Research Article

Simulation of the In Situ Spatially Varying Ground Motions and Nonlinear Seismic Response Analysis of the Cable-Stayed Bridge

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To study the nonlinear seismic behavior and seismic resistance of the long-span cable-stayed bridges subjected to earthquakes, the multidimensional and multisupported artificial ground motions are synthesized first based on the in situ site conditions of the bridge considering the coherent and traveling wave effects. Then, considering the material nonlinearity of the cable-stayed bridge, a 3D finite element model is established based on the OpenSees platform, and the nonlinear seismic response analysis of the bridge is carried out under the synthetic artificial ground motions. The nonlinear seismic response of main bridge components such as piers, towers, bearings, and cables is analyzed, and key conclusions and observations are drawn.

1. Introduction

Simulation of ground motions is always a critical issue in earthquake engineering. Using the uniform ground motions in seismic response analysis of large-span structures with large planar dimensions, such as the bridges, oil pipelines, and offshore platforms, is often not accurate. The dimension of the large-span structure cannot be ignored compared to the wavelength of the seismic waves. Therefore, it is important to consider the spatial variability of ground motion in seismic analysis of such large-span structures [1]. When seismic analysis of large-span structures is carried out, different seismic excitations at each support should be considered to ensure the seismic response accuracy of long-span structures. Meanwhile, the multidimensional and nonstationary characteristics of ground motions should also be modelled. Unfortunately, due to the scarcity of measured spatial ground motions, engineering seismic analysis of large-span structure under natural ground motions has been greatly restricted. Therefore, it is of prime significance for synthesis of spatially variable ground motions.

In terms of artificial ground motions simulation, worldwide scholars have made important contributions. Assuming that the phase angle is uniformly distributed within $(0-2\pi)$, Hao et al. [2] used the Cholesky decomposition of the power spectral matrix for synthesis of multisupport artificial seismic waves. Ramadan and Novak [3] proposed and compared the computational efficiency of simulation methods in seismic analysis of single-span and multispan simply supported beams. Qu and Wang [4, 5] used stochastic vibration theory to derive the relationship between amplitude, phase angle, and power spectrum matrix of spatially varying ground motion; however, multidimensionality of ground motions needs further research. The new version of the program, RspMatch2005 proposed by Hancock et al. [6], enabled the accelerograms to be matched to the pseudoacceleration or displacement spectral ordinates as well as the spectrum of absolute acceleration, with matching to be performed simultaneously to a given spectrum at several damping ratios. An energy-based envelope function which directly related to the Arias intensity of the ground motion was developed by Stafford

et al. [7] for utilization in the stochastic simulation of earthquake ground motion. On the other hand, a fully nonstationary stochastic model employing filtering of a discretized white-noise process for strong earthquake ground motion was developed by Rezaeian and Kiureghian [8]; specially, nonstationarity was achieved by modulating the intensity and varying the filter properties in time. Bi and Hao [9] synthesized spatially varying ground motions based on one-dimensional wave propagation theory in soil layers. Konakli and Kiureghian [10] developed a simulation method by taking into account effects of coherence, traveling wave, and local site. Yamamoto and Baker [11] proposed a wavelet packet transform approach which can be used to characterize complex time-varying earthquake ground motions and illustrated the potential benefits of such an approach in a variety of earthquake engineering applications. Jia et al. [12, 13] applied pseudoexcitation method (PEM) in seismic analysis of long-span structures under tridirectional spatially varying ground motions, and a mathematical scheme in simulating nonstationary tridirectional spatially correlated ground motions has been proposed. The virtual real-time separation method proposed by Wu et al. [14] was used to decompose the power spectrum matrix, and the ground motion correlation characteristics generated by the decomposition method are compared with the traditional cholesky decomposition method. The results showed that the method was not only fast, accurate, and feasible but also had obvious advantages in many aspects. Furthermore, Dabaghi and Kiureghian [15] developed a parameterized stochastic model of near-fault ground motion in two orthogonal horizontal directions. The major characteristics of recorded near-fault ground motions are represented to employ this model, which include near-fault effects of directivity and fling step; temporal and spectral nonstationarity; and intensity, duration, and frequency content characteristics.

In summary, a series of research results of artificial ground motion synthesis were given. Among them, the method proposed by Bi and Hao [9] fully considers the characteristics of the soil layer, and the traveling wave and coherence effect is widely used. Therefore, considering the local site layer conditions of bridges, the theoretical method of Bi and Hao [9] is used in the subsequent artificial ground motion synthesis to complete this work in the paper.

In terms of the nonlinear structural analysis, most of the previous studies focus on the high-pier continuous bridges or high-pier steel bridges and the failure location was usually predefined. However, there were very limited research literature studies on nonlinear seismic response analysis of cable-stayed bridges. Considering both the uniform and multiple-support seismic excitations, Nazmy and Abdel-Ghaffar [16] developed novel nonlinear dynamic formulation, in which the tangent stiffness and iterative scheme were utilized to capture the nonlinear seismic response. In addition, Nazmy and Abdel-Ghaffar [17] studied the nonlinear dynamic performance of a 3-D long-span cable-stayed bridge under earthquake and revealed that the multiple-support seismic excitations can have a significant effect on structural response.

