Collapse Process Analysis of Reinforced Concrete Super-Large Cooling Towers Induced by Failure of Columns

Xiang-Lin Gu, A.M.ASCE1; Qian-Qian Yu, A.M.ASCE2; Yi Li3; and Feng Lin4

Abstract: This paper presents a numerical simulation and theoretical analysis on the collapse behavior of a reinforced concrete super-large cooling tower induced by failure of columns. The finite-element model was first validated by a collapse experiment of a 1/100-scaled super-large cooling tower and was then used to study the collapse process of the prototype tower. A simplified mechanical approach was put forward to analyze the collapse modes of the prototype tower induced by failure of columns with different numbers. The results demonstrated that the collapse process of a cooling tower resulting from failure of columns could be considered as a non-fixed axis rotation problem. The collapse modes were determined by the number of failed columns, which could be further explained by comparing flexural bearing capacities, vertical bearing capacities, and horizontal bearing capacities of the remaining columns and overturning moments, vertical pressures, and horizontal thrusts, respectively. DOI: 10.1061/(ASCE)CF.1943-5509.0001023, © 2017 American Society of Civil Engineers.

Author keywords: Numerical simulation; Super-large cooling tower; Mechanical approach; Collapse mode.

Introduction

Super-large cooling towers under design and construction at inland nuclear power plants in China have now reached more than 200 m high with a maximum weight of more than 100,000 t. Their bottom diameters exceed 180 m, whereas the minimum thicknesses of the shell structures are only 300–400 mm. Because cooling towers are very close to nuclear islands, a collapse under accidental actions would yield ground vibrations that would not only affect the operational safety of nuclear facilities but could also initiate nuclear disasters (Lin et al. 2013, 2014).

Research on the collapse mechanism of cooling towers can be traced back to the investigation of the collapse of three cooling towers at the Ferrybridge power station in November 1965 (CEGB 1966). Since then, extensive research has been conducted on cooling towers, especially in the area of wind pressure properties of tower surfaces and wind-induced responses (Niemann and Pröpper 1975; Busch et al. 2002). Studies have also been conducted on the collapse-resistant behavior of cooling towers subjected to strong-motion earthquakes in earthquake-prone countries (Sabouri-Ghomi et al. 2006; Wolf and Skrikerud 1980).

With the increased dimensions of cooling towers, the collapse possibility of super-large cooling towers subjected to accidental loads in nuclear power plants increases as well. However, there is comparatively limited research on collapse modes and collapse mechanism analysis of super-large cooling towers under accidental loads.

Although there are many kinds of accidental loads inducing collapse of cooling towers, such as terrorist bombings and uneven foundation settlement, their effects can be considered as damage to cooling tower columns. When the number of failed columns reaches a threshold value, shell structures will collapse because of loss of support. The methodology to analyze damage from accidental loads, responses of remaining structures, collapse modes, and collapse mechanism of cooling towers by invalidating some columns is similar to the alternate load path method, which has been widely adopted in progressive collapse analysis (Department of Defense 2009; General Services Administration 2003; Helmy et al. 2013).

Currently, experimental study (Lew et al. 2014; Song and Sezen 2013; Jennings et al. 2015), theoretical analysis (Liu 2010; Bazzant and Zhou 2002), and numerical simulation (Gu et al. 2014; Li et al. 2014; Bandyopadhyay et al. 2015) are adopted in progressive collapse analysis of building structures under accidental loads. Ellingwood (2006) proposed a framework for considering the risk caused by low-probability, high-consequence events of building structures, and strategies for progressive collapse risk mitigation were suggested. In Baker et al. (2008), the index of robustness was adopted to establish a framework for assessing systems subjected to structural damage. A set of examples was presented to illustrate the feasibility of using this approach. Brunesi et al. (2015) and Parisi (2015) evaluated the fragility of European reinforced concrete columns under extreme loads. With respect to cooling towers, because of their industrial nature, most previous research focused on numerical simulations of the collapse behavior after blast demolition (Sun et al. 2009; Ye and Yan 2010). Few studies have been reported on collapse analysis induced by failure of columns.

In this paper, a 235-m-high natural draft cooling tower is analyzed according to the finite-element method and theoretical analysis. The numerical simulation was compared with a collapse test on a cooling tower model triggered by failure of half of the columns. Good agreement with the test data indicated that the numerical
analysis was reliable for estimating the collapse behavior of reinforced concrete super-large cooling towers. Afterward, collapse process analysis of the prototype tower was performed. In consideration of ease of use, a mechanical approach based on the collapse process is also proposed. The collapse modes and mechanism of the cooling tower are discussed. The study presented in this paper will help to clarify the collapse modes and mechanism of cooling towers induced by the failure of columns and will provide a basis for disaster prevention designs of cooling towers.

**Numerical Simulation and Verification**

**Finite-Element Model**

A three-dimensional finite-element model of the 235-m-high natural draft cooling tower is displayed in Fig. 1. The numerical simulation was similar to that reported in Yu et al. (2016). The shell structure was modeled by using the four-node shell element SHELL 163, which was capable of applying both in-plane and normal loads. The tower was divided into 171 layers along the height, and each of them contained 540 elements along the circumference. The maximum mesh size of the shell element was 0.98 × 1.31 m, and the total number of elements of the shell structure was 92,340. The variation of the thickness of the shell structure was simulated by changing the thickness of the shell elements. Every shell element was divided into 15 layers in the thickness direction. Each layer was assigned with certain materials separately (e.g., concrete, meridional, or circumferential reinforcement). The concrete and steel bars in the columns were modeled separately by the eight-node hexahedral elements SOLID164 and beam elements BEAM161, respectively, and they shared nodes at the interface so that no slip between them was considered. In total, the model had 136,500 elements.

