Review Article

Seismic Response of Base-Isolated High-Rise Buildings under Fully Nonstationary Excitation

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Stochastic seismic responses of base-isolated high-rise buildings subjected to fully nonstationary earthquake ground motion are computed by combining the pseudoexcitation and the equivalent linearization methods, and the accuracy of results obtained by the pseudoexcitation method is verified by the Monte Carlo method. The superstructure of a base-isolated high-rise building is represented by a finite element model and a shear-type multi-degree of freedom model, respectively. The influence of the model type and the number of the modes of the superstructure participating in the computation of the dynamic responses of the isolated system has been investigated. The results of a 20-storey, 3D-frame with height to width ratio of 4 show that storey drifts and absolute accelerations of the superstructure for such a high-rise building will be substantially underestimated if the shear-type multi-degree of freedom model is employed or the higher modes of the superstructure are neglected; however, this has nearly no influence on the drift of the base slab.

1. Introduction

Seismic isolation is a technology that decouples a building structure from the damaging earthquake motion. It is a simple structural design approach to mitigate or reduce potential earthquake damage. In base-isolated structures, the seismic protection is obtained by shifting the natural period of the structure away from the range of the frequencies for which the maximum amplification effects of the ground motion are expected; thus, the seismic input energy is significantly reduced. At the same time, the reduction of the high deformations attained at the base of the structure is possible, thanks to the energy dissipation caused by the damping and the hysteretic properties of these devices, further improving the reduction of responses of the structures [1]. Since the 1995 Hyogoken-nanbu earthquake, the number of seismic isolated buildings has been increasing remarkably, including residential buildings, nuclear power plants, office buildings, hospitals, and schools [2, 3]. Base isolation is also an attractive retrofitting strategy to improve the seismic performance of existing bridges and monumental historic buildings [1, 3, 4].

It is believed that isolation technology is very effective in improving the seismic performance of low- and medium-rise buildings, but it is not envisaged for high-rise buildings. However, a lot of base-isolated high-rise buildings have been built in recent decades. Sendai MT building is an 18-storey office building with a height of 84.9 m in Sendai city, which was the first base-isolated building with a height exceeding 60 m [5]; Thousand Tower, a 41-storey building with a height of 135 m, was constructed in 2002 [5]; and another super high-rise building in Japan was built in 2006, with a height of 177.4 m and height to width ratio of 5.7, which is the highest base-isolated building in the world so far [6].

A substantial amount of work has been done on base-isolated high-rise buildings. Since isolators are easily damaged by uplift when such high-rise buildings are subjected to major earthquakes, Roussis and Constantinou [7] proposed some new devices to avoid damage caused by uplift.

2. Governing Equations of Motion for the Base-Isolated System

2.1. Governing Equations of Motion for the Superstructure and Base Slab. Considering a structure with \( N \) DOFs, the governing equation of motion for the superstructure subjected to horizontal seismic ground acceleration \( \ddot{u}_g \) in the horizontal \( x \) direction is [1]

\[
M_s \ddot{u}_s + C_s \dot{u}_s + K_s u_s = -M_r \ddot{x}_b + \ddot{u}_g,
\]

where \( M_s \), \( C_s \), and \( K_s \) are the \( N \times N \) mass, damping, and stiffness matrices of the superstructure, respectively; \( \ddot{u}_s \), \( \dot{u}_s \), and \( u_s \) are the acceleration, velocity, and displacement vectors of order \( N \) relative to the base slab; \( x_b \) is the displacement of the base slab relative to the ground displacement \( u_g \); and \( r_i \) is the \( i \)-dimensional influence coefficient vector.

