Collapse Mechanism of Reinforced Concrete Superlarge Cooling Towers Subjected to Strong Winds

Qian-Qian Yu, A.M.ASCE1; Xiang-Lin Gu, A.M.ASCE2; Yi Li, Ph.D.3; and Feng Lin, Ph.D.4

Abstract: This paper presents a numerical simulation on the collapse behavior and structural stability of a reinforced concrete superlarge cooling tower subjected to strong winds. Results demonstrated that the cooling tower locally collapsed inward because of a loss of material strength rather than loss of stability. The critical wind pressure corresponding to material failure was far less than that corresponding to buckling, according to elastic stability analysis. Finally, a parametric analysis was conducted to investigate the influence of design parameters on the wind-resistant performance of the tower, including the thickness of the shell structure as well as the concrete cover, reinforcement space of the shell structure, and reinforcement ratio of the shell structure. The research presented in this paper can help to clarify the collapse mechanism of superlarge cooling towers and contribute to the improved design of future towers. DOI: 10.1061/(ASCE)CF.1943-5509.0001096. © 2017 American Society of Civil Engineers.

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Introduction

The typical cooling tower is a kind of thin-walled structure. Currently, because of increasing industry demand, the height of towers is continuously increasing. Reinforced concrete superlarge cooling towers under design and construction at inland nuclear power plants in China have reached a height of 252 m and a total weight of 120,000 t. The base diameter has exceeded 185 m, whereas the minimum thickness of the shell structures is as small as 0.35 m. Normally, nuclear power plant designers locate a cooling tower a short distance away from nuclear islands because of operational requirements. Structural failure, such as a local or entire collapse of a tower induced by strong winds, may not only affect the operational safety of nuclear facilities but also lead to catastrophic secondary disasters (Lin et al. 2013, 2014). Consequently, it is of great importance to maintain the tower’s structural integrity. The first step is to understand the collapse mode and failure mechanism of superlarge cooling towers subjected to strong winds.

In November 1965, three cooling towers with a height of 114 m at the Ferrybridge Power Station collapsed during a gale-force wind. The incident report implied that the collapse was mainly because of the underestimation of wind forces and inadequate design theories, which failed to account for the dynamic effect of wind loads and the disturbance effect of tower groups (CEGB 1966). To the best of the authors’ knowledge, this is one of the earliest significant studies on collapse-resistance performance of cooling towers. After the Ferrybridge Power Station incident, cooling towers collapsed at Ardeer Nylon Works in Ayrshire, U.K., 1973 (ICI 1973); Fiddlers Ferry Power Station in Cheshire, U.K., 1984 (CEGB 1984); Allegheny Power System in Willow Island, West Virginia, United States, 1978 (Lew et al. 1979); and in Bouchain, France, 1979 (Bamu and Zingoni 2005). All these accidents have aroused widespread concern about the safety of cooling towers (Bamu and Zingoni 2005; Pope 1994; Godoy 1984; Waszczyszyn et al. 2000).

Wind action is one of the governing parameters for cooling tower design; therefore, extensive research has been performed to investigate the wind-pressure properties on tower surfaces and wind-induced responses. Niemann and Pröpper (1975) studied fluctuating pressures on a hyperbolic cooling tower by means of full-scale measurements. The cross-correlation coefficients and RMS pressure distributions were presented. In Kawarabata et al. (1983), wind tunnel tests on both rigid and aeroelastic models were conducted to study the wind pressures outside and inside a tower surface, fluctuating wind pressures, and wind pressure spectrums. More recently, a computational fluid dynamics (CFD) analysis by Liu et al. (2006) discussed the scale effect of a single tower and the disturbance effect between two towers.

