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ESTIMATING SEISMIC DAMAGE OF COLUMN BASES IN STEEL MOMENT-RESISTING FRAMES USING MODAL DATA

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Abstract

This paper presents a method of estimating seismic damage of column bases in steel moment-resisting frames using modal data, which is identified from measured floor accelerations. The estimation algorithm is derived on the basis of a fishbone model and Bayesian model updating. Fishbone models enable explicitly identifying the rotational stiffness of column bases given the rotational stiffness of beams in steel frames using Bayesian model updating methods. The eigenvalue equation of a fishbone model governing both the translational displacements of floors and the rotations of column bases and beam-column connections is first formulated for model updating. A Bayesian model updating algorithm is implemented for identifying the rotational stiffness of column bases in fishbone models with incomplete modal data. Seismic damage of column base is estimated by comparing the identified rotational stiffness of column bases in fishbone models before and after an earthquake. The effectiveness of the proposed method is experimentally investigated through the shaking table tests of a large scale five-story steel frame testbed that can simulate multiple fractures at column bases.

Keywords: damage assessment, column base, steel frame, fishbone model, modal data

1. Introduction

Recent earthquakes highlight the need for developing rapid post-earthquake damage and safety evaluation system for building structures, such as hospitals, financial centers, and large residential apartments because such a system supports timely decision making on evacuation and reoccupation following major earthquakes [1-6]. Steel moment-resisting frames (SMRFs) have been widely built in earthquake-prone areas due to their excellent performances, such as high plastic deformation capacity. Nonetheless, as witnessed in recent earthquakes (e.g., the 2016 Kumamoto earthquake and the 2016 Taiwan Mei-Nong earthquake), some SMRFs with fixed column base configurations sustained seismic damage at column bases under severe ground shaking. The damage, including local buckling of columns, pull-out of anchor bolts, and crushing of concrete footing hindered the continued usage of the affected buildings.

This paper presents a method of estimating seismic damage of column bases in steel moment-resisting frames based on Bayesian model updating approaches using modal data. First, the eigenvalue equation of a fishbone model governing both the translational displacements of floors and the rotations of column bases and beam-column connections is formulated. Then, a hierarchical Bayesian model updating algorithm is implemented to identify the rotational stiffness of column bases in fishbone models using incomplete modal data. Seismic damage of column bases is estimated through a comparative study on the identified rotational stiffness of column bases in fishbone models before and after earthquakes. The effectiveness of the proposed method is experimentally investigated through the shaking table tests of a large scale five-story steel frame testbed.

2. Estimation method

2.1 Eigenvalue equation of fishbone model

Fishbone model is a simplified model of steel moment-resisting frames, which produces structural responses with sufficient accuracy as much as the detailed member-by-member frame model [7-11]. The analytical fishbone model of an *n*-story steel frame is shown in Fig. 1, where m_i denotes the lumped mass of the *i*th floor, k_i denotes the stiffness of the rotational spring at the *i*th floor, k_0 is the stiffness of the rotational spring at the column base, k_{ci} is the flexural rigidity of the representative column in the *i*th story, u_{ti} and u_{oi} are the lateral displacement and joint rotation of the *i*th floor, and u_0 denotes the rotation of the column base.

