

# Progressive Collapse of Hyperbolic Cooling Tower Considering the Support Inclinations

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**Abstract**—Progressive collapse of the layered hyperbolic tower shells are studied considering the influences of changes in the supporting columns' types and angles. 3-D time history analyses employing the finite element method are performed for the towers supported with I-type and  $\Lambda$ -type column. It is found that the inclination angle of the supporting columns is a very important parameter in optimization and safe design of the cooling towers against the progressive collapse. It is also concluded that use of Demand Capacity Ratio (DCR) criteria of the linear elastic approach recommended by GSA is un-conservative for the hyperbolic tower shells.

**Keywords**—Progressive collapse, cooling towers, finite element analysis, crack generation, reinforced concrete.

## I. INTRODUCTION

**P**ROGRESSIVE collapse is the mechanism in which the entire structure or part of it fails due to the sudden failure of one or more load carrying members. The sudden failure could be a result of blast, vehicle impact, terrorist bomb attack, gas explosion, foundation failure, fire, and seismic forces [1]. In this mechanism after the instantaneous removal of structural member(s), re-distribution of load carried by the members removed causes the failure of other structural elements, until the complete or partial failure takes place.

Instantaneous removal of the structural member releases internal energy, disturbing the initial load equilibrium of external loads and internal forces therefore initiating the vibration of the structure until a new equilibrium sets up or the structure collapses. Being a dynamic and nonlinear event, progressive collapse involves inelastic deformation. Detailed information regarding analysis procedures, methodologies and guidelines to resist progressive collapse is readily available in documents issued by the American Society of Civil Engineers (ASCE) [2], Federal Emergency Management Agency (FEMA) [3], [4], General Services Administration (GSA) [5], the Department of Defense (DoD) [6], British Standards Institute (BSI) [7], [8], and National building code of Canada (NBC) [9].

Collapse of the Ronan Point in Britain in 1968 attracted civil engineers' attention to study the progressive collapse; however, it could not become the problem of interest to the researchers at that time. Recently, interest in this topic has increased as a result of the terrorist attacks against the Alfred P. Murrah Building in Oklahoma City in 1995 and the World

Trade Center in New York in 2001.

Many studies have been carried out in the area of progressive collapse of steel and reinforced concrete buildings in the last few years. Kaewkulchai and Williamson [10] employing the dynamic analysis have studied the impact of failed members on other portions of a five-story, two-bay frame. In this study, it has been concluded that the impact velocity plays the most significant role in causing failure of intact beam elements below the initial column failure location. Masoero et al. [11] studied the progressive collapse of three-dimensional reinforced concrete framed structure after the sudden loss of a column. Performing the simulations using the discrete element method considering inelastic collisions between the structural elements, their results indicated that what collapse initiation and impact-driven propagation mechanisms were activated in structures with different geometric and mechanical features.

Some studies have concentrated on the methods of modeling, analysis, their advantages and limitations and design strategies involved with the progressive collapse. Marjanishvili [12] presented four successively more sophisticated analysis procedures for evaluating the progressive collapse hazard: linear-elastic static; nonlinear static; linear-elastic dynamic; and nonlinear dynamic. In this study the advantages and disadvantages of each method was discussed. It has been concluded that the linear elastic static, linear elastic time history and non-linear time history analyses are the most effective analyses procedures for progressive collapse and the simplest analysis methodology includes static linear elastic procedure, and the most exhaustive procedure is nonlinear time history analysis.

Two years later, in 2006, Shalva and Agnew [13] gave clear step-by-step descriptions of four methods for progressive collapse analysis using commercially available structural analysis software SAP2000 and compared these methods by analyzing a nine-story steel moment-resistant frame building. They demonstrated that the dynamic analysis procedures not only yield more accurate results, but are also easy to perform for progressive collapse determination. In their study, it has also shown that the current GSA performance limits for linear analysis procedures are un-conservative. Menchel et al. [14] have discussed and compared the different linear and nonlinear procedures of progressive collapse simulation given by the GSA procedure, the DoD procedure and the Load History Dependent Procedure (LHD) procedure. Yasser Alashker et al. [15] using the explicit finite-element code LS-DYNA simulated the collapse of a 10-story seismically designed steel building.

