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Effect of soil-structure interaction on the reliability of hyperbolic cooling towers

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Abstract. A semi-stochastic process model of reliability was established for hyperbolic cooling towers subjected to combined loadings of wind force, self-weight, temperature loading. Effect of the soil-structure interaction on reliability was evaluated. By involving the gust factor, an equivalent static scheme was employed to convert the dynamic model to static model. The TR combination rule was used to consider relations between load responses. An analysis example was made on the 90M cooling tower of Maoming, Guangdong of China. Numerical results show that the design not including interaction turns to be conservative.

Key words: cooling tower reliability; interaction; TR load combination rule.

1. Introduction

After the occurrence of collapse of the cooling towers at Ferrybridge Power Station of Great Britain in 1965, researchers found that the safety factor in structural design did not guarantee the safety of the cooling towers as the designer had expected. It was surveyed that the wind speed of 1 min. term in the storm reached 38.9 (m/s), which made the immense tensile force at the meridional direction in the windward region of the tower shell reach the reinforcement yielding strength. It has been recognized that the wind loading is the main load in design of reinforcement concrete cooling tower structure. The meridional reinforcement of the tower shell is determined by the net tensile force, i.e., the difference of the meridional membrane force caused by wind loading and self-weight. It is the random variation of the wind loading that may lead to underestimation of the meridional membrane force in cooling tower design.

In this paper, from the point of view of reliability, the effect of variations of the wind loading, self-weight and temperature loading on the safety of the cooling tower, as well as the soil-structure interaction, are studied. The annual failure probability of structure is given. A semi-stochastic process model is achieved for reliability anaysis and by minimizing the safety margin

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for the limit state function of the structure, the dynamic wind force is transformed to equivalent static wind force and a time-variant reliability problem is converted to a time-invariant static reliability analysis.

A typical cooling tower structure system (shown in Fig. 1) consists of the soil half-space and the upper-structure (including the ring base, discrete supporting columns and the tower shell). The column system affects not only stiffness of the tower shell bottom, but also the stress distribution at the bottom. Gould and Lee (1971) used an self-balanced boundary loading to modify the solution based on continuous elastic supporting boundary and to consider the unstrained state of the shell bottom between columns. As to the soil-structure interaction, Gould *et al.* (1986) employed a model of finite element and energy transmitting boundary to study the seismic response under the interaction; Lu and Yang (1994, 1992) developed a hybrid method that combined a finite element method with a boundary element method, which can solve not only static problem but also dynamic problem. In this paper, the effects of soil-structure interaction and the discrete columns on the structural reliability are studied. The gust response factor obtained by random vibration analysis is involved to include the dynamic property of the wind load response. The loads are modeled as random variables. The TR load combination rule is used here to consider relations between wind load, dead weight and temperature load responses. The JC iteration for first-order second-moment method, which is effective in treating equations of non-normal variables, is chosen to calculate the failure probability for the nonlinear limit state equation.

2. Structural analysis model

The hybrid method (Yang *et al.* 1994, Lu *et al.* 1992) takes advantages of both FEM in solving problems of nonuniform finite regions and BEM in solving problems of infinite regions. The tower shell is discretized by curved axisymmetric shell elements based on Novozhilov's thin shell theory, the ring base is discretized by 3-D annual beam elements, and the soil half-space is modeled by direct BEM for rotationally axisymmetric bodies. The stiffness of the discrete supporting columns is added to that of the tower shell by virtual work principle.

The structural unknowns such as displacement components are expressed as Fourier series in the circumferential coordinate θ. Firstly the FEM solving model, based on the Fourier coefficients as the generalized displacements, is achieved. In static analysis, a self-balanced boundary loading is used to reflect the unstrained state of the shell bottom between columns (Gould 1971).

The dynamic problem is solved in the frequency space.

In the BEM, the soil half-space is treated as a rotationally axisymmetric body with a infinite radius. The discrete governing BEM equation for the nth circumferential harmonic can be formulated as,

$$H_S U_S = G_S P_S \tag{1}$$

where U_S and P_S are Fourier coefficients of the nodal displacement and traction vectors; H_S and G_S are matrices of known coefficients; and the superscript n has been omitted.

