Collapse Simulation and Response Assessment of a Large Cooling Tower Subjected to Strong Earthquake Ground Motions

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Abstract: Large cooling towers in thermal power plants and nuclear power plants are likely to suffer from strong earthquakes during service periods. The resulting destructions of the cooling towers would endanger the power plants and threaten the security of the related areas. It is important to use effective means to evaluate the safety status of the cooling towers and guide further precautions as well as retrofitting efforts. This paper is therefore focused on an elaborate numerical investigation to the earthquake-induced collapses of a large cooling tower structure. A complete numerical work for simulation of material failure, component fracture, structural buckling and system collapse is presented by integrating the stochastic damage constitutive model of concrete, refined structural element models, and some other key techniques. Numerical results indicate that the damage behavior and collapse mode of the cooling tower are affected notably by the randomness specification of ground motions. The collapse mechanisms of the cooling tower are studied from the energy absorption and dissipation points of view. An effective energy-based criterion is introduced to identify the collapse of the cooling tower under ground motion excitations. While distinct collapse modes are observed, the collapse criterion can predict well the damage and failure of the cooling tower. The proposed methodology is vital to better understanding the disastrous mechanisms and potential failure paths in optimal design of the cooling towers to ensure safety.

Keywords: Cooling tower, concrete, damage mechanics, collapse criterion, responses.

1 Introduction

Large reinforced concrete (RC) cooling towers are widely used for heat dissipation and cooling in the thermal power plants and in some nuclear power plants that are built in water-deficient areas. Even in some coastal regions, an upward trend of using cooling

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towers in nuclear power plants appears to avoid thermal pollution to the marine ecosystems. With the development of the electric power industry, cooling towers over 200 m in height have come into play one after another, which greatly exceed the conventional limit of 165 m. Being one of the most important facilities, however, the safety of the cooling towers has always been a matter of people's concern. Probing into the failure likelihoods of the large cooling towers subjected to potential earthquake excitations is crucial to optimal design and reinforcement measures to guarantee security of both the power plants and the regions involved.

In building structures, plasticity extension in some elements subjected to strong ground motions is accepted [Beiraghi (2018a, 2018b, 2019)]. This matter can be investigated in the cooling towers. It is recorded that several famous collapse events of the cooling towers have occurred in history [Godoy (1984); Bamu and Zingoni (2005)]. In 1965, three cooling towers at Ferrybridge nuclear power station in UK collapsed due to the lack of design theory and the underestimation or overlook of the dynamic effect of the wind loads. Moreover, the disturbance effect of tower group had not been properly considered. In 1973, a 137 m tall cooling tower at Ardeer power plant in UK collapsed at moderate wind speeds because of the constructional defects which induced excessive circumferential stress. Before the accident, many meridional cracks had been observed on the tower. In 1979, a cooling tower in Bouchain, France, which was more than a decade old, collapsed in the breeze, possibly as the result of severe geometric errors that had accumulated during construction. In 1984, again in the UK, a 114 m high cooling tower at Fiddlers Ferry power station destroyed in high winds. Like the collapsed cooling tower at Ardeer power plant, this destruction was also identified as the construction error, i.e., the tower cylinder appeared obvious protrusion at the height of 6-13 m.

The direct causes of collapses of the cooling towers mentioned above are the wind loads. For the seismically active regions, earthquakes are the crucial threatens to the stability of the cooling towers. Thus, more attention should be paid to the earthquake resistance capability of the cooling tower. In terms of the research on seismic performance of the cooling towers, Gupta et al. [Gupta and Schnobrich (1976)] first developed the seismic analysis methods of hyperbolic cooling towers and concluded that the result of response spectrum analysis was the maximum value of the actual response. Although the seismic action was composed of three directions of excitation, it was enough for the design of cooling towers to consider only the horizontal component of the seismic ground motion. Nelson [Nelson (1981)] studied carefully the dynamics of cooling towers with column-supports; and the results showed that the real cooling tower was completely consistent with the corresponding cooling tower model. Meanwhile, the influential effect of the foundation elasticity, pillar angle, and the Poisson's ratio on the low-order vibration characteristics was roundly analyzed. Wolf [Wolf (1986)] discretized the axisymmetric shell of a hyperbolic cooling tower with high-order finite element (FE) frusta by an isoparametric expansion in meridian direction and a Fourier expansion in radial direction. It was found that accounting for the first or first two modes worked well for the linearly elastic seismic analysis of cooling towers. Makovička [Makovička (2006)] calculated the responses of an RC cooling tower under earthquakes and storms, and indicated that when considering the ductility of the tower, the impact of earthquakes is smaller than that of wind loads. However, the research on collapse analysis of cooling towers is still in the initial stage to date. Krätzig et al. [Krätzig and Zhuang (1992)] simulated the failure behavior of an RC cooling tower before total collapse under combined dead load and quasi-static wind load. The weak position of the cooling tower was found to be mainly controlled by tensioning, which would make concrete crack and steel yield instead of buckling under compression. However, the whole collapse process of the cooling tower was not reported in this paper. Li et al. [Li, Lin, Gu et al. (2014)] established a three-dimensional FE model of a super-large RC cooling tower in *LS-DYNA*, and studied the collapse modes and mechanisms of the cooling tower induced by several different accidental actions. Yu et al. [Yu, Gu, Li et al. (2016)] carried out a shaking table test on a 1/55 scaled RC super-large cooling tower. Through investigation of the failure mechanisms, the columns were found to be the weakest part of the tower. Both the acceleration and the displacement responses on the top part of the cylinder increased with the increase of the peak ground acceleration (PGA). Upon strong earthquake ground motions, the cooling tower collapsed aslant overall as a result of the failure of base columns and losing support.

Nowadays, the demands for service safety of the cooling towers are increasing constantly as some of the cooling towers under construction or design are higher and larger. However, accurate predictions of structural performances of the large cooling towers via taking fully advantages of the latest theories and technologies are still lacking. Moreover, the collapse of the cooling towers under ruinous actions may be caused either by the strength failure of some local parts or by local or global buckling. It is thus quite challenging to make precise predictions for the overall failure courses and the critical points of failure of the large cooling towers. In these regards, the present paper aims at developing a complete numerical assessment for simulation of material failure to component fracture and further to structural buckling and collapse of large cooling towers. The damage paths, collapse mechanisms, and failure modes of the cooling towers are investigated with the presence of randomness from ground motions. An effective energy-based criterion is introduced to identify the collapses of the cooling tower from the energy absorption and dissipation points of view. The proposed methodology is illustrated with application to an actual large RC cooling tower for simulation and identification of earthquake-induced collapse.

