大韓土木學會論文集
第12卷 第4號 · 1992年 12月
pp. 59~70

構造工學

R/C 쌍곡 냉각탑의 극한 거동

Ultimate Behavior of Reinforced Concrete Hyperbolic Cooling Tower

閔昌植* · 金生彬**
Min, Chang Shik · Kim, Saeng Bin

요 지

風荷重을 받는 쌍곡 냉각탑의 非彈性, 非線形 極限 擧動을 Cray Y-MP 슈퍼 컴퓨터에 開發한 有限要素 컴퓨터 프로그램으로 硏究하였다. 유한요소 망(mesh)을 각각 잘게 잘라서 3 모델을 만들고, 이 모델들을 이용하여 탄성과 비탄성 해석으로 유한요소 망의 수렴관계(mesh convergence)를 연구하였다. 연구결과 유한요소의 크기가 냉각탑의 극한거동을 예측하는데 매우 중요한 역할을 하고있음을 볼수 있었다. 비록 쌍곡 냉각탑이 風荷重에 대해서 膜應力(membrane stress)으로 저항하나, 본 硏究 結果 휨應力(bending stress)도 냉각탑의 파괴와 擧動에 매우 중요한 역할을 하고 있음을 알아 내었다. 解析한 냉각탑은 형상값(Shape factor)이 1.48에 이르렀고, 이는 냉각탑의 자오선 응력(meridional stress)이 원둘레방향 으로 상당히 재분배 되고 있음을 보여주는 것이다. 이러한 재분배에 대한 사실은 배치된 철근의 항복이 바람방향 자오선으로 부터 30°에 까지 나타난 것으로 더욱더 뚜렷하였다. 현재의 탄성해석을 이용하는 냉각탑 設計 방법은 安全측에 있음을 보여 주었으며, 1 보다 큰 형상값을 설계시에 활용하기 위해서는 더욱더 많은 연구가 선행되어야 할 것이다.

Abstract

Inelastic nonlinear behavior of a hyperbolic cooling tower under wind loading is studied using a finite element program developed on a Cray Y-MP. Convergence studies for the elastic and inelastic analyses are performed using three mesh models. It is shown that the mesh convergence plays an important role in accurately predicting the inelastic behavior of a cooling tower. Even though the cooling tower resists the applied forces through membrane stresses, it is found that the bending stresses play an important role in the failure and behavior of the cooling tower. The present analysis gives a shape factor of 1.48, which indicates a significant redistribution of meridional stresses. It is further evidenced by the distribution of meridional reinforcement yielding which reaches up to 30° from the windward meridian. The present practice of using elastic analysis for calculating the design stresses appears to be at least safe and conservative. A more comprehensive study should lead to conclusions that would allow use of a higher-than-one shape factor, thus requiring less meridional reinforcement than the present design method does.

* 정회원 · 동국대학교 공과대학 토목공학과 강사
** 정회원 · 동국대학교 공과대학 토목공학과 교수

1. Introduction

Hyperbolic cooling towers are shells of double curvature that resist the applied forces primarily through inplane membrane stresses. These shells can be more than 150 m high and may be only 20～23 cm thick for most of the height. In current design practice for hyperbolic cooling towers,[1,2,14] stresses due to design loads from elastic analysis are used to design reinforcement in concrete shells based on pointwise limit state behavior.[8,10] A possible justification for the practice - elastic analysis and pointwise limit design - can be found from the lower bound theorem of plasticity according to which the design load gives a lower bound on the true ultimate when the design is performed using stresses that are in a state of equilibrium and do not violate an appropriate yield or failure criterion. The linear elastic analysis indeed provides an equilibrium state of stresses, and the same type of design procedure is used in other reinforced concrete structures also.[1] However, the reinforced concrete is not an elastic-perfectly plastic material as in the theorem. The validity of the present practice has been established through many years of analysis, testing, and experience for other types of structures, such as reinforced concrete frames. Unlike other structures, there is little experimental evidence, with respect to reinforced concrete shells or hyperbolic cooling towers. Therefore, in recent years, the ultimate behavior of the hyperbolic cooling tower has been studied analytically with the help of the nonlinear finite element method.[11,16,18,19]

The reinforcement is provided in the meridional and the circumferential directions. In the circumferential direction, the calculated elastic stresses are relatively small and do not require much reinforcement, and for the most part of shell the reinforcement is governed by minimum reinforcement of 0.35 percent.[2,3] In the meridional direction, the reinforcement is determined by the meridional stresses along the windward meridian under the design load combination - dead load and wind load (0.9D+1.3W).[1] The elastic analysis shows that the meridional tensile stresses diminish away from the windward meridian. In case of the beam bending, the yielding of extreme fibers does not mean a failure. A redistribution of stresses leads to an ultimate load higher than the yield load. Such a redistribution may also occur in hyperbolic cooling towers that would allow yielding to spread beyond the windward meridian. By comparing the ultimate wind loads obtained from the elastic analysis and the nonlinear inelastic analysis, it has been shown that a useful redistribution of stresses occurs in hyperbolic cooling towers.[11,20]

The same cooling tower, the one designed for the Grand Gulf Nuclear Power Station, was studied by Mang et al,[16,17] Milford and Schnobrich[19,20] and Gupta and Maestrini[11] using nonlinear finite element analysis computer programs. Gupta and Maestrini[11] have compared the results from the three studies that vary considerably. These variations can be attributed to the differences in the models used by the three teams: the type of finite element, the mesh size, the crack model, the tension-stiffening effect, concrete compression behavior, and the geometric nonlinearity. Gupta and Maestrini[11] reported significantly higher shape factors (redistribution of stresses) than did Mang et al[16,17] and Milford and Schnobrich.[19,20] Gupta and Maestrini suggested that the higher value may be attributed to the crack direction change feature with the related geometric matrix. Mang et al did not include the crack direction change feature, and Milford and Schnobrich included it without the associated geometric matrix.

