Seismic Reliability Analysis of Complex Nuclear Power Plants by Explicit Time Domain Method

Baomu Li
Ph.D. Student, School of Civil Engineering and Transportation, South China University of Technology, Guangzhou, China

Cheng Su
Professor, School of Civil Engineering and Transportation & State Key Laboratory of Subtropical Building Science, South China University of Technology, Guangzhou, China
Corresponding author (email: cvchsu@scut.edu.cn)

Jianhua Xian
Ph.D. Student, School of Civil Engineering and Transportation, South China University of Technology, Guangzhou, China

Yunlun Sun
Senior Research Engineer, Beijing CNEC Energy Science and Technology Co., Ltd., Beijing, China

ABSTRACT: Seismic reliability evaluation is of great importance in nuclear power engineering. The task remains an open challenge since it will involve the dynamic reliability analysis of large-scale complex structures of nuclear power plants on a global structure level under random seismic excitations, and in particular in the presence of structural uncertainties. The traditional random vibration methods with coupling treatment of the physical and the probabilistic evolution mechanism are hardly capable of executing such a difficult task. In this study, the explicit time-domain method (ETDM) developed in recent years is applied to the seismic global reliability analysis of complex nuclear power plants in consideration of structural uncertainties. The time-domain explicit expressions of the critical responses involved are first constructed based on the impulse response functions, and on this basis, the subsequent random vibration and reliability analysis can then be conducted just focusing on the selected critical responses. The uncoupling treatment of the two sets of mechanism in ETDM will lead to a real-sense dimensional reduction in terms of degrees of freedoms and time instants involved in random vibration analysis of structures, and thus a high efficiency in dynamic reliability analysis even in the presence of large-scale structural models. The engineering application to a nuclear power plant with over 2 million degrees of freedom, which is now being built in China, shows the feasibility of the present approach.

1. INTRODUCTION

The safety problems of nuclear power plants exposed to seismic hazard have always received considerable attention because a failure of such a special structure can lead to catastrophic consequences. In view of the importance of nuclear power plants, there is a growing need for conducting the seismic reliability analysis of such complex structures. However, this remains an open challenge since the nuclear power plants are usually characterized by a huge number of degrees of freedom, leading to an unacceptable computational cost. The problems will be even more complicated when the uncertainties of structural parameters such as material properties and geometrical properties are considered. Therefore, it has been a focus to find an effective method for seismic reliability analysis of large-scale structures involving random structural parameters.

The direct Monte Carlo simulation (MCS) has been widely accepted as a versatile method for dynamic reliability assessment of general structures (Schüeller and Pradlwarter, 2009). However, it is still intractable for large-scale structures when small failure probability is of interest, because the method requires a large number of runs of large-scale model. The importance sampling technique (Au and Beck, 2001a) and the subset simulation (Au and Beck, 2001b) are usually employed to reduce the sample size. To further enhance the efficiency for seismic reliability analysis by MCS, a high-efficient sample analysis method of large-scale structures is needed.

In recent years, Su et al. have proposed and developed a family of explicit time-domain method (ETDM), which is mainly devoted to solving the nonstationary random vibration problems of linear and nonlinear large-scale structures (Su and Xu, 2014; Su et al., 2016; Hu et al., 2016; Su et al., 2018a; Su et al., 2018b). Unlike the traditional coupled physical-statistical random vibration methods, the ETDM is capable of manipulating the physical and the statistical evolution separately, which will lead to a real-sense dimensional reduction in terms of degrees of freedoms and time instants involved in random vibration analysis of structures, and thus a high efficiency in dynamic reliability analysis even in the presence of large-scale structural models. The time-domain explicit expressions of the critical responses involved are first constructed based on the impulse response functions, and on this basis, the subsequent random vibration and reliability analysis with MCS can then be conducted just focusing on the selected critical responses. In conjunction with the total probability theorem in probability theory, the method for deterministic structures is further extended to the seismic reliability analysis of stochastic structures. The engineering application to a nuclear power plant with over 2 million degrees of freedom, which is now being built in China, shows the feasibility of the present approach.