By using this BEM with a semi-analytical form, the original three-dimensional analysis is changed into a series of one-dimensional analyses. As a consequence, only the dimension in the direction of r on the surface of the soil half-space and the ring base need to be divided into boundary elements.

In this hybrid method, the governing equation for the whole system should be in the same form.

So, Eq. (1) should be transformed into the FEM form by mutiplying with $A_0 G_S^{-1}$ to get

$$K_S U_S = Q_S \tag{2}$$

where $K_S = A_0 G_S^{-1} H_S$, $Q_S = A_0 P_S$, A_0 is the diagonal matrix consisting of element areas, and constant stress elements in BEM have been chosen. In general, K_S is complex if viscous damping of the soil media is included in the material properties in dynamic analysis, Eq. (2) can be rewritten as,

$$(K_S + jC_S)\, U_S = Q_S \tag{3}$$

Based on the continuity condition of the displacement, the relation between displacement vector of the soil U_S and displacement vector of the ring base U_{rs} can be expressed as follows,

$$U_S = L_{rs} U_{rs} \tag{4}$$

where L_{rs} is a transformation matrix related to harmonic number n. Further we obtain

$$(K_{rs} + jC_{rs})\, U_{rs} = Q_{rs} \tag{5}$$

in which $K_{rs} = L_{rs}^T K_S L_{rs}$, $C_{rs} = L_{rs}^T C_S L_{rs}$, $Q_{rs} = L_{rs}^T Q_S$.

Combining the BEM equations in FEM form for the half-space and the FEM equations for the tower structure and the ring base yields the governing equations of the hybrid method of the structural system. But the frequency dependence of the soil properties is neglected here.

The wind load responses consist of two parts, one is from the mean wind speed and the other is from the fluctuating component. The stochastic wind gust response is supposed to be a stationary Gaussian process with mean zero. Finally, the rms. of the stochastic wind gust response χ is as,

$$\sigma_\chi(s, \theta) = [2\int_0^\infty S_\chi(s, \theta, f)\, df]^{1/2} \tag{6}$$

where $S_\chi(s, \theta, f)$ is the power spectral density of χ.

From above the gust response factor of χ is defined as

$$G_\chi = 1 + g\, \frac{\sigma_\chi(s, \theta)}{\bar{\chi}(s, \theta)} \tag{7}$$

where g is the peak factor, and $\bar{\chi}(s, \theta)$ is the response caused by mean speed.

3. Load model

3.1. Self-weight

The order of the meridional compressive membrane force caused by dead load is close to that of the meridional tensile membrane force caused by wind load. The self-weight of the structure is controlled by two factors, the chosen material properties and personal constructions. Its variability is far less than those of wind force and temperature loading. In this paper, the self-weight is modeled as a normal variable, with its mean value and standard deviation to be its nominal value and 0.05, respectively (Melchers 1987).

3.2. Wind load

The wind force acting on the surface of the tower shell is not only random with respect to time, but also random to both vertical and circumferential spatial coordinates. On the basis of linear elastic theory, the dynamic wind loading is converted to equivalent static loading which is assumed to have a determinate spatial contribution.

The wind pressure for cooling tower structure could be expressed as equivalent static (Niemann *et al.* 1980, American Concrete Inst. 1977). The formula for equivalent static wind loading can be written as,

$$p_e(z, \theta) = C_P(z, \theta) q_e(z) \tag{8}$$

in which z and θ are the vertical and circumferential coordinate, respectively; $q_e(z)$ is the equivalent static pressure; $C_P(z, \theta)$ is the pressure coefficient. The equivalent static pressure $q_e(z)$ as a function of height z, can be expressed as,

$$q_e(z) = \frac{1}{2}\rho[V(z) + \Delta v(t)]^2 \tag{9}$$

where $V(z)$ is the mean wind speed of 10 min. term; and $\Delta v(t)$ is the stationary fluctuating component which is assumed to be constant vertically. The computational scheme that follows will convert $\Delta v(t)$ to an equivalent static value.