With respect to the collapse mechanism of cooling towers subjected to strong winds, researchers have not achieved an agreement yet. Some researchers suspect that the towers collapse because of buckling under strong winds because of the thin-wall profile of the structures. Der and Fidler (1968), Mungan (1976, 1979), and Mungan and Lehmkamper (1979) reported comprehensive experimental studies on the aerostatic stability of cooling towers. Methods for checking their overall and local stabilities were also proposed, which have also been incorporated into the design codes of cooling towers in many countries. Abel and Gould (1981) reviewed available approaches to assess the stability of large concrete hyperboloids subject to wind loads, including scaled-up wind tunnel tests, methods based on axisymmetry, and methods not based on axisymmetry. It was demonstrated that the bifurcation calculation could estimate the wind-loaded hyperboloid buckling for


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routine design purposes with reasonable accuracy. Mahmoud and Gupta (1995) analyzed the failure mode of the Grand Gulf cooling tower under strong winds using a finite element program, where the effect of large displacement was taken into consideration. Results indicated that the failure occurred because of circumferential buckling in the vicinity of the throat rather than yielding of the meridional reinforcement.

However, some researchers advocate that the collapse of cooling towers is attributed to loss of material strength in shell structures. In Mang et al. (1983), theoretical analysis of the Port Gibson cooling tower indicated that failure was initiated by concrete cracking, followed by temporary stiffening, and, finally, by yielding of reinforcement. The buckling load obtained by linear or geometrical nonlinear analysis was far above the ultimate load-bearing capacity of the tower. Later, in 2013, Jia (2013), reinvestigated the Port Gibson cooling tower and performed a parametric study where the design parameters were changed, including the use of high-strength concrete, reduction of shell thickness, and consideration of geometrical imperfections. A consistent conclusion was drawn that the cooling tower failed by loss of material strength and buckling did not take place. This argument was also supported by Noh (2006) and Busch et al. (2002). Noh (2006) considered both material and geometrical nonlinearities in the numerical simulation of a cooling tower 150 m high and finally found that the failure of the tower was caused by the yielding of reinforcement in the windward meridian. Busch et al. (2002) numerically assessed the wind-resistant behavior of the cooling tower at Niederaussem Power Station, Germany, which is 200 m high and currently the highest cooling tower in the world, and came to a similar conclusion.

Analysis of the ultimate load-bearing capacity of cooling towers subjected to wind loads could be used to assess the wind-resistant behavior of these towers. However, further research is necessary to clarify the collapse mechanism of these towers under strong winds. Such research should also provide a basis for disaster prevention design and help maintain structural integrity over the whole service life. If failure occurs because of buckling, increasing the thickness of the shell structure may be considered; if the collapse is triggered by loss of material strength, the available solution may be improvement of material strength and the reinforcement ratio. Regarding analysis methods, the aforementioned completed research mostly checked the ultimate state of cooling towers according to the stress state of the towers or load-displacement curves of certain points. Previous research has not adequately analyzed the entire collapse process and has therefore failed to clarify the collapse mechanism. Also, current study on cooling towers with a maximum height of 200 m (Niederaussem Power Station) is insufficient for the superlarge cooling tower 252 m high presented in this study.

This study presents a numerical analysis based on the finite element method, which clarifies both the collapse process and stability behavior of a natural-draft cooling tower 252 m high under strong winds. A parametric study was then carried out to further quantify the influence of design parameters on the performance of the tower. This study extends the understanding of wind-resistant behavior and collapse mechanisms of superlarge cooling towers. The authors also provide useful suggestions for disaster prevention designs of superlarge cooling towers.

**Numerical Modeling**

**Finite Element Model**

The configuration and dimensions of the superlarge cooling tower are shown in Fig. 1. The shell structure is supported by 112 columns with a diameter of 1.6 m. The tower was constructed using concrete with a cubic compressive strength of 45 MPa and steel bars with a yield strength of 400 MPa. The inside and outside circumferential reinforcement ratios of the shell structure ranged from 0.09 to 0.63% and from 0.16 to 0.57%, respectively; the inside and outside meridional reinforcement ratios of the shell structures ranged from 0.12 to 0.30% and from 0.15 to 0.46%, respectively. Twenty-eight steel bars with a diameter of 28 mm and a yield strength of 400 MPa were used as longitudinal reinforcement for each column. The stirrup of the columns was set as steel bars with a diameter of 14 mm and a yield strength of 235 MPa at a space of 150 mm.