2. EXPLICIT EXPRESSIONS OF DYNAMIC RESPONSES

For a generic linear structural system subjected to seismic excitations, the equation of motion can be expressed as

$$\mathbf{M}\ddot{\mathbf{U}} + \mathbf{C}\dot{\mathbf{U}} + \mathbf{K}\mathbf{U} = \mathbf{L}X(t) \tag{1}$$

where $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ denote the mass, damping and stiffness matrix of the structure, respectively; $\mathbf{U}$, $\dot{\mathbf{U}}$ and $\ddot{\mathbf{U}}$ denote the time-dependent nodal displacement, velocity and acceleration vector of the structure, respectively; $\mathbf{L}$ denotes the orientation vector of the seismic excitation; and $X(t)$ denotes a random process of the ground motion acceleration.

For the linear equation of motion shown in Eq. (1), define the state vector as $\mathbf{V} = [\mathbf{U}^{\mathrm{T}}\ \dot{\mathbf{U}}^{\mathrm{T}}]^{\mathrm{T}}$. Then, with the assumption that $\mathbf{V}_0 = \mathbf{V}(0) = \mathbf{0}$, the explicit expression of the state vector at each time instant can be derived as

$$\mathbf{V}_i = \mathbf{A}_{i,0}X_0 + \mathbf{A}_{i,1}X_1 + \cdots + \mathbf{A}_{i,i}X_i \quad (i = 1, 2, \cdots, n) \tag{2}$$

where n is the number of time steps for time-history analysis; $\mathbf{V}_i = \mathbf{V}(t_i)$ and $t_i = i\Delta t$ with Δt being the time step; $X_j = X(t_j)(j = 0, 1, \cdots, i)$ are the seismic excitations at different time instants; and $\mathbf{A}_{i,0}, \mathbf{A}_{i,1}, \cdots, \mathbf{A}_{i,i}$ are the corresponding coefficient vectors, which are associated with the structural parameters and reflect the influence of structural parameters on dynamic responses. The coefficient vectors can be expressed in closed forms as

$$\begin{cases} \mathbf{A}_{1,0} = \mathbf{Q}_1,\ \mathbf{A}_{i,0} = \mathbf{T}\mathbf{A}_{i-1,0} \quad (2 \le i \le n) \\ \mathbf{A}_{1,1} = \mathbf{Q}_2,\ \mathbf{A}_{2,1} = \mathbf{T}\mathbf{Q}_2 + \mathbf{Q}_1,\ \mathbf{A}_{i,1} = \mathbf{T}\mathbf{A}_{i-1,1} \quad (3 \le i \le n) \\ \mathbf{A}_{i,j} = \mathbf{A}_{i-1,j-1} \quad (2 \le j \le i \le n) \end{cases} \tag{3}$$

where $\mathbf{T}$, $\mathbf{Q}_1$ and $\mathbf{Q}_2$ can be derived based on the Newmark-β integration scheme as (Su and Xu, 2014; Su et al., 2016)