The model that Gupta and Maestrini used did not include the effect of bending on the cracking of concrete and yielding of steel because the cooling tower is expected to resist the applied forces primarily through the membrane stresses. However, as we will see later in this paper, the circumferential bending plays an important role in the redistribution of meridional membrane stresses. Therefore, in the present study, we are using a model that accounts for the effect of bending on the cracking of concrete and yielding of steel.

None of the three teams performed a comprehensive convergence study on the finite element mesh size which may have an important effect on the results. Nonlinear finite element analyses are computationally quite intensive (the Gupta-Maestrini computer runs consumed over 30 hours of CPU on a mainframe IBM 3081). Performance of several analyses using successively refined finite element meshes would be practically impossible using the conventional mainframe computers, such as the IBM 3081. In the present study a reinforced concrete shell finite element computer program is developed on a Cray Y-MP supercomputer at the North Carolina Supercomputing Center, North Carolina, USA. The use of the supercomputer made it possible to perform a mesh convergence study that shows a significant effect of the mesh size on the computed response of the cooling tower.

1.1 Geometry and loading

The cooling tower designed for the Grand Gulf Nuclear Power Station, Port Gibson, Mississippi,[23] is used in this study. The equation of the shell meridian is presented by the following equation.[7]

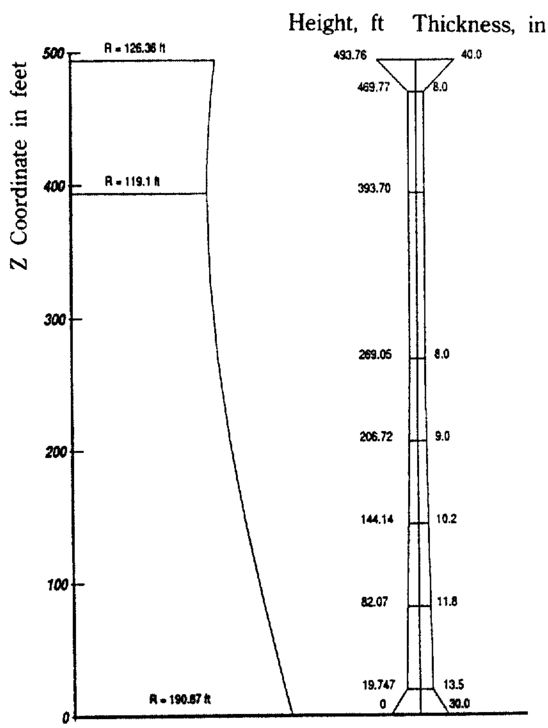
$$\frac{R^2}{a^2}-\frac{\bar{z}^2}{b^2}=1 \qquad (1)$$

where

R is the radius of the middle surface of the shell at an elevation z in meter,

$a=36.30$ m
$b=86.05$ m, above throat, $\bar{z}=z-120.0>$, and
$b=95.98$ m, below throat, $\bar{z}=z-120.0<0$.

The same tower has been used by other investigators like Mang et al,[16,17] Milford and Schnobrich[19,20] and Gupta and Maestrini.[11] The actual tower suffered a tornado accident before completion and was later repaired and completed.[7] Features included in the repaired towers are not considered here. We obtained the information about the geometry of the tower (including the thickness



Base of the Shell, Height from the Ground = 30ft

Fig. 1. Geometry and wall profile of the Grand Gulf Cooling Tower
($ft\times0.3048=m$, $in\times2.45=cm$)

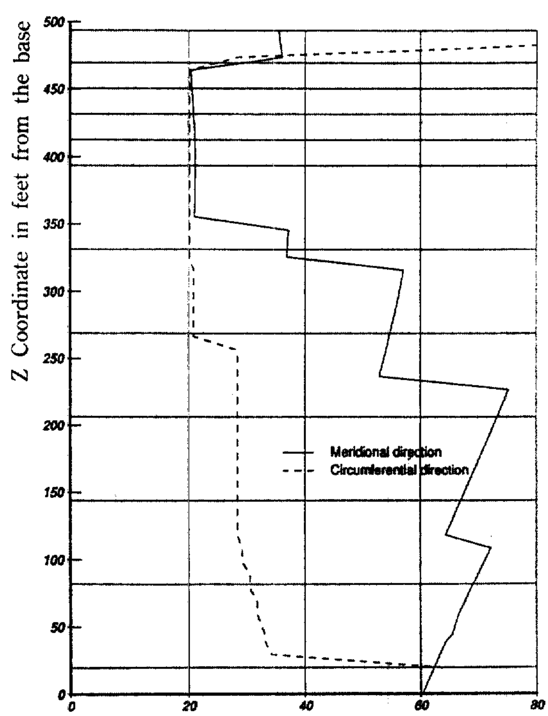


Fig. 2. Reinforcement design data from Zurn Industries
($Kips/ft\times1489.5=kg/m$)

variation) and the reinforcement distribution directly from the original design drawings[23] (see Fig. 1 and 2). This information is similar to that provided by the previous investigators, but is more detailed. The material properties used in the present study are similar to those for the previous studies.[16,19,21]

The wind loading recommended in the ACI-ASCE Committee 334 report[2] has been used. The circumferential variation of the loading is based on the coefficient given in Table 4.4.I(b) of the report. The vertical wind pressure distribution given in the report is identical to that specified in the ANSI/ASCE 7-88.[6] The table in the ACI-ASCE report gives the wind pressure coefficients at 13 angles along the circumference of the tower. The values of the coefficients for the intermediate locations are calculated by linear interpolation. The reference wind speed V_{f30} of 145 km/h (90 mph) for a 100-year return period has been used. It appears that the previous investigators did not account for the column height in calculating the wind load and assumed that the base of the shell is at the ground level which is not true. In consultation with the people on the original design team, it is assumed in the present study that the base of the shell is 9 m above the ground.