A typical three-dimensional finite element model of the cooling tower is displayed in Fig. 2. A right-handed coordinate system was adopted, where the positive direction of the x-axis is oriented downwind. The height direction from bottom to top is defined as the positive direction of the z-axis, and the y-axis is marked as crosswind. The numerical simulation here is similar to that reported by Yu et al. (2016) and Gu et al. (2017). A four-node shell element, SHELL163, with both bending and membrane capabilities, was selected to model the shell structure. In order to better reflect the hyperbolic characteristics, the tower was divided into 162 layers along the height direction, and each layer contained 728 elements along the circumferential direction. The shell structure was fine-meshed into 117,936 elements as depicted in Fig. 2(a), and the maximum mesh size was 0.76 × 1.50 m. The thickness of the shell elements in each layer was changed to simulate the varying thickness of the shell structure. Each shell element was again divided into 15 layers along its thickness direction, where certain materials were assigned separately, i.e., concrete, meridional, or circumferential reinforcement. The thickness of the material and the corresponding location were both determined according to design drawings of the cooling tower. With respect to the modeling of columns, concrete and steel bars were established separately by 8-node hexahedral element SOLID164 and beam element BEAM161, respectively [Fig. 2(c)]. A perfect bond between concrete and steel bars was assumed; hence, elements of concrete and steel bars shared nodes at the interface. The model had a total of 150,191 elements.

The *LS-DYNA* material model, *MAT_172 (*MAT_CONCRETE_EC2), was adopted to model concrete and reinforcement in the layered shell element in LS-DYNA. With different options, this model could be used to represent concrete, meridional, or circumferential reinforcement only. The constitutive relationships according to Eurocode 2 (CEN 2004) are capable of modeling cracks in tension and crushing in compression of concrete as well as yielding, hardening, and failure of reinforcement. In the columns, material models *MAT_159 (*MAT_CSCM_CONCRETE) and
Validation of the Numerical Model

The numerical model presented in this study is similar to that reported by Yu et al. (2016) and Gu et al. (2017). In Yu et al. (2016), the numerical model was adopted for elastoplastic time history analysis of a superlarge cooling tower subjected to seismic action. A shaking table test was performed on the scaled tower model to investigate the collapse mechanism and validate the numerical analysis. Acceleration and displacement responses of the tracking points, as well as the collapse process of the numerical results, compared well with the test observations. The collapse behavior of a similar reinforced concrete superlarge cooling tower induced by failure of columns was reported by Gu et al. (2017). The numerical simulation here proved to be well suited for estimating the collapse process in addition to local damage and displacement responses of a superlarge cooling tower triggered by failure of columns. It was, therefore, confirmed that the proposed numerical model was reliable to perform collapse analysis of superlarge cooling towers subjected to extreme load actions.

Wind Load

The construction site of the cooling tower belongs to wind exposure Category B [GB 50009-2012 (Ministry of Housing and Urban-Rural Development of the People’s Republic of China 2012)] and the basic wind pressure is 0.45 kPa (26.8 m/s). The wind pressure on the shell structure varies with time because of the fluctuating characteristic; therefore, its effect on the cooling tower is one kind of dynamic action. Generally, design codes of many countries amplify the wind action by a wind load factor $\beta$ to consider the effect of fluctuating and consequent resonance. The equivalent design wind pressure on the cooling tower is calculated by:

$$w_{(z,\theta)} = \beta C_{(z,\theta)} w_0$$

where $w_{(z,\theta)}$ = equivalent design wind pressure on the cooling tower (kPa); $\beta$ = wind load factor, which is taken as 1.9 in Exposure B [GB/T 50102-2003 (Ministry of Housing and Urban-Rural Development of the People’s Republic of China 2003); Huang 2001; Ke et al. 2015]; $w_0$ = basic wind pressure (kPa); $C_{(z,\theta)}$ = wind pressure distribution coefficient; $z$ = elevation; and $\theta$ = intersection angle to the windward direction. The wind load factor $\beta$ is composed of several parts, among which the amplification factor induced by resonance only takes a small portion (approximately 1.1). It is indicated by Niemann (1980) that the resonant response is small even for very high towers, and cooling towers are generally not sensitive to wind-induced vibrations. Also, the main target of this study is to investigate the collapse mechanism of cooling towers. The wind load factor based on the design code is adopted here. $C_{(z,\theta)}$ here represents the combined effects of structural height and shape, which was determined by a wind tunnel test on a rigid model of a superlarge cooling tower performed at the State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University (State Key Laboratory of Disaster Reduction in Civil Engineering 2011). A total of 36 $\times$ 12 $= 432$ points along the circumferential and meridional directions were detected. A typical group of test results (at the throat’s height of the shell structure) is compared with the calculation results based on the design codes [DL/T 5339-2006 (National Development and Reform Commission of the People’s
Republic of China 2006) and GB/T 50102-2003 (Ministry of Housing and Urban-Rural Development of the People’s Republic of China 2003), as shown in Fig. 3 (the results were normalized with reference to the wind pressure distribution coefficient at 0° by the wind tunnel test). The horizontal axis represents the angle in the circumferential direction of the shell structure, and the vertical axis represents the wind-pressure distribution coefficient. It was found that the calculation results by using the design codes could predict the wind-pressure distribution coefficient with reasonable accuracy. Therefore, in the following analysis, the wind-pressure distribution coefficients in different heights of the shell structure obtained by the design codes (as diagrammed in Fig. 4) were adopted for the sake of simplicity.

Because the frequency of wind load is always low, it is a rather static pressure that should be simulated by using implicit code. However, explicit computation was chosen in the study because the subsequent collapse progress with significantly dynamic features was concerned. The wind load was first linearly applied from zero to the wind pressure corresponding to a certain \( \lambda \) value, which lasted 5 times as long the period of the cooling tower. Afterward, a constant wind load was adopted. It this study, several assumptions were made: (1) the wind pressure distribution coefficient \( C(z, \theta) \) did not change during the collapse process and (2) the wind direction was always along the normal direction of the shell element during the collapse process.