$$\begin{cases} \mathbf{T} = \begin{bmatrix} \mathbf{H}_{11} & \mathbf{H}_{12} \\ \mathbf{H}_{21} & \mathbf{H}_{22} \end{bmatrix},\ \mathbf{Q}_1 = \begin{bmatrix} \mathbf{R}_1 \\ \mathbf{R}_3 \end{bmatrix}\mathbf{L},\ \mathbf{Q}_2 = \begin{bmatrix} \mathbf{R}_2 \\ \mathbf{R}_4 \end{bmatrix}\mathbf{L} \\ \mathbf{H}_{11} = \hat{\mathbf{K}}^{-1}(\mathbf{S}_1 - \mathbf{S}_3\mathbf{M}^{-1}\mathbf{K}) \\ \mathbf{H}_{12} = \hat{\mathbf{K}}^{-1}(\mathbf{S}_2 - \mathbf{S}_3\mathbf{M}^{-1}\mathbf{C}) \\ \mathbf{H}_{21} = a_3(\mathbf{H}_{11} - \mathbf{I}) + a_5\mathbf{M}^{-1}\mathbf{K} \\ \mathbf{H}_{22} = a_3\mathbf{H}_{12} - a_4\mathbf{I} + a_5\mathbf{M}^{-1}\mathbf{C} \\ \mathbf{R}_1 = \hat{\mathbf{K}}^{-1}\mathbf{S}_3\mathbf{M}^{-1},\ \mathbf{R}_2 = \hat{\mathbf{K}}^{-1} \\ \mathbf{R}_3 = a_3\mathbf{R}_1 - a_5\mathbf{M}^{-1},\ \mathbf{R}_4 = a_3\mathbf{R}_2 \\ \hat{\mathbf{K}} = \mathbf{K} + a_0\mathbf{M} + a_3\mathbf{C} \\ \mathbf{S}_1 = a_0\mathbf{M} + a_3\mathbf{C},\ \mathbf{S}_2 = a_1\mathbf{M} + a_4\mathbf{C},\ \mathbf{S}_3 = a_2\mathbf{M} + a_5\mathbf{C} \\ a_0 = \dfrac{1}{\beta\Delta t^2},\ a_1 = \dfrac{1}{\beta\Delta t},\ a_2 = \dfrac{1}{2\beta} - 1 \\ a_3 = \dfrac{\gamma}{\beta\Delta t},\ a_4 = \dfrac{\gamma}{\beta} - 1,\ a_5 = \dfrac{\Delta t}{2}(\dfrac{\gamma}{\beta} - 2) \end{cases} \quad (4)$$

in which $\mathbf{I}$ denotes the unit matrix, and γ and β are two parameters that can be determined according to integration stability. In this study, $\gamma = 0.5$ and $\beta = 0.25$ are used and the Newmark-β integration scheme will be unconditionally stable.

According to Eq. (3), the coefficient vectors can be arranged in the form shown in Table 1, from which it can be seen that only the coefficient vectors $\mathbf{A}_{i,0}$ and $\mathbf{A}_{i,1}(i = 1, 2, \cdots, n)$ in the first two columns need to be calculated and stored, while the other coefficient vectors in the rest columns can be directly obtained from those in the second column.

Table 1: Coefficient vectors for each time instant.

Time instant	*Coefficient vector*						
	X_0	X_1	X_2	…	X_{n-2}	X_{n-1}	X_n
t_1	$\mathbf{A}_{1,0}$	$\mathbf{A}_{1,1}$					
t_2	$\mathbf{A}_{2,0}$	$\mathbf{A}_{2,1}$	$\mathbf{A}_{1,1}$				
⋮	⋮	⋮	⋮	⋱			
t_{n-2}	$\mathbf{A}_{n-2,0}$	$\mathbf{A}_{n-2,1}$	$\mathbf{A}_{n-3,1}$	…	$\mathbf{A}_{1,1}$		
t_{n-1}	$\mathbf{A}_{n-1,0}$	$\mathbf{A}_{n-1,1}$	$\mathbf{A}_{n-2,1}$	…	$\mathbf{A}_{2,1}$	$\mathbf{A}_{1,1}$	
t_n	$\mathbf{A}_{n,0}$	$\mathbf{A}_{n,1}$	$\mathbf{A}_{n-1,1}$	…	$\mathbf{A}_{3,1}$	$\mathbf{A}_{2,1}$	$\mathbf{A}_{1,1}$