According to the design drawing of the cooling tower, concrete had a cubic compressive strength of 35 MPa, and steel bars had a yield strength of 400 MPa. The steel bars and concrete in the layered shell element were modeled by using the material model with the keyword of *MAT_CONCRETE_EC2 in LS-DYNA (Hallquist 2012). This model could be used to represent concrete, meridional, or circumferential reinforcement through different options. The constitutive relationships are capable of modeling crack in tension and compression in compression of concrete as well as yielding, hardening, and failure of reinforcement [Eurocode 2 (European Committee for Standardisation 2004)]. Because this model is only applicable to shell elements, material models with the keywords of *MAT_CSCM_CONCRETE and *MAT_PLASTIC_KINEMATIC were chosen to define the concrete and reinforcement in columns, respectively (Hallquist 2012). MAT_CSCM_CONCRETE is a continuous surface cap model, which connects the shear yield surface with the cap hardening surface by a smooth surface to describe the yield and failure criterion of concrete in columns. It takes the hardening, damage, and rate correlation of concrete into consideration and has been widely used in engineering analysis. MAT_PLASTIC_KINEMATIC is a plastic model, which could be used to simulate isotropic, kinematic, and mixed hardening with different options. The same model could also be used to consider the effect of strain rate on the material properties of steel bars. The strain rate is evaluated by using Cowper-Symonds model (Hallquist 2012) and was taken into consideration in the numerical simulation in this study.

**Validation of Numerical Simulation**

A collapse test was conducted on a scaled cooling tower model to validate the numerical simulation. The collapse was triggered by failure of half of the columns. Noncontact high-speed photography measurement technology was used during testing (Liu et al. 2015).

**Scaled Super-Large Cooling Tower**

The scaling factor of dimension $S_i$ was chosen to be 1/100, according to the working performance and site conditions at Tongji University. The minimum thickness of the prototype shell structure was 390 mm, and its corresponding scaled size was only 4 mm. Therefore, the thickness of the shell structure was enlarged twice, whereas the amount of reinforcement remained the same during
Table 1. Dimensions and Reinforcement of the Prototype and Scaled Structures

<table>
<thead>
<tr>
<th>Symbol (displayed in Fig. 2)</th>
<th>Dimension</th>
<th>Inside</th>
<th>Outside</th>
<th>Inside</th>
<th>Outside</th>
<th>Inside</th>
<th>Outside</th>
<th>Inside</th>
<th>Outside</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{31}$</td>
<td>19,815 mm</td>
<td>0.18–0.25</td>
<td>0.18–0.25</td>
<td>0.20–0.29</td>
<td>0.20–0.29</td>
<td>198 mm</td>
<td>0.18–0.25</td>
<td>0.18–0.25</td>
<td>0.20–0.29</td>
</tr>
<tr>
<td>$L_{12}$</td>
<td>29,714 mm</td>
<td>0.17–0.18</td>
<td>0.17–0.18</td>
<td>0.20–0.23</td>
<td>0.20–0.23</td>
<td>297 mm</td>
<td>0.17–0.18</td>
<td>0.17–0.18</td>
<td>0.20–0.23</td>
</tr>
<tr>
<td>$L_{20}$</td>
<td>108,721 mm</td>
<td>0.17–0.20</td>
<td>0.17–0.20</td>
<td>0.20–0.23</td>
<td>0.20–0.23</td>
<td>1,087 mm</td>
<td>0.17–0.20</td>
<td>0.17–0.20</td>
<td>0.20–0.23</td>
</tr>
<tr>
<td>$L_{72}$</td>
<td>41,402 mm</td>
<td>0.18–0.20</td>
<td>0.18–0.20</td>
<td>0.17–0.20</td>
<td>0.17–0.20</td>
<td>414 mm</td>
<td>0.18–0.20</td>
<td>0.18–0.20</td>
<td>0.17–0.20</td>
</tr>
<tr>
<td>$L_{52}$</td>
<td>17,348 mm</td>
<td>0.18–0.31</td>
<td>0.18–0.31</td>
<td>0.16–0.17</td>
<td>0.16–0.17</td>
<td>173 mm</td>
<td>0.18–0.31</td>
<td>0.18–0.31</td>
<td>0.16–0.17</td>
</tr>
<tr>
<td>Column diameter</td>
<td>1,900 mm</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>19 mm</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Column number</td>
<td>120</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>120</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

Note: Columns in the prototype structure have a longitudinal reinforcement of $50R28$ (HRB400) and a stirrup of $R20@150$ (HRB400); the columns in the scaled structure have a longitudinal reinforcement of six iron wires (20#) and stirrup of $R22@10$.

Fig. 2. Dimensions of the super-large cooling tower [the value in parentheses stands for the scaled structure (unit: millimeters; not to scale)]
Test Setup and Data Acquisition Systems

Ignoring different forms of accidental loads that may cause failure of columns, the cooling tower with half of the columns failed was set as the initial state, and the collapse process and responses of the model under the gravity force was investigated. The test setup is shown in Fig. 3, in which two pedestals, A and B, with heights of 0.3 m, were used to simulate the ground. Before testing, the cooling tower was supported by 60 columns located on Pedestal A and a supporting device. The supporting device would move downward by 180 mm (height of the columns) to simulate the failure of columns after starting the test. At this time, the elevation of the top surface of the supporting flat was the same as those of Pedestals A and B. Afterward, the cooling tower collapsed because of loss of support.

High-speed photography measurement technology was used in the experiment, as shown in Fig. 4 (Liu et al. 2015). Tracking points were positioned on the surface of the tower as displayed in Fig. 5, in which X1 and Y1 were used to measure the movement of the supporting flat.

Comparison between Numerical and Experimental Results

The finite-element modeling of the scaled cooling tower was based on the dimension of the test specimen, and the maximum mesh size of the shell element was 0.0098 × 0.0131 m. The mechanical properties of mortar and iron wires were adopted according to the test results.