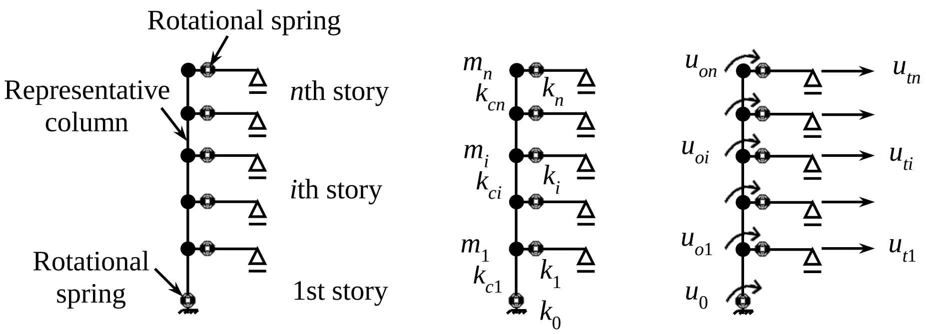


Fig. 1 – Fishbone model

The equations of motion of the *n*-story fishbone model without damping for free vibration, in terms of the lateral displacements, $\mathbf{u}_t = [u_{t1}, u_{t2}, \ldots, u_{tn}]^{\mathrm{T}} \in \mathbb{R}^n$, and the rotations of springs, $\mathbf{u}_o = [u_0, u_{o1}, u_{o2}, \ldots, u_{on}]^{\mathrm{T}} \in \mathbb{R}^{n+1}$, are expressed as

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{0} \tag{1}$$

where $\mathbf{u} = [\mathbf{u}_t^{\mathrm{T}}, \mathbf{u}_o^{\mathrm{T}}]^{\mathrm{T}} \in \mathbb{R}^{2n+1}$ is the displacement vector, and $\mathbf{M}, \mathbf{K} \in \mathbb{R}^{(2n+1)\times(2n+1)}$ are the mass and stiffness matrices. Hence, the matrix-form eigenvalue equation of fishbone model is formulated as follows [10],

$$\begin{bmatrix} \mathbf{k}_{tt} & \mathbf{k}_{to} \\ \mathbf{k}_{ot} & \mathbf{k}_{oo} \end{bmatrix} \begin{bmatrix} \phi_i^t \\ \phi_i^o \end{bmatrix} = \omega_i^2 \begin{bmatrix} \mathbf{m}_{tt} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} \end{bmatrix} \begin{bmatrix} \phi_i^t \\ \phi_i^o \end{bmatrix} \tag{2}$$

where $\mathbf{m}_{tt} = \mathrm{diag}\,(m_1, m_2, \ldots, m_n)$, and $\mathbf{k}_{to} = \mathbf{k}_{ot}^{\mathrm{T}}$; ω_i ($i = 1, 2, \ldots, n$) are the natural frequencies; $\phi_i^t \in \mathbb{R}^n$ ($i = 1, 2, \ldots, n$) are the lateral components of mode shapes; $\phi_i^o \in \mathbb{R}^{n+1}$ ($i = 1, 2, \ldots, n$) are the rotational components of mode shapes.

2.2 Bayesian model updating algorithm

In steel frames, seismic damage to column bases decreases their rotational stiffness. Thus the identification of rotational stiffness of springs at column bases in fishbone models leads to evaluate the seismic damage to column bases. An n-story fishbone model (see Fig. 1) is considered. The lumped masses and flexural rigidity of representative columns are assumed to be known, and they are estimated from the engineering drawing of the steel frame with slight uncertainties. The rotational stiffness of beams is calculated from the detection results of beam damage with the local damage evaluation method presented in [2-5]. The stiffness of rotational spring at column base k_0 in the fishbone model is identified using Bayesian model-updating methods with measured modal data.

In the fishbone model, the information of m ($\leq n$) measured modes, i.e., natural frequencies and mode shapes, are denoted as $\hat{\boldsymbol{\omega}}^2 = [\hat{\omega}_1^2, \ldots, \hat{\omega}_m^2]^{\mathrm{T}} \in \mathbb{R}^m$ and $\hat{\boldsymbol{\psi}} = [\hat{\psi}_1^{\mathrm{T}}, \ldots, \hat{\psi}_m^{\mathrm{T}}]^{\mathrm{T}} \in \mathbb{R}^{m \times s} (s \leq n)$, which are identified from the measured floor acceleration responses. Because the joint rotations are not measurable, the mode shapes $\hat{\psi}_i \in \mathbb{R}^s$ ($i = 1, 2, \ldots, m$) only contains the translational components. The modal parameters of the system modes corresponding to the measured modes are denoted as $\boldsymbol{\omega}^2 = [\omega_1^2, \ldots, \omega_m^2]^{\mathrm{T}} \in \mathbb{R}^m$ and $\boldsymbol{\Phi} = [\phi_1^{\mathrm{T}}, \ldots, \phi_m^{\mathrm{T}}]^{\mathrm{T}} \in \mathbb{R}^{m \times (2n+1)}$. Based on the Bayesian model updating method presented in [12], the posterior probability density function (PDF) for the system modal parameters $\boldsymbol{\omega}^2$ and $\boldsymbol{\Phi}$, and the stiffness of rotational spring at column base k_0 can be expressed as