The governing equation of motion of the base slab, with rotation and vertical deflection neglected, can be expressed as [1]

\[
m_b \left( \ddot{x}_b + \ddot{u}_g \right) + c_b \dot{x}_b + k_b x_b + \left( 1 - \alpha_0 \right) k_u z = 0,
\]

where \( \alpha_0 \) denotes the post- to preyielding stiffness ratio; \( z \) is a hysteretic component, which is a function of the time history of \( x_b \) and \( \dot{x}_b \); and \( m_b \), \( c_b \), and \( k_b \) are the mass, supplemental damping, and preyielding stiffness of the base slab, respectively. One has

\[
c_b = 2 \xi_b \sqrt{k_b m_b}, \quad k_d = \alpha_0 k_u; \quad m_i = r_i^T M_r r_i + m_b,
\]

where \( c_b \) is the supplemental damping ratio of the base slab; \( k_d \) is the postyielding stiffness of the base slab; and \( m_i \) is the total mass of the superstructure and base slab.

In (2) \( z \) is related to \( x_b \) and \( \dot{x}_b \) through the following nonlinear differential equation [17]:

\[
\ddot{z} = A \dot{x}_b - \left( y |\dot{x}_b| z |z|^{p-1} + \beta \dot{x}_b |z|^{q} \right).
\]

Note that in (4), \( y \) and \( \beta \) control the shape of the hysteretic loop; \( A \) controls the restoring force amplitude; and \( \eta \) controls the smoothness of the transition from elastic to plastic response. These parameters are related by \( D_y = \sqrt{A/(\beta + \gamma)} \), where \( D_y \) is the yield displacement of the isolators.

2.2. Static Correction Procedure. In properly designed base-isolated systems, the superstructure remains elastic even when subjected to a major earthquake ground motion. Therefore the modal superposition method is used to reduce the number of degrees of freedom of the system, with the first \( n \) \((n \ll N)\) modes participating in the dynamic computation. This improves the computational efficiency but introduces truncation errors because of neglecting the influence of the higher modes. The static correction procedure is employed to take into consideration the contribution of the higher modes. The displacement of the superstructure corresponding to the higher modes is obtained based on the fact that the high frequency modes react essentially in a static manner when excited by low frequencies [18]. Assume that the displacement of the superstructure \( u_s \) consists of two parts, that is, the dynamic part \( u_s^d \) and the static part \( u_s^s \) [19, 20]:

\[
u_s = u_s^d + u_s^s.
\]

The dynamic part of the displacement can be expressed as

\[
u_s^d = \Phi q = \sum_{i=1}^{n} \Phi_i q_i.
\]
where $\Phi_i$ is the mass normalised eigenvector corresponding to the $i$th eigenvalue $\omega_i^2$; frequencies and modes satisfy $K_i \Phi_i = \omega_i^2 M_i \Phi_i$, $(i = 1, 2, \ldots, n)$; $\Phi = [\Phi_1^T \Phi_2^T \ldots \Phi_n^T]$ is the $N \times n$ eigenvector matrix; and $q$ is the $N \times 1$ generalised modal displacement vector.

Premultiplying (1) by $\Phi^T$ gives

$$
\ddot{q} + C_{s} q + K_{s} q = L (\ddot{u}_y + \ddot{x}_b),
$$

where

$$
C_{s} = \Phi^T C_s \Phi,
$$

$$
K_{s} = \Phi^T K_s \Phi = \text{diag} \{ \omega_1^2, \omega_2^2, \ldots, \omega_n^2 \},
$$

$$
L = -\Phi^T M_s r_s,
$$
in which $L$ is the mode participation factor of order $n$.

The flexibility matrix of the superstructure can be expressed as

$$
K_s^{-1} = \sum_{i=1}^{n} \frac{1}{\omega_i^2} \Phi_i \Phi_i^T.
$$

The remaining static displacement corresponding to the higher modes is

$$
\ddot{u} = - \left( K_s^{-1} - \sum_{i=1}^{n} \frac{1}{\omega_i^2} \Phi_i \Phi_i^T \right) M_s \left[ \ddot{u}_y + r_s (\ddot{x}_b + \ddot{u}_y) \right],
$$