2 Description and refined modeling of the large cooling tower

2.1 Case-study specification

The cooling tower under investigation locates in northwest China. It consists of a huge RC shell barrel and 132 crossed oblique columns with a total height of 250 m. The schematic of the cooling tower is shown in Fig. 1. The shell barrel is 220 m in height, 186 m in base diameter, 118 m in top diameter, and 113 m in throat diameter. The shell thickness decreases from the bottom up with the thickest part of 1.8 m and the thinnest part of

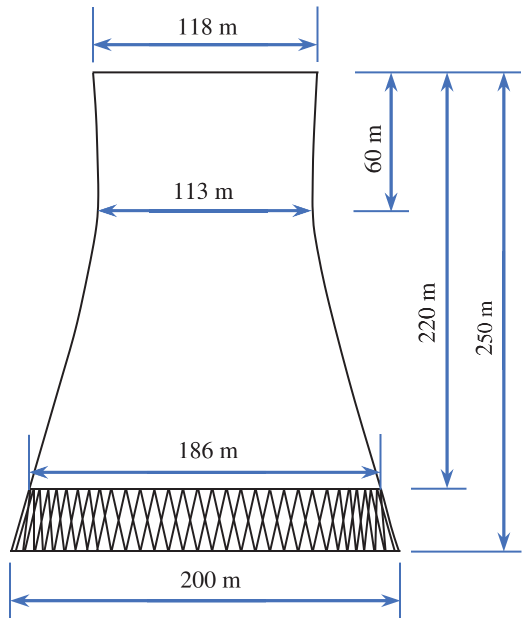


Figure 1: Schematic of the cooling tower

0.42 m. The frame-truss structure is 29 m in height with each oblique cylinder of 30.8 m in length and 1.0 m×1.7 m in rectangular cross-section. The basic reinforcement ratios of the cooling tower are shown in Tab. 1. All concrete used is of compressive strength grade C45, and the steel rebars are of type HRB335 for stirrups and HRB400 for main reinforcing bars, according to the valid Chinese standards. More details see Xu et al. [Xu and Bai (2013)].

Table 1: Reinforcement ratios of the cooling tower

Shell barrel				**Crossed oblique columns**	
Circumferential		**Meridional**		**Longitudinal**	**Stirrup**
Inner side	**Outer side**	**Inner side**	**Outer side**		
0.3-0.8%	0.4-0.9%	0.3-0.8%	0.4-0.9%	2.5%	0.5%

2.2 Numerical modeling

2.2.1 Stochastic damage constitutive law of concrete

Concrete structures subjected to strong dynamic actions may exhibit nonlinear behavior and rate-dependency. The resulting structural performance is difficult to be predicted well and truly also due to the widely existed randomness in concrete. To this end, the fiber bundle model-based stochastic damage constitutive law is employed to represent concrete in numerical modeling of the cooling tower.

In accordance with the fiber bundle model (also known as the parallel element model) [Kandarpa, Kirkner and Spencer (1996); Li and Ren (2009); Ren, Zeng and Li (2015); Zhou, Li and Ren (2016); Bažant and Le (2017)], the tensile (+) and compressive (-) random damage factors of concrete can be defined by

$$D^{\pm}(\varepsilon^{e\pm}) = \int_0^1 H[\varepsilon^{e\pm} - \Delta^{\pm}(x)]\mathrm{d}x \tag{1}$$

where $\varepsilon^{e\pm}$ are the elastic strains; $\Delta^{\pm}(x)$ are the random fields of the microscopic fracture strains with x as the coordinate; $H(\cdot)$ is the Heaviside function.

By assuming that the random fields $\Delta^{\pm}(x)$ follow the lognormal distribution, the mean and variance of the random damage evolution laws, Eq. (1), can be derived as Li et al. [Li and Ren (2009)]

$$\mu_{D^{\pm}}(\varepsilon^{\mathrm{e}\pm}) = \Phi(\Upsilon^{\pm}) \tag{2}$$

$$\mathrm{Var}[D^{\pm}(\varepsilon^{\mathrm{e}\pm})] = 2\int_0^1 (1-\eta)\Phi[\Upsilon^{\pm}, \Upsilon^{\pm}|R_{\Psi^{\pm}}(\eta)]\mathrm{d}\eta - [\Phi(\Upsilon^{\pm})]^2 \tag{3}$$

where $\Upsilon^{\pm}=(\ln\varepsilon^{\mathrm{e}\pm}-\lambda^{\pm})/\zeta^{\pm}$, and $\lambda^{\pm}$, $\zeta^{\pm}$ are the mean values and standard deviations of $\Psi^{\pm}(x)=\ln[\Delta^{\pm}(x)]$, respectively; $\Phi(\cdot)$ signifies the cumulative distribution function (CDF) of the standard normal distribution; $\eta=|x_1-x_2|$ represents an arbitrary spatial distance; $R_{\Psi}(\eta)=\exp(-\eta/l)$ is assumed to be the autocorrelation coefficient function of the process $\Psi(x)$, and l is the correlation length.

Within the framework of the damage energy release rate-based damage constitutive model [Wu, Li and Faria (2006)], the random damage evolution laws of concrete under multidimensional stress states can be determined by introducing the energy equivalent strains [Li and Ren (2009)]

$$\varepsilon_{\mathrm{eq}}^{\mathrm{e}+} = \sqrt{\frac{2Y^{+}}{E_0}} \;;\; \varepsilon_{\mathrm{eq}}^{\mathrm{e}-} = \frac{1}{(1-\alpha)E_0}\sqrt{\frac{Y^{-}}{b_0}} \tag{4}$$

where E_0 is the initial Young's modulus; b_0 is a material parameter fitted by multiaxial experimental data; α is the biaxial strength increase factor; and Y^{+}, Y^{-} are the tensile and shear damage energy release rates, respectively [Wu, Li and Faria (2006)].

Then, the multi-dimensional random damage evolution laws can be given by

$$D^{\pm}(\varepsilon_{\mathrm{eq}}^{\mathrm{e}\pm}) = \int_0^1 H[\varepsilon_{\mathrm{eq}}^{\mathrm{e}\pm} - \Delta^{\pm}(x)]\mathrm{d}x \tag{5}$$

Concrete is a strain rate sensitive material; and the dynamic behavior of concrete differentiates with its static loading behavior. This is known as the strain hysteresis effect. By invoking the representation of stochastic Stefan effect of the viscous and viscoelastic-plastic materials [Ren, Zeng and Li (2015)], the rate-dependent energy equivalent strains under constant rate loading (i.e., $\varepsilon = \dot{\varepsilon}t$) can be derived as

$$\varepsilon_{\mathrm{r}}^{\mathrm{e}\pm} = \varepsilon_{\mathrm{eq}}^{\mathrm{e}\pm} - \frac{\alpha_n \gamma_a}{\beta_n(\alpha_n + \beta_n)}\left(1 - e^{-\frac{\beta_n t}{\gamma_a}}\right)\dot{\varepsilon}_{\mathrm{eq}}^{\mathrm{e}\pm} \tag{6}$$

where α_n=$1+\phi$; β_n=$(\phi+1)/\phi$; ϕ=E_1/E_2 is the ratio between the elastic stiffness E_1 and the viscous counterpart E_2; γ_a=$\alpha_S\mu_v/(\gamma_h^2 E_k)$; E_k=$E_1E_2/(E_1+E_2)$; μ_v is the viscosity; the shape coefficient α_S=1.5, and γ_h is the aspect ratio.

Through modification of the energy equivalent strains in Eq. (5) by Eq. (6), the rate-dependency of concrete can be reasonably taken into account.

For the confined concrete due to stirrups, the compressive damage evolution needs to be revised by Li et al. [Li, Zhou and Ding (2018)]

$$D_{\mathrm{con}}^{-}(\varepsilon_{\mathrm{eq}}^{\mathrm{e}-}) = \int_0^1 H[\psi\varepsilon_{\mathrm{eq}}^{\mathrm{e}-} - \Delta^{-}(x)]\mathrm{d}x \tag{7}$$

where the reduction coefficient ψ is given by

$$\psi = 1 - \left(\frac{\upsilon\varepsilon^{\mathrm{p}-}}{\varepsilon^{\mathrm{p}-} + \vartheta/100}\right)^{\kappa} \tag{8}$$

where $\varepsilon^{\mathrm{p}-}$ is the plastic strain under axial compression with stirrup confinements; υ, ϑ, and κ are the model parameters fitted by test data.