1. Collapse process: In the experimental study, the collapse process lasted for 0.376 s. After switching on the supporting device, the flat moved downward, and half of the shell structure lost its support. The shell body became aslant under the gravity force and rotated about the center of the bottom of the shell body. The comparison between the predicted collapse process and test phenomena that is shown in Fig. 6 demonstrated that a good agreement was achieved.

2. Local damage: Figs. 7(a–e) show that severe deformations occurred to the upper part of the shell body and columns close to the failure region during the collapse process. It was observed that there was a plane of symmetry for the deformation and damage to the cooling tower, which passed through the circle center of the tower and was parallel to the collapse direction. The circular shell structure was deformed into an oval shape, in which the longitudinal axis was parallel to the collapse direction. Vertical cracks occurred on the two ends of the longitudinal

Table 2. Typical Scaling Factors of the Scaled Cooling Tower

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model/prototype</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>$S_l = 1/100$</td>
</tr>
<tr>
<td>Frequency</td>
<td>$S_w = 10$</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>$S_E = 0.3$</td>
</tr>
<tr>
<td>Stress</td>
<td>$S_s = 0.3$</td>
</tr>
<tr>
<td>Strain</td>
<td>$S_s = 1$</td>
</tr>
<tr>
<td>Acceleration</td>
<td>$S_a = 1$</td>
</tr>
<tr>
<td>Time</td>
<td>$S_t = 0.1$</td>
</tr>
<tr>
<td>Density</td>
<td>$S_d = 30$</td>
</tr>
<tr>
<td>Displacement</td>
<td>$S_d = 1/100$</td>
</tr>
<tr>
<td>Velocity</td>
<td>$S_v = 0.1$</td>
</tr>
</tbody>
</table>

Table 3. Material Properties of Galvanized Iron Wires

<table>
<thead>
<tr>
<th>Type</th>
<th>Average diameter (mm)</th>
<th>Average cross-sectional area ($\text{mm}^2$)</th>
<th>Average yield strength (MPa)</th>
<th>Average ultimate strength (MPa)</th>
<th>Average ultimate elongation (%)</th>
<th>Average elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18#</td>
<td>1.219</td>
<td>1.166</td>
<td>305.9</td>
<td>393.4</td>
<td>12.3</td>
<td>145,828</td>
</tr>
<tr>
<td>20#</td>
<td>0.914</td>
<td>0.656</td>
<td>335.5</td>
<td>397.2</td>
<td>10.9</td>
<td>145,393</td>
</tr>
</tbody>
</table>
axis, i.e., the intersection between the upper part of the shell body and the plane of symmetry. Afterward, V-shaped cracks appeared below these cracks at the front side of the tower after the shell body struck the ground [Figs. 7(a and b)]. The columns close to the failure zone failed in compression and bending, whereas no obvious damage was found for the columns far away [Figs. 7(d)]. Figs. 7(f–j) depict the results extracted from the numerical simulation, which compared well with the test results.

3. Displacement time history curve: Displacements, velocities, and accelerations of all the tracking points were obtained according to their coordinates recorded by the high-speed photography measurement technology. The collapse direction was defined as the positive direction of x-axis, and the height direction from the bottom to the top of the tower was defined as the positive direction of z-axis (Fig. 4). The displacement time history curves diagrammed in Fig. 8 show that the deviations between numerical results and test data were acceptable.

Therefore, the numerical model developed in this study proved to be a reliable analytical method for analyzing the collapse of a cooling tower induced by failure of columns. It was used to
analyze the collapse process of the prototype tower in the following section.

**Numerically Predicted Collapse Process of the Prototype Tower**

Fig. 9 shows the collapse process of the prototype tower induced by failure of half of the columns. The shell structure started to become aslant after the columns failed. The columns in the compression zone (close to the failure region) failed successively with the increasing inclination. When all the remaining columns failed, the shell structure collapsed altogether at an angle. At 3.96 s, the lower part of the shell body struck the ground and started to disintegrate because of the huge impact force, whereas the upper part of the shell body continued to fall aslant until the final collapse. Fig. 9(f) shows that there was a plane of symmetry for the deformation and damage of the cooling tower during the collapse process, which passed through the cooling tower’s center and was parallel to the collapse direction. The marking points selected on the side surface of the tower could be used to describe the motion state during the collapse process. Fig. 10 illustrates the displacement time history curves of these points in the x- and z-directions before the cooling tower struck the ground, respectively. The displacements of Marking Points 1# to 7# in the x-direction were proportional to their heights, whereas those in the z-direction were approximate to each other. The difference in the displacements of all the marking points was relatively small.
compared with the downward movements, which revealed that, during the collapse process, the shell body rotated about one point, accompanied by falling downward altogether. The displacements of Marking Points A–D in the x-direction appeared to be negative, which indicated that the motion was in the direction opposite to the collapse, i.e., the lower part of the shell body moved backward during the collapse. In the z-direction, vertical displacements occurred to Point A first and at almost the same time to Points C and D, which demonstrated that the failure moments of the columns near these two points were quite close. After $t = 3.0$ s, the displacement increments of all the marking points were approximate, which revealed that the main motion state of the shell body had transferred to be falling altogether. Therefore, the collapse mode of the cooling tower was classified as a rotary dump accompanied by tilting backward and then falling altogether.

It is interesting to find that the collapse modes of the tower model and the prototype tower were a little bit different. Detailed discussion is given in the “Comparison of Collapse Process between Scaled and Prototype Towers” section.
Simplified Mechanical Approach

The three-dimensional finite-element model of the reinforced concrete super-large cooling tower has been developed. Comparison with the test results of the scaled tower model demonstrated that this model can be successfully used to predict the collapse behavior induced by failure of columns. However, the numerical simulation required accurate modeling and fine mesh, which is complex and time-consuming. With consideration of simplicity and ease of use for engineers, a mechanical approach was proposed in this section to analyze the collapse behavior of the cooling tower.