$$
= -BM_s \left[ \ddot{u}_y + r_s (\ddot{x}_b + \ddot{u}_y) \right],
$$

where

$$
B = K_s^{-1} - \sum_{i=1}^{n} \frac{1}{\omega_i^2} \Phi_i \Phi_i^T.
$$

2.3. Equivalent Linearization Method (ELM). Using the ELM, (4) can be rewritten as follows when $\eta = 1$ [17]:

$$
\ddot{z} + c_e \dot{x}_b + k_e z = 0
$$

with

$$
c_e = \sqrt{\frac{\sqrt{E}}{\eta \pi}} \gamma \frac{E [\dot{x}_b z] + \beta \sigma_b}{\sigma_z} - A,
$$

$$
k_e = \sqrt{\frac{\sqrt{E}}{\eta \pi}} \gamma \frac{E [\dot{x}_b z]}{\sigma_z},
$$

where $E[\cdot]$ denotes the expectation operator and $\sigma_z$ and $\sigma_{x_b}$ are the standard deviations of $z$ and $x_b$, respectively.

2.4. State Space Method. Equations (2) and (7) can be compacted together into the single equation below

$$
\begin{bmatrix}
\ddot{\vec{x}_b} \\
\ddot{\vec{q}}
\end{bmatrix} + \begin{bmatrix}
C \\
K
\end{bmatrix} \begin{bmatrix}
\dot{\vec{x}_b} \\
\dot{\vec{q}}
\end{bmatrix} + \begin{bmatrix}
(1 - \alpha) k_{s_x} \\
0
\end{bmatrix} z = -\vec{M} \ddot{u}_y,
$$

where

$$
\begin{bmatrix}
\vec{M} \\
\vec{C}
\end{bmatrix} = \begin{bmatrix}
-\vec{L} & -I_n \\
-\vec{L} & -I_n
\end{bmatrix},
$$

$$
\begin{bmatrix}
\vec{K} \\
\vec{C}_s
\end{bmatrix} = \begin{bmatrix}
\alpha_0 k_{s_x} & 0 \\
0 & \vec{K}_s
\end{bmatrix} ,
$$

$$
I = [1, 0, 0, \ldots, 0]^T;
$$

in which $I_n$ is the $n \times n$ identity matrix.

The second-order (14) can be replaced by the first-order differential equations by the state space method

$$
\begin{bmatrix}
\dot{\vec{x}_b} \\
\dot{\vec{q}}
\end{bmatrix} = \begin{bmatrix}
0 & I_{n+1} \\
-\vec{M}^{-1} \vec{K} & -\vec{M}^{-1} \vec{C}
\end{bmatrix} \begin{bmatrix}
\vec{x}_b \\
\vec{q}
\end{bmatrix} + \begin{bmatrix}
0 \\
\vec{a}\end{bmatrix} \vec{z} - \begin{bmatrix}
0 \\
1\end{bmatrix} \ddot{u}_y,
$$

where

$$
\begin{bmatrix}
\vec{M} \\
\vec{C}_s
\end{bmatrix} = \begin{bmatrix}
-\vec{L} & -I_n \\
-\vec{L} & -I_n
\end{bmatrix},
$$

$$
\begin{bmatrix}
\vec{K} \\
\vec{C}_s
\end{bmatrix} = \begin{bmatrix}
\alpha_0 k_{s_x} & 0 \\
0 & \vec{K}_s
\end{bmatrix} ,
$$

$$
I = [1, 0, 0, \ldots, 0]^T.
$$

Thus (12) and (16) can be replaced by the first-order differential equation

$$
\dot{v} = H v + r \ddot{u}_y,
$$

where

$$
H = \begin{bmatrix}
0 & I_{n+1} & 0 \\
-\vec{M}^{-1} \vec{K} & -\vec{M}^{-1} \vec{C} & \vec{a} \\
0 & \vec{b} & -k_{r_z}
\end{bmatrix},
$$