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In the studies carried out for the progressive collapse, attention is mainly given to the buildings. Thin walled hyperbolic cooling towers being important parts of power plants have not gained much of interest. Nevertheless, these types of structures are also prone to sudden removal of the supporting columns; whatever the cause is can lead to the instability, loss of integrity and finally failure of the entire structure. Krätzig and Zhuang [16] numerically simulated the collapse of a cooling tower subjected to both gravity load and wind load. This research evaluated only the safety and reliability of the cooling tower and did not consider the entire process of the collapse. In the coming years, a new cooling tower with an overall height of 235 m would be built as a part of the construction of a planned nuclear power station in southern China. Feng et al. [17] followed a comprehensive approach for prediction of the ground vibration due to the collapse of this super large tower supported by 120 columns of 18m height at the bottom under two accidental loads. The accidental loads considered in this study were the sudden removal of 60 columns which could be a result of a bomb attack or foundation settlement and another was extremely strong wind load with 44.3 m/s velocity. The details of the cooling tower, the structural model as well as the collapse simulation were described by Li et al. [18]. A year later, in 2014, Yi and his co-authors [19] conducting dynamic finite element analysis continued their study on the same tower considering the nonlinear material models. In this study the focus was made on the modes and mechanisms behaviour of the collapse of the towers subjected to different accidental loads. Vehicle collision, airplane impact, local explosion and missile attack were the four simulated accidental loads to study the failure of the tower. It was found that vehicle collision, missile attack and small TNT equivalent explosives (2 kg, 20 kg, and 200 kg) might result in local failure of the cooling tower; however, the tower can still keep stable. On the other hand, large TNT equivalent explosives (2000 kg, 4500 kg) could cause severe damages in the inclined columns of the cooling tower, and lead to progressive collapse of the entire cooling tower. The airplane impacted at the throat of the cooling tower caused the local failure of shell structure of the tower, and then the progressive collapse of the cooling tower happened due to the gravitational action.

According to the available literature in the analysis and design of cooling towers supported by discrete supporting columns, it has been concluded that the structural response and load carrying capacity of these structures are strongly dependent on their supports [20]-[22]. Any change in the type, location and inclination of the supporting columns significantly alters the behaviour of the entire system therefore it plays an important role in the design of safe, reliable and economical structure. In the present paper, influences of changes in the supporting columns' types and angles on the progressive collapse mechanism of the layered hyperbolic cooling towers are studied.

## II. FINITE ELEMENT MODELLING AND SIMULATION

In the present paper, the hyperbolic cooling tower studied by [20]-[23] is considered. This cooling tower is of 176 meter height, and having the base, throat (located at the height of 125 meter) and top radius of 58.72, 36.6 and 38 meter, respectively. The tower shell thickness is varying along its height from 1.05 meter at the base to 0.24 meter at throat level and again reduces to 0.2 meter at the top. Cooling tower shell is supported on two different supports namely I-type columns (32 circular columns) and the  $\Lambda$ -Type columns (64 circular columns) of 9.17 meter vertical height. Full detail of geometry and material properties of the tower is given in [20], [22].

In recent years, with the rapid development of computational numerical simulation technique using the available commercial software packages has been applied to describe the collapse of structures, e.g., the collapse of cooling towers under blasting demolition [16] and the collapse of the World Trade Center after being hit by the planes [24]. In this study, modeling and analysis is carried out using the SAP2000 Ver 14, finite element software package. The cooling tower is modeled as the layered shell elements by defining the different layers of reinforcements and concrete by considering the material nonlinearity of each layer. This element is the 4 noded 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 at each node. The shell elements are having 2 reinforcement layers at inner face, 2 reinforcement layers at outer face and one concrete layer having the thickness of 0.3 m (i.e. totally 4 layers of reinforcement and 3 layers of concrete are used). Two noded line elements having 6 degrees of freedom at each node are used to model the supporting columns.