Besides using Eq. (3), the coefficient vectors $\mathbf{A}_{i,0}$ and $\mathbf{A}_{i,1}(i = 1, 2, \cdots, n)$ can also be determined through two deterministic time-history analyses of the structure subjected to two unit impulse excitations $p_0(t)$ and $p_1(t)$, as shown in Figure 1 and Figure 2, respectively. It can be easily observed from Eq. (2) that the coefficient vectors $\mathbf{A}_{i,0}$ and $\mathbf{A}_{i,1}(i = 1, 2, \cdots, n)$ are exactly the corresponding solutions $\mathbf{V}_i^0$ and $\mathbf{V}_i^1(i = 1, 2, \cdots, n)$ with respect to the two load cases, as also illustrated in Figure 1 and Figure 2, respectively. Therefore, the computational cost of all the coefficient vectors is equivalent to that required by two deterministic time-history analyses of the structure.

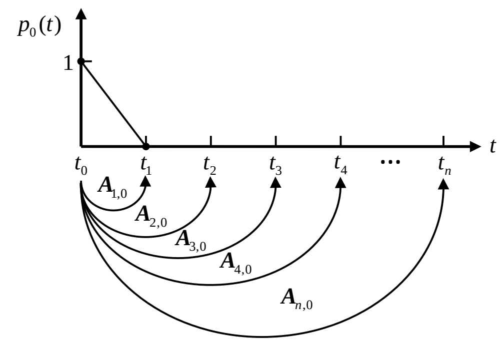

Figure 1: The unit impulse excitation $p_0(t)$.

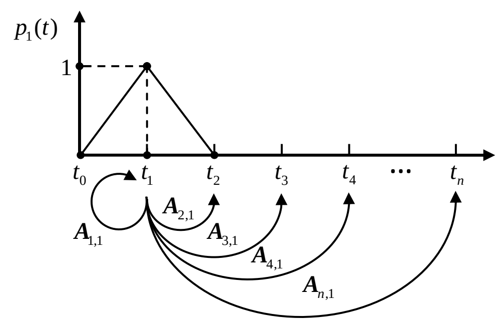

Figure 2: The unit impulse excitation $p_1(t)$.

For the purpose of seismic reliability analysis, not all structural responses are required, and only a certain number of critical responses need to be focused on. Suppose s_i is a critical response component of interest in $\mathbf{V}_i$. Then, from Eq. (2), the explicit expression of s_i can be directly obtained as

$$s_i = a_{i,0}X_0 + a_{i,1}X_1 + \cdots + a_{i,i}X_i \quad (i = 1, 2, \cdots, n) \tag{5}$$

where $a_{i,j}$ are the corresponding elements of $\mathbf{A}_{i,j}\,(j = 0, 1, \cdots, i)$ with respect to s_i.

Obviously, only a small number of elements in the coefficient vectors $\mathbf{A}_{i,0}$ and $\mathbf{A}_{i,1}(i = 1, 2, \cdots, n)$ with respect to the critical responses need to be stored. If n_s critical responses are required in time-history analysis with n time steps, the total number of coefficients that need to be stored is $N_s = 2n_s n$, which is independent of the number of degrees of freedom of the structure. Therefore, even for a complex structure with a large number of degrees of freedom, for instance, the nuclear power plant, the storage of the coefficients can be easily accomplished for construction of the explicit expressions of the required responses.

Thus far, the manipulation of the physical evolution mechanism of the structural system has been accomplished and the evolution of the critical response can be reflected using the closed-form expression shown in Eq. (5), which will lead to a real-sense dimensional reduction in terms of degrees of freedoms and time instants involved when random vibration and reliability analysis of structures are conducted.

3. SEISMIC RELIABILITY ANALYSIS FOR DETERMINISTIC STRUCTURES

Using the first passage failure criterion with symmetric double boundary value, the seismic reliability of the structural system described in Eq. (1) can be defined as

$$P_r(T) = \mathrm{P}\{\frac{|s(t)|}{b} \le 1,\ t \in [0, T]\} \tag{6}$$

where P{•} indicates the probability of the random event; T is the duration of the seismic excitation; b is the value of the symmetric boundary; and $s(t)$ is the critical response that controls the structural failure. Note that the uncertainties of the seismic excitation duration T and the symmetric boundary value b are not considered in the current study.