Ren and Obata [18] studied the plastic behavior of long-span cable-stayed bridge structures based on a 2-D finite element model. The results indicated that the deformation of the cable-stayed bridge structure caused by static load has a significant influence on its dynamic seismic response. Li et al. [19] derived the incremental dynamic equations of multidegrees-of-freedom structure under earthquake loading, and the finite element model of the long-span cable-stayed bridge was established. Furthermore, the seismic performance of a horizontally curved highway bridge was examined by Vamvatsikos and Sigalas [20] using incremental dynamic analysis (IDA) in 3D, in which IDA has been successfully applied to two-dimensional structures with a single horizontal ground motion component. Kaviani et al. [21] focused on identifying trends in seismic behavior of reinforced concrete bridges with seat-type abutments under earthquake loading, especially with respect to the abutment skew angle. In the study by Huang et al. [22], two kinds of finite element models based on implicit and explicit integral are established, respectively. The seismic response characteristics and failure modes of the single tower cable-stayed bridge are tested. Li et al. [23] investigated the seismic vulnerability of a five-span cable-stayed bridge in the longitudinal direction. The geometry and material nonlinearity of the cable-stayed bridge structure were considered comprehensively, and a 3-D finite element model was established based on the OpenSees platform. Omrani et al. [24] proposed a probabilistic framework to achieve the sensitivity of the seismic performance of a typical reinforced concrete overpass bridge to variations in the parameters of its abutment model. Han et al. [25] studied the seismic behavior of a reinforced concrete single-tower cable-stayed bridge equipped with sliding friction bearings under 2-D seismic excitations. Based on the OpenSees platform, a three-dimensional numerical model was established, and the nonlinear seismic response and seismic performance of the single-tower cable-stayed bridge were systematically studied. Recently, Wang et al. [26, 27] evaluated the potential benefits of using buckling-restrained braces (BRBs) to seismically rehabilitate straight bridges and assessed the collapse capacity and failure modes of skewed bridges retrofitted with buckling-restrained braces (BRBs) at the column bent.

The OpenSees software, led by the Pacific Earthquake Engineering Research Center, is the dominant one in the current structural nonlinear time history analysis. Compared with large commercial finite element software ANSYS, ABAQUS, etc., OpenSees can easily call a variety of complex nonlinear stress-strain materials in a wider range. At present, the software has been widely used in nonlinear seismic response analysis of long-span bridges. Since the last century, many scholars have done a lot of research on the constitutive relations of common confined concrete and proposed different constitutive relation models. Some constitutive relation models are based on theoretical derivation, other models are based on statistical analysis of numerical calculations, and few models are based on the semiempirical and semitheoretical results. At present, there are mainly popular models: Mander model [28]; Kent-Park model [29]; and Hoshikuma model [30].

For the seismic response analysis of reinforced concrete structures, it is especially important to accurately simulate the hysteretic behavior of reinforced bars under earthquake loading. In view of this, scholars from various countries had proposed a variety of mechanical calculation models, such as Ramberg-Osgood steel constitutive model [31]; Richard steel constitutive model [32]; and modified Giuffre-Menegotto-Pinto steel hysteresis constitutive model [32]. These models are widely used in nonlinear seismic response analysis.

Most of the study focused on the nonlinear seismic response of high-pier continuous girder bridges with predetermined failure locations. Generally, plastic hinges are assumed to achieve the shape deformation capability of the components and to obtain the nonlinear seismic response of the structural members. For the complex structure, such as long span cable-stayed bridges, it is rare to consider the material nonlinearity in the nonlinear seismic analysis considering the in situ site condition.

Following above discussion, the objective of this paper is to study the seismic behavior of long-span cable-stayed bridges under spatial earthquake ground motions. In Section 2, the multidimensional and multisupport artificial ground motions are synthesized first based on the in situ site conditions of the bridges considering coherent and traveling wave effects. Then, considering the material nonlinearity of the cable-stayed bridge, a 3-D finite element model is established based on OpenSees in Section 3. The nonlinear seismic response analysis of the cable-stayed bridge is carried out, and the seismic response of main structural components such as piers, towers, bearings, and cables is analyzed in Section 4. The major conclusions and findings are drawn in Section 5.

2. Simulation of In Situ Spatially Varying Ground Motions

2.1. One-Dimensional Wave Theory. For a single-frequency harmonic excitation, the wave dynamic response equation can be written as [33]

$$\nabla^2 e = \frac{\omega^2}{c_p^2} e \quad (1)$$

$$\text{or } \nabla^2 \{\Omega\} = \frac{\omega^2}{c_s^2} \{\Omega\},$$

where $\nabla^2 e$ and $\nabla^2 \{\Omega\}$ are the Laplace operator of the volumetric strain amplitude and the rotational strain vector, respectively, and c_p and c_s are the P wave and S wave propagation velocity, respectively.

The stiffness matrices $[S_{SH}]$ and $[S_{P-SV}]$ can be expressed as

$$\begin{aligned} [S_{SH}]\{\mu_{SH}\} &= \{P_{SH}\} \\ \text{or } [S_{P-SV}]\{\mu_{P-SV}\} &= \{P_{P-SV}\}, \end{aligned} \quad (2)$$

where $\{\mu_{SH}\}$ and $\{P_{SH}\}$ are the out-of-plane displacements and load vector and $\{\mu_{P-SV}\}$ and $\{P_{P-SV}\}$ are the in-plane displacements and load vector.

The soil layer transfer function $H(\omega)$ is

$$H(\omega) = \frac{v_t}{v_0} = \frac{1}{\cos kt^L d + (i/p) \sin kt^L d}, \quad (3)$$

where $p = t^R G^{*R} / t^L G^{*L}$.