$$\begin{aligned} p(\boldsymbol{\omega}^2, \boldsymbol{\Phi}, k_0 \mid \hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}) &\propto p(\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}} \mid \boldsymbol{\omega}^2, \boldsymbol{\Phi}, k_0)\, p(\boldsymbol{\omega}^2, \boldsymbol{\Phi} \mid k_0)\, p(k_0) \\ &\propto \exp\left\{ -\frac{1}{2} \left[\left(\begin{bmatrix} \hat{\boldsymbol{\omega}}^2 \\ \hat{\boldsymbol{\psi}} \end{bmatrix} - \begin{bmatrix} \boldsymbol{\omega}^2 \\ \boldsymbol{\Gamma\Phi} \end{bmatrix} \right)^{\mathrm{T}} (\boldsymbol{\Sigma}_\varepsilon)^{-1} \left(\begin{bmatrix} \hat{\boldsymbol{\omega}}^2 \\ \hat{\boldsymbol{\psi}} \end{bmatrix} - \begin{bmatrix} \boldsymbol{\omega}^2 \\ \boldsymbol{\Gamma\Phi} \end{bmatrix} \right) \right. \right. \\ &\qquad \left. \left. + \frac{1}{\sigma^2} \sum_{i=1}^{m} \left\| (\mathbf{K}(k_0) - \omega_i^2 \mathbf{M}) \phi_i \right\|^2 + \frac{1}{\xi^2} (k_0 - \bar{k}_0)^2 \right] \right\} \end{aligned} \tag{3}$$

where $p(\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}} \mid \boldsymbol{\omega}^2, \boldsymbol{\Phi}, k_0)$ is the likelihood function; $p(\boldsymbol{\omega}^2, \boldsymbol{\Phi} \mid k_0)$ is the prior distribution for the system modal parameters $\boldsymbol{\omega}^2$ and $\boldsymbol{\Phi}$; $p(k_0)$ is the hyper-prior probability distribution for the rotational stiffness k_0; $\boldsymbol{\Gamma} \in \mathbb{R}^{ms \times m(2n+1)}$ is a selection matrix with elements of 0 or 1, which picks the components of the system mode shapes $\boldsymbol{\Phi}$ that correspond to the measured mode shapes $\hat{\boldsymbol{\psi}}$; $\boldsymbol{\Sigma}_\varepsilon$ is a covariance matrix of measurement error for modal parameters; σ^2 is a prescribed system error variance; $\bar{k}_0$ represents the nominal values of the stiffness of rotational spring at column base; ξ^2 is a variance of the hyper-prior distribution of the rotational stiffness k_0, where it is taken to be a very large value that can give a non-informative prior; the stiffness matrix $\mathbf{K}$ is a function of the rotational stiffness k_0 as follow:

$$\mathbf{K}(k_0) = \mathbf{K}_B + \mathbf{K}_C + k_0\mathbf{K}_{cb} \tag{4}$$

where $\mathbf{K}_B$ and $\mathbf{K}_C$ are parts of the global stiffness matrix associated with the contributions of the rotational springs of beams and representative columns respectively; $\mathbf{K}_{cb}$ is the substructure stiffness matrix corresponding to the rotational spring at column base.