It was known from the numerical simulation that the collapse mode of the cooling tower induced by failure of columns mainly depended on the bearing capacities of the remaining columns. Therefore, the theoretical analysis was performed by comparing the flexural, vertical, and horizontal bearing capacities of the remaining columns and overturning moments, vertical pressures, and horizontal thrusts, respectively.

Stress-Strain Relationships of Concrete and Steel Bars

Because there were close-packed spiral stirrups in the columns of cooling towers, the constitutive relationship of confined concrete was selected for concrete (Mander et al. 1988; Zheng et al. 2007), and the trilinear model was adopted for steel bars, as shown in Fig. 11.

Basic Assumptions

Some basic assumptions were made as follows (Zheng et al. 2007; Li et al. 1999):
1. There was a plane of symmetry for the deformation and damage of the cooling tower during collapse, which passed through the circle center of the cross section of the shell body and was parallel to the collapse direction;
2. The vertical deformations of the remaining columns obeyed the plane cross section assumption;
3. Before the tower struck the ground, the influence of the deformation of the shell body on the position of its center of gravity was ignored; and
4. The effect of the inclinations of the columns on their internal forces was ignored.

Collapse Analysis of the Cooling Tower

After some columns failed, the remaining ones were considered to be a whole to bear the load transferred from the shell structure.
The remaining columns were partitioned into three zones according to their possible damage: elastoplastic compression and damage of concrete (Zone 1), residual stress of concrete (Zone 2), and tensile stress of concrete (Zone 3), as presented in Fig. 11. The partition was determined by the self-weight of the shell body, the number of the remaining columns, and the constitutive relationship of the remaining columns. The boundary between Zones 1 and 2 was located at the point where the compressive strain was $\varepsilon_{cu}$, and the boundary between Zones 1 and 3 was located where the strain was 0, i.e., the neutral axis.

According to Assumption 1, only half of the cooling tower was studied, as shown in Fig. 11. The columns were numbered counterclockwise from the collapse side. The start and end numbers of the failed columns were 1 and $(j - 1)$, respectively; the start and end numbers of the remaining columns were $j$ and $n$, respectively, where $n$ equaled half of the total number of the columns.

The damage/collapse modes and collapse mechanism of the cooling tower based on the bearing capacities of the remaining columns could be determined by following the steps given in Fig. 12.

**Check of Compressive Bearing Capacity of the Remaining Columns**

The vertical bearing capacity of the remaining columns could be calculated by Eq. (1)

$$N_{cu} = \varphi(n - j + 1)(f_{cc}A + f_{su}A_s)$$

where $\varphi$ = stability factor [GB 50010-2011 (MOHURD 2011)], which is used to consider the effect of the slenderness ratio to the load-carrying capacity of a long column; $f_{cc}$ = peak stress of confined concrete (Mander et al. 1988); $A$ = cross-sectional area of core concrete in one column; $f_{su}$ = ultimate strength of steel bars; and $A_s$ = area of longitudinal reinforcement in one column.

The impact factor of dynamic loads $k_d$ in calculating the sudden vertical load $N_{cu}k_dG$ was obtained by Eq. (2) (Zheng et al. 2007)

$$k_d = 1 + \frac{(j - 1)}{n}$$

where $k_d$ = impact factor of the dynamic load; $(j - 1)$ = half the number of failed columns; and $n$ = half the number of columns. It is found from Eq. (2) that when no column failed, $(j - 1)$ is taken as 0, and thus $k_d$ is equal to 1, i.e., it becomes a static load. When all the columns fail, $(j - 1)$ is taken as $n$, and thus $k_d$ is equal to 2. $G = mg$ is the half weight of the shell body, where $m$ is half mass of the shell body, and $g$ is the acceleration of gravity.

If $N_{cu} < k_dG$, the remaining columns would be crushed, and the cooling tower would fall down and collapse on site; otherwise, the remaining columns would not be crushed, and the cooling tower may fall aslant under the global overturning moment.

**Check of Flexural Bearing Capacity of the Remaining Columns**

The flexural bearing capacity of the remaining columns $M_d$ was calculated by Eq. (3). The stress state of the columns was considered to be at the elastoplastic stage, and the ultimate state was taken as the moment when the steel bars of the outermost column in the tension zone reached the ultimate strength or the concrete of the innermost column in the compression zone reached the ultimate compressive strain $\varepsilon_{cu}$.

$$M_d = \frac{N_{cu}K_s}{A}$$

where $A$ = cross-sectional area of core concrete in one column. $K_s$ = stress concentration coefficient in the adjacent column.
The overturning moment $M_d$ at the initial time is calculated by Eq. (4), according to Fig. 13

$$M_d = G r_{c,j} \sin \phi_{j}$$

(4)

where $G =$ half-weight of the shell structure; $r_{c,j} =$ radius of gyration, i.e., the distance between the center of gravity and the neutral axis in the $x, o, z$-plane coordinate system; $\phi_{j} =$ initial inclination of the cooling tower, i.e., the angle between the line connected from the center of gravity to the neutral axis and the vertical axis.

If $M_d \geq M_g$, the cooling tower may fall aslant a little but keeps stable. If $M_d < M_g$, the cooling tower would rotate altogether, resulting in failure of the remaining columns, and afterward, the cooling tower would collapse progressively.

**Check of Bearing Capacities of the Remaining Columns in Progressive Collapse**

**Calculation of Kinematical Parameters.**

The collapse of the shell structure is a dynamic continuous process. As the angles of inclination of the columns increased and the remaining columns failed successively, the neutral axis of the remaining columns moved backward. The distance between the center of gravity and the rotation axis increased as well as the moment of inertia. Meanwhile, the rotation angular velocity of the shell body changed. The collapse process was actually classified as a non-fixed axis rotation problem. On the other hand, the support reaction force also changed as a result of the moving neutral axis.