In the linear elastic range, the auto-power spectrum of the soil layer at any position and the cross-power spectrum between any two points can be expressed by the following formula:

$$\begin{aligned} S_{jj}(\omega) &= |H_j(i\omega)|^2 S_g(\omega), \quad j = 1, 2, L, n, \\ S_{jk}(\omega) &= H_j(i\omega) H_k^*(i\omega) S_g(\omega) \gamma_{j'k'}(d_{j'k'}, i\omega), \quad j, k = 1, 2, L, n, \end{aligned} \quad (4)$$

where $H_j(i\omega)$ and $H_k(i\omega)$ are the site transfer functions at supports j and k , respectively. $S_g(\omega)$ is the ground motion power spectral density at the base rock; $\gamma_{j'k'}(d_{j'k'}, i\omega)$ is the coherency loss function of spatial ground motions at the base rock, where the bedrock power spectrum values are based on the modified Tajimi-Kanai power spectrum model; the formula is as follows:

$$\begin{aligned} S_g(\omega) &= |H_p| S_0(\omega) = \frac{\omega^4}{(\omega_f^2 - \omega^2)^2 + (2\omega_f \omega \xi_f)^2} \\ &\times \frac{1 + 4\xi_g^2 \omega_g^2 \omega^2}{(\omega_g^2 - \omega^2)^2 + 4\xi_g^2 \omega_g^2 \omega^2} \Gamma, \end{aligned} \quad (5)$$

where: $|H_p|$ is a high-pass filter function. $S_0(\omega)$ is the Tajimi-Kanai power spectral density. ω_g and ξ_g are the central frequency and damping ratio of the Tajimi-Kanai power spectral density function. ω_f and ξ_f are the central frequency and damping ratio of the high-pass filter.

2.2. Multidimensional Multisupport Power Spectrum Matrix. The acceleration power spectrum matrix of m ground support point excitations is expressed as [12]

$$S(i\omega) = \begin{bmatrix} S_{11}(i\omega) & S_{12}(i\omega) & S_{13}(i\omega) & \cdots & S_{1m}(i\omega) \\ S_{21}(i\omega) & S_{22}(i\omega) & S_{23}(i\omega) & \cdots & S_{2m}(i\omega) \\ S_{31}(i\omega) & S_{32}(i\omega) & S_{33}(i\omega) & \cdots & S_{3m}(i\omega) \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ S_{m1}(i\omega) & S_{m2}(i\omega) & S_{m3}(i\omega) & \cdots & S_{mm}(i\omega) \end{bmatrix}, \quad (6)$$

Here,

$$S_{kl}(i\omega) = p_{kl}(i\omega) \sqrt{S_{kk}(i\omega) S_{ll}(i\omega)}, \quad (7)$$

where, $S_{kk}(i\omega)$ and $S_{ll}(i\omega)$ are the auto-power spectral density function at supports k and l , respectively, $S_{kl}(i\omega)$ is the cross power spectral density function, and $p_{kl}(i\omega)$ is the coherent function and can be expressed as

$$p_{kl}(i\omega) = |p_{kl}(i\omega)| \exp\left(\frac{-i\omega d_{kl}^L}{v_{app}}\right), \quad (8)$$

where $|p_{kl}(i\omega)|$ is the modulus for coherent function; $\omega d_{kl}^L/v_{app}$ is the angle of $p_{kl}(i\omega)$; and v_{app} is the apparent wave velocity.

Considering the multidimensional characteristic of ground motions, the power spectral density matrix of each points can be expressed as [12]

$$S_{kl}(i\omega) = \begin{bmatrix} S_{klxx}(i\omega) & S_{klxy}(i\omega) & S_{klxz}(i\omega) \\ S_{klyx}(i\omega) & S_{klyy}(i\omega) & S_{klyz}(i\omega) \\ S_{klzx}(i\omega) & S_{klzy}(i\omega) & S_{klzz}(i\omega) \end{bmatrix}, \quad (9)$$

$$S_{xx}(i\omega) = S_{yy}(i\omega) = S_{xy}(i\omega) = S_{yz}(i\omega), \quad (10)$$

$$\begin{aligned} S_{xz}(i\omega) &= S_{yz}(i\omega) = 0.6\sqrt{S_{xx}(i\omega)S_{zz}(i\omega)} \\ &= 0.6\sqrt{S_{yy}(i\omega)S_{zz}(i\omega)}. \end{aligned} \quad (11)$$

Through equations (9)–(11), the matrix of the cross-power spectral density function is dimensionally extended.

2.3. Multidimensional and Multipoint Nonstationary Ground Motion Synthesis. The $(3m \times 3m)$ -dimensional matrix is decomposed into the product of the lower triangular matrix and the upper triangular matrix:

$$S(\omega) = P^* P^T = [e^{-i\omega T}]^* [S_{\ddot{x}}] [R] [S_{\ddot{x}}] [e^{-i\omega T}], \quad (12)$$

where

$$[e^{-i\omega T}] = \text{diag} \left[e^{-i\omega T_{1x}} e^{-i\omega T_{1y}} e^{-i\omega T_{1z}} e^{-i\omega T_{2x}} e^{-i\omega T_{2y}} \right. \\ \left. \cdot e^{-i\omega T_{2z}} \dots e^{-i\omega T_{mx}} e^{-i\omega T_{my}} e^{-i\omega T_{mz}} \right], \quad (13)$$

and $[S_{\ddot{x}}]$ can be expressed as

$$[S_{\ddot{x}}] = \text{diag} \left[\sqrt{S_{\ddot{x}_{1x}}} \sqrt{S_{\ddot{x}_{1y}}} \sqrt{S_{\ddot{x}_{1z}}} \sqrt{S_{\ddot{x}_{2x}}} \sqrt{S_{\ddot{x}_{2y}}} \right. \\ \left. \cdot \sqrt{S_{\ddot{x}_{2z}}} \dots \sqrt{S_{\ddot{x}_{mx}}} \sqrt{S_{\ddot{x}_{my}}} \sqrt{S_{\ddot{x}_{mz}}} \right]. \quad (14)$$