1.2 Method of analysis

As stated above a finite element computer program for reinforced concrete shells is developed on the Cray Y-MP supercomputer. The present computer program is a comprehensively revised version of the program developed by Akbar and Gupta[5] that was used in the Gupta and Maestrini[11] study. In addition to the supercomputing vector algorithms that were added, the present computer program discretized the 4-node superparametric shell elements into several concrete and steel layers to account for the effect of bending on the cracking of concrete and yielding of steel.[12,15] As did the original Akbar-Gupta program, the present computer program includes the rotating crack model proposed by Gupta and Akbar.[9] Appropriate selective integration schemes are used to avoid shear locking behavior before cracking of concrete[4,22] and after cracking.[9] The original computer program sets the transverse shear modulus parallel to a crack in concrete equal to zero. This particular feature introduces numerical instability in the solution. Further, as discussed by Min and Gupta,[21] it is known[13] and reasonable to assume that the transverse shear part of the deformation remains small relative to the bending deformation even after the concrete cracks. Therefore, in the present program the same unreduced transverse shear modulus is used before and after cracking. The present computer program also has an added feature of the shear retention factor for inplane shear parallel to the cracks. Even though the shear retention factor should not have any effect in a converged solution by definition,[21] it has been pointed out by many researchers[12,15] that including the factor improves the numerical stability of the solution process. However, in the present study we were not experienced any numerical instabilities possibly due to the computer program in use[21] includes the algorithm for the crack change[9] and the solution is driven by the displacement increment. Therefore, for the cooling tower study presented here the factor is taken to be zero. The variable convergence tolerance, 1%, 0.1% and 0.01% were used to see the effect of the criterion on the ultimate behavior. As the tolerance is getting finer the number of iteration is increased markedly, but has little effect on the overall bahavior of the load-deflection curves. The convergence tolerance was taken to be 1% of the maximum residual nodal forces at any load step.

The biaxial behavior of uncracked concrete and the uniaxial behavior of the cracked element are assumed to be linear elastic in compression and in tension. Reinforcing steel is assumed as an elastic-perfectly plastic material in tension and in compression, and is treated as an equivalent uniaxial material which can be smeared into the proper place in concrete layers. The effects of large deformation and tension-stiffening are not considered. The tension-stiffening would somewhat increase the cracked concrete stiffness,[13] but will not expect to affect the calculation of the ultimate load. The solution is driven by displacement increment (applied at the throat of the cooling tower) that can capture not only the ascending portion of the load-deflection curve, but also the descending portion of the behavior, which possess better stability when the solution reaches near the ultimate. The stiffness matrix is updated at each iteration to minimize the problems associated with abrupt changes in stiffness properties. The finite element program is verified by nonlinear analysis on several example problems and by comparison with experimental results.[21]

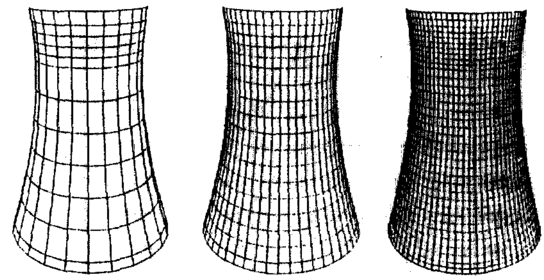


Fig. 3. Finite element models for the cooling tower
(a) 12×12 mesh (b) 24×18 mesh
(c) 36×36 mesh

2. Convergence Study

2.1 Finite element model

Only one half of the cooling tower needs to be analyzed because of the symmetry of the wind load about the windward meridian and the axisymmetry of the cooling tower geometry. Three models summarized in Table 1 are considered to study the mesh convergence. The half-circumference of the tower in the three models is discretized into 12, 24 and 36 elements (each element covering 15, 7.5 and 3.75 degrees), and the height of the tower into 12, 18 and 36 elements, respectively. Fig. 3 shows the cooling tower finite element models, 12×12 mesh, 24×18 mesh and 36×36 mesh, respectively. In the present study and in the study by the other investigators[11,16,19] the shell is assumed to be hinged on a rigid foundation. Even though the actual tower is supported on several inclined columns, this assumption is not likely to change the overall behavior of the shell.

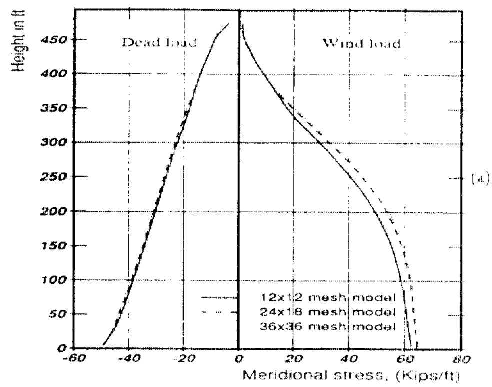
2.2 Elastic analysis

To show the convergence among the three mesh models, we first obtain the results from the elastic analysis by applying wind load and dead load separately. From the viewpoint of the present study the meridional stresses are most significant. Therefore, only meridional stresses from the three models will be compared. The meridional variation of the meridional stresses due to dead and wind loads from the three mesh models is shown in Fig. 4(a).

