stresses are the meridian stresses along the height of the tower and hoop stresses in circumference of the tower shell. The maximum values of meridian and hoop tensile stresses are demonstrated for the elements located between the angles θ=0° to θ=11.25° (along the meridians m1 and m2) by referring to key diagram given in Fig. 5.

The Fig. 10 shows that the maximum meridian stresses for I-type column supports are at the bottom of the shell at the junction of the shell and supporting columns and is almost same for all

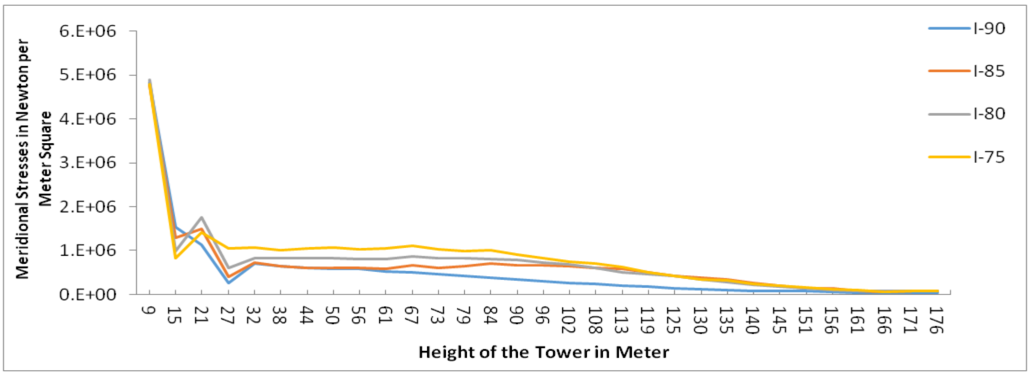

Fig. 10 Meridian stresses along the height of the tower at θ=0^{o} for I-type column supports

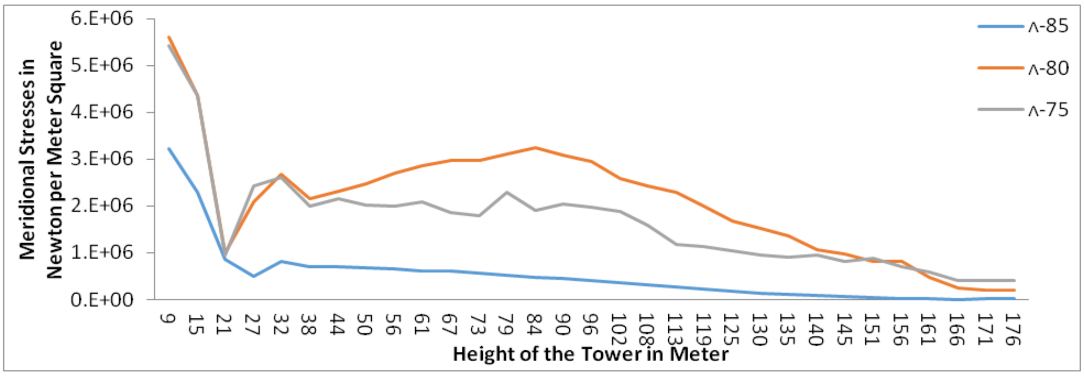

Fig. 11 Meridian stresses along the height of the tower at θ=0^{o} for Λ-type column supports

types of columns having the different angles. There is a sudden decrease in the meridian stresses after the junction (which is the location of stress concentration) between the tower and columns up to 15meter and then fluctuation from 15 to 32 meter in the stress values and then the stresses are gradually decreasing up to the top of the tower wherein the stresses are the minimum.

For the towers with Λ-types of the supports, it can be seen from the Fig. 11 that the maximum meridian stresses are at the junction of the tower and supporting columns which is almost same for Λ-85 and Λ-75 at this location and is almost 1.75 times that of for Λ-85. After the sudden decrease up to 21 meter, along the tower height values are the maximum for Λ-80 except after the height of 160.