The expression of (6) is equivalent to

$$P_r(T) = \mathrm{P}\{\max_{t\in[0,T]} \frac{|s(t)|}{b} \le 1\} \tag{7}$$

Generally speaking, the failure of the structural system is controlled by several critical responses rather than only one critical response. In this case, with the weakest link assumption, the seismic reliability of the structural system can be defined as

$$P_r(T) = \mathrm{P}\{\max_{j=1,2,\cdots,n_s} [\max_{t\in[0,T]} \frac{|s_j(t)|}{b_j}] \le 1\} \tag{8}$$

where n_s is the number of critical responses that control different structural failure modes, and $s_j(t)$ and $b_j(j = 1, 2, \cdots, n_s)$ are the critical responses and the corresponding boundary values, respectively. Then, the failure probability of the structural system can be obtained as $P_f(T) = 1 - P_r(T)$.

It can be seen from Eq. (5) that the structural dynamic responses can be expressed as a linear function of excitation values at different time instants. Based on the explicit expression of dynamic responses, the MCS can be easily conducted for seismic reliability analysis of structures without the need for repetitively solving the equation of motion shown in Eq. (1), leading to a high computational efficiency.

For the sake of clarity, the procedures of the ETDM-based MCS for seismic reliability analysis of structural systems are summarized as follows:

(1) Determine the critical responses that control the failure modes of the structural system. Assume n_s critical responses are considered. Then, the critical responses and the corresponding values of symmetric boundary are taken as $s_j(t)$ and $b_j(j = 1, 2, \cdots, n_s)$, respectively.

(2) Generate a sufficient number of samples of seismic excitations with the given power spectral density function of ground motion acceleration through numerical simulation. Suppose N samples of seismic excitations are obtained.

(3) For a given sample of seismic excitation, calculate the critical responses $s_j(t)(j=1,2,\cdots,n_s)$ using Eq. (5). If $\max_{j=1,2,\cdots,n_s}[\max_{t\in[0,T]}\frac{|s_j(t)|}{b_j}]>1$, a failure of the structural system is observed. Repeat the above calculation for each sample of seismic excitation until all samples have been considered. Suppose the number of structural failure is N_0. Then, the failure probability of the structural system can be obtained as $P_f(T)=N_0/N$.

4. SEISMIC RELIABILITY ANALYSIS FOR STOCHASTIC STRUCTURES

In general stochastic dynamic systems, structural random parameters and random excitation parameters are regarded as mutually independent due to the influence of various physical phenomena. Although random responses of the structure are complex functions of structural random parameters and random excitation parameters, the two sets of random parameters can be decoupled in the probability sense. Therefore a sensible strategy for seismic reliability analysis of stochastic structures is to address the random excitation parameters first and then the structural random parameters. The conversion relationship between conditional probability and total probability in probability theory provides the mathematical tools to solve the seismic reliability of stochastic structures from the seismic reliability of deterministic structures.

For the linear structural system shown in Eq. (1), assume that the uncertain structural parameters are denoted by an n_p-dimensional random vector $\mathbf{\Theta}=[\Theta_1\ \Theta_2\ \cdots\ \Theta_{n_p}]^T$. Then, the equation of motion (1) can be rewritten as

$$\mathbf{M}(\mathbf{\Theta})\ddot{\mathbf{U}}+\mathbf{C}(\mathbf{\Theta})\dot{\mathbf{U}}+\mathbf{K}(\mathbf{\Theta})\mathbf{U}=\mathbf{L}(\mathbf{\Theta})X(t) \quad (9)$$