The stresses in each element are calculated at five points: at four points of the 2×2 Gaussian quadrature, plus at one point of the 1×1 Gaussian quadrature. For each column of elements in the present models, we can study the stress variation along three meridians, two meridians going through the 2×2 quadrature points, and one going through the 1×1 quadrature point. To study the convergence, the stresses are plotted in Fig. 4(a) along a 2×2 quadrature meridian closest to the windward meridian. For the three models, the three meridians are at 3.17°, 1.58° and 1.06°, respe-

Table 1. Parameters of the finite element models

Model	12×12	24×18	36×36
Number of elements in circumferential direction	12	24	36
Number of elements in meridional direction	12	18	36
Total number of elements	144	432	1,296
Number of nodes	169	475	1,369
Number of degrees of freedom	743	2,201	6,551
Semi-bandwidth	70	130	190



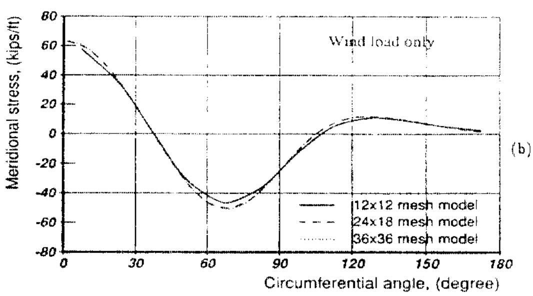


Fig. 4. (a) Vertical distribution of the meridional stress due to dead and wind loads at 3.17°, 1.58° and 1.06° from the windward meridian; (b) Circumferential distribution of the meridional stress due to wind load at 15.5 m, 17.7 m and 18.0 m from the bottom of the shell, respectively- Elastic analysis

ctively. Since the meridional stresses caused by the wind load are varying circumferentially, the location of the meridian at where the highest wind-induced stresses are inspected introduces an additional parameter in the convergence study. The figure shows that the meridional stresses due to wind load along the inspection meridian in the 24×18 mesh and the 36×36 mesh models are quite close. For the dead load, on the other hand, both the applied forces and the stresses are axi-symmetric. Therefore, the circumferential location of the inspection meridian does not play a role. As such, even the 12×12 mesh model gives meridional stresses that are in good agreement with those given by the 24×18 and the 36×36 mesh models.

Table 2. Comparison of horizontal throat displacement and ultimate wind load factor obtained by elastic analysis

	Horizontal throat displacement	Ultimate wind load factor -elastic analysis
12×12 mesh	6.65 cm	1.77 (6.6%)*
24×18 mesh	6.81 cm	1.68 (1.2%)
36×36 mesh	6.78 cm	1.66

*Ratio increased from wind load factor 1.66.

The circumferential variation of meridional stresses due to wind load from the three mesh models is shown in Fig. 4(b). It is not possible to pick up the stresses at the same height in the three models. However, it can be observed from Fig. 4 (a) that the variation in the meridian stresses among the three heights, 15.5 m, 17.7 m and 18.0 m, for the three models, respectively, is negligible. Again, the curves for the 24×18 mesh match very closely with those of the 36×36 mesh.

Table 2 shows a comparison of the horizontal throat displacement under the wind load only. The table also shows the ultimate wind load factors (λ_{ult}) given by: $\lambda_{ult}=(A_s f_y - N_y^D)/N_y^W$, where A_s is steel area, f_y is yield stress of steel, N_y^D is the meridional stress resultants due to dead load, and N_y^W is the meridional stress resultants due to wind load. The horizontal throat displacements due to wind load are very similar from the three models. The wind load factor is only slightly higher for the 12×12 and 24×18 meshes than that for the 36×36 mesh. From the elastic analysis point of view, we can conclude that the 24×18 mesh gives stresses that are practically converged and that the 12×12 mesh gives results that are acceptable.

2.3 Inelastic analysis - Multi-layer model

The bending element ("multi-layer model") is divided into ten concrete layers and four steel layers. The concrete cross-section area is not reduced to compensate for the presence of the reinforcing steel. For the inelastic analysis with the multi-layer model we use the same models: 12×12 mesh, 24×18 mesh and 36×36 mesh models.

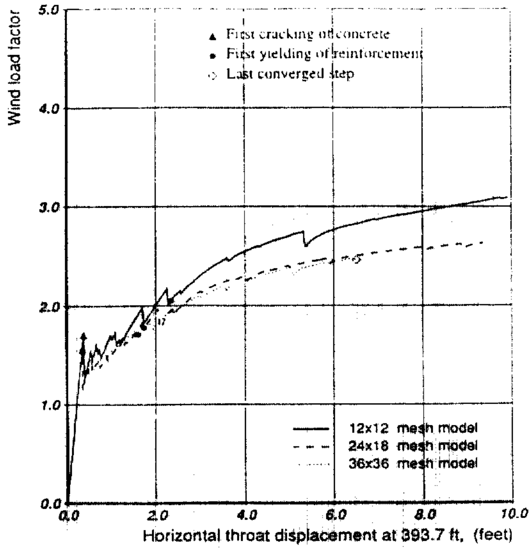


Fig. 5. Load-deflection curves for the multi-layer models

Fig. 5 shows a comparison of load-deflection curves obtained by the inelastic analysis with the multi-layer 12×12, 24×18 and 36×36 mesh models. It also shows the positions for the initial concrete cracks and the initial reinforcement yieldings for three models. The diamond symbol at the end of the curve for the 36×36 mesh model indicates the last converged step for the model.