Comparing the towers with I-type and Λ-types of the supports, it can be realized that Λ-types are more dependent of the change of the angle of the inclination of the supporting columns for meridian stresses.

It can be observed from the Fig. 12 that, the variation of the hoop stresses for I-Types of supports are more or less same for all the cases. The maximum values are at the base level of the towers. It also can be realized that after the height of 21meter the variation of the stresses are not much sensitive for the change of the inclination angle of the supporting columns. A comparison of the hoop stresses at the top of the towers reveals that, with increase in the inclination angle of the supporting columns the stresses increase at the top of the tower which is the maximum for I-75 and the minimum for I-90.

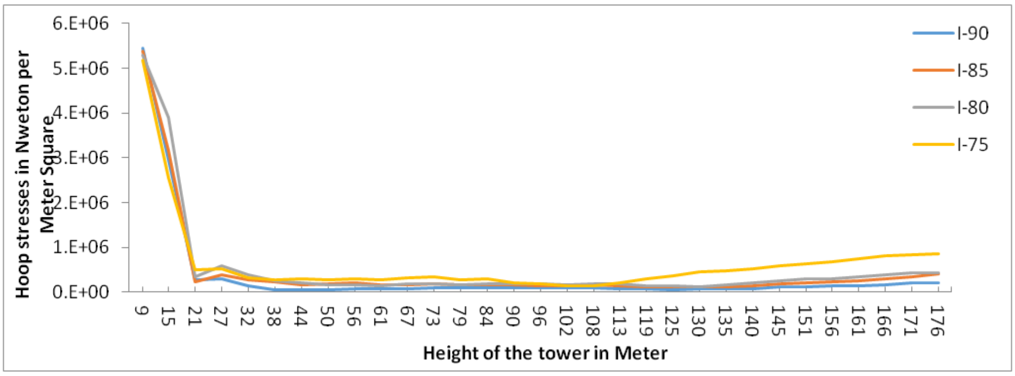

Fig. 12 Hoop stresses along the height of the tower at $\theta=0^o$ for I-type column supports

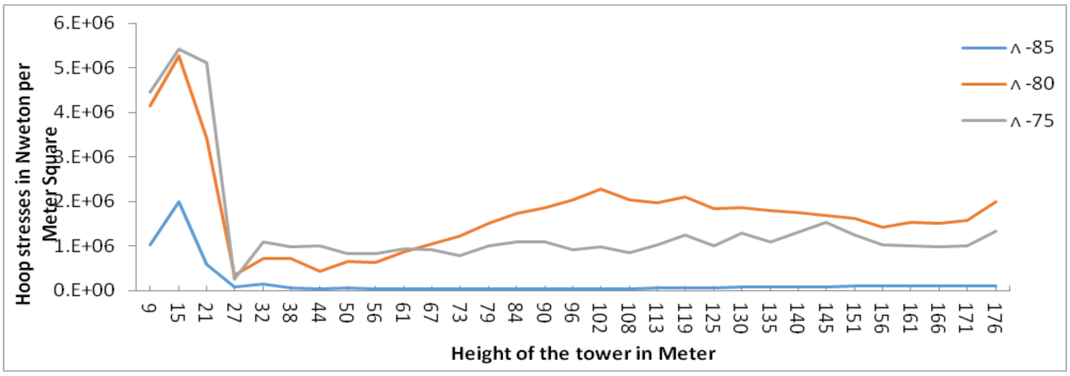

Fig. 13 Hoop stresses along the height of the tower at $\theta=0^o$ for Λ-type column supports

For towers supported on I-type columns from the Figs. 10 and 12, the maximum values of meridian and hoop tensile stresses exceeded the tensile strength of the concrete therefore the tower shell would be cracked (up to 5 meter from tower lintel). For all the towers supported on I-type columns, the cracks would appear at the base of the tower shell in the vicinity of the supporting columns. The cracks would be oblique and the angle of the cracks would be different for different supporting columns. For instance in the shell element above zero meridian I-90 (vertical) column, cracks would have their inclination angle of 140.3 degrees with tower lintel.