R is the lagged coherence matrix given by

$$R = \begin{bmatrix} 1 & |p_{12}| & L & |p_{1N}| \\ |p_{21}| & 1 & L & |p_{2N}| \\ M & M & O & M \\ |p_{N1}| & |p_{N2}| & L & 1 \end{bmatrix}, \quad (15)$$

where $N = 3m$ and R is a definite or semidefinite symmetric matrix. The rank of R is greater than 1, and it can be decomposed as the summation of nonzero eigenvalues α_j and the corresponding normalized eigenvectors $\{\varphi\}_j$ ($j = 1, 2, \dots, r, r \leq 3m$), which is given by

$$S(\omega) = P^* P^T = [e^{-i\omega T}]^* [S_{\ddot{x}}] [Q] [Q]^T [S_{\ddot{x}}] [e^{-i\omega T}], \quad (16)$$

where, $[e^{-i\omega T}]$, $[S_{\ddot{x}}]$, and $[Q]$ characterize the traveling wave effect, the site effect, and the coherence effect, respectively. After obtaining $P = [e^{-i\omega T}] [S_{\ddot{x}}] [Q]$, the stationary time series $u_{m_{dx}}(t)$ can be simulated in the time domain as

$$u_{m_{dx}}(t) = \sum_{b=1}^{3m} \sum_{k=0}^{N-1} a_{mb}(\omega_k) \cos[\omega_k t + \theta_{mb}(\omega_k) + \varphi_{bk}], \quad (17)$$

where

$$\begin{aligned} a_{mn} &= \sqrt{4\Delta\omega} |l_{mn}(i\omega_k)|, \\ \theta_{mn} &= \arctan \frac{I[l_{mn}(i\omega_k)]}{\text{Re}[l_{mn}(i\omega_k)]}, \end{aligned} \quad (18)$$

where a_{mb} and θ_{mb} indicates the magnitude and phase angle of the k th frequency component at points m and b . φ_{bk} is the random phase angle which follows the uniform distribution over the interval $(0, 2\pi)$. $l_{mn}(i\omega)$ represents the element of the P matrix in any direction. I and Re represent the imaginary part and the real part, respectively. The nonstationary ground motion is mainly characterized by nonstationary amplitude and frequency. Specially, the nonstationary amplitude is simulated by a smooth process ground motion multiplied by a deterministic intensity envelope function, so nonstationary ground motion in any direction at any point can be expressed as

$$a(t) = f(t) \sum_{j=1}^3 d_j(t) u_j(t), \quad (19)$$

where $u_j(t)$ represents the stationary process of the j time period with different frequency components in different time periods; $d_j(t)$ represents a rectangular function with a value of 1 in the j time period and 0 in the other two time periods; and $f(t)$ represents an intensity envelope function and describes the nonstationarity of the amplitude, which can be expressed as

$$f(t) = \begin{cases} (t/t_0)^2, & 0 \leq t \leq t_0, \\ 1, & t_0 < t \leq t_n, \\ \exp[-0.155(t - t_n)], & t_n < t \leq T. \end{cases} \quad (20)$$

In this paper, $t_0 = 5$ s and $t_n = 13$ s. Because the actual vibration is a process varying from weak to strong, then from strong to weak, the trapezoidal window function in equation (20), in which T_1 , T_2 , and T_3 are the preshock period, strong earthquake period, and the post-weak period, respectively. Taking 1/10 of the strong earthquake period as the transition time as shown in Figure 1 (in this example, the values of T_1 , T_2 , and T_3 are 5 s, 8 s, and 7 s, respectively).

2.4. Multidimensional and Multisupport In Situ Ground Motion Synthesis. The basic theory about multidimensional and multisupport nonstationary ground motion synthesis is given in Sections 2.1–2.3. Figure 2 shows the detailed soil parameters at the local site of the cable-stayed bridge, in which 1 and 1°, respectively, indicate the bottom position of the left tower and the position corresponding to the bedrock and 2 and 2° represent the bottom position of the right tower and the position corresponding to the bedrock. Among the soil layer parameters, h indicates the thickness of the soil

FIGURE 1: Trapezoidal window connection diagram.

FIGURE 2: Parameters of each soil layer at the bridge site.

layer, G indicates the shear modulus of the soil layer, ρ indicates the corresponding soil layer density, and ξ and ν are the damping ratio and Poisson's ratio, respectively.

In this paper, the peak ground acceleration of the bedrock is 2.94 m/s^2 (0.3 g), the corresponding bedrock acceleration (white noise) self-spectral density Γ is $0.0115 \text{ m}^2/\text{s}^4$, and the ground motion duration T is 40.96 s . After the parameters of the soil layer at the bridge site are determined, according to the multidimensional and multisupport nonstationary ground motion synthesis theory and considering the randomness of phase angle, three longitudinal, horizontal, and vertical artificial ground motions are synthesized, as shown in Figures 3–5, respectively.

It can be seen from Figure 3 that the peak acceleration ground motions at 1# site can reach 5.21 m/s^2 and that at 2# site can even reach 7.98 m/s^2 , which can be seen on the surface of the soil layer. The peak acceleration at ground can reach 0.81 g which is much larger than that at the bedrock, indicating the amplification effect of ground soil being evident.