The maximum meridional membrane force $MaxN_\varphi$ in the windward region is the controlling factor for RC cooling tower shell wind resistant design. And $MaxN_\varphi$ is in direct proportion to not only $q_e(z)$, but also the overturning moment of tower shell caused by the wind force. The later relation can be expressed as,

$$\frac{MaxN_\varphi}{\overline{N}_\varphi} = G_{N_\varphi} = \frac{\int_{H_1}^{H} C_F(z) q_e(z) z d(z) dz}{\int_{H_1}^{H} C_F(z) \overline{q}(z) z d(z) dz} \tag{10}$$

where G_{N_φ} is the gust response factor of meridional membrane force; the ratio $(MaxN_\varphi / \overline{N}_\varphi$ represents the dynamic factor of membrane force N_φ; $\overline{N}_\varphi$ is the response caused by mean wind pressure $\overline{q}\ (z)$; $C_F(z) = \int_0^{2\pi} C_P(z, \theta)\cos\theta d\theta$ is the sectional damping factor of the tower shell; $d(z)$ is the diameter of z height; H is the total height of the tower; and H_1 is the height of the supporting columns. Solving the equation above about $\Delta v(t)$, the equivalent static pressure can be written as,

$$q_e(z) = \frac{1}{2}\rho V_{10}^2 \left(\frac{z}{10}\right)^{2\alpha} \left[1 + 2\left(\frac{H}{2}\right)^{\alpha} K(\alpha, G_{N_\varphi}) + \left(\frac{H}{2}\right)^{2\alpha} K^2(\alpha, G_{N_\varphi})\right] \tag{11}$$

where α is the exponent of wind speed profile; the factor $K(\alpha, N_\varphi)$ is a function of α and G_{N_φ}; and V_{10} is the mean wind speed at 10m height. The exponent α is supposed to be 1/7, corresponding to open terrain. The gust factor G_{N_φ} is treated here as the specific value which represents the dynamic property of the cooling tower system of lower soil and upper structure.

3.3. Temperature loading

The meridional membrane forces corresponding to temperature loading are obtained as follows,

$$N_\varphi = N_\theta = -\frac{Eh}{1-\upsilon}\alpha \times t_0(z,\theta) \tag{12}$$

in which E and υ are the Young's Modulus and Poisson's ratio, respectively; α is the coefficient of thermal expansion; h is the thickness of the shell at z height; and $T_0(z, \theta)=1/2(T_{ex.}+T_{in.})$ is the average temperature of interior and exterior wall of the tower shell, whose calculation refers to [Code NDGZ5-88]. Suppose that the interior wall temperature has a determinate vertical contribution, so $T_0(z, \theta)$ can be expressed as,

$$T_0(z,\theta) = T \times a(z) + b(z) \tag{13}$$

where T is the air temperature outside the tower; $a(z)$, $b(z)$ are two spatial contribution coefficient. According to the local meteorological statistical figures, T is assumed to be a normal vairable with its mean value and standard deviation to be 23.1°C and 13.3°C, respectively. So, the temperature load is formulated as linear expression of random variable T.

4. Limit state and load combination rule

Through analysis of limit bearing capacity of the cooling tower (Lu *et al.* 1989), it is clear that the failure mechanism of the tower shell is of strength instead of buckling. The limit state for the cooling tower failure is defined here as the membrane tensile force at the meridional direction in the windward region on the bottom of the tower shell exceeding the tensile yield strength of the meridional reinforcement, which represents the dominant failure mode of the cooling tower. The limit state equation is written as,

$$g\ (N_\varphi,\ r) = R - N_\varphi = A_g\ \sigma_y - (N_{\varphi w} - |N_{\varphi g}| + N_{\varphi t}) = 0 \tag{14}$$

in which N_φ is the load response of meridional membrane force; and R is the meridional resistance; A_g is the reinforcement area of unit circumferential are length, σ_y is the reinforcement yield strength; and $N_{\varphi w}$, $N_{\varphi t}$ and $N_{\varphi g}$ are the wind load response, temperature load response, and dead load response at opposite direction, respectively. Further we can obtain $N_{\varphi w}=V_{10}^2 N_{\varphi w0}$, in which $N_{\varphi w0}$ is the load response when V_{10}=1.0 (m/s); The temperature load response is as

$$N_{\varphi t} = T \times N_{\varphi ta} + N_{\varphi tb} \tag{15}$$

where $N_{\varphi ta}$ and $N_{\varphi tb}$ represent the load responses when $T_0(z, \theta)=a(z)$ and $T_0(z, \theta)=b(z)$, respectively.