$$
r = - \begin{bmatrix}
0 & 0 & 0
\end{bmatrix}^T.
$$

in which $(n+1)$-dimensional vector $a$ is given by

$$
a = \vec{M}^{-1} [-(1 - \alpha_0) k_{s_x}, 0, 0, \ldots, 0]^T.
$$

3. Nonstationary Stochastic Analysis by PEM

Because of the uncertainty of earthquakes, stochastic seismic analysis is a powerful tool in earthquake engineering and has experienced extensive development in recent decades. Jangid [21] studied the performance of the isolated buildings and bridges, and the stochastic responses of the isolated structures subjected to uniformly modulated nonstationary earthquake excitations were obtained by solving Lyapunov equation. As the stochastic response of a nonlinear system is strongly affected by the nonstationary behaviour of an earthquake [22], the fully nonstationary earthquake model proposed by Conte and Peng [23] is adopted in this paper.
3.1. Fully Nonstationary Earthquake Excitation Model. The earthquake excitation model is considered a sigma-oscillatory process [23], which is a sum of $p$ zero-mean, independent, uniformly modulated Gaussian processes. Each uniformly modulated process consists of the product of a deterministic time modulating function, $A_k(t)$, and a stationary Gaussian process, $Y_k(t)$. Thus, the earthquake ground motion $\ddot{u}_g(t)$ is defined as [23]

$$
\ddot{u}_g(t) = \sum_{k=1}^{p} X_k(t) = \sum_{k=1}^{p} A_k(t) Y_k(t). \tag{22}
$$

In (22), the modulating function $A_k(t)$ is defined as

$$
A_k(t) = \alpha_k (t - \zeta_k)^{\beta_k} e^{-\gamma_k (t - \zeta_k)} H(t - \zeta_k), \tag{23}
$$

where $\alpha_k$ and $\gamma_k$ are positive constants; $\beta_k$ is a positive integer; $\zeta_k$ is the “arrival time” of the $k$th subprocess, $X_k(t)$; and $H(t)$ is a unit step function.

The $k$th stationary Gaussian process, $Y_k(t)$, is characterized by its autocorrelation function

$$
R_{Y_kY_k}(\tau) = e^{-\gamma_k |\tau|} \cos(\eta_k \tau) \tag{24}
$$

and its power spectral density (PSD) function

$$
S_{Y_kY_k}(\omega) = \frac{\gamma_k}{2\pi} \left[ \frac{1}{\gamma_k^2 + (\omega + \eta_k)^2} + \frac{1}{\gamma_k^2 + (\omega - \eta_k)^2} \right] \tag{25}
$$

in which $\gamma_k$ and $\eta_k$ are two free parameters representing the frequency bandwidth and predominant frequency of the process, $Y_k(t)$, respectively.
3.2. PEM Computational Procedure. The procedure of computing stochastic responses of base-isolated systems is summarized below.

Step 1. Constitute the pseudoexcitation at instant \( t_k \) as in \([16]\)

\[
\ddot{u}_g(t_k) = \sqrt{S_{\ddot{u}_g}(\omega, t_k)} \exp(i\omega t_k),
\]

where \( i = \sqrt{-1} \). Substitute \( \ddot{u}_g(\omega, t_k) \) into \((19)\), with \( c_e \) and \( k_e \) in \((13)\) and \((19)\) given an initial value at \( t = 0 \).

Step 2. Compute the pseudoresponse \( \ddot{v}(\omega_j, t_k) \) by Runge-Kutta method.

The corresponding nonstationary random vibration response analysis is transformed into an ordinary direct dynamic analysis. Thus \( \ddot{v}(\omega_j, t_k) \) can be evaluated at a series of equally spaced frequency points \( \omega_j = j\Delta \omega \) \((j = 1, 2, \ldots, N_\omega)\) at \( t_k \) by Runge-Kutta method, where \( \Delta \omega \) is the frequency step and \( N_\omega \) is the total number of frequency steps.