The most probable values of the parameters $\boldsymbol{\omega}^2$, $\boldsymbol{\Phi}$, and k_0 are obtained by minimizing the objective function as follows:

$$J(\boldsymbol{\omega}^2, \boldsymbol{\Phi}, k_0) = -2\ln p(\boldsymbol{\omega}^2, \boldsymbol{\Phi}, k_0 \,|\, \hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}) \tag{5}$$

The minimization of $J(\boldsymbol{\omega}^2, \boldsymbol{\Phi}, k_0)$ is conducted through a iterative procedure of three linear optimization problems introduced in [12]. The most probable values of the parameters $\boldsymbol{\omega}^2$, $\boldsymbol{\Phi}$, and k_0 are denoted as $\tilde{\boldsymbol{\omega}}^2$, $\tilde{\boldsymbol{\Phi}}$, and $\tilde{k}_0$. The posterior PDF can be approximated by a Gaussian distribution with mean equal to the most probable values $(\tilde{\boldsymbol{\omega}}^2, \tilde{\boldsymbol{\Phi}}, \tilde{k}_0)$, and covariance matrix equal to the inverse of the Hessian matrix of $J(\boldsymbol{\omega}^2, \boldsymbol{\Phi}, k_0)/2$ calculated at the most probable values $(\tilde{\boldsymbol{\omega}}^2, \tilde{\boldsymbol{\Phi}}, \tilde{k}_0)$.

2.3 Damage assessment

Seismic damage of column bases is estimated by comparing the identified rotational stiffness k_0 in the fishbone model before and after earthquakes. The damage index is defined as follows:

$$d = \frac{k_0^o - k_0^d}{k_0^o} \times 100\% \tag{6}$$

where k_0^o and k_0^d are the identified rotational stiffness of the spring at the column base in the fishbone model before and after an earthquake.

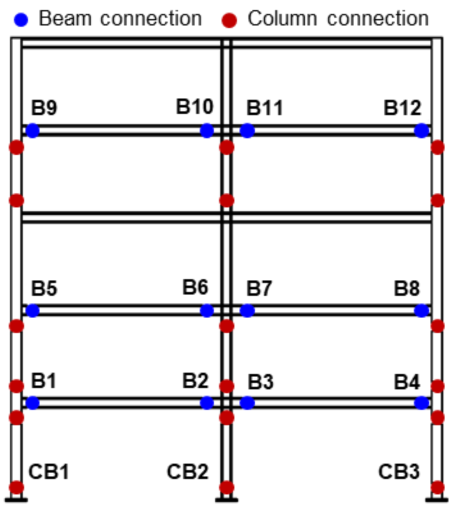
3. Experimental investigation

The estimation method was experimentally investigated through the shaking table tests of a five-story steel frame (see Fig. 2(a)) at the Disaster Prevention Research Institute (DPRI), Kyoto University. The dimensions of the steel frame were 1.0 × 4.0 × 4.4 m. Its plan was one bay by two bays. In steel frames, the exterior column lines had column sections oriented about the minor axis, while the interior column lines had column sections oriented about the major axis. The specimen had two longitudinal steel frames. In each longitudinal frame, there were 12 and 18 removable steel connections at beam and column ends respectively (see Fig. 2(b)), located at the first, second and fourth stories. The removable connection was made of four steel links at the flanges and one pair of links at the web (Fig. 2(c)). The detailed information of the testbed was reported in [2].

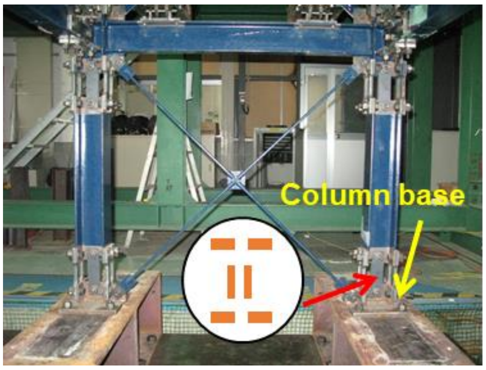
By removing the links, fracture damage to column bases and beam ends was simulated. Fig. 3 illustrates the undamaged state of the removable connection and damage patterns of column bases and beam ends. Damage CL1 and CL2 respectively simulated fracture damage to half of the column flange about the major and minor axes at the column base. The reduction in the bending stiffness was 72.5% for damage CL1 and 71.9% for damage CL2. Damage BL1 and BL2 simulated the fracture of the whole bottom flange, and the fracture of the bottom flange and half the web, respectively. The reduction in the bending stiffness about the major axis of the beam section was 68.5% for damage BL1 and 99.8% for damage BL2.