By comparing the trends in Fig. 13, it can be seen that for the towers supported by Λ-types of the supports, variation of the hoop stresses are much sensitive to the change of the inclination angle as compared to the I-Types. It can be seen that the maximum stresses are at the height of 15 meter which has been shifted by 6 meter as compared to the I-Types.

More investigation on the figures clarifies that, the stresses in the towers supported by Λ-columns are more sensitive to the change of the inclination angles as compared to the towers supported by I-Type columns. By comparing the peak magnitudes of the stresses for I-Types (5387937 N/m^2 for I-80) and Λ-types (5415475 N/m^2 for Λ-75) it can be realized that there is not much difference; But one may state that for I-Types the stresses along the height of the tower shell is less than those of for Λ-types.

For towers supported on Λ-type columns from the Figs.11 and 13, the oblique cracks (up to 10 meter from tower lintel) would appear at the base of the tower shell in the vicinity of the supporting columns for all the cases however for Λ-80 type the cracks would also appear at higher levels (from the height of almost 55 meter up to almost 97 meter) and again the angle of the cracks would be different for different supporting columns' angles. The cracks would also generate in the supporting columns for both I-type and Λ-types at the initial time of the earthquake before the generation of the first cracks in the tower shells.

It is also worth mentioning that the crack patterns and heights are highly sensitive to the types of the supporting columns and also the inclination angle of the supporting columns.

6.3 Base reactions

The base reactions experienced in the Bhuj earthquake are shown in Figs. 14 to 23 for nonlinear time histories and are depicted in global *X* direction for I-type and Λ-type of column supports. The maximum and minimum values of the base reactions in global coordinate system are represented in Table 5 for all the cases.

Base reactions are three force components (FX and FY are base shears in global *X* and *Y* directions and FZ is vertical force component in global *Z* direction) and three moment components (MX, MY and MZ are the moments about the global *X*, *Y* and *Z* directions) at the base of columns. Moment MZ produces the torsion in the supporting columns.

The nonlinearity started with the cracking of the concrete columns in tension due to the combination of biaxial bending and axial tensile force of the columns. The cracking of the columns happens at the different time steps for the different cases. For I-90, I-85, I-80 and I-75