According to the total probability formula (Wang, 2007), the seismic reliability $P_r(T)$ of the stochastic structural system shown in Eq. (9) can be expressed as

$$P_r(T)=\int_{-\infty}^{+\infty}P_r(T\,|\,\boldsymbol{\theta})f_{\mathbf{\Theta}}(\boldsymbol{\theta})\mathrm{d}\boldsymbol{\theta} \quad (10)$$

where $P_r(T\,|\,\boldsymbol{\theta})$ is the conditional seismic reliability under $\mathbf{\Theta}=\boldsymbol{\theta}$ and $f_{\mathbf{\Theta}}(\boldsymbol{\theta})$ is the joint probability density function of the random vector $\mathbf{\Theta}$. The solution of $P_r(T\,|\,\boldsymbol{\theta})$ falls into the seismic reliability problem of deterministic structures, which can be directly obtained through the solution procedures described in Section 3.

As the analytical form of $P_r(T\,|\,\boldsymbol{\theta})$ with respect to $\boldsymbol{\theta}$ is difficult to derive, it is obtained by numerical fitting using the response surface method in this study. By a limited number of numerical experiments, $P_r(T\,|\,\boldsymbol{\theta})$ can be expressed in quadratic polynomial form in terms of the structural random parameters $\mathbf{\Theta}=[\Theta_1\ \Theta_2\ \cdots\ \Theta_{n_p}]^T$ as

$$P_r(T\,|\,\boldsymbol{\theta})=a+\sum_{j=1}^{n_p}b_j\theta_j+\sum_{j=1}^{n_p}c_j\theta_j^2 \quad (11)$$

where a, b_j and c_j $(j=1,2,\cdots,n_p)$ are undetermined coefficients. To determine the $(2n_p+1)$ coefficients in Eq. (11), the same number of experimental points or seismic reliability analyses of deterministic structures are needed. The specific procedures are described as follows:

(1) Select $(2n_p+1)$ numerical experimental points according to the experimental design method suggested by Bucher and Bourgund (1990). They include the mean point

$(\mu_1, \mu_2, \cdots, \mu_{n_p})$ and $2n_p$ axial points $(\mu_1, \cdots, \mu_j \pm f\sigma_j, \cdots, \mu_{n_p})(j=1,2,\cdots,n_p)$, in which μ_j and σ_j are the mean and the standard deviation of the j-th structural random parameter $\Theta_j(j=1,2,\cdots,n_p)$, respectively, and f is generally taken as 2 - 3. In the present study, f is assumed to be 2.5. For convenience, these $(2n_p+1)$ numerical experimental points are denoted by $\boldsymbol{\theta}_k(k=1,2,\cdots,2n_p+1)$.

(2) Calculate the mass matrices $\mathbf{M}(\boldsymbol{\theta}_k)$, the damping matrices $\mathbf{C}(\boldsymbol{\theta}_k)$, the stiffness matrices $\mathbf{K}(\boldsymbol{\theta}_k)$ and the orientation vectors $\mathbf{L}(\boldsymbol{\theta}_k)$ with respect to the numerical experimental points $\boldsymbol{\theta}_k(k=1,2,\cdots,2n_p+1)$. Then, the conditional seismic reliabilities $P_r(T|\boldsymbol{\theta}_k)(k=1,2,\cdots,2n_p+1)$ can be obtained by the solution procedures stated in Section 3.

(3) Solve for the undetermined coefficients in Eq. (11) by the known conditional seismic reliabilities $P_r(T|\boldsymbol{\theta}_k)(k=1,2,\cdots,2n_p+1)$, and obtain the expression of $P_r(T|\boldsymbol{\theta})$.

(4) Substitution of the expression of $P_r(T|\boldsymbol{\theta})$ into Eq. (10) yields the seismic reliability of the stochastic structural system as

$$\begin{aligned} P_r(T) &= \int_{-\infty}^{+\infty}(a+\sum_{j=1}^{n_p} b_j\theta_j + \sum_{j=1}^{n_p} c_j\theta_j^2) f_{\boldsymbol{\Theta}}(\boldsymbol{\theta})\mathrm{d}\boldsymbol{\theta} \\ &= a+\sum_{j=1}^{n_p} b_j\mu_j + \sum_{j=1}^{n_p} c_j(\sigma_j^2+\mu_j^2) \end{aligned} \quad (12)$$

It can be seen from the above procedures that the seismic reliability analysis of stochastic structures is based on that of deterministic structures.

