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A novel control strategy for reproducing the floor motions of high-rise buildings by earthquake-simulating shake tables

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A novel control strategy for reproducing the floor motions of high-rise buildings by earthquake-simulating shake tables

Abstract: To enable the experimental assessment of the seismic performance of full-scale nonstructural elements with multiple engineering parameters (EDPs), a three-layer testbed named Nonstructural Element Simulator on Shake Table (NEST) has been developed. The testbed consists of three consecutive floors of steel structure. The bottom two floors provide a space to accommodate a full-scale room. To fully explore the flexibility of NEST, we propose a novel control strategy to generate the required shake table input time histories for the testbed to track the target floor motions of the buildings of interest with high accuracy. The control strategy contains two parts: an inverse dynamic compensation via simulation of feedback control systems (IDCS) algorithm and an offline iteration procedure based on a refined nonlinear numerical model of the testbed. The key aspects of the control strategy were introduced in this paper. Experimental tests were conducted to simulate the seismic responses of a full-scale office room on the 21st floor of a 42-story high-rise building. The test results show that the proposed control strategy can reproduce the target floor motions of the building of interest with less than 20% errors within the specified frequency range.

Keywords: shake table test, nonstructural element, high-rise building, open-loop IDCS algorithm, off-line iteration.

1. Introduction

The aftermath of recent earthquakes has witnessed damage to both structural and
nonstructural elements in building structures (Fierro et al. 2011, Hisada et al. 2012, Miranda et al. 2012, Rodgers et al. 2021). Despite these observations, a significant knowledge gap exists in the literature regarding the seismic behavior of nonstructural elements in high-rise buildings, highlighting the need for further research. Shake table testing is a feasible approach for simulating the seismic behavior of structural systems and nonstructural elements in high-rise buildings. However, unlike the structural system of high-rise buildings which can be experimentally tested by reduced-scale specimens (e.g., Lu et al. 2016, Chen et al. 2021), it is difficult to make reduced-size nonstructural elements. An alternative is to put the nonstructural elements on the shake table and excite them with floor motions. It then becomes a crucial problem to make the shake table reproduce the target floor motions with high accuracy. Numerous studies have been devoted to improving the acceleration tracking accuracy of shake table systems (e.g., Spencer and Yang 1998, Kuehn et al. 1999, Conte and Trombetti 2000, Nakata 2010, Phillips et al. 2014), while only a few of them specifically focused on the reproduction of floor motions which usually feature long-period response in high-rise buildings (Chen et al. 2020). In addition, the floor motions of high-rise buildings exhibit large lateral drift with respect to the ground that easily exceeds the capacity, such as the stroke and the oil supply, of most existing shake tables that are primarily designed to replicate ground rather than floor motions.

Some new testing facilities were developed to meet the demand for floor motion reproduction. A nonstructural component simulator was developed at the University at Buffalo, namely, UB-NCS (Figure 1(a)), which is composed of a two-level steel frame driven by very slender actuators of a plus/minus 1-meter stroke to simulate the motions of two consecutive...
floors in multi-story buildings (Mosqueda et al. 2009). The large stroke of the actuators compromises the dynamic stability of the simulator, resulting in an operating frequency range of 0.2 Hz to only 5 Hz. In addition, the simulator is not yet capable of bidirectional loading. Shimizu Corporation in Japan completed a novel three-dimensional shake table, E-Spider (Figure 1(b)), which is a Stewart platform driven by six long-stroke electric actuators (Kaneko et al. 2016). Despite the plus/minus 1.5-meter stroke, the small payload of 3 tons of E-Spider is not sufficient for civil infrastructures. Kajima Corporation in Japan developed an advanced three-dimensional two-layer shake table system, W-DECKER (MTS Systems Corporation 2013) (Figure 1(c)). The small table at the upper layer is specifically designed to reproduce large floor motions in high-rise buildings while the large table at the lower layer is intended to generate ground motions. Nonetheless, the upper small table's payload capacity of roughly 5 tons also remains inadequate for civil infrastructure testing purposes. Despite the limited payloads, it is also considered uneconomical to construct such new testing facilities in most of the university laboratories.