Generally, it is difficult to solve a non-fixed axis rotation problem. Consequently, the collapse process was decomposed into a superposition of several time periods. The schematic diagram of a cooling tower rotation at the time period $j$ is displayed in Fig. 14(a).

Although the neutral axes of the remaining columns (rotation axis of the shell structure) at time periods $j$ and $j + 1$ were different for a dynamic continuous process, the end moment of time period $j$ was the starting moment of time period $j + 1$; hence, the relationship of kinematical parameters between different time periods was determined according to Fig. 14(b).
Time period \( j \) started from the initial failure of the columns to the moment when the strain of the \( j \)th column reached the ultimate value \( \varepsilon_{cu} \). At the start moment of time period \( j \), all the columns were under uniform compression. \( \theta^j \) represents the angle between the normal direction of the connected circle centers for each column and the vertical line, which was equal to 0. At the end moment of time period \( j \), the strain of the \( j \)th column reached the ultimate value \( \varepsilon_{cu} \), and the angle \( \theta^j \) changed to \( \theta^b \), which is expressed by Eq. (5). During this time period, the shell structure rotated about the neutral axis (the projections of the neutral axis in \( x, o, z \)-plane coordinate system were \( x = x_{bj} \) and \( z = z_{bj} \)), and the rotation angle \( \Delta \theta_j \) is expressed by Eq. (6). At the end moment of time period \( j \), the inclination of the shell body \( \phi_j \) changed from \( \phi^j \) to \( \phi^b \), as expressed by Eq. (7).

\[
\theta^b = \arctan \left( \frac{\varepsilon_{cu}h}{x_j - x_{bj}} \right) \tag{5}
\]

\[
\Delta \theta_j = \theta^b - \theta^j \tag{6}
\]

\[
\phi^b = \phi^j + \Delta \theta_j \tag{7}
\]

After the center of gravity rotated about the neutral axis with the rotation radius of \( r_{cj} \) by an angle \( \Delta \theta_j \), its position changed from \( C_j \) to \( C_{j+1} \). The movements along the \( x \)- and \( z \)-axes \( \Delta x_{Gj} \) and \( \Delta z_{Gj} \) could be obtained by Eqs. (8) and (9), respectively. The coordinates of the center of gravity \( C_{j+1} \) are given by Eqs. (10) and (11).

\[
\Delta x_{Gj} = r_{cj}(\sin \phi^b_j - \sin \phi^j_j) \tag{8}
\]

\[
\Delta z_{Gj} = r_{cj}(\cos \phi^b_j - \cos \phi^j_j) \tag{9}
\]

\[
x_{G(j+1)} = x_{Gj} + \Delta x_{Gj} \tag{10}
\]

\[
z_{G(j+1)} = z_{Gj} + \Delta z_{Gj} \tag{11}
\]

When the strain of the \( j \)th column exceeded the ultimate value \( \varepsilon_{cu} \), the increased vertical bearing capacity caused a movement of the neutral axis toward the tension zone to maintain the balance. Then, the collapse process proceeded to time period \( j+1 \), which started from the moment that the strain of the \( j \)th column reached \( \varepsilon_{cu} \) and ended at the moment the strain of the \( (j+1) \)th column reached \( \varepsilon_{cu} \). During this time period, the shell body rotated about a new axis with the projection coordinates of \( x = x_{b(j+1)} \) and \( z = z_{b(j+1)} \). Afterward, the end moment of time period \( j+1 \) was the start moment of time period \( j+2 \), and so on. During time period \( j+1 \), the center of gravity rotated about the neutral axis with the rotation radius of \( r_{ci(j+1)} \) by an angle \( \Delta \theta_{j+1} \), and its position changed from \( C_{j+1} \) to \( C_{j+2} \), as shown in Fig. 14(b).

Different from the start moment of time period \( j \), the angle between the normal direction of the connected circle centers of all the columns and the vertical line was not equal to 0 and is expressed by Eq. (12). The other kinematic parameters were calculated the same as that for time period \( j \).

\[
\theta^j_{j+1} = \arctan \left[ \frac{\varepsilon_j(x_{j+1}) - \varepsilon_j(x_{b(j+1)})}{x_{b(j+1)} - x_{bj}} \right] \tag{12}
\]

where \( \varepsilon_j(x_{j+1}) \) and \( \varepsilon_j(x_{b(j+1)}) \) are strains where the \( X \)-coordinates equal \( x_{j+1} \) and \( x_{b(j+1)} \) at time period \( j \), respectively; and \( h \) is height of the columns.

![Fig. 15. Schematic diagram of the radial and tangential reaction forces at time moment \( i \)](image)

**Calculation of Dynamic Parameters.** During time period \( i \), the radial reaction force \( N_i \) and tangential reaction force \( R_i \) along the connection of the neutral axis to the center of gravity are given in Fig. 15, and they are calculated by Eqs. (13) and (16), respectively.

\[
N_i = G \cos \phi_i - m r_{ci}(\omega_i)^2 \tag{13}
\]

where \( N_i \) is radial reaction force of the remaining columns; \( \phi_i \) is inclination of the shell body at one moment during time period \( i \); \( \omega_i \) is rotation angular velocity of the center of gravity about the neutral axis. Actually, \( m r_{ci}(\omega_i)^2 \) in Eq. (13) is the centrifugal force at that moment.