In this investigation, 11 models (Figs. 1 and 2) are created for various angles of inclination of the columns. Out of these, five models are with I-type columns and six are with  $\Lambda$ -type columns. These two groups of models are identical with respect to cross sectional geometry of their elements and materials. However, inclination angle of the supporting columns has been varied. To have consistent results the tower shells have been discretized into 32 elements in circumferential direction and into 30 elements in meridian direction.

For I-type columns namely, I-90, I-85, I-80, I-75 and I-T are considered. The columns are having the 0o, 5o, 10o, 15o and 18.31o degrees inclination towards the axis of the tower, respectively. However, the I-T type columns are tangential to the respective meridian at the base of the tower shell.

For  $\Lambda$ -type columns, each column of the  $\Lambda$  pair in  $\Lambda$ -85 type having the inclination of 5o in circumferential direction of the tower thus having the angle of 10o between the columns of the  $\Lambda$  pair and therefore this angle would be 20o and 30o degrees for  $\Lambda$ -80 and  $\Lambda$ -75 types, respectively.  $\Lambda$ -75-T type columns are inclined towards the axis of the tower with their angle bisectors tangential to the respective meridian at the base of the shell. Thus  $\Lambda$ -75-T type columns are inclined circumferentially and towards the axis of the shell. The  $\Lambda$ -J-Z

type columns are only circumferentially inclined but joined at the foundation level. The  $\Lambda$ -J-T type columns are also the  $\Lambda$ -J-Z type but inclined towards the axis of the tower having the angle bisector tangential to the respective meridian at the base of the tower shell. The material properties for concrete and steel used in these models are given in Table I.

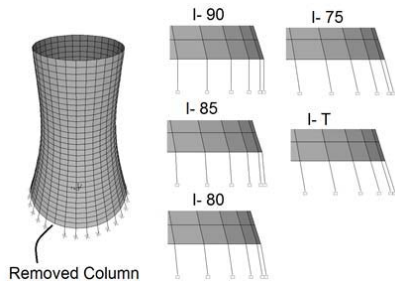


Fig. 1 Towers supported on I-type columns

For  $\Lambda$ -type columns, each column of the  $\Lambda$  pair in  $\Lambda$ -85 type having the inclination of  $50^\circ$  in circumferential direction of the tower thus having the angle of  $100^\circ$  between the columns of the  $\Lambda$  pair and therefore this angle would be  $200^\circ$  and  $300^\circ$  degrees for  $\Lambda$ -80 and  $\Lambda$ -75 types, respectively.  $\Lambda$ -75-T type columns are inclined towards the axis of the tower with their angle bisectors tangential to the respective meridian at the base of

the shell. Thus  $\Lambda$ -75-T type columns are inclined circumferentially and towards the axis of the shell. The  $\Lambda$ -J-Z type columns are only circumferentially inclined but joined at the foundation level. The  $\Lambda$ -J-T type columns are also the  $\Lambda$ -J-Z type but inclined towards the axis of the tower having the angle bisector tangential to the respective meridian at the base of the tower shell. The material properties for concrete and steel used in these models are given in Table I.

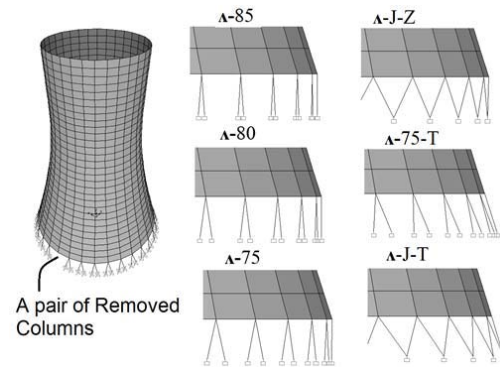


Fig. 2 Towers supported on  $\Lambda$ -type columns

TABLE I  
MATERIAL PROPERTIES

Concrete		Reinforcement		Supporting Columns	
Elastic Modulus	34GPa	Elastic Modulus	206GPa	Moment of Inertia (I)	0.0597m <sup>4</sup>
Poisson's Ratio	0.167	Yield Stress	500MPa	Area	0.8659m <sup>2</sup>
Density	0.0023kg/cm <sup>3</sup>	Tensile Stress	750MPa	Distance from Neutral Axis	0.525
Compressive Strength	36MPa	Poisson's Ratio	0.167	Elastic Section modulus (Z)	0.1136m <sup>3</sup>
Tensile Strength	2.7Mpa				