Through necessary consideration of the rate-dependency and the stirrup confinement effect, the generalized multi-dimensional random damage constitutive relationship of concrete can be given by Li et al. [Li, Wu and Chen (2014)]

$$\boldsymbol{\sigma} = (1 - \mathrm{D}^{+})\bar{\boldsymbol{\sigma}}^{+} + (1 - \mathrm{D}^{-})\bar{\boldsymbol{\sigma}} - = (\mathbb{I} - \mathbb{D}) : \bar{\boldsymbol{\sigma}} = (\mathbb{I} - \mathbb{D}) : \mathbb{E}_0 : (\boldsymbol{\varepsilon} - \boldsymbol{\varepsilon}^{\mathrm{p}}) \tag{9}$$

where $\bar{\boldsymbol{\sigma}}$ is the effective stress tensor with its positive/negative components $\bar{\boldsymbol{\sigma}}^{\pm}$; $\mathbb{E}_0$ is the initial undamaged elastic stiffness; $\mathbb{I}$ is a fourth-order unit tensor; $\mathbb{D}$=$D^{+}\mathbb{P}^{+}$+$D^{-}\mathbb{P}^{-}$ is a fourth-order damage tensor with $\mathbb{P}^{\pm}$ as its positive/negative projection (PNP) tensors [Wu and Cervera (2018)]; and $\boldsymbol{\varepsilon}$ is the total strain tensor. The incremental evolution of plastic strain $\boldsymbol{\varepsilon}^{\mathrm{p}\pm}$ are empirically given by Ren et al. [Ren, Zeng and Li (2015)]

$$\dot{\boldsymbol{\varepsilon}}^{\mathrm{p}\pm} = H[\dot{D}^{\pm}]\varsigma_{\mathrm{p}}^{\pm}(D^{\pm})^{n_{\mathrm{p}}^{\pm}}\dot{\boldsymbol{\varepsilon}}^{\mathrm{e}\pm} \tag{10}$$

where $\varsigma_{\mathrm{p}}^{\pm}$ and $n_{\mathrm{p}}^{\pm}$ are the material parameters identified by experiments.

2.2.2 Stochastic structural modeling of the cooling tower

The stochastic structural modeling of the cooling tower is implemented invoking the two-scale random fields synthesis strategy [Zhou, Li and Spencer (2019)]. First, the fracture strain random field at the microlevel is modeled to obtain the random damage constitutive relation samples. Further, the resulting macroscale strength random field of any concrete member or structure is required to follow a certain distribution under the covariance constraint in terms of the macroscopic scale of fluctuation of the random field. Then, both the microscopic random damage evolution of concrete and the fluctuation of macroscopic structural responses can be fully represented taking advantage of the stochastic FE method [Contreras (1980); Vanmarcke and Grigoriu (1983); Liu, Belytschko and Mani (1986); Sudret and Der Kiureghian (2000); Stefanou (2009)].

In FE modeling of the cooling tower, the multilayer shell element and the fiber beam element are employed to simulate the shell and the oblique columns, respectively. As each fiber owns a separate constitutive relationship, the fiber beam element can simulate one-dimensional (1-D) components with cross sections of arbitrary shape and conveniently consider the stirrup confinement effect [Spacone, Filippou and Taucer (1996)]. Likewise, the multilayer shell element composed of membrane elements and shell elements allows including layers with various thicknesses and material properties [Lin and Scordelis (1975)]. For example, it is possible to smear the reinforcement bars into one or more layers per their positions and orientations. Integrating the above stochastic damage constitutive model of concrete into the two refined structural elements, the random damage, degradation, and softening behaviors of concrete can be captured through the stochastic FE analysis, with fair accuracy and efficiency.

It should be noted that only the vertical spatial variability of concrete strength is considered in structural modeling. The coefficient of variation of strength generally takes the value of 15.6%. The distribution and correlation parameters of the two-scale random fields are listed in Tab. 2. Besides, the scale of fluctuation of the macro strength random field is 0.6 m. The stochastic harmonic function method [Chen, Sun, Li et al. (2013)] is adopted to simulate the random fields of the microscopic fracture strain of concrete. Other material parameters involved for random damage modeling of concrete are given in Tab. 3.

To consider the collapse simulation of the cooling towers, the elastic-plastic with hardening and progressive damage constitutive model [Johnson and Cook (1985)] is adopted to model the reinforcement bars. Because the variability of steel is much smaller compared to that of concrete, the deterministic mean parameters of the steel properties are used, as shown in Tab. 4.

The refined FE model of the cooling tower is established in *ABAQUS*, as shown in Fig. 2. Totally, 190,468 nodes, 23,528 multilayer shell elements, and 1,914 fiber beam elements are developed. All the bottom nodes are fixed to the ground as a rigid boundary though, the constraint in the direction of the one-way excitation is released for earthquake input. The first ten basic frequencies of the cooling tower are 0.505 Hz, 0.512 Hz, 0.611 Hz, 0.651 Hz, 0.771 Hz, 0.841 Hz, 0.895 Hz, 0.933 Hz, 0.968 Hz, and 1.055 Hz, respectively. The stochastic damage constitutive model is introduced to simulate concrete

Table 2: Distribution parameters of two-scale random fields of concrete

Random fields	Mean value	Standard deviation	Scale of fluctuation
Tensile fracture strain	$\lambda^{+}=4.9722$	$\zeta^{+}=0.4127$	$l^{+}=1/100$
Compressive fracture strain	$\lambda^{-}=7.6683$	$\zeta^{-}=0.5891$	$l^{-}=1/120$
Macroscopic strength	$\mu_S=39.8$ MPa	$\sigma_S=6.0$ MPa	$\theta=0.6$ m

Table 3: Deterministic material parameters of concrete

Material parameters	Tension	Compression
Elasticity	$E_0=33500$ MPa, Poisson's ratio	$v_c=0.2$
Biaxial strength increase	N/A	$f_{bc}/f_c=1.16$
Plasticity	$\varsigma_{\mathrm{p}}^{+}=0.3128$	$\varsigma_{\mathrm{p}}^{-}=0.3907$
	$n_{\mathrm{p}}^{+}=0.3892$	$n_{\mathrm{p}}^{-}=0.4758$
Dynamic damage	$\gamma_a^{+}=8$	$\gamma_a^{-}=1.5$
	$\phi^{+}=0.0002$	$\phi^{-}=0.0005$
Confinement	N/A	$\upsilon=0.8718$
	N/A	$\vartheta=0.4816$
	N/A	$\kappa=0.6652$

Table 4: Mean material parameters of steel

Material parameters	Symbol (Unit)	HRB335	HRB400
Modulus of elasticity	E_s (MPa)	200000	200000
Poisson's ratio	v_s	0.3	0.3
Yield strength	f_y (MPa)	335	400
Ultimate strength	f_u (MPa)	455	540

with the user-defined material subroutine in *ABAQUS*. The structural damping of the cooling tower is determined by the equivalent modal damping ratios [Sivandi-Pour, Gerami and Khodayarnezhad (2014); Sivandi-Pour, Gerami and Kheyroddin (2015)].

3 Damage and collapse analysis of the cooling tower under seismic actions

3.1 Critical techniques for collapse simulation

Since strong nonlinearities including material degradation and softening, element fracture, and contact-impact are widely involved in structural collapse, some relevant techniques are thus requisite to address these issues.