For the 12×12 mesh and the 24×18 mesh models, the analyses are stopped after the models gained relatively large horizontal throat displacements without an apparent failure. The slope of load-deflection curves toward the end is quite small as compared to the slope at the early stages of the loading. That means the shell may continue to deform without resisting much more load. Thus, the cooling tower must be close to failure. Further, the present model does not consider the effect of large deformations. Therefore, it is considered prudent to not go beyond the present 2.7~3 m (9~10 ft) throat displacement. The 36×36 mesh model reaches the ultimate wind load factor 2.46 with the horizontal throat displacement 1.98 m (6.5 ft). We should use the 36×36 mesh model to evaluate the ultimate load. We may use the 24×18 mesh model for an approximate analysis.

3. Behavior of cooling tower

3.1 Crack and yield patterns

Fig. 6 and 7 give the progressive development of cracks on the inside surface and the outside surface concrete layers, respectively. In Figures 8 and 9, the progressive yield patterns on the outer and the inner circumferential reinforcement layers are shown, respectively, and Fig. 10 and 11 give the yield patterns on the outer and the inner meridional reinforcement layers, respectively.

The cracks formed in the elements close to the windward meridian are relatively horizontal, and in those away from the windward meridian, the cracks become progressively inclined (about 45° with respect to meridional direction). Slightly inclined cracks in the windward row of elements are necessary for transferring shear from the windward meridian direction to the circumferential direction. The cracks in this area (at circumferential angles less than 45° and below the throat of the shell) are mainly membrane through cracks. Beyond the 45° circumferential angle and above the throat of the shell, on the other hand, the cracks are formed mainly due to the bending deformations. We can observe a practically horizontal series of cracks in the 17th strip of elements in the crack patterns for the 36×36 mesh model (at 1.98 m throat displacement). These horizontal cracks lead to the nonconvergence in this model. These cracks cause a loss of capability in the shell to redistribute the meridional stresses in the circumferential direction.

The yield patterns indicate also a similar combination of membrane and bending deformations in the cooling tower. For the two circumferential reinforcement layers, the different yield regions indicate that the shell primarily deforms in bending in the circumferential direction. On the other hand, for the meridional reinforcement layers in Figures 10 and 11, the yield regions are very similar, which indicates that the meridional deformation is predominately membrane. In two meridional reinforcement layers, the yielding zones are

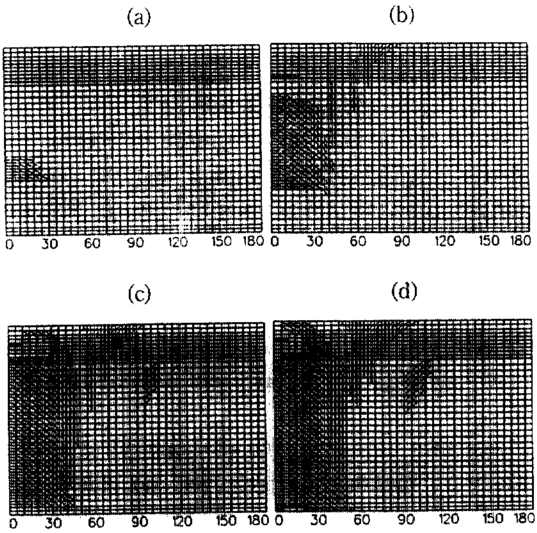


Fig. 6. Crack patterns on the outermost concrete layer at throat displacement (a) 0.11 m, (b) 0.49 m, (c) 0.94 m, and (d) 1.98 m

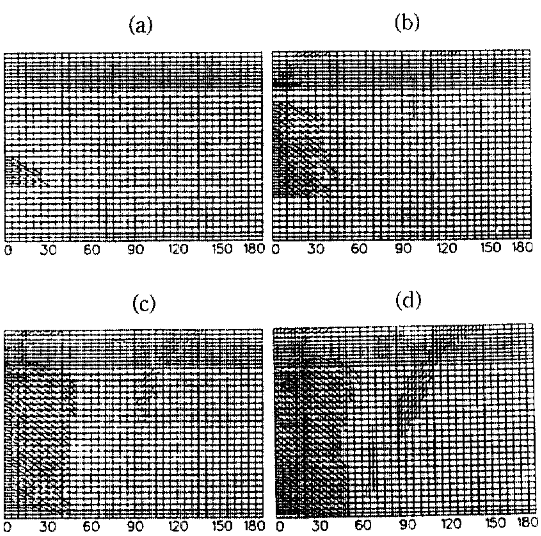


Fig. 7. Crack patterns on the innermost concrete layer at throat displacement (a) 0.11 m, (b) 0.49 m, (c) 0.94 m, and (d) 1.98 m

formed in horizontal strips. The formation of strip-like yielding zones is attributed to the uneven design reinforcement quantity.[21] The failure mode for the hyperbolic cooling tower under the wind

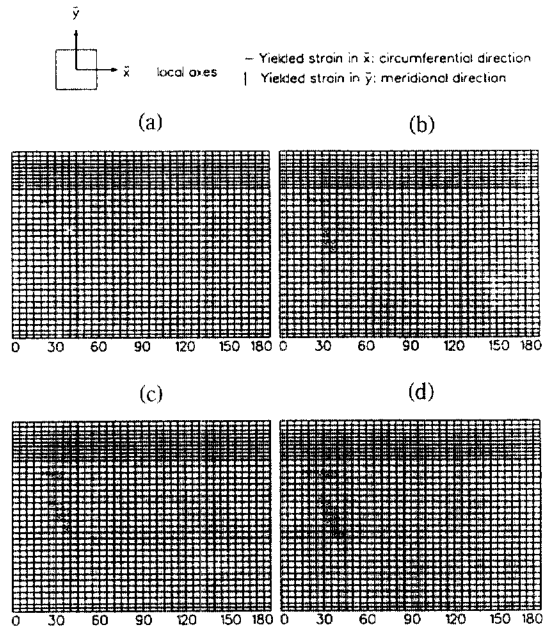


Fig. 8. Yield patterns on the outer circumferential reinforcement layer at throat displacement (a) 1.46 m, (b) 1.71 m, (c) 1.85 m, and (d) 1.98 m

loading can be viewed as a membrane meridional failure precipitated by the loss of circumferential bending capacity.