**Collapse Process Simulation of the Cooling Tower Subjected to Strong Winds**

In the numerical analysis, vertical gravity load \( G \) and wind load perpendicular to the tower surface \( \lambda w(z, \theta) \) were applied to the cooling tower, where \( w(z, \theta) \) was calculated by Eq. (1) as aforementioned and \( \lambda \) is an amplification coefficient. In the analysis, \( \lambda \) was added from 1.0 with a step of 10% until the tower finally collapsed. The wind-resistant capacity of the cooling tower was consequently determined by the \( \lambda \) value at collapse. Numerical results implied that when \( \lambda \) increased from 1.0 to 1.9, the strong wind load resulted only in deformation of the shell structure, whereas the stress did not reach material strength, and collapse of the cooling tower did not occur. The windward side of the tower became concave in the \( x \)-direction, whereas the two sides across-wind became convex, which was consistent with the distribution of the wind pressure along the circumferential direction. The maximum deformation, which occurred at the throat region, was attributed to its minimum thickness, and the deformation above and below the throat decreased gradually as the shell thickness increased. As the amplification coefficient \( \lambda \) increased, the deformation of the shell structure increased and the general distribution was kept the same. At this stage, the tower remained stable. When \( \lambda \) reached 2.0, the shell structure lost its structural integrity and finally collapsed. At this time, the wind pressure at 10 m high and 0° to the windward side was \( \lambda \beta \) times the basic wind pressure \( w_0 \), i.e., 1.71 kPa (52.3 m/s). At the early stage of loading, the deformation of the shell structure was similar to that when \( \lambda \) was less than 2.0. However, as the loading time increased, the deformation continuously increased, and the shell structure could not hold its shape anymore. At 1.3 s, two vertical cracks appeared.
at 157.5 m, denoted as a and b in Fig. 5. Fig. 6 compares the horizontal Section 1-1 before and after deformation. It was demonstrated that the region between a and b deformed inward along the windward direction, whereas the region near a and b deformed outward along the across-wind direction. Significant curvature change was found in a and b. The distance between a and b in the y-direction was approximately 49.1 m, and the corresponding central angle was 60°. The maximum deformation identified as d in Fig. 6 was as much as 1.553 m. The deformation of the shell structure was related to the wind pressure on the surface, which indicated that the region between a and b was in compression, whereas the remaining portion was in tension. The stress and strain of concrete and reinforcement in both meridional and circumferential along the thickness direction of the shell structure were extracted from the numerical results attributed to the layered shell element modeling. It was found that, when cracking, the outer circumferential reinforcement in a and b had already reached the ultimate strength and the tensile strain had exceeded the ultimate strain; the outer meridional reinforcement was also in tension but had not yielded. Both the inner circumferential and meridional reinforcement were in compression and had not yielded. Taking a and b before deformation as the support of the region between a and b, the failure here could be considered a loss of material strength at the support as a result of a negative moment.

Fig. 7 displays the collapse process of the cooling tower subjected to strong winds. As the wind pressure increased, the width and number of the cracks on the windward side of the shell structure increased. The lower part of the windward surface first deformed concave and then collapsed into the center of the tower. At the same time, the upper part with an angle from −30° to +30° to the windward direction moved outward because of tension and finally collapsed out of the shell structure. With the development of cracks and extension of the damaged region, vertical and circumferential cracks soon occurred to the leeward surface under the effect of wind action and self-weight. Finally, the upper shell structure disintegrated and collapsed. Because the radius of the lower part was larger than that of the upper part, most of the blocks fell inside the shell structure and only a few scattered around the columns during the collapse process, as shown in Fig. 7(e). The residual appearance of the tower reported here is similar to the three collapsed towers at the Ferrybridge Power Station (CEGB 1966). The failure modes of these four cooling towers were approximately the same, where the upper part of the shell structure collapsed while the columns and the upper portion (approximately 1/5 of the shell structure) remained stable, which belonged to a local collapse.

Fig. 7. Collapse process of the cooling tower subjected to strong winds: (a) t = 6.0 s; (b) t = 10.0 s; (c) t = 18.0 s; (d) t = 22.0 s; (e) t = 32.0 s
Stability Analysis of the Cooling Tower Subjected to Strong Winds

Cooling towers are considered prone to losing stability when subjected to strong winds because of the profile of thin-walled structures. Generally, the thickness of the shell structure is increased to improve the wind-resistant behavior. However, this solution has a few issues that have not been adequately addressed:

The thickness of the shell structure is an important design factor that affects structural integrity and involves both economic and time costs. It may be expensive and time consuming to improve the overall and local stability by blindly increasing the shell thickness. In addition to buckling failure, a cooling tower may also collapse because of loss of material strength. In this case, increasing of thickness of the shell structure may not be an effective way to prevent collapse.