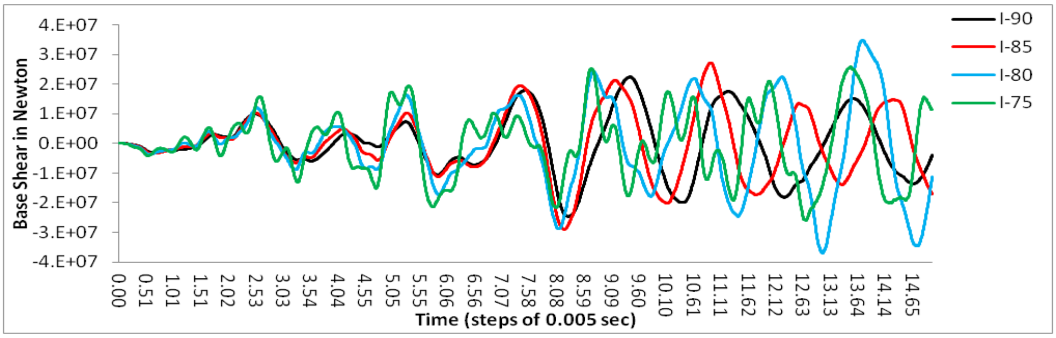

Fig. 14 Base shear force under earthquake excitation for I-type column supports

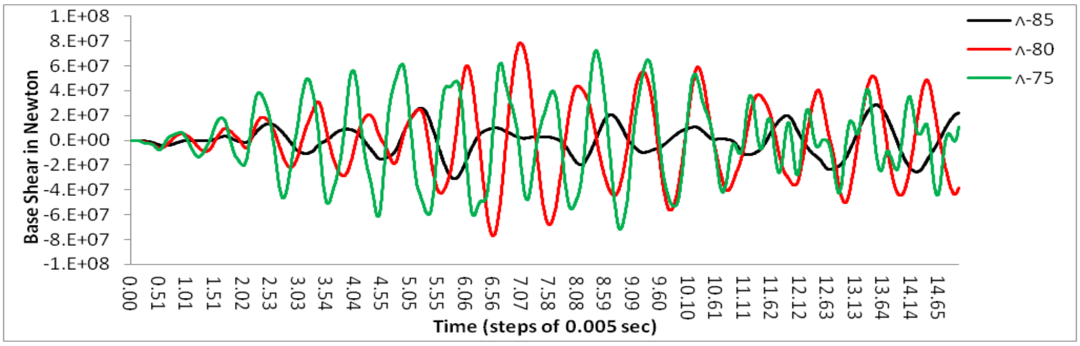

Fig. 15 Base shear force under earthquake excitation for Λ-type column supports

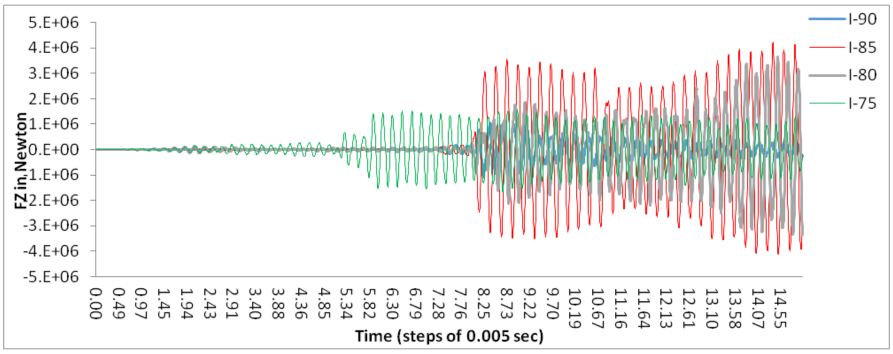

Fig. 16 Base reactions (FZ) for earthquake excitation, I-type column supports

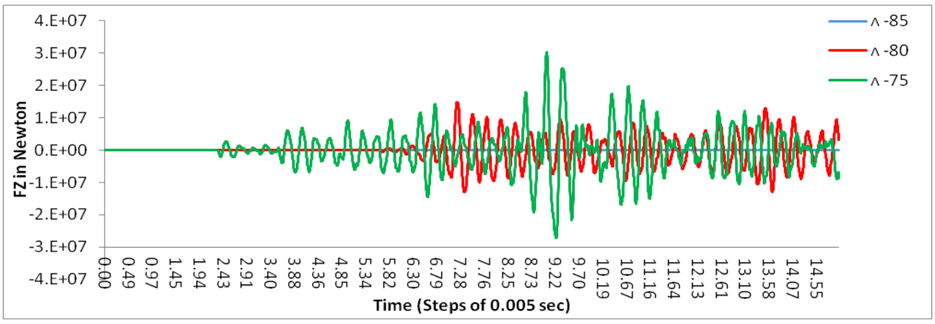

Fig. 17 Base reactions (FZ) for earthquake excitation, Λ-type column supports

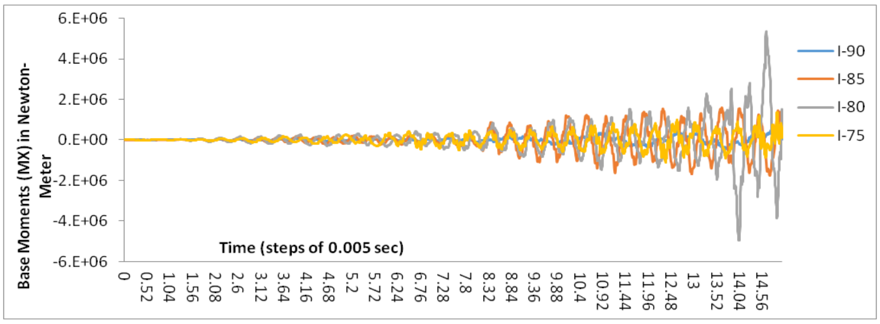

Fig. 18 Base reactions (MX) for earthquake excitation, I-type column supports

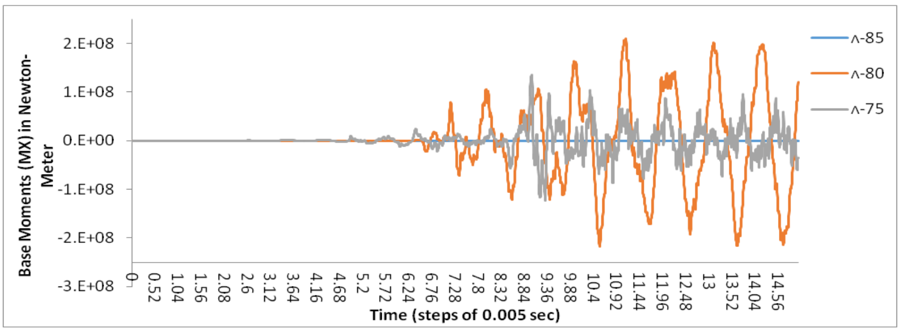

Fig. 19 Base reactions (MY) for earthquake excitation, Λ-type column supports

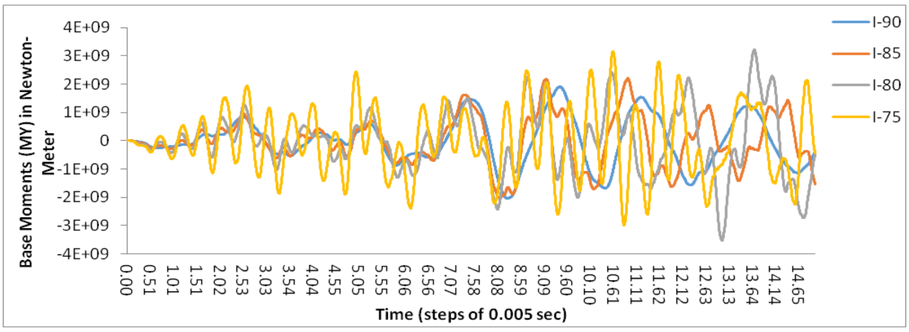