4. Comparison with other results

There are four sets of analyses for the same Grand Gulf Cooling Tower: Mang et al,[16,17] Milford and Schnobrich,[19,20] Gupta and Maestrini,[11] and the present study. As discussed earlier, the ultimate load reported by Gupta and Maestrini is based on a numerical instability which is not related to the actual failure mode. Therefore, their results are not compared here. There is a small variation in the cracking strength of concrete used in each study, 32.5 kg/cm^2 in Mang et al and the present study and 30.6 kg/cm^2 in Milford and Schnobrich. This minor difference in the cracking strength is not likely to affect the results noticeably.

The ultimate wind load factor predicted by the

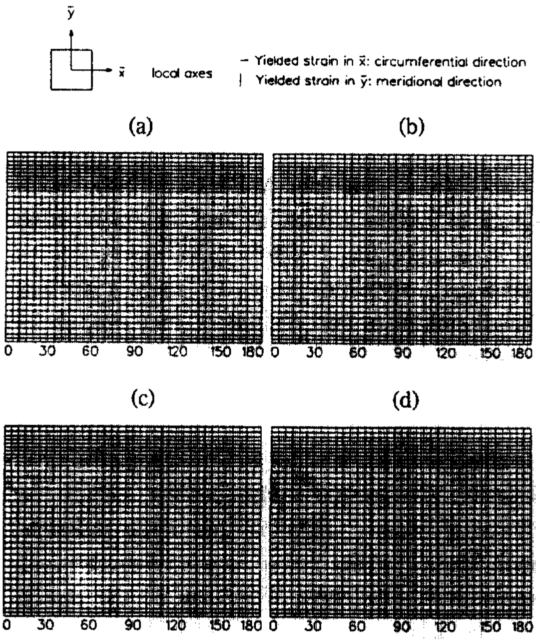


Fig. 9. Yield patterns on the inner circumferential reinforcement layer at throat displacement (a) 0.94 m, (b) 1.46 m, (c) 1.71 m, and (d) 1.98 m

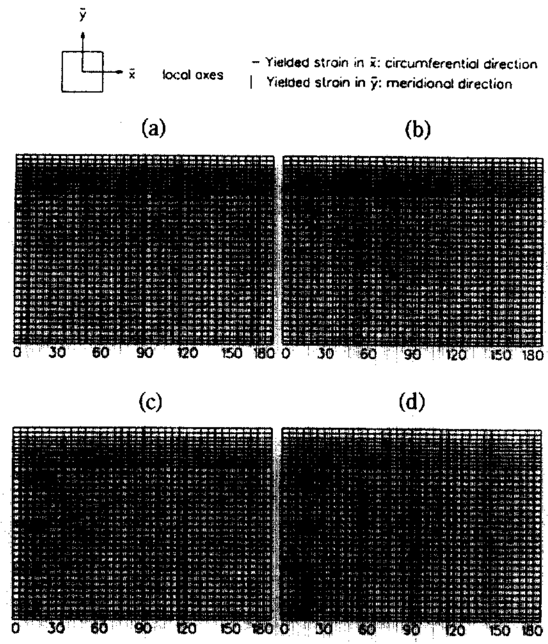


Fig. 10. Yield patterns on the outer meridional reinforcement layer at throat displacement (a) 0.55 m, (b) 0.94 m, (c) 1.46 m, and (d) 1.98 m

Table 3. Comparison of the shape factors from various nonlinear analyses for Grand Gulf Cooling Tower

	Variables				
	Geometric nonlinerity	Tension stiffening	Concrete cracking strength	Crack direction change	Shape factor
Mang et al.[16,17]	X	X	Full		1.0
	X		Full		0.95
		X	Full		1.17
Milford & Schnobrich[19,20]	X	20%	Full	X+	1.38
		20%	Full	X+	1.45
	X	5%	Full	X+	1.15
	X	10%	Half	X+	0.89
Present study			Full	X	1.48

+ A geometric matrix is not included.

elastic analysis performed by the three teams is 1.66, 1.52 and 1.66 for Mang et al, Milford-Schnobrich and the present study, respectively. There are differences in the three sets of analys is related to various computational and material factors. As discussed earlier, for the present study, we adopt the original design data from Zurn Industries,[23] which is slightly different from that used

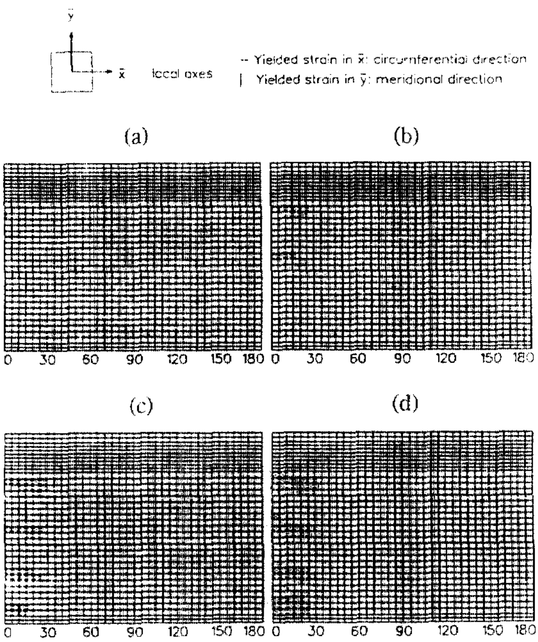


Fig. 11. Yield patterns on the inner meridional reinforcement layer at throat displacement (a) 0.49 m, (b) 0.94 m, (c) 1.46 m, and (d) 1.98 m

by others. We also consider the height of the supporting columns to calculate the wind load.