Step 3. Obtain the cross- and auto-PSD of the responses by PEM at \( t_k \) and \( \omega_j \).

\[
\begin{align*}
S_{\ddot{x}_b \ddot{y}_g}(\omega_j, t_k) & = \ddot{v}^*_b(\omega_j, t_k) \ddot{v}(\omega_j, t_k), \\
S_{\ddot{z} \ddot{x}_b}(\omega_j, t_k) & = \ddot{z}^*_b(\omega_j, t_k) \ddot{x}_b(\omega_j, t_k), \\
S_{\ddot{z} \ddot{z}}(\omega_j, t_k) & = \ddot{z}^*_b(\omega_j, t_k) \ddot{z}(\omega_j, t_k), \\
S_{\ddot{x}_b \ddot{x}_b}(\omega_j, t_k) & = \ddot{x}^*_b(\omega_j, t_k) \ddot{x}_b(\omega_j, t_k).
\end{align*}
\]

In \((28)\), \( \dddot{x}_b(\omega_j, t_k) \) and \( \dddot{z}(\omega_j, t_k) \) are the pseudoresponses obtained by Step 3; \( \dddot{x}_b^*(\omega_j, t_k) \) and \( \dddot{z}^*(\omega_j, t_k) \) are the complex conjugate of \( \dddot{x}_b(\omega_j, t_k) \) and \( \dddot{z}(\omega_j, t_k) \), respectively.
Figure 4: Comparison of the RMS storey drifts: (a) base slab, (b) 3rd storey, (c) 13th storey, and (d) 19th storey, evaluated by PEM with those obtained by Monte Carlo simulation.

Step 4. Compute the covariance and variance of the responses by Wiener-Khintchine theorem at \( t_k \).

The quantities \( E[\dot{x}_b z] \), \( \sigma_z \), and \( \sigma_{\dot{x}_b} \) used in (13) are given by

\[
E[\dot{x}_b z] = \int_{-\infty}^{+\infty} S_{\dot{x}_b z}(\omega, t_k) \, d\omega \\
= \Delta \omega \sum_{j=1}^{N_x} \left[ S_{\dot{x}_b x}(\omega_j, t_k) + S_{\dot{x}_b \dot{x}}(\omega_j, t_k) \right],
\]

and

\[
\sigma_z^2 = 2 \int_0^{+\infty} S_{zz}(\omega, t_k) \, d\omega = 2\Delta \omega \sum_{j=1}^{N_x} S_{zz}(\omega_j, t_k),
\]

\[
\sigma_{\dot{x}_b}^2 = 2 \int_0^{+\infty} S_{\dot{x}_b \dot{x}_b}(\omega, t_k) \, d\omega = 2\Delta \omega \sum_{j=1}^{N_x} S_{\dot{x}_b \dot{x}_b}(\omega_j, t_k).
\]

(29)

Step 5. Evaluate \( c_e \) and \( k_e \) by (13).

When the corresponding responses become convergent, \( t_k \) is replaced by \( t_{k+1} \), and Steps 1–5 are repeated for the next time step. Equations (13), (19), (28), and (29) make
up the complete formulation of the isolated system. The computational procedure is shown in Figure 1.

4. Numerical Study

In the base-isolated frame structure shown in Figure 2, each of the 20 storeys is 3.6 m in height, so the total height of the frame structure is 72 m, with 18 m in width and 15 m in depth in the x and z directions, respectively. Thus its height to width ratios are 4 and 4.8 in the x and z directions, respectively. The reinforced concrete beams are all identical, with width of 0.6 m and depth of 0.8 m. The reinforced concrete columns are all of square cross-sections, with side length \( d \). The column properties, that is, their side length \( d \), extensional rigidities \( EA \), and flexural rigidities \( EI \) for the in-plane behavior, are in three different values, depending upon the storey number, as shown in Table 1. The total mass of each storey is distributed uniformly as a lumped mass at each of its nodes, as also shown in Table 1. The damping ratios of the superstructure and isolation slab are 0.03 and 0.10, respectively; the fundamental period of the base-fixed superstructure is \( T_s = 1.66 \) s, and the fundamental period of the base-isolated system is \( T_d = 3.50 \) s. Other values used are the post- to preyielding stiffness ratio \( \alpha_0 = 0.1 \); and the yielding displacement \( D_y = 0.01 \) m.