5. ENGINEERING APPLICATION

To demonstrate the feasibility of the present approach for complex nuclear power plants, a seismic reliability analysis is conducted for the Huaneng high-temperature gas-cooled reactor nuclear power plant now being built in China. The nuclear power plant considered herein is composed of the reactor plant, the spent fuel plant and the nuclear auxiliary plant.

The finite element model of the nuclear power plant is established using the general-purpose finite element software ANSYS. The whole model consists of 7,122 beam elements, 265,523 shell elements and 179,058 solid elements, leading to 474,432 nodes and a total number of 2,141,352 (about 2.14 million) degrees of freedom for the whole structure. A total number of 150 mode shapes are considered with the damping ratio of each mode being $\zeta = 0.07$.

The nonstationary ground acceleration process $X(t)$ is assumed to be a uniformly modulated nonstationary zero-mean random process expressed as

$$X(t) = g(t)x(t) \quad (13)$$

with $g(t)$ being a modulation function and $x(t)$ being a stationary random process with zero mean. The modulation function is set to be

$$g(t) = \begin{cases} (t/t_a)^2 & 0 \le t < t_a \\ 1 & t_a \le t < t_b \\ \mathrm{e}^{-\lambda(t-t_b)} & t_b \le t \le t_c \end{cases} \quad (14)$$

with $t_a = 3.0$ s, $t_b = 11.0$ s, $t_c = 25.0$ s and $\lambda = 0.15$.

Consider a level of earthquake with the average peak ground acceleration being $4.5\,\mathrm{m/s^2}$. The corresponding design acceleration response spectrum is shown in Figure 3. The power spectral density function of $x(t)$ compatible with the design acceleration response spectrum is presented in Figure 4, by which one can generate a set of samples using the spectral representation method of a stochastic process (Shinozuka, 1972). Substitution of the samples of $x(t)$ into Eq. (13) yields the samples of nonstationary ground acceleration process $X(t)$, one of which is presented in Figure 5. As the compatible power spectrum of ground motion acceleration is used, the average acceleration response spectrum corresponding to such seismic excitation samples will be identical to the prescribed design acceleration response spectrum.

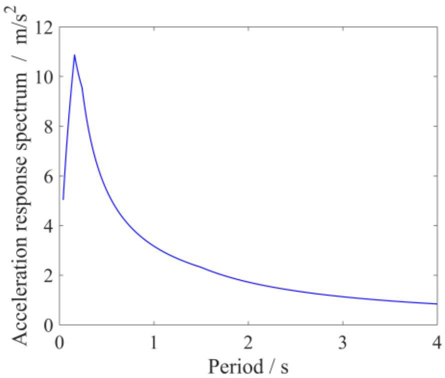

Figure 3: The design acceleration response spectrum.

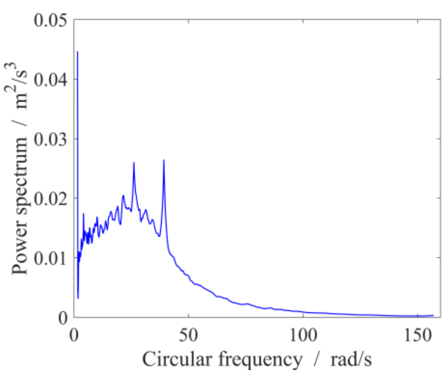

Figure 4: The compatible power spectrum of ground motion acceleration.