The random variation of the total load response comes mostly from the wind load response because of the two following reasons

I. the variability of dead load is far less than those of the wind and temperature load;

II. the temperature load contribution is very small in the sum total of responses.

The wind load becomes the controlling factor in the load combination. The TR load combination rule, proposed by Turkskra *et al.* has been chosen in this load combination case, we maximize the total load response in 1 year time and obtain as,

$$\underset{t\in[0,T_0]}{Max}\ N_\varphi(t) = \underset{t\in[0,T_0]}{Max}\ V_{10}^2(t) \times N_{\varphi w0} - |N_{\varphi g}| + T \times N_{\varphi ta} + N_{\varphi tb} \tag{16}$$

in which T_0=1 year; $\underset{t\in[0,T_0]}{Max}\ V_{10}$ is the 10 min. annual maximum mean wind speed of height 10m; T is the random variable of average air temperature in a year; and $N_{\varphi g}$ is the random variable of dead load response.

The annual maximum mean speed $\underset{t\in[0,T_0]}{Max}\ V_{10}$ is generally described by an extreme value type I distribution, whose cumulative distribution function is $F_I(V)=\exp\{-\exp[-\alpha(V\text{-}\mu)]\}$, in which α and μ are the mode and slope of the function, respectively.

The combination rule of Eq. (16) has two indicated meanings as follows

I. the dead load and temperature load are permanent loads acting continuously on the structure;

II. the annual maximum wind speed occurs independently of the lowest temperature in a year.

Besides, the resistance $R=A_g\sigma_y$ is also controlled by the materials used in construction and permanent installations. Here the random variable R is modeled as a normal variable with a coefficient of variation to be 0.03 and a mean value to be its nominal (Melchers 1987).

5. Reliability evaluation

The first-order second-moment reliability method (so-called FOSM method) used in this paper is the most popular with those wishing to obtain the safety index β in reliability analysis. For the nonlinear limit state equation, we choose JC iteration method, which proved especially effective in this case, to calculate the final index β and annual failure probability. The comprehensive limit state equation can be written as,

$$g(MaxV_{10}, T, N_{\varphi t}, R) = R - MaxV_{10}^2 N_{\varphi w0} - |N_{\varphi g}| + TN_{\varphi ta} + N_{\varphi tb} = 0 \tag{17}$$

which is treated as a equation with 4 random variables.

In JC iteration, firstly the original variables x_i are transformed to equivalent normal variables. The transformed normal variables have the same CDF (Cumulative Distribution Function) value and PDF (Probability Density Function) value as the original distribution at the design point. Then the design point on the failure surface in the basic variable space is checked. The final safety index β equals to the distance from the design point to the orign, and the failure probability is as $P_f=\Phi(-\beta)$.

6. Numerical results

The 90m natural-draught hyperbolic cooling tower at the Maoming Power Station, Guangdong, China, whose design period is 60 years, is chosen to be analyzed. Its geometry and dimensions including upper structure and lower soil are shown in Fig. 1. The shell meridian is defined by $r^2=19.4^2+0.177\times z^2$, and its thickness is determined by $h=0.14+0.36\times\exp[-2\times(66.2+z)/19.4]$. The number of pairs of supporting columns is 40. And reinforcement area unit length Ag on the bottom section of the chosen tower shell is 22.5 cm^2/m, and the tensile yield strength $\sigma_y=2.9\times10^5$ (KN/M^2). All concrete members have the following properties: Young's modulus, $E=2.7\times10^7$ (KN/M^2); Poisson's ratio, $\nu=0.167$; and weight density, $\gamma=24$ (KN/M^3). The soil media has the following properties: Young's modulus, $E_S=5.88\times10^4$ (KN/M^2); Poisson's ratio, $\nu=0.167$; weight density, $\gamma=24$ (KN/M^3); and internal viscous damping ratio $\eta=0.1$.