3. Nonlinear Finite Element Simulation of the Cable-Stayed Bridge

A large railway cable-stayed bridge is located at the Wenzhou, China, with a span combination of $(51 + 91 + 300 + 91 + 51) \text{ m}$. The ratio of the side span to middle span is 0.473 , and the total length of the bridge is 584 m . The girder is a prestressed concrete box beam with a single box and two rooms. The tower above the bridge deck has an inverted Y shape, and the column below the bridge deck has a diamond shape. The height of the tower is 118 m ,

with the height of the bridge above the deck being 75 m . The longitudinal width of the tower is extended from 6.5 m at the top of the tower to 8 m at the bottom of the tower. The cable has the standard galvanized parallel wire having a tensile strength of 1860 MPa . The layout of main bridge is shown in Figure 6.

The reinforcement arrangement of the pier is as follows: the main reinforcement bar is HRB400 steel with the nominal diameter of 16 mm and the basic spacing of 150 mm ; the pier stirrup is also HRB335 steel with the nominal diameter of 12 mm , the spacing of the hoop of 100 mm in the 6 m range at bottom of the pier and the rest being 150 mm ; and the concrete cover depth of the pier is 45 mm . To the towers, the main reinforced bar adopts the HRB400 steel bar with a diameter of the steel bar of 32 mm and the basic distance of 200 mm . The stirrup of the tower uses HRB335, with the nominal diameter of the steel bar of 20 mm , the spacing of the stirrup at the bottom of 100 mm , and the rest of 125 mm . The concrete cover depth of the tower is 45 mm , which is the same as that of the pier.

In order to obtain the nonlinear seismic response of the cable-stayed bridge, a 3-D nonlinear finite element model of cable-stayed bridge is established based on OpenSees platform.

3.1. Pier and Tower. As the main vulnerable members of the cable-stayed bridge under earthquake, the plastic hinge at the bottom section of the piers and towers is relatively easy to be developed, which are simulated by the nonlinear beam-column element. The fiber section of the piers and towers can be divided into three parts, including the confined

FIGURE 3: Longitudinal artificial ground motions at two points. (a) 1# site. (b) 2# site.

FIGURE 4: Transverse artificial ground motions at two points. (a) 1# site. (b) 2# site.

concrete, unconfined concrete, and reinforced bar section, with different materials.

3.1.1. Concrete Material Model. The Concrete02 material model is based on the uniaxial constitutive relationship of plain concrete defined by Kent-Scott-Park [28]; the model changes the peak strain value of the concrete compression skeleton curve. The concrete tension capacity takes into account the tensile hardening of the material and the stiffness degradation effect after the initial cracking with the increase of the maximum tensile strain. The hysteresis curve of the material is determined according to the hysteresis rule

defined by Karsan-Jirsa [34]. The stress-strain relationship and hysteresis curve of the concrete material model are shown in Figure 7.

In the Concrete02 material model, the ascending section is described by a quadratic polynomial, the descending section is described by a straight line, and finally a residual stress is retained. The model can be expressed as

$$\begin{aligned} \sigma_c = & \left\{ K f'_c \left[2 \left(\xi_c / \xi_0 \right) - \left(\xi_c / \xi_0 \right)^2 \right], \xi_c \right. \\ & \leq \xi_0, K f'_c \left[1 - Z \left(\xi_c - \xi_0 \right) \right], \xi_0 < \xi_c \leq \xi_u, 0.20 f'_c, \xi_c > \xi_u, \end{aligned} \quad (21)$$

FIGURE 5: Vertical artificial ground motions at two points. (a) 1# site. (b) 2# site.

FIGURE 6: Layout of the cable-stayed bridge (unit/m).

FIGURE 7: Concrete material model. (a) Stress-strain relationship. (b) Hysteresis curve.

where

$$\begin{aligned}\xi_0 &= 0.002K, \\ K &= 1 + \frac{\rho_{sv} f_{yv}}{f'_c}, \\ Z &= \frac{0.5}{3 + 0.29 f'_c / (145 f'_c - 1000) + 0.75 \rho_s \sqrt{h'} / (s_h - 0.002K)},\end{aligned}\tag{22}$$

where K represents the amplification factor for the hoop effect, f'_c represents the maximum compressive stress of the concrete, ξ_0 is the strain corresponding to the maximum compressive stress of the concrete, and Z is the strain softening slope, which ξ_u is the ultimate strain of the concrete. ξ_u is the volume ratio of the stirrup, and f_{yv} is the yield strength of the stirrup, h' is the height of the concrete core, and s_h is the pitch of the stirrup. The ascending and descending sections of the concrete material are straight, and the maximum tensile stress f'_t and softening stiffness E_{ts} of the concrete are

$$f'_t = -0.07K f'_c, E_{ts} = \frac{f'_t}{0.002}.\tag{23}$$

3.1.2. Reinforcement Material Model. The reinforcement material is defined by the Steel02 material in the OpenSees program. The Steel02 material uses the modified Giuffre-Menegotto-Pinto steel hysteresis constitutive model [31, 32], which can consider two-way Bauschinger effect and the isotropic strengthening effect in the structures. It is widely used in nonlinear simulation of materials, and the model can be expressed as

$$\sigma^* = b\xi^* + \frac{(1-b)\xi^*}{(1 + \xi^{*R})^{1/R}},\tag{24}$$

where

$$\begin{aligned}\xi^* &= \frac{\xi - \xi_r}{\xi_0 - \xi_r}, \\ \sigma^* &= \frac{\sigma - \sigma_r}{\sigma_0 - \sigma_r},\end{aligned}\tag{25}$$

where b is the strain hardening coefficient, R is the parameter considering the Bauschinger effect, σ_0 and ξ_0 are the stress and strain corresponding to the intersection of the asymptote and the postyield asymptote at the initial time; and σ_r and ξ_r are the stress and strain at the loading point of the reaction. The stress-strain relationship and hysteresis curve of the reinforced bar material model are shown in Figure 7.