Fig. 20 Base reactions (MY) for earthquake excitation, I-type column supports

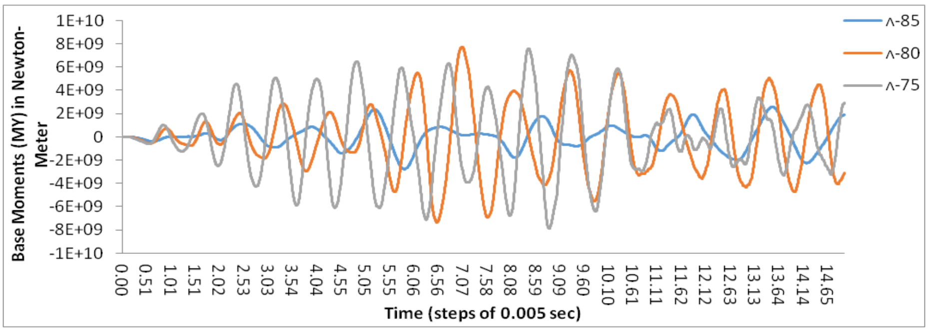

Fig. 21 Base reactions (MY) for earthquake excitation, Λ-type column supports

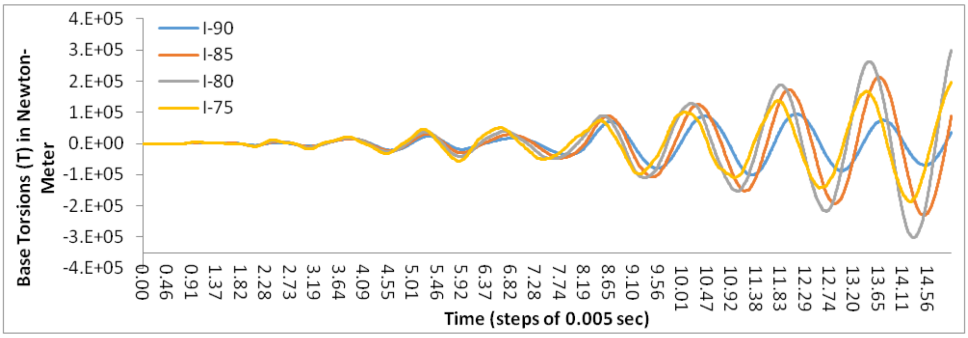

Fig. 22 Base torsions (T) for earthquake excitation, I-type column supports

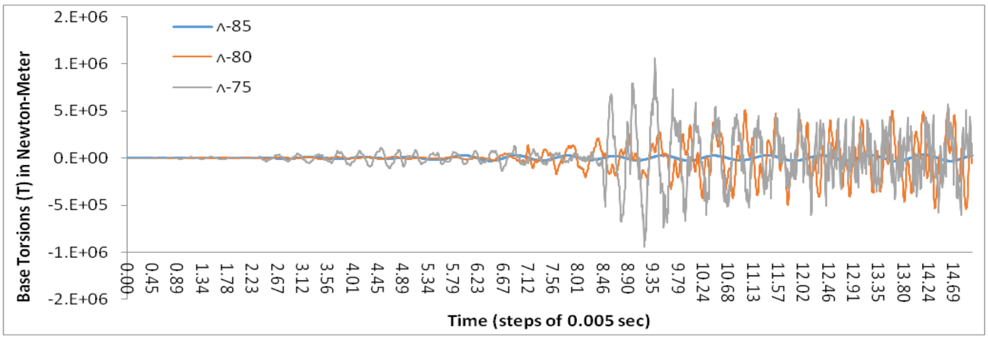

Fig. 23 Base torsions (T) for earthquake excitation, Λ -type column supports

Table 5 Maximum and minimum base reactions for different supporting columns

		Global FX	Global FY	Global FZ	Global MX	Global MY	Global MZ
		N	N	N	N-m	N-m	N-m
I-90	Max	2.24E+07	3.38E+03	1.05E+06	4.77E+05	1.91E+09	9.44E+04
	Min	-2.46E+07	-3.56E+03	-1.07E+06	-4.56E+05	-2.04E+09	-1.01E+05
