Seismic Performance Analysis of the Cooling Tower Structure in Kunshan High-tech Industrial Heritage Park

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Abstract. The cooling tower, standing at a height of 70 meters, features a reinforced concrete hyperbolic cylindrical wall structure. Seismic performance analysis was conducted using Rhino and ABAQUS software, considering multi-directional seismic time-history effects. The dynamic elastic-plastic time-history analysis under multi-directional seismic actions demonstrated that the yield development of the structure is reasonably progressive, indicating good overall seismic performance, with all structural components meeting the required performance levels.

Keywords: Cooling Tower, Seismic Performance, Time History Analysis, Static ElasticPlastic Analysis, Finite Element Simulation.

1 Introduction

China's 20th-century industrialization spurred the construction of numerous thermal power plants, with cooling towers playing a key role in heat dissipation. These hyperbolic, reinforced concrete structures, though now obsolete due to technological advancements, have become significant industrial heritage sites, often preserved and repurposed in urban renewal projects.

Due to their structural complexity and height, cooling towers are particularly vulnerable to seismic forces, making seismic performance assessment crucial for their adaptive reuse. Previous studies have explored various aspects of their seismic behavior, including soil-structure interaction, combined horizontal and vertical seismic loads, and different supporting systems [1-4]. Methods like time-history and response spectrum analyses have been widely used to evaluate their seismic response.

This paper conducts a comprehensive seismic performance analysis of the cooling tower in Kunshan High-tech Industrial Heritage Park using finite element modeling (FEM). The study assesses structural integrity under multi-directional seismic loads, proposes retrofitting strategies, and provides a scientific basis for its preservation and

functional adaptation within urban redevelopment. The results aim to enhance understanding of cooling towers' seismic behavior and inform future conservation efforts.

2   Overview of the Cooling Tower in the Kunshan Xinyuan Power Plant Retrofit Project

The cooling tower is located on the north side of Xiaolin West Road in Kunshan, within the northwest corner of the central ring road, and south of Zhangjiagang River. It was originally part of the Kunshan Xinyuan Thermal Power Plant project, which has since been decommissioned. The cooling tower stands at 70 meters and adopts a hyperbolic reinforced concrete cylindrical wall structure, which ensures efficient heat dissipation and structural stability. The cylindrical wall is made of reinforced concrete, with a double-layer frame structure inside. The base of the cylindrical wall utilizes a concrete foundation and supporting columns to ensure the tower's stability and durability.

3   Research Methodology

In accordance with the "Design Code for Industrial Circulating Water Cooling Systems," natural draft cooling towers must consider the effects of self-weight, wind load, and seismic action. Consequently, Time-History Analysis and Response Spectrum Analysis were selected as the primary methods for dynamic structural analysis.

3.1   Time-History Analysis

Time-History Analysis is a numerical method that evaluates the response of structures under dynamic loads. This analysis involves solving the dynamic equilibrium equation:

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = F(t) \tag{1}$$

Where M is the mass matrix, C is the damping matrix, and K is the stiffness matrix, $u(t)$ is the displacement vector, $\dot{u}(t)$ is the velocity vector, $\dot{u}(t)$ is the acceleration vector, and $F(t)$ represents the time-varying external force, such as seismic loads. Numerical integration methods like the Newmark-β or the Wilson-θ [5] method are applied to solve this equation incrementally over time, allowing the computation of time-varying structural responses such as displacement, velocity, and acceleration. This method effectively models the nonlinear behavior of materials, including yielding, damage, and failure.

3.2 Response Spectrum Analysis

Response Spectrum Analysis [6] simplifies the seismic response evaluation by using the structure's modal properties. The equation of motion is first decoupled into individual modal equations through eigenvalue analysis:

$$M\ddot{u} + C\dot{u} + Ku = 0 \tag{2}$$

The solution to each modal equation provides the maximum response for that mode, which is then combined using modal superposition techniques, such as the square root of the sum of the squares (SRSS) or complete quadratic combination (CQC) methods, to obtain the total response of the structure. The response spectrum provides the peak response values (displacement, velocity, acceleration) as a function of the natural frequency, derived from extensive seismic records and statistical analysis. This method is particularly effective in preliminary design stages, where a simplified and computationally efficient assessment of seismic response is required.