Figure 2: FE model of the cooling tower

First, the multi-scale failure criteria from material to element and further to component are employed to simulate the damage, fracture, and buckling behaviors of the cooling tower. The failure of steel is identified by a limit strain, e.g., 0.05, whereas for concrete by a limit damage factor, e.g., 0.95. Once a failure criterion is met, the material stiffness is degraded according to the specified damage evolution law. An element would be terminated providing that the maximum degradation at all material points is reached. Therefore, the simulation of failure of the members and structures is achieved via the element removal strategy [Li, Zhou and Ding (2018)].

Second, the contact-impact phenomenon exists extensively among scattered debris once a collapse is initiated. This probably would affect the collapse behavior of the overall structure of the cooling tower. To this end, the penalty contact model and the general contact algorithm in *ABAQUS* are employed to account for the contact interactions. A linear or nonlinear spring stiffness is generally applied between segments to compute the contact force and the penetration distance.

Finally, a robust integral algorithm is indispensable to achieve an accurate and stable numerical solution to the above highly nonlinear problems. In this regard, the central difference method is used, which avoids the convergence issue due to iterative solving procedure in an implicit algorithm. However, the stabilized time step must be taken to refrain the solution from unbounded growth [Belytschko, Liu, Moran et al. (2014)]. A further advantage of the explicit algorithm is that it allows a parallel computing procedure signifying a faster processing in application to collapse analysis of large cooling towers.

Integrating all the above theories and techniques, a complete numerical platform for collapse simulation of large cooling towers has been established, by which the random damage and failure process, as well as the underlying mechanisms can be investigated in the system.

3.2 Seismic damage and collapse simulation of the cooling tower

The ground motions recorded at Chi-Chi, Taiwan, 9/20/1999, earthquake are selected as the seismic inputs to the cooling tower under investigation herein. Many seismograph stations have recorded complete seismic wave data in this earthquake. Considering the effects of the epicentral distance, site feature, and the impulse component characteristic, the ground acceleration sequences recorded at four different seismograph stations are adopted in the following. In accordance with the original database [PEER (Pacific Earthquake Engineering Research Center) (2019)], they are marked as TCU049EW (EQ1), TCU065EW (EQ2), TCU075EW (EQ3), and TCU087NS (EQ4), respectively. The suffixes EW and NS signify the record orientations are east-west and south-north, respectively. The acceleration time histories and the corresponding response spectra are shown in Fig. 3, where the damping ratio takes the value of 2%. The recorded PGAs for the considered records are 0.28 g, 0.40 g, 0.34 g, and 0.23 g, respectively, where g is the unit of the gravitational acceleration with the approximate value of 9.8 m/s^2. To investigate the damage and collapse behavior of the cooling tower, the four selected ground motion series are magnified to one (named as EQ1-EQ4), two (named as EQ1a-EQ4a), and three (named as EQ1b-EQ4b) times its original level in terms of their PGAs, respectively. All 12 seismic waves are input unidirectionally to the cooling tower for damage and collapse analysis in succession after the gravity analysis.

By using the developed analysis model for structural damage and collapse in *ABAQUS*, the nonlinear whole-process dynamic response analyses of the cooling tower subjected to 12 different earthquake records are performed. It is found that the cooling tower does not collapse under the earthquake excitation without applying scale factor, i.e., the original four records (EQ1-EQ4). For the double magnified level records (EQ1a-EQ4a), collapse of the cooling tower is observed only in the case under EQ2a record. The collapse of the cooling tower under EQ2a ground motion record initiates with the damage and failure of some local crossed oblique columns. Then, the global buckling happens from the throat of the upper shell barrel. A mixture mode combining the strength failure with the dynamical instability failure results in the final collapse of the cooling tower under EQ2a seismic wave. While for the three times amplified level records, except for the case EQ1b, collapse occurs in all the other three cases. The damage and collapse processes and failure modes are depicted in Fig. 4, in which the compression damage of concrete is the basic variable of the nephograms.

From Fig. 4, it is seen that the cooling tower collapses following with quite different damage paths and failure patterns. Some are due to the strength failure of the bottom crossed oblique columns, while some are caused by the buckling of the upper shell barrel. On the other hand, the randomness from the earthquake excitations may also contribute to the variation of the

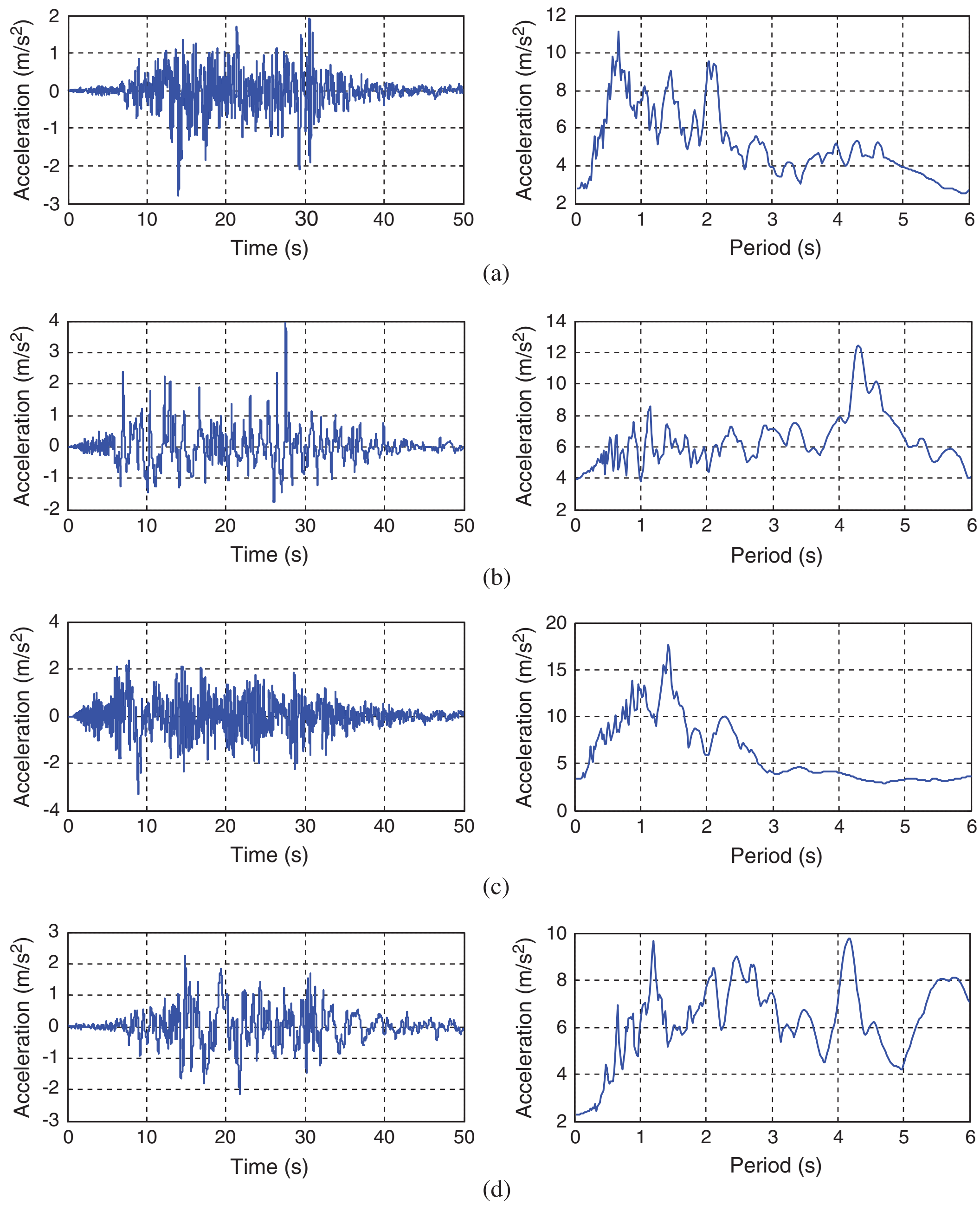


Figure 3: The acceleration time histories and response spectra of Chi Chi earthquake. (a) EQ1: TCU049EW (b) EQ2: TCU065EW (c) EQ3: TCU075EW and (d) EQ4: TCU087NS

damage behavior of the cooling tower. These aspects may probably shock the traditional cognitions in designing the cooling towers. It is generally recognized that the bottom crossed oblique columns are the weakest members of cooling towers under the seismic action. However, in this study, it is found that the cooling tower would hardly collapse under the design level of ground motions. Moreover, beyond the designed level, the cooling tower may fail with quite different paths, or stand safely still. This indicates that accurate modeling, analysis, and assessment of the large cooling towers are quite necessary for their optimal design, exploring their failure mechanisms, and establishing precaution measures to avoid hazard risks. The present damage modeling and collapse analysis method makes a useful attempt to this point.