In China, a stability check now requires both an overall and local elastic stability check. For the entire shell body, it is required that the safety coefficient $K_B$ be no less than 5, where $K_B$ is defined as the ratio of critical buckling wind pressure $q_{cr}$ to the designed wind pressure $q$ at the top of the tower. A local elastic stability check is carried out according to the work presented by Mungan (1976, 1979) and Mungan and Lehmkamper (1979). The corresponding safety coefficient is also required to be no less than 5. More details can be found in DL/T 5339-2006 (National Development and Reform Commission of the People’s Republic of China 2006) and GB/T 50102-2003 (Ministry of Housing and Urban-Rural Development of the People’s Republic of China 2003). For the cooling tower in this study, the calculated overall stability coefficient was 6.63 and the minimum local stability coefficient was 5.63, both of which are in compliance with the design codes. In the current stage, it was only assured that under the designed wind pressure, the cooling tower could not buckle. When the wind action exceeds the design load, the overall or local stability coefficient may be less than 5. Therefore, it is necessary to know more about the wind-resistant behavior of the cooling tower.

When the tower loses stability, an inflection point will occur in the load-displacement curve of the shell structure. The displacement increases rapidly while the wind load keeps constant or increases very slowly. In Zhang (2011), a stability analysis of a cooling tower was performed based on the load-displacement curve by the explicit dynamic method. A similar approach was adopted in this study to investigate the critical buckling wind pressure. In the numerical analysis, a linear elastic material model was selected and the geometrical nonlinearity was taken into consideration. The loads $G$ and $\lambda_{w(c,\theta)}$ were applied to the structure. The load-displacement curves at different $\lambda$ values were obtained and the critical buckling wind pressure was determined based on the inflection point of the curve. The results showed that displacement in the $x$-direction (windward) occurred in the upper part of the shell structure. Taking a node (No. 61141) in the throat height of the windward side as a tracking point, its windward displacement versus windward displacement is depicted in Fig. 8. The amplification factor corresponding to the inflection point is 23.0. When $\lambda$ further increased, the deformation increased rapidly; therefore, the load at $\lambda = 23.0$ was considered the critical buckling wind pressure, which was also approximate to the value presented in Zhang (2011). Figs. 9 and 10 display the deformation of the cooling tower at $\lambda = 23.0$ and 23.5, respectively. It was indicated in the figure that at the critical buckling wind pressure ($\lambda = 23.0$), the deformation of the shell structure was relatively small and the displacement in the $x$-direction (windward) was 1.95 m. When the amplification coefficient was enhanced to 23.5, the displacement of the cooling tower was much larger. Apparent local buckling was observed in the vicinity of point P as depicted in Fig. 10(a). P was located at 0.33 times the whole height of the cooling tower and 62° to the windward direction. It was, therefore, concluded that in the elastic stability analysis, the region around P represents the structural weakness, which underwent local buckling first when the load reached the critical buckling wind pressure. A similar conclusion was also drawn by

![Fig. 8. Amplification coefficient $\lambda$ versus windward displacement](image)

![Fig. 9. Deformation of the cooling tower under wind load at $\lambda = 23.0$: (a) side view; (b) bird’s-eye view](image)

![Fig. 10. Deformation of the cooling tower under wind load at $\lambda = 23.5$: (a) side view; (b) bird’s-eye view](image)
Zhao et al. (2008). In the ultimate bearing-capacity analysis of a 176-m-high cooling tower, buckling occurred in the region with a height 0.286 times that of the cooling tower with a 70° angle to the windward direction. The local stability coefficient around the area also appeared to be minimal, which implied that the local buckling should also initiate from here.

Analysis of Collapse Mechanism

Based on the aforementioned analysis, an apparent difference was observed between collapses because of loss of material strength and loss of stability. In the first case, a nonuniform tensile and compressive stress was observed along the circumferential direction generated by the wind load. When the stress of the outside circumferential reinforcement in the upper region of the shell structure reached the ultimate strength triggered by the nonuniformly distributed wind pressure along the circumferential direction, the cooling tower began to fracture, and finally approximately 4/5 of the shell structure collapsed. In the buckling case, when the wind action reached the critical value, local buckling occurred in the region that was approximately 0.3 times the height of the shell structure and 60°–70° to the windward direction. The calculated amplification coefficient $\lambda$ in the first case was far less than that in the second case, which demonstrated that as the wind pressure increased, the cooling tower would fail because of loss of material strength rather than buckling. The conclusion of failure because of loss of material strength could be further explained by the following:

1. During the numerical simulation of the collapse process, elastic-plastic material models were adopted. In the material dimension, the mechanical properties of both concrete and reinforcement were more accurately modeled, including elastic modulus variation, strength degradation, and differences in tension and compression of concrete, as well as the yielding and strain hardening of reinforcement. In the structural dimension, the variation of internal force as a result of plastic deformation or concrete crack was taken into account. The geometric nonlinearity was also considered using the explicit dynamic analysis technique. Therefore, the collapse analysis of the cooling tower was more accurate;

2. The critical amplification coefficient $\lambda$ determined from the numerical analysis was equal to 2.0, which was approximate to some previous research on similar topics. In Milford and Schnobrich (1984), the effects of tension stiffness, rotating crack, and geometric nonlinearity were considered, and the corresponding $\lambda$ value was 2.1. Mahmoud and Gutpa (1995) reported a $\lambda$ value of 1.73 with the introduction of large displacement. Noh (2005, 2006) performed a nonlinear analysis of the Port Gibson cooling tower in Mississippi and obtained a $\lambda$ value of 1.925. The highest cooling tower belonging to the RWE Power Station in Niederaussem, Germany, was investigated by an elastic-plastic model, and its critical amplification coefficient was 2.31. It was therefore believed that the result obtained in this study was reasonable; and

3. The check of overall or local buckling required by the design codes is actually to control the minimum thickness of the shell structure and consequently to prevent buckling failure. These codes [DL/T 5339-2006 (National Development and Reform Commission of the People’s Republic of China 2006) and GB/T 50102-2003 (Ministry of Housing and Urban-Rural Development of the People’s Republic of China 2003)] check only the elastic stability, where the elastic-plastic behavior of concrete is not considered. The ultimate load-bearing capacity is also influenced by construction errors and geometrical imperfections. In order to have a sufficient safety coefficient, both the overall and local stability coefficients are required to be no less than 5. Consequently, the critical wind pressure with respect to the buckling failure is far larger than the one with respect to the material failure. Therefore, it was indicated that a cooling tower that met the requirement of the design codes would fail because of loss of material strength rather than buckling failure.

Parametric Analysis

Design codes indicate the stability of a cooling tower is directly related to the thickness of the shell structure, whereas the reinforcement ratio has little effect. If the cooling tower collapses because of loss of stability, the ultimate bearing capacity could be directly increased by improving the shell thickness. On the contrary, for a cooling tower that fails by loss of material strength, if the thickness of the shell structure is increased while the layout and amount of the reinforcement are kept the same, the ultimate bearing capacity cannot be enhanced. The ultimate bearing capacity would be relevantly improved if the reinforcement space along the thickness direction is increased as well as the thickness of the shell structure (the thickness of the concrete cover is kept the same).

Parametric analysis was conducted based on the numerical model to further investigate the effects of shell thickness, reinforcement space in the thickness direction of the shell structure, and reinforcement ratio on the ultimate bearing capacity and failure mode of the cooling tower. Six models were developed, as listed in Table 1. For the specimen nomenclature, M0 is the original model; M1 and M2 identify the models with a thicker shell structure but the same reinforcement as M0; M3 denotes the model with a larger reinforcement ratio but the same shell thickness as M0; and A and B represent improvement percentages of 25 and 50%, respectively. In M1, the reinforcement layout is kept as same as M0 and the thickness of the concrete cover is consequently enhanced. However, the thickness of the concrete cover in M2 is equal to that