4 Finite Element Model Construction

The cooling tower consists of numerous components, making it challenging to create the geometric model directly in ABAQUS. Therefore, the geometric model was first constructed in Rhino software (as shown in Figure 3) and then imported into ABAQUS for finite element analysis.
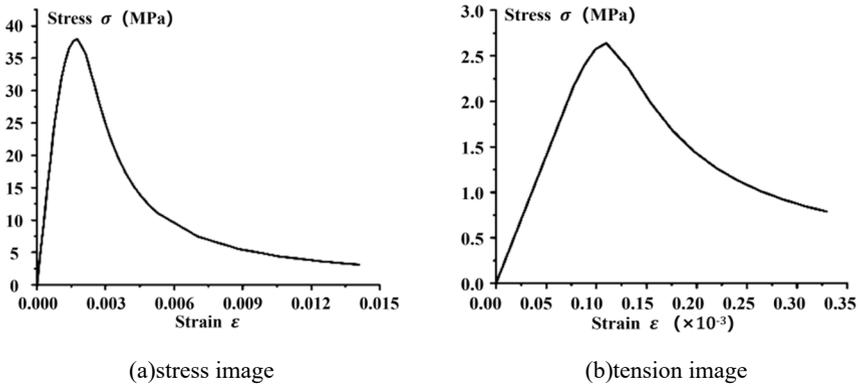
In the model, shear walls, slabs, and ventilation shell parts were modeled using layered shell elements with reduced integration four-node shell elements (H4R), where the reinforcement was equivalent to steel plates of equal volume acting together with the concrete. Beams and columns were modeled using truss elements, with the three-dimensional two-node truss elements (T3D2) chosen for the element type. The layered shell elements are based on composite material mechanics principles, dividing a shell element into multiple layers, each with different thicknesses and material properties (e.g., concrete, reinforcement) [7] . Each layer can define several cross-sectional integration points, where stress and strain are calculated independently during finite element analysis, allowing precise capture of nonlinear material behavior. Additionally, the layered shell elements consider the coupling between in-plane bending, in-plane shear, and out-of-plane bending, thoroughly reflecting the spatial mechanical performance of shell structures.

For concrete materials, ABAQUS provides the Concrete Damaged Plasticity (CDP) model, which is a model developed based on plasticity and continuum damage theory, widely used for analyzing the behavior of reinforced concrete structures under monotonic, cyclic, and dynamic loads. The CDP model is particularly suitable for simulating the complex mechanical behavior of reinforced concrete structures and is one of the most commonly used models in this field. Its failure mechanism is achieved by simulating damage evolution in concrete under tension and compression. The model introduces tensile and compressive stiffness damage factors (DAMAGET

and DAMAGEC) to simulate material performance degradation during loading, de-fined by *Concrete Tension Damage and *Concrete Compression Damage.

When using this model, the stress-strain relationships of concrete under uniaxial compression and tension conditions need to be defined. The stress-strain curves can be calculated using the formulas provided in the appendix of the "Code for Design of Concrete Structures" (GB50010-2010) and then input into ABAQUS for numerical analysis.

In the experiment, the concrete grade was C40, with an elastic modulus of 32500 MPa. According to the "Code for Design of Concrete Structures" (GB50010-2010), the Poisson's ratio was 0.2, and the density was 2400 kg/m³. The constitutive relation-ship obtained from the above equations is shown in Figure 1. In addition, the CDP model requires defining five plastic parameters: ϕ, e, $\frac{f_{bo}}{f_{co}}$, K_c, and λ.

(a)stress image   (b)tension image

Fig. 1. Concrete Constitutive Relationship.

In this study, the constitutive model for steel reinforcement adopts a bilinear iso-tropic hardening model, which divides the stress-strain relationship into an elastic stage and a hardening stage, with a linear relationship for each stage. The elastic modulus in the hardening stage is taken as 1/100 of the elastic stage's modulus. This model effectively reflects the stress-strain relationship of steel reinforcement while improving the convergence of the calculation. The stress-strain curve for steel rein-forcement is shown in Figure 2 and calculated by the following equation:

$$\sigma = \begin{cases} \varepsilon E_s & \varepsilon \le \varepsilon_y \\ f_y + k(\varepsilon - \varepsilon_y) & \varepsilon > \varepsilon_y \end{cases} \tag{3}$$

where E_c is the elastic modulus of steel, f_y is the yield strength, and ε_y is the yield strain.

The constitutive model for concrete adopts the CDP model, while the constitutive model for reinforcement steel uses a bilinear plasticity model. In the model, the rein-forcement in the beam-column elements is connected to the concrete through shared nodes, and the walls and beam-columns are connected via fine mesh grids at common

nodes. Fully fixed boundary conditions are applied at the base of the tower foundation and the bottoms of internal structural columns, i.e., with zero displacements and rotation constraints in the X, Y, and Z axes at the bottom of the shear walls.
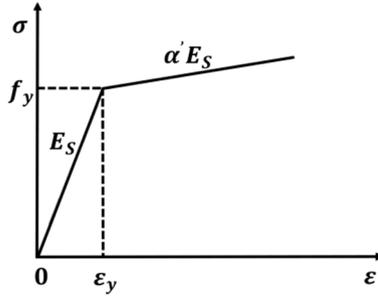