However, when it comes to the safety evaluation of the cooling towers, one cannot get around the challenge of identifying synthetically the structural failure in stochastic dynamic analysis. This leads to another issue to be solved in the following.

4 Collapse identification of the cooling tower using an effective energy-based criterion

Collapse prevention is a vitally important performance objective in aseismic design of the large cooling towers. Although the incremental dynamic analysis (IDA) [Vamvatsikos and Cornell (2002)] is widely used to assess the collapse likelihood of engineering structures, the performance indicators, e.g., the inter-storey drift ratio (ISDR), can hardly represent the global performance. Moreover, the prediction results are found to be sensitive to the pre-defined thresholds [Huang, Ren and Li (2017); Deniz, Song and Hajjar (2017)]. The lower computational efficiency is another concern that should not be ignored.

Considering the random damage evolution of materials and the notable redistribution of internal forces of members before structural collapse, significant variability can be observed in the failure paths and failure modes of structures [Li, Zhou and Ding (2018)]. This means the classical criteria for dynamic stability [Bernal (1998); Miranda and Akkar (2003); Xie (2006); Krylov (2007); Bažant and Cedolin (2010)] are not applicable to identify the collapse critical point of the cooling towers. Attention is therefore paid to the aggregated energy quantities [Smyth and Gjelsvik (2006); Szyniszewski and Krauthammer (2012)]. For example, the structural collapse can be determined while the gravity energy abruptly exceeds the dynamic input energy [Deniz, Song and Hajjar (2017)].

Alternatively, the dynamic stability of hardening structural systems can be identified by comparison of the structural intrinsic energy with the total external work [Xu and Li (2015)]. Both the variation of structural properties and the change of external actions are included in exploring the system stability. However, the Xu-Li energy criterion is not applicable for deteriorating structural systems [Zhou and Li (2019)]. On the strength of the effective energy, the Xu-Li criterion was further improved and extended to the collapse identification of deteriorating structural systems [Zhou and Li (2017)]. This new effective energy-based criterion is therefore introduced in the following to identify the collapse of the large cooling tower structures.

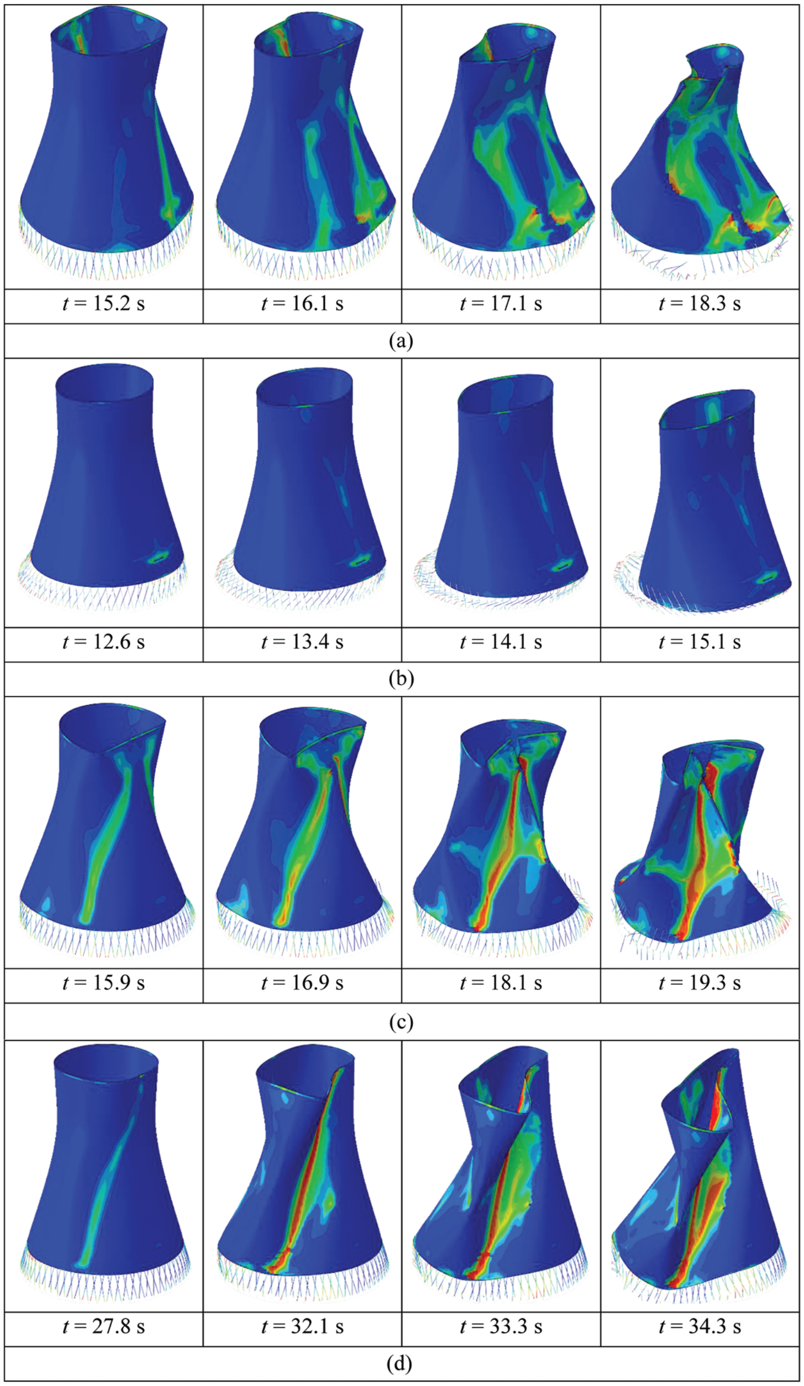


Figure 4: The seismic damage processes and collapse modes of the cooling tower. (a) Damage phases under EQ2a: TCU065EW (two times EQ2) (b) Damage phases under EQ2b: TCU065EW (three times EQ2) (c) Damage phases under EQ3b: TCU075EW (three times EQ3) and (d) Damage phases under EQ4b: TCU087NS (three times EQ4)

4.1 Effective energy-based collapse criterion

For a general dynamic system, the equation of motion can be expressed as

$$\mathbf{M}\ddot{\mathbf{u}}(t)+\mathbf{C}\dot{\mathbf{u}}(t)+\mathbf{f}(\mathbf{u},t)=\mathbf{F}(t) \tag{11}$$

where $\mathbf{M}$ is the mass matrix, $\mathbf{C}$ is the damping matrix, $\mathbf{f}(\mathbf{u},t)$ is the stiffness force, $\mathbf{F}(t)$ is the dynamic action, $\mathbf{u}(t)$, $\dot{\mathbf{u}}(t)$, and $\ddot{\mathbf{u}}(t)$ are the displacement, velocity, and acceleration vectors, respectively.