As introduced by Gupta and Maestrini,[11] we are also going to use the 'shape factor' for the purpose of comparison. The shape factor is the ratio of the ultimate wind load given by a nonlinear analysis and that obtained from an elastic analysis. A shape factor of unity means that there is no useful redistribution of meridional stresses. The shape factors from the three sets of analyses along with various variables are summarized in Table 3. Gupta and Akbar[9] have shown that the absence of the crack direction change feature can result in underestimating the ultimate load even in an element. Mang et al[16,17] do not include this feature. Milford and Schnobrich[19,20] on the other hand, include the feature, but do not include a geometric matrix[9] which is needed to 'drive' the crack direction change beyond reinforcement yielding.

The displacement increment drive used in the present study gives a computational algorithm that is much more stable than does the load increment drive because the displacement increment drive can predict not only the load-deflection curves with positive slopes but also those with negative slopes. Mang et al and Milford and Schnobrich appear to use the monotonic load increment drive for their algorithm, which is more susceptible to numerical instabilities when the analysis approaches the ultimate load. Therefore, they may underestimate the ultimate loads.

The results reported by Mang et al and Milford-Schnobrich have included and excluded the effect of tension-stiffening and the geometric nonlinearity. The present study does not include the effects of tension-stiffening and large deformations. Mang et al seemed to have a relatively weak tension-stiffening effect. This may be the reason why the ultimate strength is reduced by only 5% when they remove the tension-stiffening. On the other hand, Milford and Schnobrich found that the tension-stiffening has a more pronounced effect. When they reduced the tension-stiffening factor from 20% to 5%, the ultimate strength is reduced by 17%. Both Mang et al and Milford-Schnobrich found some effect of geometric nonlinearity. Milford and Schnobrich reported 5% of decrease in the ultimate load when the geometric nonlinearity is included. Mang et al have a greater decrease of 15%. It appears that Mang et al performed a limited mesh convergence study (only for elastic analysis) and that Milford and Schnobrich did not perform any. As is apparent from the present study, the mesh convergence plays an important role in accurately predicting the inelastic behavior of a cooling tower.

We believe that the other teams found the tension-stiffening and the concrete cracking strength to be relatively important parameters in the behavior and the ultimate strength of the tower because of the absence of the crack direction change feature or of the related geometric matrix. As shown by Min and Gupta,[21] the reduced cracking strength changes the stiffness of the tower in the initial stage of the loading, but has practically no effect on the ultimate strength. Similarly, we believe that adding tension-stiffening would somewhat increase the cracked concrete stiffness, but will not increase the ultimate strength. On the other hand, the reduction in the ultimate strength due to geometric nonlinearity observed by the two teams in the range of 5~15% appears to be reasonable. As stated earlier, the present study does not include the effect of the geometric nonlinearity; therefore, our shape factor should be appropriately reduced to account for the particular effect.

5. Conclusions

The ultimate behavior of the Grand Gulf Cooling Tower under wind loading has been examined by inelastic finite element analysis consisting of a 4-node superparametric membrane-bending shell element. It is found that the failure of cooling tower occurs by the yielding of meridional reinforcement when the meridional stresses can no longer redistribute because of the yielding of circumferential reinforcement in bending. Even though a cooling tower resists the applied forces primarily through membrane stresses, the failure is a mixed membrane and bending one.

Elastic and inelastic analyses are performed to study mesh convergence using three progressively refined models, 12×12, 24×18 and 36×36. The elastic analysis shows that the 24×18 mesh model gives results that are quite close to those from the 36×36 mesh model, and that the results from the 12×12 mesh model are acceptable. In the inelastic analysis, only the 36×36 mesh model is able to predict the failure. Although the 24×18 mesh model does not predict the failure, the load-deflection curve from the model is quite close to that from the 36×36 mesh model. The 12×12 mesh model generally exhibits a behavior similar to that exhibited by the two refined models. However, the 12×12 mesh model is relatively stiff and does not predict the failure.

The inelastic analysis predicts an ultimate wind load factor of 2.46 for a multi-layer 36×36 mesh model. In terms of shape factor, the present analysis gives a shape factor of 1.48, which indicates a significant redistribution of meridional stresses. When we compare the results obtained by Mang et al,[16] Milford and Schnobrich,[20] and in the present study, our shape factor of 1.48 is higher than those given by the other teams. There are various factors to explain to why the present study gives a higher shape factor. As we show in the present study, the mesh convergence plays an important role in accurately predicting the inelastic behavior of a cooling tower. It appears that the other teams did not perform any or performed a limited mesh convergence study. The other teams did not include the crack direction change feature or the related geometric matrix and also used the monotonic force increment drive. Both may result in underestimating the ultimate load. Both Mang et al and Milford and Schnobrich found some effect of geometric nonlinearity in the range of 5-15% reduction when they included the effect. The present study does not include the effect of the geometric nonlinearity. Therefore, our shape factor should be appropriately reduced to account for this effect.