3.2. Bearing Model. The bearing is a critical component to connect the superstructure and the substructure. Under normal conditions, the constant load of the superstructure is effectively transmitted to the substructure. Under earthquake loading, the bearing transmits the inertial force of the substructure. As the first defense line for earthquakes, the bearing plays an essential role on the safety of the bridges. Therefore, it is very important to accurately simulate the mechanical characteristics of the bearing in the seismic response analysis of the bridge structure. The GPZ series of pot-type rubber bearings are used in this bridge structure, which has a capacity ranging from 1000 kN to 50000 kN. GPZ series pot-type rubber bearings have low friction coefficient, large bearing capacity, and strong corrosion resistance, which can not only greatly improve the capacity of

FIGURE 8: Bilinear ideal elastic model.

FIGURE 9: Stress-strain relationship curve of the hardening material.

TABLE 1: Summary of materials and elements used in OpenSees.

Component	Materials	Element
Girder	Concrete02	Elastic beam column
Pier	Concrete02 Steel02	Nonlinear beam column
Tower	Concrete02 Steel02	Nonlinear beam column
Cable	Elastic Initial strain material MinMax material	Truss element
Bearing	Hardening	Two-node link

the bearing and the life of the rubber but also ensure that the bearing has flexible rotation performance and good cushioning performance.

The movable pot-type rubber bearing can be simulated by a bilinear ideal elastic spring element, and its resilience model is shown in Figure 8. Critical sliding friction force $F_{\max}$ (KN) of movable pot rubber bearing can be expressed as

$$F_{\max} = \mu_d R. \quad (26)$$

TABLE 2: Main parameters used in the finite element model.

Component	Variable	Description	Value
Pier	Ec1	Young's modulus of concrete	3.25E10 Pa
	Fc1	Compressive strength of concrete	2.86E7 Pa
	Fy	Yield strength of steel	3.35E8 Pa
Girder	Ec2	Young's modulus of concrete	3.55E10 Pa
	Fc2	Compressive strength of concrete	3.93E7 Pa
Tower	Ec3	Young's modulus of concrete	3.25E10 Pa
	Fc3	Compressive strength of concrete	2.86E7 Pa
	Fy2	Yield strength of steel	4.00E8 Pa
Cable	Fy4	Extreme strength of cable	1.86E9 Pa
	Ec4	Young's modulus of cable	1.95E11 Pa
Bearing	m	Friction factor	0.02

TABLE 3: Comparison of the dynamic characteristics of the first ten modes of cable-stayed bridge.

Model	Period (s)	Finite element software		
		OpenSees	ANSYS	MIDAS
Model 1	T_1	5.8493	5.9613	5.5677
Model 2	T_2	2.8453	2.6304	2.5913
Model 3	T_3	2.3457	2.3640	2.2162
Model 4	T_4	2.1144	2.0483	1.9415
Model 5	T_5	1.7406	1.6539	1.7550
Model 6	T_6	1.3870	1.2126	1.3980
Model 7	T_7	0.9897	0.9463	1.0605
Model 8	T_8	0.9087	0.8952	0.9351
Model 9	T_9	0.7758	0.7699	0.7790
Model10	T_{10}	0.6817	0.6984	0.6956

The initial stiffness is

$$k = \frac{F_{\max}}{x_y}, \quad (27)$$

where μ_d is the sliding friction coefficient, generally taken as 0.02; R is the upper structure gravity (kN) assumed on the support; and x_y is the movable pot-type bearing yield displacement, generally taken as 0.002–0.005 m.

The hardening material model in the OpenSees is used to simulate the bilinear ideal elastic shape. The stress-strain relationship curve of the hardening material is presented in Figure 9.

3.3. Girder. Under the earthquake loading, the girder is normally in the elastic state and is simulated by elastic beam column elements. The mechanical parameters of the elastic beam column element mainly include the sectional area, the moment of inertia, and the elastic modulus. These parameters are calculated according to the material properties and the section size of the structure. And the second-stage load such as bridge decking is considered when simulating the superstructure.

3.4. Cables. In this section, two material models are used for the simulation of the mechanical properties of the cables.

FIGURE 10: Displacement time history of bearing under 1# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 11: Displacement time history of bearing under 2# ground motion. (a) Longitudinal. (b) Transverse.

The initial strain material model is used to model the initial strain of the cables. The MinMax material model is used to achieve the stress threshold of the cable, after which the cable element will automatically be withdrawn from the work state.

According to the aforementioned detailed descriptions of the elements and materials for piers, towers, bearings, girders, and cables, the materials and elements used in the simulation are listed in Table 1. The values of the main parameters used in the finite elements are presented in Table 2.

It should be noted that (1) the influence of pile-soil interaction on its seismic response is not considered in the

model, and the bottom positions of the towers and piers are assumed being fixed; (2) the seismic response analysis mainly considered the nonlinearity of the structural material, while the geometric nonlinearity of the cable-stayed bridge is not considered for brevity.