Integrating Eq. (11) with respect to the displacement yields the energy balance equation [Beiraghi (2017, 2018c, 2018d)]:

$$E_{\mathrm{k}}(t)+E_{\mathrm{d}}(t)+E_{\mathrm{e}}(t)+E_{\mathrm{p}}(t)=E_{\mathrm{input}}(t) \tag{12}$$

where $E_{\mathrm{k}}(t)$, $E_{\mathrm{d}}(t)$, $E_{\mathrm{e}}(t)$, $E_{\mathrm{p}}(t)$ and $E_{\mathrm{input}}(t)$ are the kinetic energy, dissipated damping energy, elastic strain energy, plastic strain energy, and the total input energy, respectively.

Since the dynamic stability is generally related to both the system property and the dynamic action, Xu et al. [Xu and Li (2015)] developed an energy criterion for dynamic stability through comparison of the structural intrinsic energy versus the total external input energy. The structural intrinsic energy is defined as Xu et al. [Xu and Li (2015)]

$$E_{\mathrm{intr}}(t)=\left|\mathbf{f}^{\mathrm{T}}(\mathbf{u},t)\mathbf{u}(t)-\int_0^{\mathbf{u}}\mathbf{f}^{\mathrm{T}}(\mathbf{u},t)\mathrm{d}\mathbf{u}\right| \tag{13}$$

where the superscript T denotes a vector transpose.

The dynamic instability occurs on condition that

$$E_{\mathrm{intr}}(t)>E_{\mathrm{input}}(t) \tag{14}$$

at a certain time instant t while the dynamic equilibrium state cannot be further maintained.

If the intrinsic energy is always less than the input energy, i.e.,

$$E_{\mathrm{intr}}(t)<E_{\mathrm{input}}(t) \tag{15}$$

in the time period [0, t], then the system is dynamically stable.

The Xu-Li criterion is applicable for predicting the onset time of dynamic instability of the hardening structural systems. However, while it was applied to the deteriorating structural systems considering the degradation and softening effects of the related materials, the Xu-Li criterion gave wrong results in several cases [Zhou and Li (2017, 2019)]. The reason may lie in the discrepancy of the unstable critical states between hardening and deteriorating structural systems. In this regard, the effective energy criterion [Zhou and Li (2017)] was proposed to further extend the applicability of the Xu-Li criterion. Specifically, two effective energy-based indices, i.e., the effective intrinsic energy and the valid external work were newly defined to generalize the Xu-Li criterion for identification of the dynamic stability of both hardening and deteriorating structures.

The effective intrinsic energy of structures is redefined from that of the intrinsic energy as

$$E_{\mathrm{eff\_intr}}(t) = \left|\mathbf{f}^{\mathrm{T}}(\mathbf{u},t)\mathbf{u}(t) - E_{\mathrm{e}}(t)\right| = \left|\mathbf{f}^{\mathrm{T}}(\mathbf{u},t)\mathbf{u}(t) - \frac{1}{2}\int_V \boldsymbol{\sigma} : \dot{\boldsymbol{\varepsilon}}_{\mathrm{e}} \mathrm{d}V\right| \tag{16}$$

where $\boldsymbol{\sigma}$ is the stress tensor, $\dot{\boldsymbol{\varepsilon}}_{\mathrm{e}}$ is the elastic strain rate tensor, V denotes the solution domain, and $E_{\mathrm{eff\_intr}}(t)$ is transient, signifying the absorbing energy due to the system vibration at time t. It can be seen that subtracting the plastic strain energy $E_{\mathrm{p}}(t)$ from the intrinsic energy in Eq. (13) gives the effective intrinsic energy herein.

On the other hand, both the plastic strain energy and the damping energy are irreversible. This means the energy effectually input to the system at time t does not count in $E_{\mathrm{d}}(t)$ and $E_{\mathrm{p}}(t)$ which have already been dissipated. To this end, the effective input energy is defined by

$$\begin{aligned} E_{\mathrm{eff\_inp}}(t) &= E_{\mathrm{input}}(t) - E_{\mathrm{d}}(t) - E_{\mathrm{p}}(t) \\ &= \int_0^t \mathbf{F}^{\mathrm{T}}(t)\mathrm{d}\mathbf{u}(t) - \int_0^t \dot{\mathbf{u}}^{\mathrm{T}}(t)\mathbf{C}\dot{\mathbf{u}}(t)\mathrm{d}t - \int_0^t \left(\int_V \boldsymbol{\sigma} : \dot{\boldsymbol{\varepsilon}}_{\mathrm{p}}\, \mathrm{d}V\right)\mathrm{d}t \end{aligned} \tag{17}$$

where $E_{\mathrm{eff\_inp}}(t)$ is also recognized as the valid external work. Considering the energy balance Eq. (12), another form of the valid external work can be given by

$$E_{\mathrm{eff\_inp}}(t) = \frac{1}{2}\dot{\mathbf{u}}^{\mathrm{T}}(t)\mathbf{M}\dot{\mathbf{u}}(t) + \frac{1}{2}\int_V \boldsymbol{\sigma} : \dot{\boldsymbol{\varepsilon}}_{\mathrm{e}}\, \mathrm{d}V \tag{18}$$

Eq. (18) indicates that the elastic deformation energy and kinetic energy are free to be transformed from one moment to another, which fits with the common sense.

The collapse of complex structures subjected to seismic actions is generally attributed to the degradation and softening induced dynamic instability. Analogy to the Xu-Li energy criterion, an effective energy criterion for identification of dynamic stability of generalized structural systems is developed as follows

$$\begin{cases} E_{\mathrm{eff\_intr}}(t) \le E_{\mathrm{eff\_inp}}(t) & \text{Dynamic stability} \\ E_{\mathrm{eff\_intr}}(t) > E_{\mathrm{eff\_inp}}(t) & \text{Dynamic instability} \end{cases} \tag{19}$$

This new criterion states that a system is dynamically stable providing that the effective intrinsic energy is less than the valid input energy all the time. Once the effective intrinsic energy is observed to exceed the valid external work for the first time, the system is signified to go into dynamic instability. For convenience, a collapse critical function for general structures subjected to dynamic actions can be defined by

$$S(t) = E_{\mathrm{eff\_inp}}(t) - E_{\mathrm{eff\_intr}}(t) \tag{20}$$

whereby $S(t) \ge 0$ maintains for $t \ge 0$ indicates the structure would not collapse during the whole process of excitation, and at the first observation of $S(t) < 0$ the structure is believed to start to collapse and crumble.

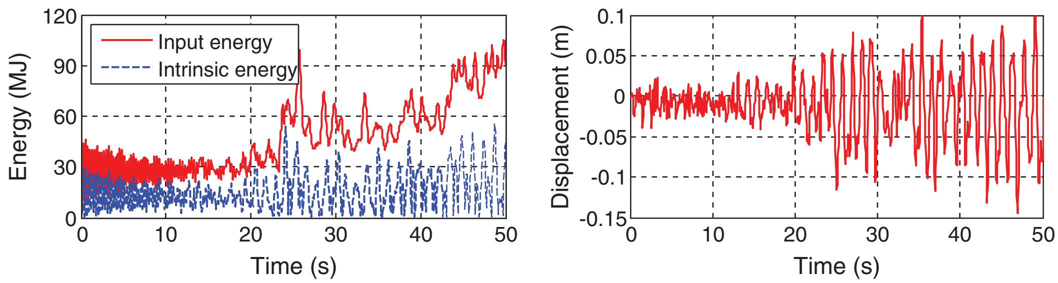
The foregoing effective energy criterion does not violate the energy balance principle, but provides a critical condition at which the structure collapse or not. Since the whole-process dynamic stability of the structure is under careful inspection, the exact onset time of structural collapse induced by dynamic actions can be quantitatively determined by the proposed effective energy criterion. The criterion has been successfully applied to several different structural systems [Zhou and Li (2017, 2019)].