I-85	Max	2.72E+07	6.00E+03	4.22E+06	1.58E+06	2.21E+09	2.13E+05
	Min	-2.90E+07	-5.54E+03	-4.11E+06	-1.74E+06	-2.04E+09	-2.30E+05
I-80	Max	3.47E+07	2.30E+04	3.64E+06	5.35E+06	3.23E+09	3.00E+05
	Min	-3.70E+07	-2.74E+04	-3.35E+06	-4.97E+06	-3.51E+09	-3.02E+05
I-75	Max	2.57E+07	5.12E+03	1.60E+06	1.36E+06	3.15E+09	1.96E+05
	Min	-2.59E+07	-6.84E+03	-1.67E+06	-1.11E+06	-2.98E+09	-1.87E+05
Λ-85	Max	2.85E+07	7.11E+02	1.23E+05	2.50E+05	2.58E+09	3.72E+04
	Min	-3.11E+07	-6.72E+02	-1.18E+05	-2.36E+05	-2.77E+09	-3.52E+04
Λ-80	Max	7.85E+07	1.98E+06	1.47E+07	2.10E+08	7.73E+09	5.13E+05
	Min	-7.69E+07	-1.97E+06	-1.30E+07	-2.18E+08	-7.36E+09	-5.46E+05
Λ-75	Max	7.24E+07	1.14E+06	3.04E+07	1.35E+08	7.58E+09	1.06E+06
	Min	-7.17E+07	-9.51E+05	-2.72E+07	-1.22E+08	-7.90E+09	-9.43E+05