Figure 1. Sketch of existing testing facilities for floor motion reproduction: (a) UB-NCS (b) E-spider (c) W-DECKER.

To take full advantage of existing shake tables that are already built in many structural
engineering laboratories, Chen et al. (2020) developed a class of linear and nonlinear control strategies to let a friction pendulum-and-mass system closely track the seismic floor response of the roof acceleration of a 34-story building. Experimental validation highlighted the control performance of the close-loop nonlinear controllers. However, the application is limited to a simple structure loaded by a uniaxial shake table [Figure 2(a)]. Ji et al. (2009) adopted an open-loop IDCS algorithm (Tagawa and Fukui 1994, Tagawa et al. 2011) through model matching and $H_{\infty}$ controller to generate the shake table input for their rubber-mass system with the aim of achieving the large displacement and velocity responses at the roof of a 30-story high-rise building to earthquake excitations. The control strategy was verified through a series of full-scale substructure shake table tests conducted on the E-Defense shake table (Figure 2(b)).

Figure 2. Testing method based on existing shake tables for floor motion reproduction proposed by: (a) Chen et al. (2020) (b) Ji et al. (2009) (c) proposed system.

Many nonstructural elements (e.g., the in-plane and out-of-plane behaviors of partition walls) are sensitive to multiple engineering demand parameters (EDP) (Singhai and Rai 2016, Onat et al. 2018, Xie et al. 2021). To provide various EDPs such as story drift, floor acceleration, and floor velocity for the experimental testing on nonstructural elements, a three-floor testbed named Nonstructural Element Simulator on Shake Table (NEST) has been developed at the
Institute of Engineering Mechanics (IEM). NEST can host a full-scale room space of 2.8 m high and 3.5 m-by-4.5 m in plan and consisted of braced steel frames, friction pendulums (FPs), springs, and natural rubber bearing (NRBs) (Figure 2(c)). Various types of nonstructural elements which are sensitive to different kinds of EDPs can be tested by NEST.

To leverage the flexibility of NEST, this paper proposes a control strategy to generate the required shake table input time histories for the NEST testbed to track the target floor motions of the buildings of interest with high accuracy. In the subsequent sections, we first mathematically define the problem, then introduce the control strategy, and conclude the paper with a trial test on a 42-story high-rise building.

2. Problem definition

Consider an $N$-story archetype building subjected to uniform acceleration excitation $\ddot{u}_g(t)$ at the base, Eq. (1) defines the equation of motion in the absolute coordinate.

$$
[M]{\ddot{y}_{ab}(t)} + [C]{\dot{y}_{ab}(t)} + [K]{y_{ab}(t)} = \begin{bmatrix} 1 \\ 0 \\ \vdots \\ 0 \end{bmatrix} (c_1 \ddot{u}_g(t) + k_1 u_g(t))
$$

where \{1\} is the $N \times 1$ identity matrix, $[M]$, $[C]$, and $[K]$ are the mass, damping, and stiffness matrix of the archetype building, respectively, $k_1$ and $c_1$ are stiffness and damping coefficients at the first floor of the archetype building, \begin{bmatrix} 1 \\ 0 \end{bmatrix} is the location vector with $N$ rows and 1 column, and \{y_{ab}(t)\}, \{\dot{y}_{ab}(t)\}, \{\ddot{y}_{ab}(t)\} are the absolute displacement, velocity, and acceleration response of the archetype building, respectively.

\{y_{ab}(t)\} can be represented as a linear combination of the absolute modal responses:
\[ \{ y_{ab}(t) \} = [\phi_p] \{ p(t) \} \]  

(2)

where \([\phi_p]\) and \(\{ p(t) \}\) are the time-invariant mass-normalized mode shape and the time-varying function of the modal absolute response. By substituting Eq. (2) into Eq. (1), one gets:

\[
\{ \ddot{y}(t) \} + [\ddot{C}] \{ \dot{y}(t) \} + [\ddot{K}] \{ y(t) \} = [\phi_p]^T \begin{bmatrix} 1 \\ 0 \end{bmatrix} (c_1 \ddot{u}_g(t) + k_1 u_g(t))
\]  

(3)

where \([\ddot{C}]\) and \([\ddot{K}]\) are the Rayleigh damping matrix and stiffness matrix of the archetype building in the modal coordinates, \(a_0\) and \(a_1\) are the parameters of Rayleigh damping determined by the first and third modal frequencies of the archetype building, \(\omega_i\) is the \(i^{th}\) circular frequency of the archetype building.