4.2 Collapse identification of the cooling tower

In the following, the effective energy criterion is employed to recognize whether the cooling tower collapses or not in numerical simulation. The energy-based identification results are ascertained by the corresponding relative displacement responses at the top of the tower. During the damage and collapse simulation, the energy indices involved in the effective energy criterion are computed and processed via Python script. The comparisons of the time histories of effective input energy and intrinsic energy in different cases are shown in Figs. 5-14, along with the corresponding relative displacement time history. For the cases no collapse is observed, quite similar results can be obtained as well. Therefore, some of the analysis results are not presented herein for simplicity.

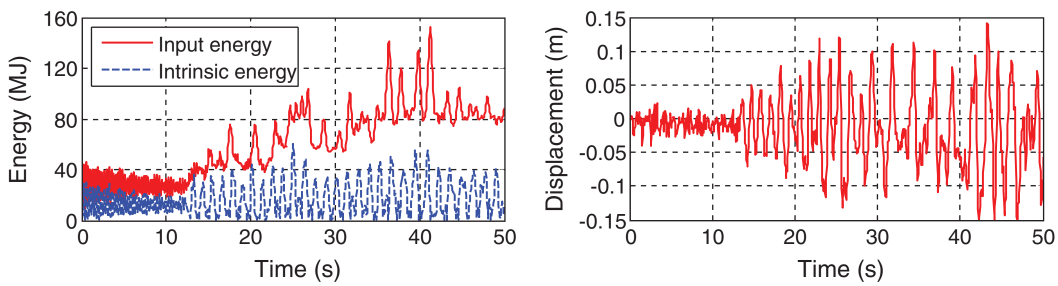
From the mentioned results, it is seen that the effective input energy is always less than the effective intrinsic energy of the cooling tower during the whole process of the earthquake excitation and no collapse occurs. This coincides well with the displacement-based predictions as no abrupt drifting is observed; See Figs. 5-7 and 9-11. Nevertheless, the effective intrinsic energy would exceed the valid input energy once the collapse of the cooling tower is triggered. In this case, an abrupt drifting can be generally observed in the top displacement; See Figs. 8 and 12-14. It is worth noting that the first exceeding time of the effective intrinsic energy over the valid external work is basically coincident with the time that a sudden change in the top displacement is observed.

The considerable displacement at a certain local point may be necessary for the collapse of the cooling tower through, it is not sufficient. Sometimes the failure of a local component does not lead to the overall failure of the structure, while not vice versa. Similar research findings can be referred to Huang et al. [Huang, Ren and Li (2017); Li, Zhou and Ding (2018)]. That's why the effective energy criterion is introduced herein. It is believed that an aggregated energy quantity probably would be much more effective and useful in predicting the global performances of the large complex structures. Moreover, the developed effective energy criterion is promisingly applicable to the dynamic reliability analysis of the large cooling towers.



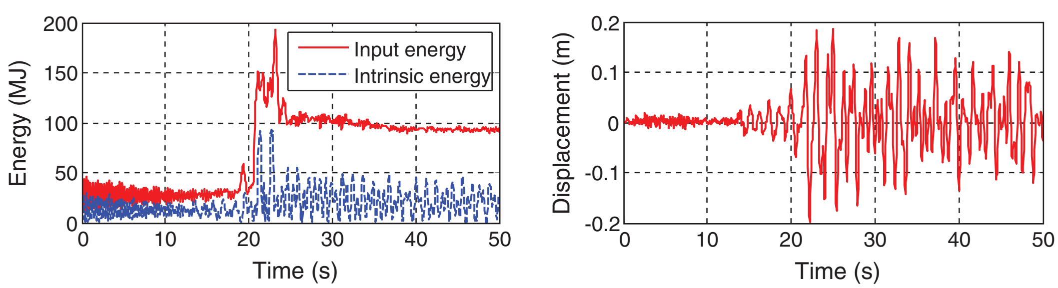
(a) (b)

Figure 5: The effective energy and the top displacement time histories under EQ1. (a) Effective energy time histories and (b) Displacement time history



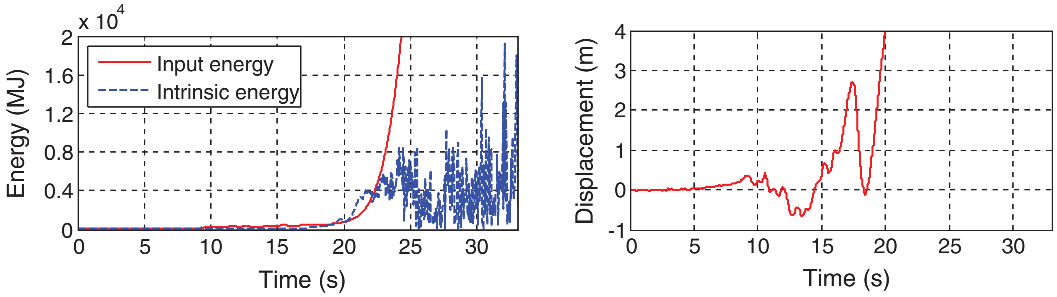
(a) (b)

Figure 6: The effective energy and the top displacement time histories under EQ2. (a) Effective energy time histories and (b) Displacement time history



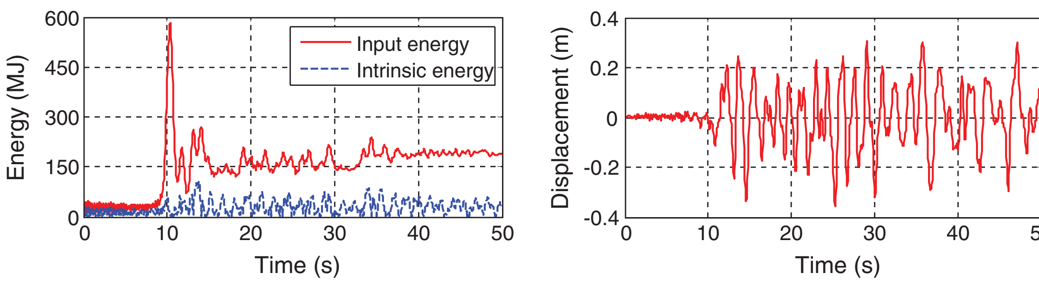
(a) (b)

Figure 7: The effective energy and the top displacement time histories under EQ1a. (a) Effective energy time histories and (b) Displacement time history



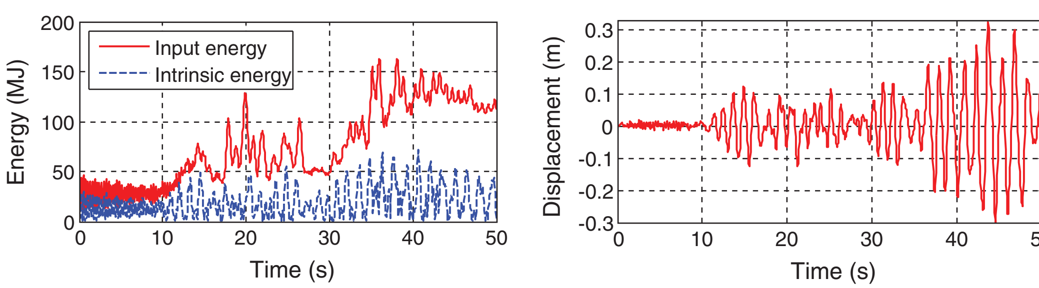
(a) (b)