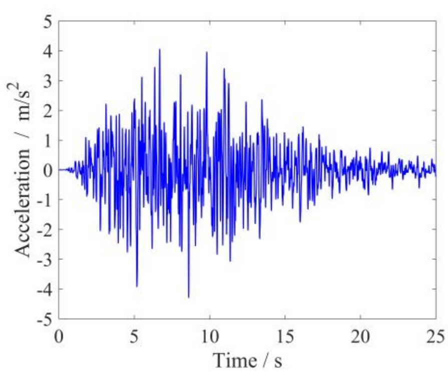

Figure 5: A sample of ground acceleration.

The critical responses of the nuclear power plant are taken as the shear forces per length of the 664 shear wall elements, as shown in Figure 6. The structural failure occurs when any of the critical responses exceeds its corresponding boundary value, which is set to be the bearing capacity of the corresponding shear wall element. Both cases of deterministic and stochastic structure are taken into account in this example. For the case of stochastic structure, the Young's modulus E and the density ρ of concrete are assumed to be mutually independent random variables with the probabilistic information listed in Table 2, while for the case of deterministic structure, the Young's modulus and the density of concrete are taken to be the mean values in Table 2. The seismic reliability analyses of the deterministic and the stochastic structure are conducted using the present methods described in Section 3 and Section 4, respectively. For the above two cases, the number of samples of seismic excitations is taken as $N = 100,000$ and the duration of the time-history analysis is set to be $T = 25$ s with the time step being $\Delta t = 0.01$ s.

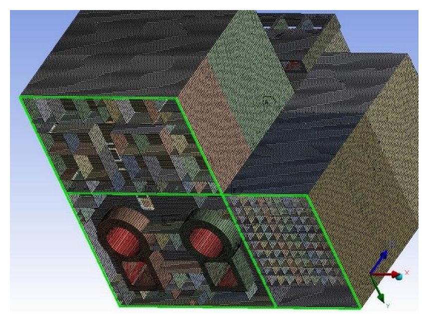

Figure 6: The highlighted shear wall elements for seismic reliability analysis.

Table 2: Probabilistic information of the random structural parameters.

Random parameter	*Distribution type*	*Mean*	*COV*
E (GPa)	*Normal*	*31.5*	*0.167*
ρ (kg/m^3)	*Normal*	*2,400*	*0.167*

Note: COV = coefficient of variation

The failure probabilities of the structural system corresponding to the two cases are shown in Figure 7. It can be observed that the statistical variations of structural parameters tend to increase the failure probability of the structural system. It is worth noting that, for seismic reliability analysis of this complex nuclear power plant with over 2 million degrees of freedom, the elapsed times of the present approach are around 1.5 hours and 7.5 hours for the cases of deterministic and stochastic structure, respectively, which is acceptable in practice for seismic reliability analysis of such a large-scale structure.

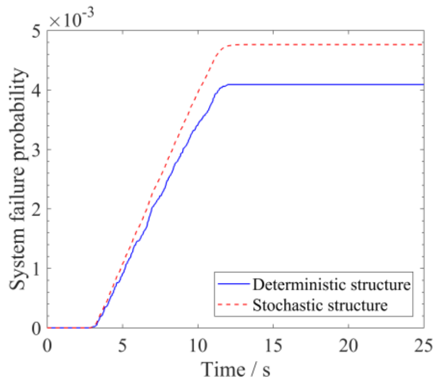

Figure 7: System failure probability under seismic excitations.

6. CONCLUSIONS

The ETDM-based MCS has been developed for the seismic reliability analysis of complex nuclear power plants in consideration of the uncertainties of both seismic excitations and structural parameters. Using ETDM, the explicit expressions of the critical responses are constructed through two impulse response time-history analyses of the structure, and on this basis, the MCS can be readily carried out just focusing on the critical responses, leading to a high computational efficiency. In conjunction with the total probability theorem, the method for seismic reliability analysis of the deterministic structures is extended to that of the stochastic structures. The engineering application to a nuclear power plant with over 2 million degrees of freedom shows the feasibility of the present approach.

7. ACKNOWLEDGEMENTS

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