3.5. Dynamics Verification of the OpenSees Model. In order to verify the correctness and rationality of the OpenSees dynamic analysis model, the finite element models of the cable-stayed bridge was also established in ANSYS and MIDAS CIVIL platforms. Table 3 shows dynamic characteristics of the first ten modes of the bridge under different finite

FIGURE 12: Displacement time history of bearing under 3# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 13: Hysteresis curve of the bottom section of 1# pier under 1# ground motion. (a) Longitudinal. (b) Transverse.

element models. It can be seen that the free vibration periods of the cable-stayed bridge obtained by OpenSees software are close to that derived from ANSYS and MIDAS CIVIL. The maximum relative error is less than 10%, which indicates that the finite model based on the established OpenSees platform is reasonable.

4. Nonlinear Seismic Response Analysis

The nonlinear seismic response analyses are conducted in this section based on the finite element model of the established cable stayed bridge. The seismic responses of the

bearings, piers, towers, and cables are given below under three ground motions, and the response results are analyzed, respectively.

4.1. Bearing Response. The longitudinal and transverse displacement responses of each bearing under 1# ground motion are presented in Figures 10(a) and 10(b), respectively. It should be noted that the bearing number starts from 1# at the left pier and is arranged to the right in turn and double bearings are set at the tower section. It can be seen from Figure 10 that the displacement of each bearing on the same tower is almost the same in longitudinal and transverse

FIGURE 14: Hysteresis curve of the bottom section of 2# pier under 1# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 15: Hysteresis curve of the bottom section of 3# pier under 1# ground motion. (a) Longitudinal. (b) Transverse.

directions, so only one of double bearings is represented in the next study.

Specifically, it can be seen from Figure 10(a) that the longitudinal displacement responses at each bearing is basically consistent, with the peak response being 0.112 m, because the girder is longitudinally drifted under the ground motions and the displacement of each bearing is displaced by the girder and the substructure. As seen from Figure 10(b), it can be concluded that the transverse displacement response of 8# bearing is the largest, with the response value of 0.230 m, and response at 1# bearing being the second largest with the value of 0.123 m. The longitudinal and transverse displacement responses of each bearing under 2# and 3#

ground motions are presented in Figures 11 and 12, respectively, from which it can be seen that the longitudinal displacement responses of each bearing are basically consistent and the transverse displacement responses at the 3# and 5# bearings are smaller than those of the others, as a result of the transverse bending of the girder.

4.2. Pier Response. For brevity, the hysteresis curve of the bottom section of piers under 1# ground motion is presented in Figure 13.

It can be seen from Figures 13–16 that the longitudinal and transverse hysteresis curves of the bottom section of all

FIGURE 16: Hysteresis curve of the bottom section of 4# pier under 1# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 17: Time history of the piers curvature under 1# ground motion. (a) Longitudinal. (b) Transverse.

piers under 1# ground motion are opposite. Specifically, the maximum bending curvature at the bottom of 1# pier is $4.49 \times 10^{-4} \text{ m}^{-1}$, with the corresponding maximum bending moment of $2.22 \times 10^8 \text{ N}\cdot\text{m}$ in the longitudinal direction. However, the maximum bending curvature in the transverse bridge direction is $1.78 \times 10^{-4} \text{ m}^{-1}$, with the corresponding bending moment being $2.48 \times 10^8 \text{ N}\cdot\text{m}$. This is because the bottom of 2# pier is under 1# ground motion. It should be noted that the bending moment curvatures in the longitudinal and transverse directions of the bridge are basically the same as those of the left side pier.

Due to the asymmetry of the ground motion input at the left and right piers and towers, the seismic demand of the

right pier is presented herein. The bending curvature at 3# pier bottom is $2.01 \times 10^{-4} \text{ m}^{-1}$ which is the largest in the longitudinal direction, with the corresponding bending moment of $1.52 \times 10^8 \text{ N}\cdot\text{m}$. The bending curvature in the transverse direction of the bridge is $7.52 \times 10^{-5} \text{ m}^{-1}$ which is the largest, with the corresponding bending moment being $1.58 \times 10^8 \text{ N}\cdot\text{m}$. Similarly, the variation curve of the hysteresis curve at the bottom section of 4# pier is almost the same as that of the 3# pier.

It can be seen from Figures 13–16 that, under 1# ground motion, the energy and residual deformation of the 3# and 4# piers are very small, indicating that the right piers are basically in elastic working state, while the energy dissipation

FIGURE 18: Time history of the piers curvature under 2# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 19: Time history of the piers curvature under 3# ground motion. (a) Longitudinal. (b) Transverse.

of the left pier (1# and 2# piers) is obviously larger than that of the right side piers, which is mainly caused by the asymmetry of the ground motion input. In conclusion, there is no significant energy dissipation in each pier member, indicating that the piers are basically in elastic state with minor plastic deformation.

Figures 17–19 show the time history curves of the longitudinal and transverse curvatures at the bottom section of each pier under the ground motions. The following observations can be made: (1) Under the excitation of 3-D ground motions, the curvature response at 1# and 2# piers (left side) is larger than that of 3# and 4# piers (right side) in both longitudinal and transverse directions. This is due to

the asymmetry of ground motion inputs at the left and right sides, which takes the effects of the site, coherence, and traveling wave into account. (2) Longitudinal curvature response value is larger than that in the lateral direction in terms of the curvature of the bottom section of the same pier. This is mainly due to the cross-sectional shape of the piers and the vertical input considered.