### III. LINEAR STATIC ANALYSIS

To perform the analysis, GSA progressive collapse guidelines [5] have been followed. GSA mandates several loss scenarios for the sudden removal of the structural elements, however according to these guide lines only one element is required to be removed at a time. GSA guidelines recommend alternate load path method of evaluating the progressive collapse potential. It means that the cause of the element failure has not been taken into account. It has only stated that the element is suddenly unable to carry the load which gives the advantage that the results are independent of the initiating load therefore, the solution would be valid for any abnormalities causing the structural member loss [12], [13]. Knowing that the sudden removal of the columns is a dynamic event the combination of dead and live loads given in (1) are multiplied by a factor of two as dynamic amplification factor to include the dynamic response when doing the static analysis.

For static analysis procedure:

$$\text{Load} = 2 \times (\text{DL} + 0.25\text{LL}) \quad (1)$$

In accordance with one of GSA's required scenarios, one column is removed from I-type columns (i.e. 31 columns are intact) and a pair from the  $\Lambda$ -Type columns (62 columns are intact). Being an axisymmetric problem the location of the column removed would not affect the analyses results. 3-D finite element models of the tower after removal of the column are given in Figs. 1 and 2. In this study the dead load is considered as the self weight of the reinforced concrete (unit weight of 23 kN/m<sup>3</sup>) members of cooling tower structure. In the present case the live load is not considered because it will only increase the magnitude of the applied load and will not influence the collapse mechanism.

### IV. DISCUSSION OF THE RESULTS

#### A. Demand Capacity Ratio (DCR)

To investigate the influence of the change in inclination angle of the supporting columns the DCR values are calculated for the columns next to the removed ones. Knowing the fact that the acceptable DCR values in GSA are given for primary and secondary structural elements of buildings and not for cooling towers, therefore only the concept is applied here. Likewise, if calculated DCR values after the removal of

columns for higher inclination angle increases it means that the potential for progressive collapse becomes higher (i.e. higher is the DCR, more is the potential for progressive collapse).

As per GSA 2003, the acceptance criteria in linear static approach for structural components are given by (2):

$$DCR = \frac{Q_{UD}}{Q_{CE}} \quad (2)$$

In (2),  $Q_{UD}$  is the action forces (axial, shear, moment and other possible combinations) or determined demand forces and  $Q_{CE}$  is the expected ultimate capacity (un-factored capacity or theoretical capacity) for the components. Due to the self weight of the structure, the supporting columns of the cooling tower are experienced axial, torsion and two components of shear force and bending moments. To evaluate  $Q_{CE}$ , the section designer available in SAP2000 is used. In section designer, reinforced concrete column of circular section (supporting columns) is modeled and interaction curves have been plotted using the Caltrans Interaction Surface command. Using this command, the code specified strength reduction factors are ignored (i.e., set to 1.0) when creating the interaction curves, therefore the un-factored capacity of the column can be evaluated. The un-factored capacity obtained from the software is verified for the case of axial load by hand calculation using the theoretical ultimate strength given in (3).

$$P_n = 0.85f_c(A_g - A_{st}) + f_y A_{st} \quad (3)$$

Using (3) the evaluated ultimate capacity of an axially loaded column is  $P_n = 2.84 \times 10^6$  N and using the section designer  $P_n = 2.82 \times 10^6$  N, which indicates that the difference is negligible.