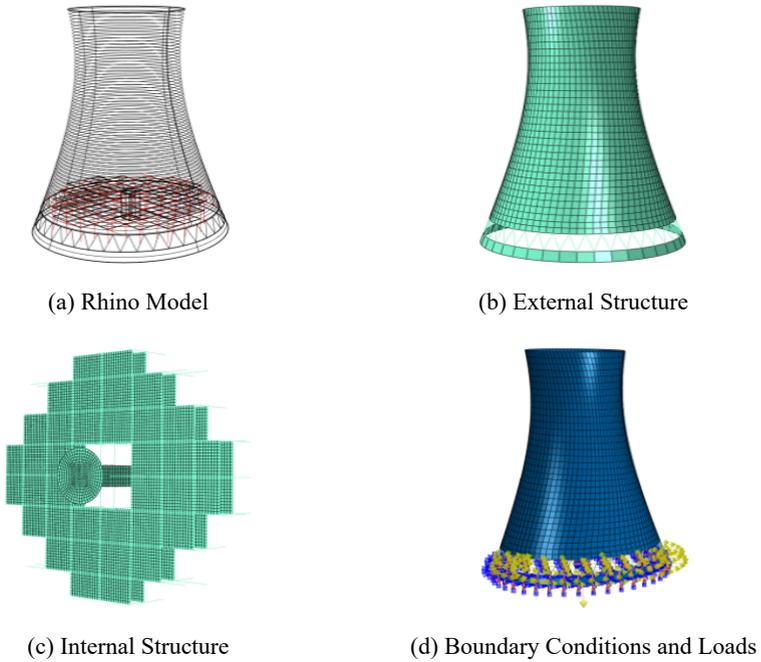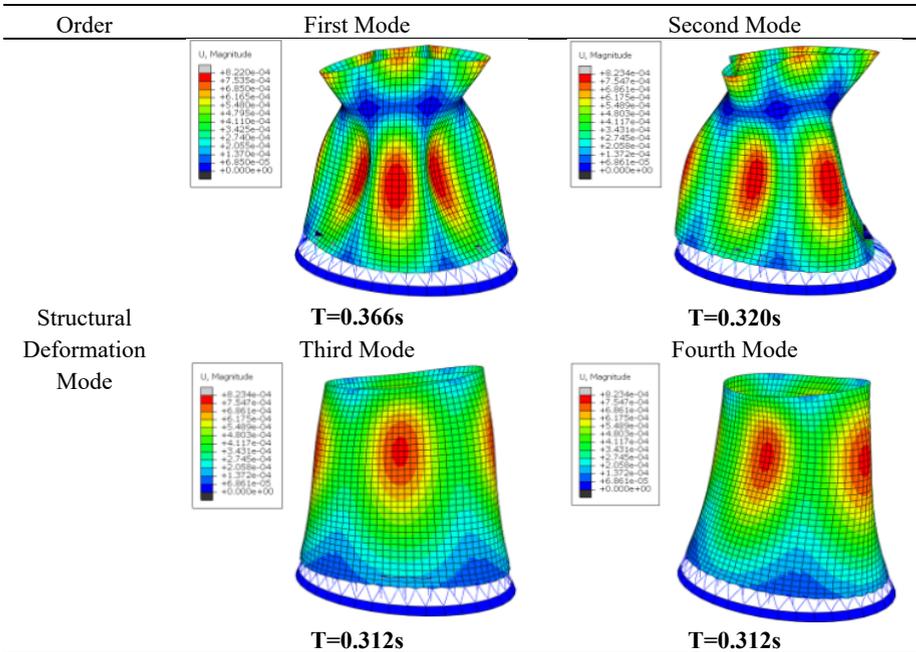**Fig. 2.** Bilinear Kinematic Hardening Model.

(a) Rhino Model

(b) External Structure

(c) Internal Structure

(d) Boundary Conditions and Loads

Fig. 3. Finite Element Model Establishment.

5   Seismic Performance Analysis

5.1   Period and Mode Shapes

The first four natural periods and mode shapes of the structure are shown in Table 1, providing a clear illustration of the structure's deformation characteristics.

Table 1. Modal Analysis of the Cooling Tower.

Order	First Mode	Second Mode
Structural Deformation Mode	**T=0.366s**	**T=0.320s**
	Third Mode	Fourth Mode
	T=0.312s	**T=0.312s**

5.2   Selection and Input of Earthquake Waves

To verify the seismic performance of the cooling tower structure, the dynamic elasto-plastic time history analysis method was used to study the response of the model under seismic waves. The structure is designed for seismic intensity 7 (0.1g), based on the provisions of the "Code for Seismic Design of Buildings" GB50011-2010. According to this code, when using the time history analysis method, actual strong earthquake records (natural waves) and artificially simulated acceleration time-history curves (artificial waves) should be selected based on the site classification and the seismic design group. In this study, five natural waves and two artificial waves were selected for the time history analysis. The target response spectrum is the design response spectrum for rare earthquakes corresponding to seismic intensity 7 in the first seismic design group and Class III site in China, with the peak ground acceleration of the rare earthquake set at 0.22g. After scaling the selected earthquake waves to 0.22g, the average seismic influence coefficient curve of the acceleration response spectrum

was compared to the target response spectrum. The difference at the periods corresponding to the main modes of the structure was less than 20%, meeting the code requirements. The selected earthquake waves are listed in Table 2.