According to calculations in the study (Yang and Lu 1992), the gust response factor G_{N_φ} equals to

$$G_{N_\varphi}=2.10 \text{ (interaction included)}$$

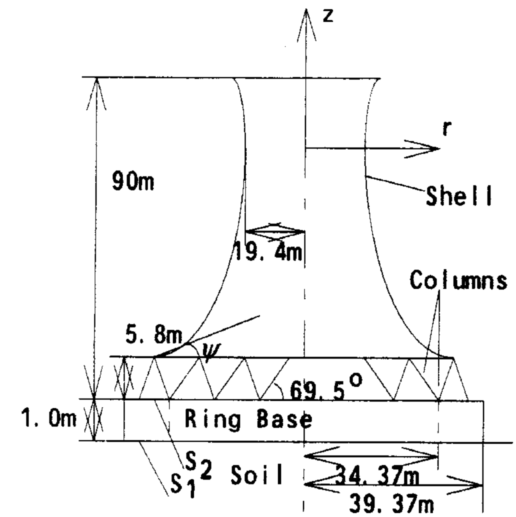

Fig. 1 Cooling tower structure system $PG(z)=q_e(z)/(1/2)\rho V_{10}^2$

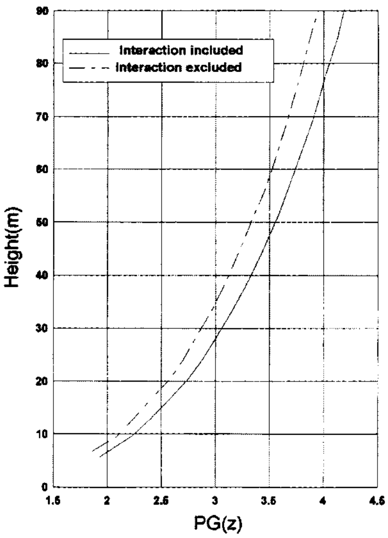

Fig. 2 Equivalent static pressure

$$G_{N_\varphi}=2.24 \text{ (interaction excluded)}$$

whose two curves of equivalent static pressure are shown in Fig. 2.

A simplified scheme for reliability computation has been proposed to check the results, in which the $N_{\varphi t}$ caused by temperature load is neglected, and $N_{\varphi g}$ caused by self-weight and the resistance R are treated as determinate. The simplified limit state equation is expressed as,

$$g(MaxV_{10}) = R - MaxV_{10}^2\ N_{\varphi w0} - |N_{\varphi g}| = 0 \tag{18}$$

This simplification is based on the facts that the variability of self-weight load as well as the resistance R is much less than that of the wind load, and the load response contribution from temperature is much (about 10^{-2} order) less than that of the self-weight and wind load.

The annual failure probability for simplified limit state of Eq. (18) under assumptions including continuous elastic supporting boundary and discrete column supporting boundary, are shown in Table 1. Two extreme value type I models for annual maximum wind speed $\underset{t\in[0,T_0]}{Max} V_{10}$ are chosen here as,

i) model (1) α=0.450, μ=17.78 (m/s) (Zhu 1972)
ii) model (2) α=0.382, μ=19.25 (m/s) (London 1971)

Table 1 Annual failure probability for simplified limit state equation

Annual failure probability Pf=$\Phi(-\beta)$		Interaction included G_{N_φ}=2.10	Interaction excluded G_{N_φ}=2.24
Model (1)	Continuus support	0.0013	0.0069
α=0.382 μ=19.25	Discrete support	0.0386	0.0609
Model (2)	Continuous support	0.00009	0.00021
α=0.450 μ=17.78	Discrete support	0.0114	0.0196

Table 2 Annual failure probability for comprehensive limit state equation

Discrete support	Model (2) α=0.450 μ=17.78 (m/s)
Interaction included	0.0088
Interaction excluded	0.0154

The later model is from the local statistic figures.

Results for comprehensive limit state Eq. (17) are given in Table 2, where the later model (2) is used.

7. Conclusions

i) The continuous elastic supporting assumption makes the N_φ on the bottom section of the tower shell more smoothly circumferentially distribute, therefore reduces evidently the failure probabilities.

ii) A part of the energy caused by action of external loadings has been absorbed by the lower soil when considering the structure-soil interaction. This also reduces the structural failure probabilities. Reliability assessments regardless of the interaction tend to be conservative.

The whole analysis above falls generally within the so-called level II category within the framework of reliability analysis.

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