Supporting columns are under combined action of axial forces and biaxial bending moments. To make the comparisons understandable, the axial capacity ( $=17650372$  N) and resultant biaxial bending moment capacities (BMC, sum of the square root of the uniaxial bending capacity  $=\sqrt{M_{ux}^2 + M_{uy}^2} = 4806534$  Nm) for the balanced section of the columns are computed. To obtain the DCR values, axial forces and biaxial moments are calculated separately. The calculated DCR are presented in Table II.

Comparison of the DCR values given in Table II for I-type columns shows a decrease in the values with increase in the inclination angle of the supporting columns. For supporting columns I-90 to I-T, the DCR decreases by almost 10 % for axial forces and by almost 92 % for resultant moments. The rate of decrease for DCR values calculated for axial forces and resultant bending moments are different (higher for resultant moments). From the numerical results represented in Table II it can be seen that all the demand forces (axial and shear in x and y directions) and moments (torsion and bending) in the supporting columns decrease with increase in their inclination angles, therefore the structure becomes safer. Further investigation in the columns compressive stresses also confirms that increase of inclination angle decreases the progressive collapse potentiality. These observations in the columns' forces and calculated DCR values from linear static analyses lead to a general statement that, for the cooling tower of the present study more is the inclination angle of the I-type supporting columns toward the axis less is the potential for the progressive collapse. Increase in inclination angle towards the axis of the tower of the I-type supporting columns strengthens the columns more significantly in the bending than under axial compression.

TABLE II  
DCR VALUES

Types of Supports	P	Vx	Vy	T	Mx	My	M Resultant	DCR (axial)	DCR (Resultant Moments)	Compressive Stresses
	N	N	N	N-m	N-m	N-m	N-m			N/m <sup>2</sup>
I-90	-3.4E+07	-1.56E+06	-5.84E+05	-2.76E+06	-2.55E+06	-6.71E+06	7.18E+06	1.91	1.49	-1.20E+08
I-85	-3.3E+07	1.12E+06	1.97E+05	-2.48E+06	9.64E+05	4.78E+06	4.88E+06	1.87	1.02	-8.88E+07
I-80	-3.2E+07	5.90E+05	1.28E+05	-2.03E+06	7.18E+05	2.45E+06	2.55E+06	1.82	0.53	-6.50E+07
I-75	-3.1E+07	9.43E+04	5.89E+04	-1.23E+06	3.99E+05	3.12E+05	5.07E+05	1.74	0.11	-4.18E+07
I-T	-3E+07	-1.02E+05	2.87E+04	-6.92E+05	2.18E+05	-5.10E+05	5.55E+05	1.72	0.12	-4.16E+07
A-85	-1.6E+07	-1.87E+05	9.16E+05	-1.62E+06	4.68E+06	-8.12E+05	4.75E+06	0.88	0.99	-6.63E+07
A-80	-1.4E+07	-2.18E+05	8.17E+05	-1.49E+06	4.41E+06	-8.94E+05	4.50E+06	0.82	0.94	-6.34E+07
A-75	-1.4E+07	-2.48E+05	7.34E+05	-1.34E+06	4.17E+06	-9.79E+05	4.29E+06	0.79	0.89	-6.14E+07
A-J-Z	-1.5E+07	-3.69E+05	5.49E+05	-9.66E+05	3.63E+06	-1.45E+06	3.91E+06	0.83	0.81	-6.16E+07
A-75-T	-1.6E+07	-1.78E+05	-2.52E+04	-2.58E+05	-1.20E+05	-6.60E+05	6.71E+05	0.89	0.14	-2.50E+07
A-J-T	-1.7E+07	-2.35E+05	-2.06E+04	-2.58E+05	-2.46E+04	-7.92E+05	7.92E+05	0.97	0.16	-2.69E+07

From the values given in Table II for the towers supported on A-type columns, there is a decrease in DCR and compressive stresses from A-85 to A-75 with increase of inclination angle in circumferential direction of the tower. Despite the decrease in progressive collapse potentiality from A-85 to A-75, it can be observed that the further increase of inclination angle of the supporting columns in circumferential

direction slightly increases both the DCR values and compressive stresses.