Table 2. Selection of Earthquake Waves.

	ID	Seismic Wave	Peak Value/g	Duration/s	Time Step/s
	E1	Big Bear-01_NO_902,Tg(0.49)	0.225	45	0.02
	E2	Coalinga-01_NO_351,Tg(0.46)	0.071	40	0.01
natural waves	E3	New Zealand-02_NO_587,Tg(0.43)	0.255	22	0.02
	E4	San Fernando_NO_59,Tg(0.46)	0.019	15	0.01
	E5	Cape Mendo-cino_NO_829,Tg(0.45)	0.549	36	0.02
artificial	E6	ArtWave-RH2TG045,Tg(0.45)	0.1	30	0.02
waves	E7	ArtWave-RH4TG045,Tg(0.45)	0.1	30	0.02

During the analysis, bidirectional seismic actions were considered. The primary direction (X-direction) earthquake waves were uniformly scaled to 0.22g, while the orthogonal direction (Y-direction) earthquake waves were set at 0.85 times the primary direction. The bidirectional seismic motions were input simultaneously without any time lag.

5.3 Seismic Response Analysis

The five natural waves (E1-E5) and two artificial waves (E6-E7) were input into the model for seismic response analysis, and the maximum average inter-story drift and maximum displacement were obtained, as shown in Table 3.

Table 3. Maximum Displacement Differences Between Stories and Maximum Displacement

Height(m)	Seismic Wave Maximum Displacement Difference (mm)	E1	E2	E3	E4	E5	E6	E7
0.00-20.00	X Direction	17.7	10.9	6.4	9.2	13.9	10.7	7.2
	Y Direction	15.1	9.3	5.4	7.8	11.8	9.1	6.1
20.00-45.00	X Direction	30.9	16.9	9.1	12.4	20.3	17.2	12.9
	Y Direction	26.3	14.4	7.7	10.5	17.3	14.6	11.0
45.00-70.00	X Direction	42.2	23.7	13.7	19.3	28.8	25.6	18.2
	Y Direction	36.9	20.1	11.7	16.4	24.5	21.8	14.5
maximum displacement	X Direction	79.0	55.7	153.0	121.7	41.6	356.1	367.9
	Y Direction	67.2	47.3	130.1	10.3	35.4	303.1	312.7

Based on the data above, the inter-story displacements and drifts were calculated, as illustrated in Figure 4.
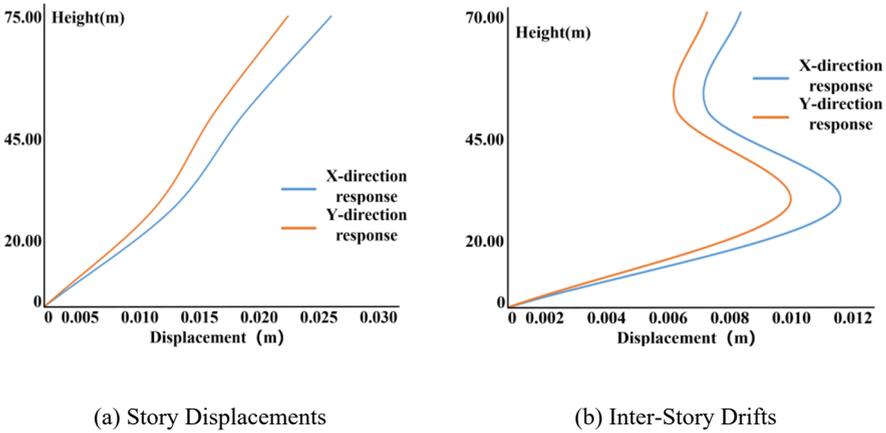
(a) Story Displacements   (b) Inter-Story Drifts

Fig. 4. Structural Story Displacements and Inter-Story Drifts

6   Conclusions

This paper focuses on the cooling tower in the renovation project of the Kunshan Power Plant. Based on the architectural and structural construction drawings, relevant standards, and design specifications, a corresponding finite element model was established in the ABAQUS software. Seven seismic waves, representing rare earthquake levels for the region, were input into the model for nonlinear dynamic time history analysis. Structural deformation was used as the evaluation criterion, and the results were compared with the code-specified limits. The verification results indicate that the cooling tower's design meets the seismic design requirements.

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