A versatile, fully nonstationary earthquake ground-motion model proposed by Conte and Peng [23] is employed here, and this stochastic earthquake model is applied to an actual earthquake, N00W (N-S) component of the San Fernando earthquake of February 9, 1971, recorded at the Orion Boulevard site. The corresponding parameters of the sigma-oscillatory process estimated are given in Table 2 [23]. The model parameters are determined by adaptively least-squares fitting the analytical time varying (or evolutionary) PSD function of the proposed model to the evolutionary PSD function estimated from the actual earthquake accelerogram.
The PSD function of the earthquake excitation is shown in Figure 3. Obviously, this earthquake model can capture the time variation of both the intensity and the frequency content of the earthquake record at the target.

Figure 4 compares the root mean square (RMS) storey drifts of the isolated structure evaluated by PEM with those given by Monte Carlo simulation (500 samples are used). The relationship between the restoring force and the drift of the isolators is described by the Bouc-Wen model, so each sample of Monte Carlo simulation is a nonlinear time history analysis of the isolated system. Clearly both the drift of the base slab and the storey drifts of the superstructure agree well with the two sets of results, so that the accuracy of the results by the PEM is demonstrated.

The peak RMS of the storey drifts and absolute accelerations of the base-fixed structure and those of the base-isolated one are shown in Figure 5. It demonstrates that the responses decrease significantly after isolation, so they reveal that the isolation technology can still protect the structures from damage during earthquakes even for high-rise buildings with a proper design of the isolators employed.

The improvement of accuracy of the storey drifts by the static correction procedure is given by Figure 6. It shows that the accuracy of the response is improved with employment of this method.

Figure 7 illustrates the influence of flexure of the superstructure on the peak RMS storey drifts and absolute accelerations of the storeys. The FE model and shear-type MDOF
model are each applied to the superstructure, with the first 20 modes participating in the dynamic computation. It shows that storey drifts and absolute accelerations increase when extensions of the columns and flexure of the beams are suitably taken into account in the FE model.

Figure 8 shows the influence of the number of the modes participating in the computation of the response of the superstructure. The results are obtained with $n = 3, 10, \text{ and } 40$ modes, respectively. They reveal that the storey drifts of the superstructure will be substantially underestimated if only the first few modes are included in the dynamic computation from Figure 8(a), while Figure 8(b) demonstrates that the absolute accelerations vary significantly with the number of modes participating in the dynamic computation. So the influence of the higher modes on the responses of the superstructures should not be neglected for the base-isolated high-rise buildings.

Figure 9 shows the influence of the number of the modes participating in dynamic computation on the (a) drift and (b) absolute acceleration of the base slab. The number of the modes
participating in the dynamic computation is found to have a small influence on the drift of the base slab but a remarkable influence on the absolute acceleration of the base slab.

5. Conclusions

The stochastic responses of a base-isolated high-rise building subjected to fully nonstationary ground excitations are analyzed by combining the PEM and the ELM. The conclusions can be drawn as follows.

1. The results obtained by the PEM agree well with those obtained by the Monte Carlo method and the accuracy of the results of such hysteretic systems evaluated by the PEM is verified.

2. The static correction procedure is employed for considering the contributions of the higher modes of the structure which causes almost no increase of the computational effort.

3. An FE model and a shear-type MDOF model are implemented for the superstructure of such high-rise buildings. It is found that the storey drifts and absolute accelerations are underestimated if the flexural deformation of the beam components and the axial deformations of the column components are neglected in the shear-type MDOF model used. The peak RMS storey drifts of the superstructure could be underestimated by about 60%, and the peak RMS absolute accelerations of the superstructure could be underestimated by about 7%.

4. The storey drifts and the absolute accelerations could be underestimated if the higher modes of the superstructure are neglected in the FE model used; and the number of the modes participating in the dynamic computation has a small influence on the response of the base slab but a remarkable influence on its absolute acceleration, and sometimes the absolute acceleration of base slab could be overestimated by about 40%.

Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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