Further investigation from Table II for the towers supported on A-type columns reveals that, for A-75 type having the lowest DCR and compressive stresses further increase of inclination angle toward the axis of tower leads to increase of these values. Surprisingly it is observed that for the towers



supported on a pair of columns (i.e.  $\Lambda$ -type) increase of inclination angle toward the axis of the tower does not improve the capacity against progressive collapse as in the case of towers supported on a single column (i.e. **I**-type).

The observations made from the numerical analyses lead to a general statement that for the towers supported on single columns increase of inclination angle towards the tower axis and for the towers supported on a pair of columns only increase of inclination angle in circumference of the tower improves the safety of the total structure against the progressive collapse.

To validate the applicability of GSA formulas given in (2), resultant compressive stresses generated in the columns are studied. From the numerical results it can be realized that the compressive stresses in all the supporting columns for all types of the supports exceeded the compressive strength of the concrete (given in Table I), therefore the columns concrete is crushed.

In the present study of the cooling tower supported by **I**-type and  $\Lambda$ -type columns, DCR values are found to be

considerably less than the value recommended by GSA guidelines [5] for the framed or flat plate structures but resultant compressive stresses of the columns exceed their permissible values. These anomalous findings challenge the completeness of the GSA guidelines and therefore open up new chapter of research on safety of structure under progressive collapse.

### B. Resultant Displacements

In 3-D analysis, due to the self of the tower shell the resultant displacements are one vertical and two radial components along global Z, X and Y directions, respectively. In the present analyses the column along the X axis is removed (Figs. 1 and 2), therefore the resultant displacement in X-Z plane have been considered. The resultant displacements (sum of the square roots of the radial and vertical displacements) along the height of the towers are illustrated in Figs. 3 and 4. These are plotted for the nodes of the tower shells located along the meridian of the removed columns.

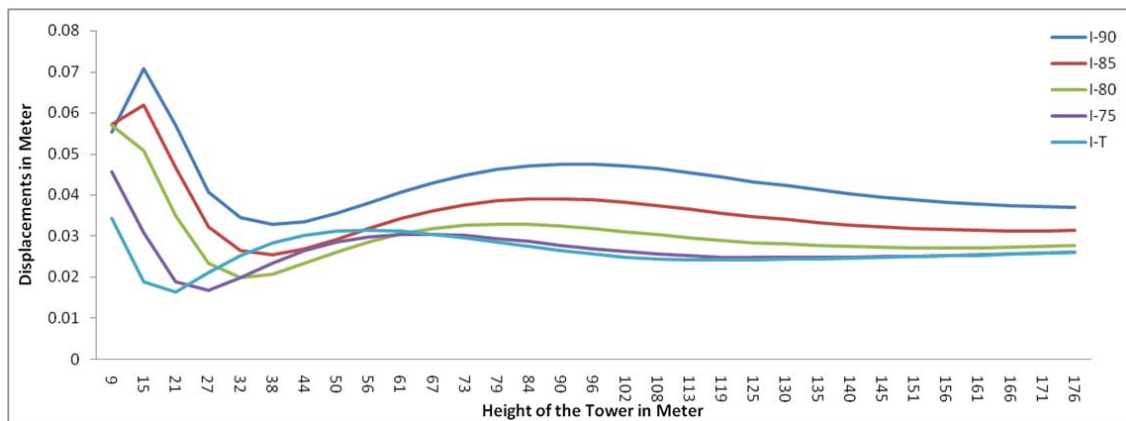


Fig. 3 Resultant displacements along the tower height for **I**-type supports

Comparison of the trends shown in Fig. 3 for the towers supported on **I**-type columns reveals that, from **I**-90 to **I**-T the maximum radial displacements decreases with increase of inclination angle of the supporting columns toward the axis of the tower (as it was indicative from DCR variation). Further investigation clarifies that the location of maximum displacements is dependent of inclination angle of supporting columns. The maximum resultant displacements are 0.071, 0.062, 0.057, 0.046 and 0.034 meter for **I**-90, **I**-85, **I**-80, **I**-75 and **I**-T, respectively. Location of the maximum displacements changes from the height of 15 meter (for **I**-90 and **I**-85) to the base (for **I**-80 and **I**-75) and to the height of 50 (for **I**-T). It is also worth mentioning that displacements are in the form of the waves (concave-convex) along the meridian of the tower shell.