On the other hand, Eq. (6) gives the equation of motion of the 3DOF testbed of NEST in the absolute coordinates.

\[
[M_s] \{ \dddot{x}_{ab}(t) \} + [C_s] \{ \ddot{x}_{ab}(t) \} + [K_s] \{ x_{ab}(t) \} = \begin{bmatrix} 1 \\ 0 \end{bmatrix} (c_{1s} \ddot{u}_s(t) + k_{1s} u_s(t))
\]  

(6)

where \([M_s]\), \([C_s]\), and \([K_s]\) are the mass, damping, and stiffness matrix, respectively, \(k_{1s}\) and \(c_{1s}\) are stiffness and damping coefficients at the first floor of the 3DOF system, \(\ddot{u}_s(t)\) is the acceleration input at the base of the testbed, \(\{ x_{ab}(t) \}\), \(\{ \ddot{x}_{ab}(t) \}\), \(\{ \dddot{x}_{ab}(t) \}\) are the absolute displacement, velocity, and acceleration response of the testbed to \(\dddot{u}_s(t)\).

This paper aims to propose a control strategy to minimize the discrepancy of the first two rows of the testbed response \(\{ x_{ab}(t) \}\), \(\{ \ddot{x}_{ab}(t) \}\), \(\{ \dddot{x}_{ab}(t) \}\) from the first two rows of the
respective target response \( \{y_{abT}(t)\}, \{\dot{y}_{abT}(t)\}, \{\ddot{y}_{abT}(t)\} \). The discrepancy is evaluated by the relative error in the Fourier amplitude spectra of the testbed and target responses. The error \( E_t \) is defined for each response quantity in Eq. (7).

\[
E_t = \frac{\sum_{f} |\mathcal{F}(Y(f))| - |\mathcal{F}(X(f))|}{\sum_{f} |\mathcal{F}(Y(f))|}^2
\]

where \( Y(t) \) is the target response, \( X(t) \) is the response of the testbed, \( \mathcal{F}(Y(t)) \) and \( \mathcal{F}(X(t)) \) represent the Fourier amplitude spectrum of the target response and that of the testbed response.

3. Control strategy

The proposed control strategy is illustrated in Figure 3. It consists of two parts. Firstly, an LQR (linear quadratic regulator) controller-based-IDCS algorithm is utilized to generate a specific shake table input that enables the testbed to trace the target floor motions of the archetype building during specific ground motions. Secondly, an offline iteration procedure based on the nonlinear FEM of the testbed is employed to address the nonlinearity primarily introduced by the friction pendulums.
Figure 3. Concept of the proposed control strategy.

### 3.1 LQR control-based IDCS algorithm

The displacement error between the target responses of the archetype building and the responses of the testbed, \( \{e\} = y_{abT}(t) - x_{ab}(t) \), is taken as the controlled variable in the first-step control, where \( \{y_{abT}(t)\} \) is the absolute displacement time history vector at the target \( n^{th} \), \((n+1)^{th}\), and \((n+2)^{th}\) floor of the archetype building. By defining an augmented state vector, \( \{Z\} = \{e; y_{ab}(t); \dot{e}; \dot{y}_{ab}(t)\} \), Eq. (8) gives the state equation of the archetype-testbed system.