types, the nonlinearity starts after 0.43, 0.42, 0.42 and 0.42 seconds, respectively and for Λ-85, Λ-80 and Λ-75 types, the nonlinearity starts after 0.445, 0.475 and 0.78 seconds, respectively.

The stresses in the concrete columns are recorded along the depth. Whenever the tensile stress at a particular fiber exceeds the tensile strength of the concrete, the effective depth of the columns in finding out the moment of inertia is modified to take into account the depth of the crack. For this purpose the column is divided into number of elements and for each element the tensile stresses are recorded. For calculating the moment of inertia of the elements, transformation of moment of inertia of each element after cracking to a common neutral axis is made.

Maximum and minimum values of the base reactions for the cooling towers with the different supporting columns are represented in Table 5. The maximum values are given by positive sign and the minimum values by negative sign in the table.

Numerical results reveal that, the base reactions are very sensitive to the change of the inclination angle of the supporting columns. By comparing the figures it can be observed that, values of the base reactions are not consistent with increasing inclination of the supporting columns during the earthquake.

By comparing the numerical values given in Table 5, it can be noticed that for the towers with I-type column supports, the smallest base reactions are for the I-90 supporting columns and the greatest are for I-80 however, for the towers with Λ-type column supports, the smallest base reactions are for the tower with Λ-85 supporting columns and the greatest are for Λ-80.

It is of interest to mention that the greatest values (for both positive and negative signs) of the base reactions are for Λ-type column supports as compared to those of for the I-types.

7. Conclusions

Numerical results of analyses of the cooling towers supported on the discrete supporting columns have been carried out to study the influence of the inclination angle of the supporting columns on the dynamic response of the cooling tower structure. The modal and nonlinear time history analyses are undertaken in this study and it has been observed that, the dynamic response of the total structure is quite sensitive to the type of the supporting columns which are I-type column and **Λ**-type column supports and also to the change of the inclination angle of these supporting columns. In practical design either earthquake or the wind governs the design criteria. The findings are of great consequence to the safe design of the cooling towers against earthquake excitations. The numerical results of modal analysis show that the change of the angle of the inclination in each type of the supporting columns significantly alters the characteristics of both circumferential and lateral modes and therefore the resistance of the cooling tower against the earthquake loading as it has been observed that the stiffness of the structure system increases with increase in inclination angle of the supporting columns, resulting in decrease of the period of the structural system. It also has been observed from the numerical results that the location of the maximum displacements in the tower shell is very sensitive to the types of the supporting columns considered in this study. It is also conscious from numerical results that the participation of the stiffness of the tower in structural response of the cooling tower are dependent of the change in the angle and even in the types of the supporting columns; as it has been noticed that for I-type column supports the tower is stiffer at top level and flexible at bottom whereas it is stiffer at bottom level and flexible at the top for **Λ**-type columns. With increase in the stiffness of the supports, the tower shell becomes more deformable at the higher levels. For all kinds of supports (four cases for I-type columns and three for **Λ**-type columns), the nonlinear time history results are in good agreement with the results of modal analyses and that the towers with **Λ**-type column supports are more efficient for earthquake loading as they have the minimum period and minimum displacements. Therefore, it is concluded that the hyperbolic structure of the cooling tower can be optimized by finding the optimum inclination angle of the supports.

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