### C. Failure of the Tower Shells

The tower shell failure could be either tensile or compressive. In the form of the tensile failure, the cracks are generated in the reinforced concrete due to the excessive tensile stresses beyond the tensile strength of the concrete shell elements. In the form of the compressive failure, the reinforced concrete shell elements are crushed due to the excessive compression beyond the compressive strength. To evaluate the failure of the towers, the principal stresses (shown in Fig. 5) are presented in Tables III and IV for the shell elements along the height of the towers.

In Fig. 5 the transverse shear stresses are not shown. The direct stresses shown in Fig. 5 are either tensile (positive) or compressive (negative) and represent the magnitudes on the top (outer) surface of the elements.

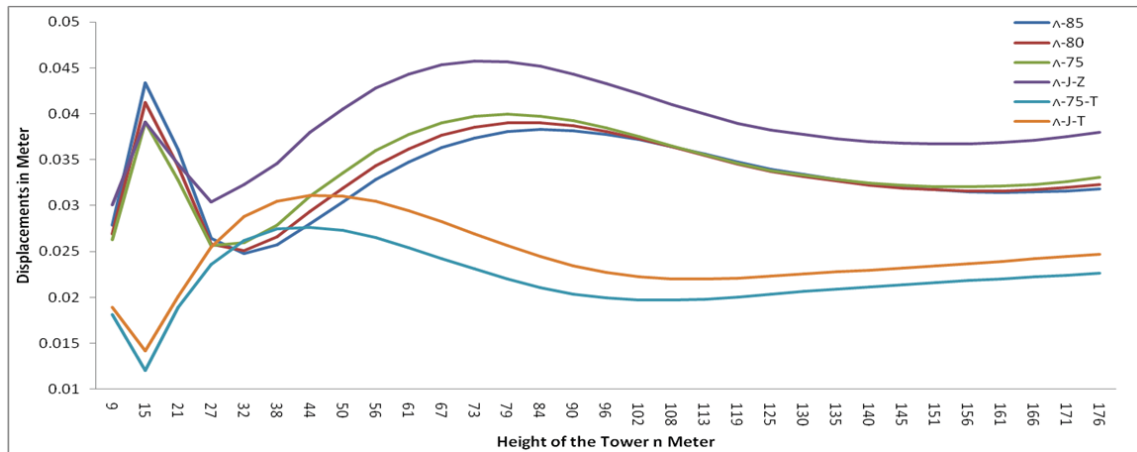


Fig. 4 Resultant displacements along the tower height for A-type supports

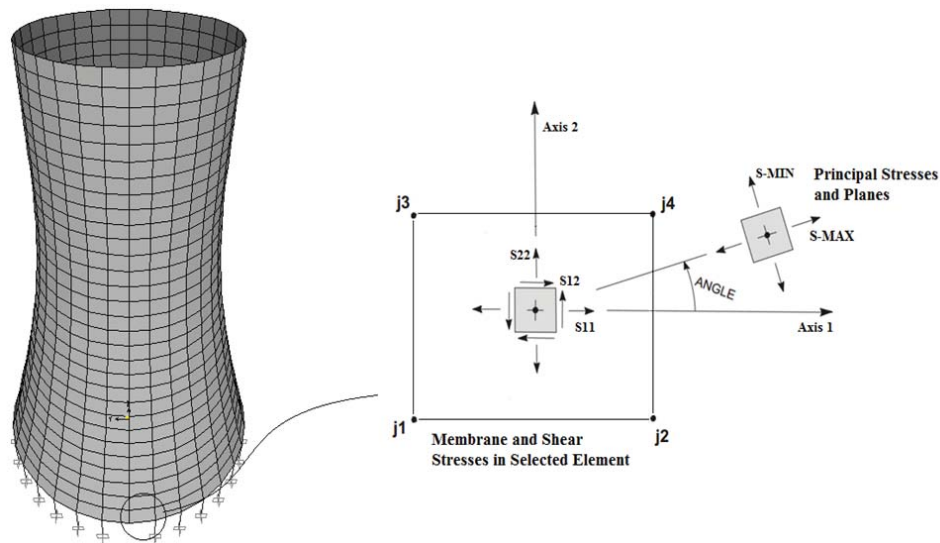


Fig. 5 Principal stresses for the shell element

TABLE III  
PRINCIPAL STRESSES FOR THE TOWERS SUPPORTED ON I-TYPE COLUMNS

<b>I-90</b>	SMaxTop	N/m2	2.0E+07	2.1E+06
	SMinTop	N/m2	-3.6E+06	-8.7E+06
	SAngleTop	Degrees	3.4	44.146
<b>I-85</b>	SMaxTop	N/m2	1.3E+07	2.7E+06
	SMinTop	N/m2	-1.3E+06	-6.9E+06
	SAngleTop	Degrees	7.646	51.957
<b>I-80</b>	SMaxTop	N/m2	7.7E+06	3.7E+06
	SMinTop	N/m2	4.0E+05	-5.2E+06
	SAngleTop	Degrees	20.688	60.479
<b>I-75</b>	SMaxTop	N/m2	5.4E+06	4.9E+06
	SMinTop	N/m2	-5.9E+05	-3.4E+06
	SAngleTop	Degrees	55.676	67.912
<b>I-T</b>	SMaxTop	N/m2	5.9E+06	5.7E+06
	SMinTop	N/m2	-2.2E+06	-2.3E+06
	SAngleTop	Degrees	66.063	70.494

TABLE IV  
PRINCIPAL STRESSES FOR THE TOWERS SUPPORTED ON A-TYPE COLUMNS

<b>A-85</b>	SMaxTop	N/m2	2.0E+07	3.3E+06
	SMinTop	N/m2	-3.9E+06	-8.5E+06
	SAngleTop	Degrees	2.367	38.472