\[
\{\dot{Z}\} = [A]\{Z\} + [B]U_s(t) + [D]U_g(t) \tag{8}
\]

\[
\{Z\} = \begin{pmatrix} e \\ y_{ab}(t) \\ \dot{e} \\ \dot{y}_{ab}(t) \end{pmatrix}, \quad \begin{pmatrix} [B_F]y_{ab}(t) - x_{ab}(t) \\ y_{ab}(t) \\ [B_F]\dot{y}_{ab}(t) - \dot{x}_{ab}(t) \end{pmatrix} = \begin{pmatrix} [B_F][\phi_p]p(t) - x_{ab}(t) \\ [\phi_p]p(t) \\ [B_F][\phi_p]\dot{p}(t) - \dot{x}_{ab}(t) \end{pmatrix} \tag{9}
\]

\[
[A] = \begin{bmatrix}
0_{3\times3} & 0_{3\times N} & I_{3\times3} & 0_{3\times N} \\
0_{N\times3} & 0_{N\times N} & 0_{N\times3} & I_{N\times N} \\
0_{N\times3} & -[AA_p] & 0_{N\times3} & -[CC_p]
\end{bmatrix} \tag{10}
\]

\[
\{B_F\} = \{0_{3\times(n-1)} \  I_{3\times3} \  0_{3\times(N-n-2)}\}, \quad \{B\} = \begin{bmatrix} 0_{3\times1} \\ 0_{N\times1} \\ 1 \\ 0 \\ 0 \end{bmatrix}, \quad \{D\} = \begin{bmatrix} 0_{3\times1} \\ 0_{N\times1} \\ 0_{3\times1} \end{bmatrix} \tag{11}
\]

\[
U_s(t) = (c_{15}u_s(t) + k_{15}u_s(t))/m_{15}, \quad U_g(t) = (c_1u_g(t) + k_1u_g(t))/m_1 \tag{12}
\]

\[
[AA_s] = [M_s]^{-1}[K_s], \quad [CC_s] = [M_s]^{-1}[C_s] \tag{13}
\]

\[
[AA_p] = [B_F][\phi_p][\bar{K}], \quad [CC_p] = [B_F][\phi_p][\bar{C}] \tag{14}
\]
where \( \{B_F\} \) is the location vector of the target floors in the archetype building, \( m_1 \) is the mass of the first floor of the archetype building, \( m_{1s} \) the mass of the first floor of the testbed, \( U_s(t) \) is the shake table control force at the base of the testbed, \( \{B\} \) and \( \{D\} \) are the control force and the seismic force location vectors, respectively.

In the LQR algorithm, the \( \{e\} \) and \( \{\dot{e}\} \) in the state variable \( \{Z\} \) shown in Eq. (9) were used as the feedback signal to calculate the control force. The commonly used performance function in Eq. (15) (Kurata et al. 1999, Nagashima et al. 2001, Li and Ou 2006, Shi et al. 2013) is adopted.

\[
J = \int_0^\infty [(Z)^TQ(Z) + U_s(t)^T R U_s(t)] \, dt
\]

where \( R=1 \) is a positive weighting value for the control force, \( Q \) is a \( 2(N+3) \times 2(N+3) \) weighting matrix for the structural responses. Since we only need to minimize the first two rows of \( \{e\} \) and \( \{\dot{e}\} \), the weighting matrix \( Q \) can be written as Eq. (16).

\[
Q = \text{diag} \left[ \begin{array}{c} 2 \\ 2 \\ 0_{(N+1) \times 1} \\ 2 \\ 2 \\ 0_{(N+1) \times 1} \end{array} \right]
\]

The control force \( U_s(t) \) is calculated by Eq. (17) where \( G \) is the feedback gain. The shake table input \( u_s(t) \) can then be obtained by solving the first-order differential equation in Eq. (18).

\[
U_s(t) = -G\{Z\}
\]

\[
U_s(t) = c_{1s}\ddot{u}_s(t) + k_{1s}u_s(t)
\]