Figure 8: The effective energy and the top displacement time histories under EQ2a. (a) Effective energy time histories and (b) Displacement time history



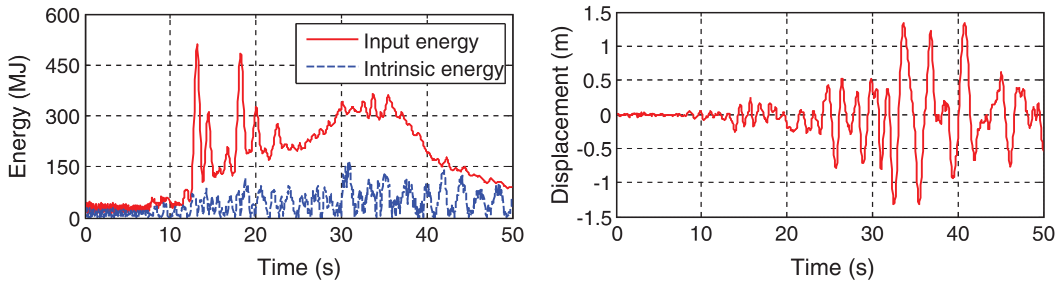
(a) (b)

Figure 9: The effective energy and the top displacement time histories under EQ3a. (a) Effective energy time histories and (b) Displacement time history



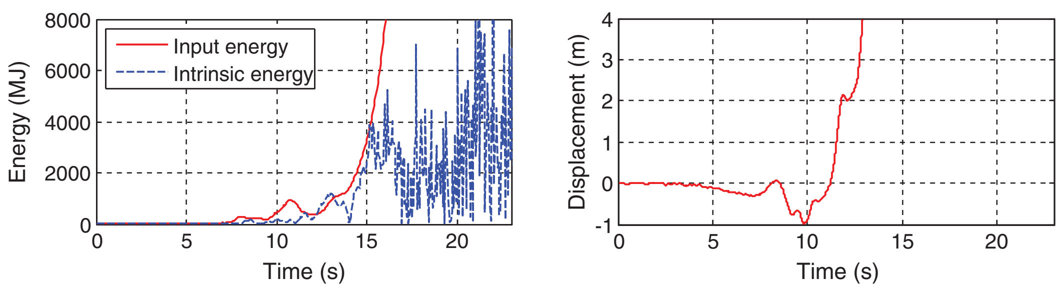
(a) (b)

Figure 10: The effective energy and the top displacement time histories under EQ4a. (a) Effective energy time histories and (b) Displacement time history



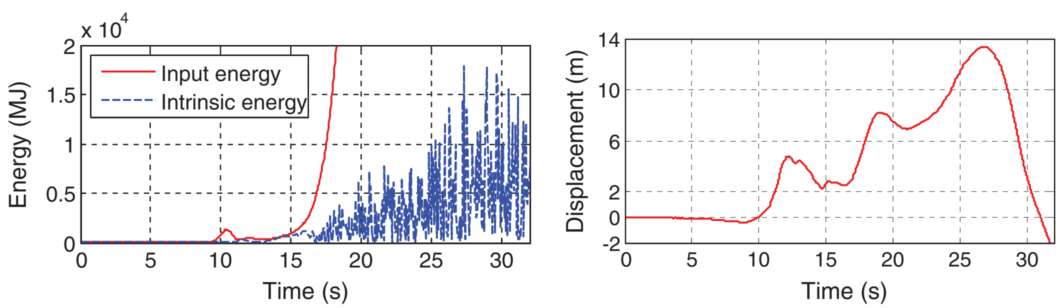
(a) (b)

Figure 11: The effective energy and the top displacement time histories under EQ1b. (a) Effective energy time histories and (b) Displacement time history



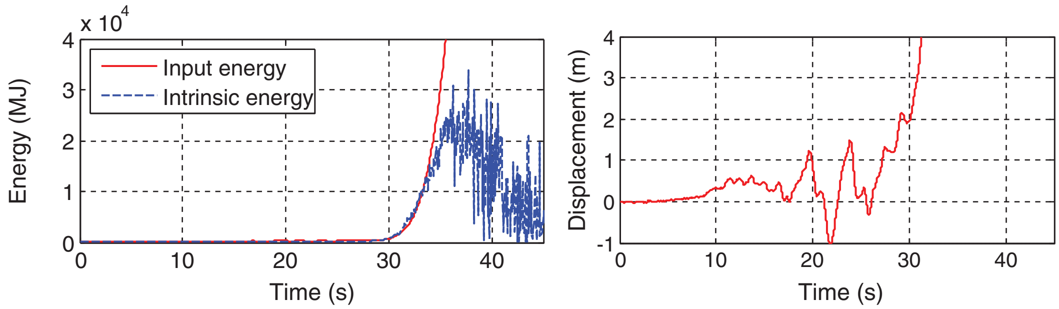
(a) (b)

Figure 12: The effective energy and the top displacement time histories under EQ2b. (a) Effective energy time histories and (b) Displacement time history



(a) (b)

Figure 13: The effective energy and the top displacement time histories under EQ3b. (a) Effective energy time histories and (b) Displacement time history



(a) (b)

Figure 14: The effective energy and the top displacement time histories under EQ4b. (a) Effective energy time histories and (b) Displacement time history

5 Conclusions

This paper presents a refined damage and collapse assessment of a large RC cooling tower subjected to earthquake ground motions. A complete numerical model is developed for simulating the nonlinear seismic behavior of the cooling towers, including the material failure, component fracturing, and the structural buckling or collapse process. The collapse mechanisms of the cooling tower are studied from the energy absorption and dissipation points of view. An effective energy-based collapse criterion is recommended to identify the dynamic stability of the cooling tower instead of the local response-based predictions. The following conclusions may be drawn:

1. Elaborate structural modeling is the basis for accurate response analysis of the cooling towers under extreme dynamic excitations. The softening, degradation, and other essential properties of concrete can be fully represented by the dynamic stochastic damage model, which serves as a critical tool for reproducing the seismic collapse of the large cooling towers. The integrated approach can help better understand the disastrous mechanisms and be extensively applied to the optimal design and analysis of large RC cooling towers.
2. Although the bottom crossed oblique columns are the intended weakest part of the cooling tower for seismic fortification, the cooling tower would collapse following with quite different damage paths and failure patterns. With random material damage evolutions, some of the collapses are caused by the strength failure of the bottom crossed oblique columns, while some are due to the buckling of the upper shell barrel. Moreover, the variation in the earthquake record characteristics and intensities may also contribute to the diversity and pattern of the damage behavior of the cooling tower. It is therefore necessary to account for the potentially multiple collapse modes in designing the large cooling towers, by taking the randomness of ground motions into consideration.
3. The effective energy-based collapse criterion works well in quantitatively identifying the dynamic stability of the cooling towers. Two aggregated energy indices are much more

effective and useful than the local structural response in predicting the global performances of the cooling towers. While different collapse modes are observed, the collapse criterion can predict accurately the critical time of the collapse initiation. The proposed methodology is vital to further reliability assessment of the actual large RC cooling towers.

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