Because \( N_i \) could not be obtained directly from Eq. (13), the kinetic energy theorem expressed by Eq. (14) was introduced for the simultaneous solution.

\[
\frac{1}{2} J_i(\omega_i)^2 - \frac{1}{2} J_i(\omega^b_i)^2 = Gr_{ci}(\cos \phi_i - \cos \phi^b_i) - M_{di} \Delta \theta_i \tag{14}
\]

where \( J_i \) is moment of inertia of the shell body about the neutral axis at time period \( i \); \( \omega^b_i \) and \( \omega_i \) are angular velocities of the shell body at the start moment and during one moment of time period \( i \), respectively. \( M_{di} \) is resisting moment of the remaining columns in time period \( i \).

At the initial moment of time period \( j \), \( \omega^b_j = 0. \omega^b_j \) which corresponds to the end moment of time period \( j \), could be obtained by Eq. (14). However, because of the change of the rotation radius at the next time period, \( \omega^b_{j+1} \) at the start moment of time period \( j+1 \) was not equal to \( \omega^b_j \), as expressed by Eq. (15).

\[
\omega^b_{j+1} = \omega^b_j \cos(\phi^b_{j+1} - \phi^b_j) \tag{15}
\]

Similarly, the angular velocities at the start and end moments of each time period could be obtained. Substituting \( \phi^b_i \) and \( \omega^b_i \) into Eq. (13) results in the radial reaction force \( N_i \) at the end moment of the time period.

\[
R_i = G \sin \phi_i - m \frac{d\omega_i}{dt} r_{ci} \tag{16}
\]

The relationship between moment of inertias and moments is given by Eq. (17).
Simultaneously solving Eqs. (16) and (17) gives the reaction force at any moment during time period $i$. The tangential reaction force $R_i$ at the end moment of the time period could be calculated by substitution of $\phi^p_i$.

The vertical compression $N_{hi}$ and horizontal thrust $R_{hi}$ acting on the remaining columns could be evaluated by decomposing the radial reaction force $N_i$ and triangle reaction force $R_i$ in the $x$- and $z$-directions, as expressed by Eqs. (18) and (19), respectively.

$$N_{hi} = N_i \cos \phi_i + R_i \sin \phi_i$$  \hspace{0.5cm} (18)

$$R_{hi} = N_i \sin \phi_i + R_i \cos \phi_i$$  \hspace{0.5cm} (19)

### Relationship between Inclination and Time.

The time corresponding to rotation angle $d\theta_i$ could be evaluated by integration as expressed by Eq. (20).

$$\omega_i = \sqrt{\frac{2Gr_{ci}(\cos \phi_i^p - \cos \phi_i)}{J_i} - \frac{2M_{di}\Delta \theta_i}{J_i} + (\omega_i^p)^2}$$  \hspace{0.5cm} (20)

Since $\omega_i = d\theta_i/dt$ and $dt = d\theta_i/\omega_i$, the relationship between inclination $\theta_i$ and time $t$ is written as

$$t = \int_{\theta_i^p}^{\theta_i} \frac{1}{\sqrt{\frac{2Gr_{ci}(\cos \phi_i^p - \cos \phi_i)}{J_i} - \frac{2M_{di}\Delta \theta_i}{J_i} + (\omega_i^p)^2}} d\theta_i$$  \hspace{0.5cm} (21)

The relationship between inclination $\phi_i$ at one moment during time period $i$ and the rotation angle $\theta_i$ at the same moment is presented by Eq. (22).

$$\phi_i = \phi_i^p + \theta_i - \theta_i^p$$  \hspace{0.5cm} (22)

By substituting Eq. (22) into Eq. (21), time $t$ could be obtained by integration.

### Check of Compressive and Shear Bearing Capacities of the Columns during the Collapse Process.

As the neutral axis moved backward and the remaining columns failed successively, when the vertical compressive load $N_{hi}$ exceeded the vertical bearing capacity of the remaining columns $N_{cu}$ [Eq. (23)], the remaining columns could not support the shell body anymore, and thus the shell body fell down altogether during rotation.

$$N_{hi} > N_{cu} = \sum_{i=j}^{m} \sigma_i A_i + \sum_{i=j}^{m} \sigma_i' A_i' - \sum_{i=m+1}^{n} \sigma_i A_i$$  \hspace{0.5cm} (23)

If $N_{cu}$ was always larger than $N_{hi}$ during the collapse process, and the horizontal bearing capacity $V_u$ was larger than the horizontal thrust $R_{hi}$, the columns would not fail under the horizontal thrust, and the cooling tower would rotate until collapse. If $R_{hi} \geq V_u$, shear failure would occur to the remaining columns resulting from the horizontal thrust. The cooling tower would move forward or backward, resulting in a collapse mode of falling aslant accompanied by horizontal moving (forward or backward).

On the basis of Assumption 4, the effect of inclination was neglected; the horizontal thrust was undertaken by the shear bearing capacity of the columns.

The shear bearing capacity of the remaining columns is expressed as

$$V_u = \sum_{i=m}^{n} V_{ui}$$  \hspace{0.5cm} (24)

where $V_{ui}$ = shear bearing capacity of one column, which could be calculated according to GB 50010-2010 (General Administration of Quality Supervision, Inspection and Quarantine of the People’s Republic of China 2010)

$$V_{ui} = \frac{1.75}{\lambda + 1} f_y b h_0 + f_y f_{yv} A_{sv} h_0 + 0.07 N_c$$  \hspace{0.5cm} (25)

where $\lambda = \text{shear span ratio of eccentric compression members,}$ taken as $M/Vh_0$; $f_y = \text{tensile strength of concrete;}$ $f_{yv} = \text{yield tensile strength of stirrups;}$ $A_{sv} = \text{cross-sectional area of all the legs of one stirrup at one cross section;}$ $b = \text{width of the cross section, which is taken as 1.76r for circular sections;}$ $h_0 = \text{effective depth, which is taken as 1.6r for circular sections;}$ $s = \text{spacing of stirrups;}$ $N_c = \text{axial compression, which is taken as 0.3f_y A;}$ and $A = \text{cross-sectional area.}$

If the columns are under tension, $V_{ui}$ is calculated by using Eq. (26)

$$V_{ui} = \frac{1.75}{\lambda + 1} f_y b h_0 + f_y A_{sv} h_0 - 0.2 N_t$$  \hspace{0.5cm} (26)

where $N_t = \text{tensile force.}$

### Collapse Modes of the Cooling Tower Induced by Failure of Columns

There are six possible collapse modes of the cooling tower according to the mechanical analysis model:

1. When $N_{cu} \geq K_f G$ and $M_d \geq M_y$, the remaining columns could bear both the sudden vertical load and overturning moment. Therefore, the remaining columns would not fail, and the cooling tower would keep stable without or with only a small inclination.