### 3.2 Off-line iteration procedure based on the nonlinear numerical model of the testbed

In the NEST testbed, friction pendulums were used on the 1\textsuperscript{st} and 3\textsuperscript{rd} floors to ensure the
required large stroke with finite lateral stiffness (see Figure 2(c)). They introduce nonlinearity to the testbed and have a non-negligible detrimental effect on the reproduction accuracy. The dynamic responses of the testbed are better simulated by a nonlinear FEM than the linear model in the first-step control. To solve this problem, an outer-loop offline iteration procedure was proposed based on the nonlinear FEM of the testbed to update the linearized state matrix of the testbed in the LQR-IDCS algorithm, which constitutes the second part of the control strategy in Figure 3. The procedure can be described by the following steps: (1) Generate the table motion $\ddot{u}_s(t)$ according to the linear 3DOF model of the testbed with the initially assigned damping ratio, $h=1\%$. (2) Input the table motion $\ddot{u}_s(t)$ to the nonlinear FEM of the testbed and get its simulated responses. (3) Obtain the Fourier spectra errors $E_\ell$ between the nonlinear FEM’s simulated responses and the archetype building responses within the concerned frequency range. (4) Apply the system identification procedure to identify the equivalent linearized modal parameters of the testbed by the ARX method (Pakzad and Fenves 2009, Ji et al. 2011) from the nonlinear FEM responses and update the state matrix if the errors, $E_\ell$, are larger than a prescribed tolerance $\varepsilon$.

By updating the state matrix in the LQR-IDCS algorithm, a new table motion $\ddot{u}_s(t)$ is generated. The iterative procedure proceeds until the error $E_\ell$ is lower than $\varepsilon$. Since the nonlinearity caused by the Coulomb friction of the FPs has a non-negligible effect only on the equivalent damping of the linearized 3DOF model for the LQR-IDCS control, we only updated the equivalent damping ratio of the first and second modes of the linear 3DOF model during the iterative procedure.

The error $E_\ell$ in Eq. (7) is evaluated in the concerned frequency range $[f_L, f_U]$ of a specific
response quantity. In this study, $f_L = 0.1$ Hz for all the response quantities, while $f_U$ is determined by Eq. (19) for the acceleration and velocity responses, and by Eq. (20) for the inter-story drift where $T_i$ is the $i^{th}$ modal period of the testbed.

$$f_{UAV} = \frac{2}{(T_2 + T_3)}$$  \hspace{1cm} (19)  

$$f_{UD} = \frac{2}{(T_1 + T_2)}$$  \hspace{1cm} (20)

4. Implementation to a high-rise building

4.1 Archetype building and ground motion

We take an archetype building of a 42-story reinforced concrete (RC) frame-core wall structure (Lu et al 2015) to illustrate the strategy. It is located on a site of Soil Type 2 in a region of the high seismic intensity of 8.5 according to the Chinese seismic design code for buildings (GB50011-2010, 2010). It corresponds to a 0.3g peak ground acceleration ($PGA$) for design basis earthquake (DBE).

The archetype building is modelled by a planar discrete shear-flexural spring model proposed by Xiong et al. (2016) to simulate the shear and flexural coupling effect of the frame-core wall dual system (Figure 4). The vibrational periods of the first two modes of the planar model, $T_1$ and $T_2$, are derived from a refined 3-dimensional member-by-member model (Lu et al. 2015). The ratio $T_1/T_2$ enables the solution of an intermediate parameter $\alpha_0$ and in turn the flexural and shear stiffness $EI$ and $GA$ by the equations proposed by Miranda and Taghavi (2005). The $\alpha_0$ represents the flexural-shear stiffness ratio and takes 9.7 in this specific case.
Figure 4. Archetype building model and lateral force distribution (a) flexural-shear spring model (b) first mode shape distribution (c) triangular distribution (d) Ai distribution.

The vibration periods and modal effective mass ratios of the discrete shear-flexural spring model are presented in Table 1, where $T_i$ and $M_i$ are the period and effective mass of the $i^{th}$ mode, respectively, and $\sum m$ is the total mass. The modal shapes are depicted in Figure 5. The $P-\Delta$ effect was not considered in this study.

Table 1 Periods and modal effective mass ratios of the archetype building.

<table>
<thead>
<tr>
<th>No. mode $i$</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_i$ (s)</td>
<td>2.695</td>
<td>0.851</td>
<td>0.466</td>
<td>0.297</td>
<td>0.205</td>
</tr>
<tr>
<td>$M_i/\sum m$</td>
<td>0.763</td>
<td>0.095</td>
<td>0.040</td>
<td>0.023</td>
<td>0.015</td>
</tr>
</tbody>
</table>

Figure 5. Mode shapes of the archetype building.