4.3. Tower Response. It can be seen from Figures 20(a) and 20(b) that, under 1# ground motion, the bottom section of the left tower has the maximum curvature of $1.09 \times 10^{-4} \text{ m}^{-1}$ in the longitudinal direction, with the

FIGURE 20: Hysteresis curve of the bottom section of left tower under 1# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 21: Hysteresis curve of the bottom section of right tower under 1# ground motion. (a) Longitudinal. (b) Transverse.

corresponding bending moment being $1.76 \times 10^8 \text{ N}\cdot\text{m}$. The maximum curvature value at the left tower in the transverse direction is $3.98 \times 10^{-4} \text{ m}^{-1}$, with the corresponding bending moment being $2.32 \times 10^8 \text{ N}\cdot\text{m}$. As can be seen from Figures 21(a) and 21(b), the maximum curvature of the bottom section of the right pier is $9.98 \times 10^{-5} \text{ m}^{-1}$ in the longitudinal direction, with the corresponding bending moment being $1.74 \times 10^8 \text{ N}\cdot\text{m}$, while the bending curvature in the transverse direction is $3.69 \times 10^{-4} \text{ m}^{-1}$, with the corresponding bending moment of $2.22 \times 10^8 \text{ N}\cdot\text{m}$.

Figures 22–24 show the curvature time history at the bottom section of the towers under three ground motions. It can be seen from Figures 22(a) and 22(b) that, under 1# ground motion, the curvature response in the longitudinal direction at the bottom section of right tower is larger than that of the left tower. In the transverse direction, the law is exactly opposite. The curvature response at the bottom section of the left tower is larger than that of the right tower. The same conclusion can be observed in Figures 23 and 24. This is due to the asymmetrical ground motions input and the considered effects of the local site, coherence, and traveling wave.

FIGURE 22: Curvature time history of the towers under the 1# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 23: Curvature time history of the towers under the 2# ground motion. (a) Longitudinal. (b) Transverse.

4.4. Cable Response. As a critical component of the cable-stayed bridge, it is necessary to study the axial force response of the cable under earthquake loading. Figure 25 shows the axial force time history of each cable under the 1# ground motion without considering the initial axial force. It can be seen that the axial force time-history curve of each cable is consistent. Furthermore, Figure 26 shows the maximum axial force of each cable under the ground motions considering the initial axial force of the cable. The maximum value is 694980 N, which occurs under the 2# earthquake. The corresponding cable area is 0.00858 m^2 , and the maximum stress of pull-down cable is 81 MPa, which is much

smaller than the ultimate stress value indicating that the cable does not break.

5. Conclusion

To study the nonlinear seismic behavior and seismic resistance of the long-span cable-stayed bridges subjected to ground motions, the multidimensional and multisupported artificial ground motions are synthesized first based on in situ condition of the cable-stayed bridge considering the coherent and traveling wave effects. Then, considering material nonlinearity, a three-dimensional finite element model was

FIGURE 24: Curvature time history of the towers under the 3# ground motion. (a) Longitudinal. (b) Transverse.

FIGURE 25: Axial force time history under 1# ground motion.

FIGURE 26: Axial force maximum value.

established based on the OpenSees platform, and the non-linear seismic response analysis of bridge was carried out under the synthetic artificial ground motions. The critical conclusions and observations are drawn in the following:

- (1) The longitudinal displacement responses at each bearing are basically consistent with each other because the girder is longitudinally drifted under the ground motions. The transverse displacement responses at the 3# and 5# bearings are smaller than that of others', which is due to the transverse bending of the girder. It can be seen that the maximum value of the displacement response of the bearings under three ground motions is greater than 0.10 m whether in the longitudinal or transverse directions, which is much larger than the critical value of the bearings sliding displacement equal to 0.005 m. Therefore, the seismic

performance of the bearings shows nonlinear characteristics, which means the linear elastic simulation of bearings cannot truly reflect the mechanical properties and nonlinear modelling is necessary.

- (2) Under 3D ground motions action, the curvature seismic response of left side piers (1# and 2#) is larger than that of right side piers (3# and 4#), both in the longitudinal and transverse directions. This is due to the asymmetry of ground motion inputs at the left and right sides, which accounted for effects of the local site, coherence, and traveling wave. To the same pier, the longitudinal curvature response is larger than that in the transverse direction at the bottom section. This is mainly due to the cross-sectional shape of the piers

and the vertical input considered. It can be seen from the pier hysteresis curves that each pier shows different degrees of energy consumption, which means the piers enter nonlinear working stage. Generally speaking, the curvature response and energy consumption of the pier shows that the left side is larger than that of the right side and the longitudinal direction is greater than the transverse.

- (3) The curvature response in the longitudinal direction at the bottom section of the right tower is larger than that of the left tower. In the transverse direction, the law is opposite. The curvature response at the bottom section of the left tower is larger than that of the right tower. This is due to the asymmetrical ground motion input which takes into account the effects of the site, coherence, and traveling wave. It can be seen from the hysteresis curve of the bottom section of towers that the residual deformation of the towers after the earthquake is small and the energy consumption of the left and right towers in transverse direction is better than that in the longitudinal direction, which depend on the sectional characteristics of the left and right towers.
- (4) The maximum axial force of all the cables under the ground motions considering the initial axial force is 694980 N, which occurs under the 2# earthquake with the corresponding cable area 0.00858 m^2 , so maximum tensile stress of the cable is 81 MPa, which is very much smaller than the ultimate stress value indicating that the cable does not break. Therefore, for the normal seismic analysis of cable-stayed bridges, the cables are not seismic vulnerable components. Furthermore, in general seismic response studies, the cables can be directly modelled with ideal elastic materials without considering their failure modes, which can improve the calculation efficiency while ensuring the calculation accuracy.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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