2. Although $N_{cu} \geq K_f G$ and $M_d \geq M_y$, the deformation of the shell body could not be stabilized because of a large region of failed columns. Afterward, local collapse would occur to the shell body close to the failure zone. At the same time, several columns near the failure zone that were under complex stress state were subjected to failure.

3. When $N_{cu} < K_f G$, the remaining columns could not bear the sudden vertical load; hence, all the remaining columns would be crushed immediately, and the cooling tower would fall down on site.

4. When $M_d < M_y$ and $N_{cu} \geq N_{hi}$, the remaining columns could not bear the overturning moment; consequently, the remaining columns would be crushed successively. If the remaining columns could bear the self-weight of the shell body during the collapse process, the cooling tower would become aslant until collapse but would not fall down.

5. In the situation of Case 4, as the inclination of the shell body and the overturning movement increased, the remaining columns would fail successively. When the remaining columns could not bear the vertical load anymore, i.e., $N_{cu} < N_{hi}$, the cooling tower would fall down with an inclination, which is classified as the collapse mode of being aslant and finally falling down.

6. During the process of falling aslant, a horizontal thrust or tension would occur to the remaining columns. If these forces exceeded the horizontal bearing capacity of the columns ($N_{cu} \geq N_{hi}$ and $V_u < R_{hi}$), the remaining columns would fail quickly, and the shell body would move forward or backward.
This collapse mode was considered as falling aslant accompanied by moving forward or backward and finally falling down altogether. It is concluded that the first case would not cause collapse of the cooling tower; the second case showed a local collapse, and Cases 3–6 showed a complete collapse.

For the prototype tower discussed in the numerical simulation, the collapse process is analyzed as follows. The geometry and dimensions of the cooling tower is given in Table 1. The half mass of the shell body was 6.33 × 10⁷ kg. The height of the center of gravity $H_3$ was 92.89 m. Half of the rotational inertia of the shell body about the center of gravity can be expressed as $J_3 = J_{31} = 2.96 \times 10^{11}$ kg · m². The check of the load-bearing capacities of the remaining columns on the basis of the mechanical analysis model is presented in Fig. 16.

Because half of the cooling tower was considered because of symmetry, the corresponding number of columns was 60. The vertical bearing capacity of one column $N_{vui}$ was calculated as 82.3 MN according to Eq. (1). The number of the columns corresponding to the intersection point in Fig. 16(a) was 47, which indicated that when the number of failed columns exceeded $2 \times 47$, the remaining columns could not bear the sudden load, and the cooling tower would fall down on site; otherwise, the columns would not be crushed immediately, but the cooling tower may fall aslant under the eccentric load. Fig. 16(b) shows the results concerning the moments. When half of the columns failed, the overturning moment $M_\alpha$ ($2.73 \times 10^4$ MN · m) exceeded the flexural bearing capacity of the remaining columns $M_d$ ($2.628 \times 10^4$ MN · m). Consequently, the cooling tower would fall aslant. As the remaining columns failed successively, the difference between the overturning moment and the flexural bearing capacity increased. The intersection point between the vertical load $N_{hi}$ and the bearing capacity of the remaining columns $N$ presented in Fig. 16(c) is located between 58 and 59, which demonstrated that, when the number of failed columns was less than $2 \times 58$, the vertical bearing capacity of the remaining columns was larger than the vertical load, and the cooling tower would not fall down during the collapse process. In terms of the horizontal load, as the number of failed columns increased, the horizontal bearing capacity of the remaining columns $V_u$ decreased, whereas the horizontal thrust $R_{hi}$ increased first and then decreased. When the number of failed columns exceeded $2 \times 58$, the horizontal thrust transferred into horizontal tension force. The first intersection point in Fig. 16(d), which was located between 42 and 43, denoted that, when the number of failed columns exceeded $2 \times 42$, the remaining columns could not bear the horizontal thrust resulting from the rotation of the shell body. The cooling tower became aslant and fell down backward, i.e., the remaining columns and the lower part of the shell body moved in the direction opposite to the collapse. If the remaining columns failed after the horizontal force had changed the direction, the cooling tower would move forward, i.e., the remaining columns and the lower part of the shell body moved in the same direction as that of the collapse.

From the preceding discussions, it can be concluded that the collapse modes of the cooling tower induced by failure of columns could be determined on the basis of the mechanical analysis. When half of the columns failed, the cooling tower would become aslant and finally fell down accompanied by moving backward. This was mainly because at the initial moment, the overturning moment...
exceeded the flexural bearing capacities of the remaining columns, and then the shell body fell aslant, followed by successive failure of the remaining columns caused by the redistribution of internal forces. When the number of failed columns was larger than 2 × 42, the horizontal bearing capacity of the remaining columns was less than the horizontal thrust, and the columns would consequently fail under the horizontal thrust. The remaining columns and the lower part of the shell body would move backward. Finally, when all the columns failed, the shell body would fall down at a certain angle.