<b>A-80</b>	SMaxTop	N/m2	2.0E+07	3.4E+06
	SMinTop	N/m2	-4.0E+06	-8.4E+06
	SAngleTop	Degrees	2.436	38.02
<b>A-75</b>	SMaxTop	N/m2	2.0E+07	3.5E+06
	SMinTop	N/m2	-4.0E+06	-8.4E+06
	SAngleTop	Degrees	2.512	37.389
<b>A-J-Z</b>	SMaxTop	N/m2	2.2E+07	3.4E+06
	SMinTop	N/m2	-4.8E+06	-8.7E+06
	SAngleTop	Degrees	2.515	34.655
<b>A-75-T</b>	SMaxTop	N/m2	6.2E+06	6.2E+06
	SMinTop	N/m2	-2.6E+06	-1.6E+06
	SAngleTop	Degrees	67.699	69.379
<b>A-J-T</b>	SMaxTop	N/m2	5.8E+06	5.7E+06
	SMinTop	N/m2	-1.5E+06	-1.8E+06
	SAngleTop	Degrees	62.364	66.001

## V.CONCLUSION

In the present paper, influences of changes in the supporting columns' types and angles on the progressive collapse mechanism of the layered hyperbolic cooling towers are numerically studied. It is found that the potential for the progressive collapse of the hyperbolic tower is significantly sensitive to the type of the supporting columns and also to the change of the inclination angle of these columns. From the numerical results it is observed that for the towers supported on single columns increase of inclination angle towards the tower axis and for the towers supported on a pair of columns only increase of inclination angle in circumference of the tower improves the safety of the total structure against the progressive collapse. Another important conclusion is that the GSA recommendations for the safety of the framed or flat plate structures are un-conservative for hyperbolic tower shells and will not provide a safe structure against the progressive collapse. The findings are of great consequence to the safe design of the cooling tower shells and it is strongly recommended that more research is indispensable on safety of hyperbolic shell structures under progressive collapse.

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