<table>
<thead>
<tr>
<th>Model</th>
<th>Improvement of shell thickness (%)</th>
<th>Variation of concrete cover</th>
<th>Variation of reinforcement space</th>
<th>Improvement of reinforcement ratio (%)</th>
<th>Amplification coefficient $\lambda$ at collapse</th>
<th>Improvement of $\lambda$ (%)</th>
<th>Failure mechanism</th>
</tr>
</thead>
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<tr>
<td>M0</td>
<td>0</td>
<td>No</td>
<td>No</td>
<td>0</td>
<td>2.0</td>
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<tr>
<td>M1-A</td>
<td>25</td>
<td>Yes</td>
<td>No</td>
<td>0</td>
<td>2.0</td>
<td>0</td>
<td>Loss of material strength</td>
</tr>
<tr>
<td>M1-B</td>
<td>50</td>
<td>Yes</td>
<td>No</td>
<td>0</td>
<td>2.0</td>
<td>0</td>
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</tr>
<tr>
<td>M2-A</td>
<td>25</td>
<td>No</td>
<td>Yes</td>
<td>0</td>
<td>2.1</td>
<td>5</td>
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</tr>
<tr>
<td>M2-B</td>
<td>50</td>
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<td>Yes</td>
<td>0</td>
<td>2.2</td>
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<tr>
<td>M3-A</td>
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<td>No</td>
<td>25</td>
<td>2.3</td>
<td>15</td>
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</tr>
<tr>
<td>M3-B</td>
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<td>No</td>
<td>No</td>
<td>50</td>
<td>2.6</td>
<td>30</td>
<td>Loss of material strength</td>
</tr>
</tbody>
</table>

Note: All the calculation is based on the model M0; the improvement ratio is defined as $(\text{M}^* - \text{M0})/\text{M0} \times 100\%$. 

in M0, and the reinforcement space in the thickness direction of the shell structure is improved. Table 1 shows that increasing the shell thickness is not beneficial for the ultimate bearing capacity. In models M1-A and M1-B, the shell thickness was increased by 25 and 50%, respectively, whereas the critical wind pressure was kept the same with model M0. The collapse mode of model M1-B is shown in Fig. 11, which shows the initial cracks and final collapse mode to be similar to model M0. Comparison among models M0, M2-A, and M2-B implied that it was possible to improve the wind-resistant behavior by increasing the reinforcement space, but the efficiency was limited. When the reinforcement ratio was enhanced in models M3-A and M3-B, the critical wind pressure was significantly enhanced by 15 and 30%, respectively. However, the improvement of $\lambda$ was less than that of the reinforcement ratio. It was, therefore, confirmed that it is feasible to improve the wind-resistant behavior by improving the flexural bearing capacity of the shell structure. All six models had the same crack initiation position and final collapse mode as M0 (Figs. 11–13). All of them failed by loss of material strength, and the only difference was that the collapse process of models M3-A and M3-B lasted slightly longer than that of M0.

**Conclusions**

In this paper, a three-dimensional finite element model was developed to investigate the collapse behavior of a reinforced concrete
superlarge cooling tower subjected to strong winds. Failure because of loss of material strength and loss of stability was compared. Afterward, a parametric analysis was performed to further investigate the effect of some design variables. The following observations and conclusions can be made:

1. The numerical model proposed in this paper is suited for collapse analysis of cooling towers subjected to strong winds;
2. Under the wind action, the cooling tower, which met the requirement of design codes, failed because of loss of material strength rather than loss of stability. The critical wind pressure based on elastic stability analysis was far larger than that corresponding to material failure;
3. Improving the flexural bearing capacity of the shell structure is more beneficial than increasing the thickness of the shell structure for the wind-resistant behavior of a cooling tower; and
4. During the failure process, the upper part of the shell structure collapsed into the center of the tower. This was mainly because of the stress of the outer circumferential reinforcement in the upper windward side of the shell structure reaching the ultimate strength because of the nonuniform distribution of wind pressure along the circumferential direction. Afterward, fracture and breakage of the shell structure appeared as a result of material failure and the damaged region kept spreading. Finally, 4/5 of the shell body collapsed under the combined effects of wind load and self weight.

Future study is planned to take the wind-induced vibration effect directly into consideration when preparing the numerical simulation.

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