**Parametric Analysis of Collapse Modes of the Prototype Tower**

To verify the mechanical approach and to better understand the collapse mechanism of the cooling tower, the collapse behavior of the prototype tower induced by failure of columns with different numbers was analyzed by both theoretical solution and numerical simulation. The results showed that the number of failed columns determined the collapse modes and that the corresponding failure mechanisms were also different, as listed in Table 4. Also, the conclusions drawn from the mechanical approach were conformed with the numerical results. The cooling tower kept stable when 10 columns failed, which accounted for 8.3% of the total number of the columns, as shown in Fig. 17. When the number of failed columns increased to 20, 30, 40, and 50, i.e., 16.7, 25.0, 33.3, and 41.7%, respectively, the cooling tower collapsed locally. A typical case with 30 failed columns is shown in Fig. 18. When the failed columns were 60, 80, and 90, i.e., 50, 66.7, and 75%, respectively, the cooling tower became aslant and fell down altogether. The tower with 80 failed columns is presented in Fig. 19. When the number of failed columns was 100 (83.3% of the total number), all the remaining columns were crushed, and the shell body fell down with a small inclination, as shown in Fig. 20.

**Comparison of Collapse Process between Scaled and Prototype Towers**

The collapse modes corresponding to failure of 50% of the columns were observed to be different for the test scaled tower and simulated prototype tower. According to the aforementioned collapse modes, the test tower collapsed locally, whereas the prototype tower became aslant and fell down altogether. These two different collapse modes could be determined by comparing the flexural bearing capacity of the remaining columns $M_d$ and the overturning moment $M_g$. If $M_d \leq M_g$, an overall collapse would happen; if $M_d > M_g$,

### Table 4. Results of Parametric Analysis

<table>
<thead>
<tr>
<th>Number of failed columns</th>
<th>Percentage of failed columns</th>
<th>Collapse mode</th>
<th>Collapse mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>8.3</td>
<td>No collapse</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>20</td>
<td>16.7</td>
<td>Collapsed locally</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>30</td>
<td>25.0</td>
<td>Collapsed locally</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>40</td>
<td>33.3</td>
<td>Collapsed locally</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>50</td>
<td>41.7</td>
<td>Collapsed locally</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>60</td>
<td>50.0</td>
<td>Became aslant and fell down altogether</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>80</td>
<td>66.7</td>
<td>Became aslant and fell down altogether</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>90</td>
<td>75.0</td>
<td>Became aslant and fell down altogether</td>
<td>$N_{cu} &gt; k_d G; M_d &gt; M_g$</td>
</tr>
<tr>
<td>100</td>
<td>83.3</td>
<td>Fell down altogether</td>
<td>$N_{cu} &lt; k_d G$</td>
</tr>
</tbody>
</table>

![Fig. 17. Collapse process of the cooling tower induced by failure of 10 columns: (a) failure zone of the cooling tower (side view; $t = 0.0$ s); (b) displacement contours in the $z$-direction (side view; $t = 1.5$ s)](attachment:fig17.png)
Fig. 18. Collapse process of the cooling tower induced by failure of 30 columns: (a) failure zone of the cooling tower (side view; \( t = 0.0 \) s); (b) collapse moment of the cooling tower (front view; \( t = 6.1 \) s); (c) failure of local columns (side view; \( t = 6.1 \) s)

Fig. 19. Collapse process of the cooling tower induced by failure of 80 columns: (a) failure zone of the cooling tower (side view; \( t = 0.0 \) s); (b) collapse moment of the cooling tower (front view; \( t = 3.25 \) s); (c) failure of local columns (side view; \( t = 0.9 \) s)

Fig. 20. Collapse process of the cooling tower induced by failure of 100 columns: (a) failure zone of the cooling tower (side view; \( t = 0.0 \) s); (b) collapse moment of the cooling tower (front view; \( t = 2.3 \) s); (c) failure of local columns (side view; \( t = 0.65 \) s)
the tower would keep stable or collapse locally. The calculation on the scaled tower indicated $M_J = 18.3$ kN·m and $M_C = 16.5$ kN·m, i.e., $M_J > M_C$. Therefore, local collapse took place in the experimental study. This was because the strength of the mortar used for construction was comparatively high. The strength ratio of mortar to concrete according to the similarity theory was 0.3, whereas the actual value reached 0.5. Recalculation with the mortar strength that is based on the similarity theory resulted in a contrary conclusion, i.e., $M_J < M_C$, and the tower collapsed altogether. Therefore, it was concluded that, if the strength of the mortar for construction of the scaled tower was exactly the value obtained from the similarity theory, the collapse mode should be consistent with the prototype tower.

Conclusions

A three-dimensional finite-element model was developed in this paper to study the collapse behavior of a reinforced concrete super-large cooling tower induced by failure of columns. The numerical simulation was validated by a collapse experiment of a scaled cooling tower. Afterward, a theoretical solution was proposed to analyze the collapse modes of the prototype tower for ease of use. The following observations and conclusions can be made:

1. The proposed numerical model proved reliable to analyze the collapse process of cooling towers induced by failure of columns;
2. The proposed mechanical analysis approach could be used to predict different collapse modes of cooling towers induced by failure of columns with different numbers and to explain the corresponding collapse mechanism; and
3. The numerical simulation and theoretical analysis indicated the following: When the number of failed column was 8% or less of the total number of the columns, the cooling tower did not collapse; when the number increased to 16.7–41.7%, local collapse occurred to the cooling tower; when the number reached to 50–75%, the cooling tower became aslant and fell down altogether; when the number exceeded 83.3%, the cooling tower fell down altogether. The reasons for differing results in these collapse mode calculations are as follows: When the flexural bearing capacity of the remaining columns was less than the overturning moment, the tower would fall aslant; when the vertical bearing capacity of the remaining columns was less than the vertical load, the tower fell down; when the horizontal bearing capacity of the remaining columns was less than the horizontal thrust, the tower moved forward or backward; otherwise, the tower remained stable or collapsed locally.

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References