The present inelastic analyses show that the results obtained from the elastic analysis are conservative and safe because the elastic analysis cannot account for the redistribution of stresses circumferentially that we see in the inelastic analysis. We may not be able to reduce the reinforcement in the direct proportion of the 1.48 shape factor. However, we can make the design more realistic by making use of the information obtained from an analysis of the type used in the present study. As supercomputers are becoming widely available, and as the cost of performing nonlinear analysis is coming down, it may be economically practical to perform an analysis of the type used in the present study in the actual design stage of a tower.

References

1. ACI 318-89. (1989). *Building Code Requirements for Reinforced Concrete* (ACI 318-89). American Concrete Institute, Box 19150, Redford Station Detroit, Michigan 48219.
2. ACI-ASCE. (1977). "Reinforced Concrete Cooling

Tower Shells - Practice and Commentary." *ACI Journal*, 22-31, January. ACI-ASCE Committee 334, Title No. 74-2.
3. ACI-ASCE. (1984). "Reinforced Concrete Cooling Tower Shells - Practice and Commentary." *ACI Journal*, November-December. ACI-ASCE Committee 334, Title NO. 81-52.
4. Ahmad, Sohrabuddin, Irons, Bruce M., and Zienkiewicz, O. C. (1970). "Analysis of Thick and Thin Shell Structures by Curved Finite Elements", *Int. J. for Numer. Meth. in Eng.*, 2, 419-451.
5. Akbar, Habibollah, and Gupta, Ajaya K. (1985). "*Membrane Reinforcement in Concrete Shells: Design Versus Nonlinear Behavior*". North Carolina State University, Raleigh, North Carolina 27695-7908, January. Reinforced Concrete Shell Research Report.
6. ASCE Standard. (1990). "Minimum Design Loads for Buildings and Other Structures". *Technical Report ANSI/ASCE* 7-88, American Society of Civil Engineers, NY, NY 10017-2398, July.
7. Gupta, A. K. (1978). "Grand Gulf Cooling Tower Study". *Technical report*, IIT Research Institute, Chicago, Illinois, December. Project J8325/Report No. 78J063.
8. Gupta, A. K. (1981). "Membrane Reinforcement in Shells". *J. Struct. Div.*, ASCE, 107(1), 41-56.
9. Gupta, A. K., and Akbar, H. (1984a). "Cracking in Reinforced Concrete Analysis". *J. Struct. Engrg.*, **110**(8), 1735-1746.
10. Gupta, A. K. (1984b). "Membrane Reinforcement in Concrete Shells: A Review". *Nuclear Engineering and Design*, **82**, 63-75.
11. Gupta, A. K., and Maestrini, S. (1986). "Investigation on Hyperbolic Cooling Tower Ultimate Behavior". *Engrg Struct.*, **8**, 87-92, April.
12. Hand, Frank R., Pecknold, David A., and Schnobrich, William C. (1973). "Nonlinear Layered Analysis of RC Plates and Shells". *J. Struct. Div.*, **99**(7), 1491-1505.
13. Hu, Hsuan-Teh, and Schnobrich, William C. "Nonlinear Analysis of Plane Stress State Reinforced Concrete Under Short Term Monotonic Loading". *Technical Report*, University of Illinois, Urbana, Illinois 61801, April 1988.
14. IASS Working group No. 3. (1979). "Recommendations for the design of hyperbolic or other similarly shaped cooling towers". Technical report, IASS, Madrid.
15. Lin, Cheng-Shung, and Scordelis, Alexander C. (1975). "Nonlinear Analysis of RC Shells of General Form". *J. Struct. Div.*, 101(3), 523-538.
16. Mang, H. A., Floegl, H., Trappel, F., and Walter, H., "Wind-Loaded Reinforced-concrete Cooling Towers: Buckling or Ultimate Load?" *Engrg Struct.*, **5**, 163-180, July (1983).
17. Mang, H. A., and Trappel, F. (1984). "Physically Linear Buckling Analysis of Reinforced Concrete Cooling Towers - Design Necessity or Academic Exercise?" In P. L. Gould, W. B. Kratzig, I. Mungan, and U. Wittek, editors, Natural Drought Cooling Towers, 279-297, Ruhr-Universitat Bochum, Germany, September 5-7. *Proceedings at the 2nd International Symposium.*
18. Meschke, Gunther, Mang, Herbert A., and Kosza, Peter, "Finite Element Analysis of Cracked Cooling Tower Shell". *J. Struct. Engrg., ASCE*, **117**(9), 2620-2639 (1991).
19. Milford, R. V. and Schnobrich, W. C. (1984). "Nonlinear Behavior of Reinforced Concrete Cooling Towers". Technical report, University of Illinois, Urbana-Champaign, Illinois 61801, May. *Structural Research Series* No. 514.
20. Milford, R. V., and Schnobrich, W. C. (1986). "The Effect of Cracking on the Ultimate Strength of Reinforced Concrete Cooling Towers". *Engrg Struct.*, **8**, 46-54, January.
21. Min, Chang Shik, and Gupta, Ajaya K. (1992). "A Study of Inelastic Behavior of Reinforced Concrete Shells Using Supercomputers". *Technical report*, Department of Civil Engrg., North Carolina State Univ., Raleigh, North Carolina 27695-7908, March 1992. Reinforced Concrete Shell Research Report.
22. Zienkiewicz, O. C., Taylor, R. L., and Too, J. M. (1971). "Reduced Integration Technique in General Analysis of Plates and Shells". *Int. J. for Num. Meth. Eng.*, **3**, 275-290.
23. Zurn Ind., (1977). *Design blue lines and copies: Shell Reinforcing-Tower 1 & 2, Grand Gulf Nuclear Station*, November. Provided by private communication from Zurn Balcke-Durr Industries, Inc., Cooling Tower Division, Tampa, FL 33631-3718

(接受 : 1992. 8. 19)