For the example implementation for the high-rise building, we take the strong ground motion recorded at the MZQ station during the 2008 M8.0 Wenchuan earthquake in China as the input. The record is featured by the pulse-like characteristics and rich long-period contents (Qu and Shi 2016). The motion of both directions of MZQ are scaled proportionally to let the
$PGA$ in the EW direction equal $5.1 \text{m/s}^2$. The time history and acceleration spectra of the record are presented in Figure 6. The building's $x$-direction was assumed to align with the EW component.

Figure 6. MZQ motion: (a) time history, (b) acceleration spectra (5% damping ratio).

### 4.2 Necessary structural parameters of the archetype building

The flexural-shear spring model in Figure 4(a) does not provide direct access to the lateral stiffness of the first story of the archetype building, which is a necessary ingredient in Eq. (8) for the LQR control. A pushover analysis is performed on the archetype building's numerical model in OpenSees (McKenna 2011) to obtain the stiffness $k_1$ of the first story. The lateral force distribution is applied to the archetype building through the first mode shape distribution, inverted triangle distribution, and the Ai distribution (BCJ 1997) to account for higher mode effects (Figure 4). As can be seen in Table 2, the $k_1$ values corresponding to the different lateral force distributions are very close to each other. Therefore, their average value is taken as $k_1$ in this study. The corresponding damping coefficient factor $c_1$ of the first story is then obtained by Eq. (21), where $h_0$ is the damping ratio of the archetype building, $\omega_i$ is the $i$th circular frequency of the archetype building.

<table>
<thead>
<tr>
<th>First mode shape</th>
<th>Triangular</th>
<th>Ai distribution</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2. $k_1$ obtained from the pushover procedure.
<table>
<thead>
<tr>
<th>$k_1$</th>
<th>$2.56 \times 10^4$ kN/mm</th>
<th>$2.46 \times 10^4$ kN/mm</th>
<th>$2.47 \times 10^4$ kN/mm</th>
<th>$2.50 \times 10^4$ kN/mm</th>
</tr>
</thead>
</table>

In this study, the first-step control is performed in MATLAB. And the dynamic response of the archetype building can be obtained by solving Eq. (8). The solution is compared with the results of the time history analysis in OpenSees (McKenna 2011) in Figure 7. The identical results justify the accuracy of the proposed pushover-based method for obtaining $k_1$ and $c_1$.

![Comparison of time history analysis results between OpenSees and MATLAB.](image)

**4.3 Structural properties of the testbed**

The floor masses and story stiffnesses of the NEST testbed were determined to be identical in both x- and y-directions and are listed in Table 3. Friction pendulums were installed on the 1\textsuperscript{st} and 3\textsuperscript{rd} floors with their tangent stiffness serving as the story stiffnesses. A pretest on the friction pendulums (FPs) was conducted to determine their friction coefficient, which was
found to be 2.3%. Natural rubber bearings were installed on the 2nd floor. To simulate the testbed's nonlinear behavior, a 3-dimensional member-by-member nonlinear FEM was developed in OpenSees (McKenna 2011). The friction pendulums were modelled by the `singleFPBearing` element. The natural rubber bearings were modeled by the `twoNodeLink` element. And the beams and columns of the steel frame were modeled by the `elasticBeamColumn` element.

Table 3. Structural properties of the testbed.

<table>
<thead>
<tr>
<th>Floor mass (ton)</th>
<th>Floor stiffness (kN/mm)</th>
<th>Maximum story drift (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1F</td>
<td>4.5</td>
<td>0.118 ± 600</td>
</tr>
<tr>
<td>2F</td>
<td>7.41</td>
<td>1.995 ± 64</td>
</tr>
<tr>
<td>3F</td>
<td>7.41</td>
<td>0.231 ± 190</td>
</tr>
</tbody>
</table>

For the LQR control, a 3DOF linear lumped mass model was also established for the testbed. The first and second natural periods of vibration in the same direction are computed to be 2.69 s and 0.85 s, respectively. The transfer functions, depicting the transfer ratios between the ground motion acceleration and the absolute acceleration, velocity at the first floor, and inter-story drift between the first and second floors of the testbed, are illustrated in Figure 8. The major frequency response of the absolute acceleration and the velocity response of the testbed falls within the frequency range from 0.1 Hz to $f_{U,AV}$ (Eq. (19)), and the major frequency response of the inter-story response of the testbed falls within the frequency range from 0.1 Hz to $f_{U,D}$ (Eq. (20)), where $T_i$ is the $i^{th}$ modal period of the testbed.
Figure 8. Transfer ratios between the ground motion acceleration and (a) the absolute acceleration, (b) velocity at the first floor, (c) and inter-story drift between the first and second floors of the testbed.

4.4 Results in the numerical domain

For the 42-story high-rise archetype building in this study, the reproduction errors of the concerned responses are all minimized by under 5% after nine iterations in the numerical domain. Figure 9 gives a detailed exhibition of comparisons between the target responses and those of the testbed achieved when the errors of floor accelerations reduced to below 5%. Take the errors of the 1st floor acceleration as an example, Figure 10 indicates that the errors changed between positive errors and negative errors alternatively but generally present a decreasing tendency.

Figure 9. Fourier spectrum comparisons of target and testbed responses after final iteration (iteration 9) in the x-direction.
Figure 10. Fourier spectrum of acceleration at 1st floor during the iteration procedure in the x-direction.

4.5 Experimental results

A full-scale trial test was conducted on the 5 m-by-5 m shake table in the IEM. The test bed, NEST, hosted a full-scale room space of 2.8 m high and 3.5 m-by-4.5 m in plan. As shown in Figure 11(a), the 3-story testbed consisted of braced steel frames, FPs, springs, and NRBs. The stiffness of the braced steel frame was very large so that it would behave as a rigid body during the vibration. The structural responses between the target and the testbed are compared in this paper.
Figure 11. Overview and drawings of testbed: (a) general view (b) elevation in the x direction (c) arrangement of the accelerometers (d) arrangement of the displacement transducer (unit: mm).

During shaking, the accelerations and the displacements of each floor were recorded by three-way accelerometers and displacement transducers at an interval of 0.001 s, respectively. The data were resampled to 200 Hz during the data processing. Figure 12 showed comparisons between the target responses and those of the testbed achieved in the x-direction under MZQ motion. The maximum error was 15.4%.
Figure 12. Fourier amplitude spectrum comparisons of experimental and target floor accelerations, velocities, and inter-story drift in the x-direction.

As shown in Figure 12, the reproduction errors in the test were larger than those in the numerical examples. It occurred because of the shake table’s control and the modeling error for the testbed, in addition to the measurement noise. Although a refined nonlinear FEM was conducted to well simulate the seismic responses of the testbed, the modeling error for the testbed, e.g., the potential velocity dependency of friction coefficient at the 1st and 3rd floor, was speculated to be the main source of reproduction errors.

5. Conclusion

This paper proposed a novel control strategy to reproduce floor motions in the middle elevation of a high-rise building as the input for various layouts of nonstructural elements. The control strategy contains an open-loop LQR-IDCS algorithm and an offline iteration procedure based on the nonlinear FEM of the testbed. A trial shake table test was then conducted to validate the proposed method as well as investigating the seismic performance of various nonstructural elements in a typical 42-story high-rise building. Results showed that the
proposed open-loop LQR-IDCS algorithm with offline iteration procedure can reproduce target floor motions of the archetype high-rise building with less than 20% errors. Further experimental investigations can be conducted for larger amplitude excitations.

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References


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Conflict of interest

We declare that we have no financial and personal relationships with other people or organizations that can inappropriately influence our work, there is no professional or other personal interest of any nature or kind in any product, service and/or company that could be construed as influencing the position presented in, or the review of, the manuscript entitled “A novel control strategy for reproducing the floor motions of high-rise buildings by earthquake-simulating shake tables”.