Five damage cases were considered to evaluate the performance of the presented method. Damage cases were summarized in Table 1. In Case 1, no damage was simulated at column bases. In Cases 2 and 4, damage CL1 was simulated at column base CB2. In Cases 3 and 5, damage CL1 and CL2 were separately simulated at column bases CB2 and CB1. In order to examine the influence of beam damage on the estimation of damage to column bases, beam connections B1, B3, B5, and B8 had damage BL2, BL1, BL2, and BL2 in Cases 1, 4, and 5.

(a)

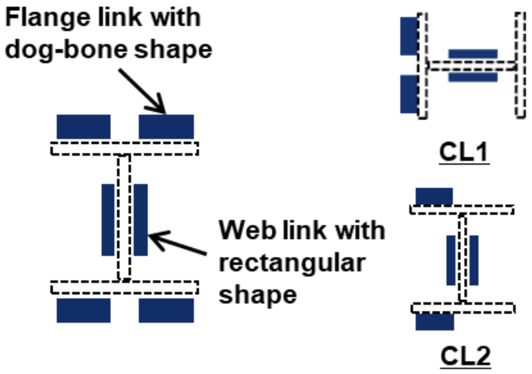


(b)

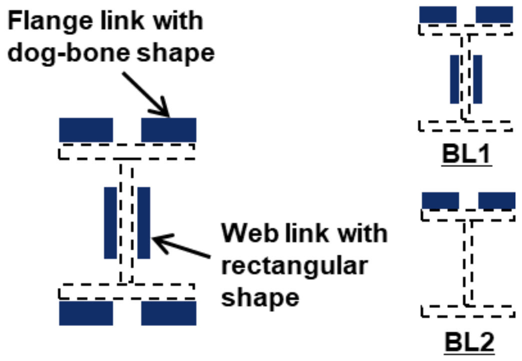


(c)

Fig. 2 – Five-story steel frame testbed: (a) overview; (b) beam and column connections; (c) steel removable connection [2]



(a)



(b)

Fig. 3 – Undamaged state and damage patterns: (a) column base; (b) beam connection [2]

Table 1 – Damage cases

Cases	Column bases (damage pattern)	Beam connections (damage pattern)
Case 1	—	B1 (BL2), B3 (BL1), B5 (BL2), B8 (BL2)
Case 2	CB2 (L1)	—
Case 3	CB1(L2), CB2 (L1)	—
Case 4	CB2 (L1)	B1 (BL2), B3 (BL1), B5 (BL2), B8 (BL2)
Case 5	CB1(L2), CB2 (L1)	B1 (BL2), B3 (BL1), B5 (BL2), B8 (BL2)

The specimen was excited by white noise excitations in the longitudinal direction. The floor accelerations were recorded by the accelerometers that were deployed at the centers of the floor plans. The natural frequencies and mode shapes of the structure were identified using the ARX method. The identified frequencies of three dominant modes of the undamaged structure were 3.14, 8.29, and 10.53 Hz. The changes in the identified frequencies in damage cases were shown in Fig. 4. The decreases in the first frequency in Cases 1 to 5 were 8.4%, 1.0%, 1.7%, 9.4%, and 9.9%. The decreases in the second and third frequencies in Cases 1 to 5 were less than 2.2% and 1.7%. The reductions of natural frequencies induced by the damage of column bases in Cases 2 to 5 were minimal. Beam damage in Cases 1, 4, and 5 reduced the first frequency by about 8.4%. The identified mode shapes of three modes were summarized in Fig. 5. Compared with the undamaged state, the identified mode shapes remained nearly identical in Cases 1 to 5, which indicated that the lateral mode shapes changed very slightly when the steel frame sustained fracture damage at column bases.

Beam damage was evaluated by the local damage evaluation method presented in [2-5] with the measured strain responses. In the fishbone model, the rotational stiffness of beams was estimated from the detection results of beam damage. The rotational stiffness of the column base in the fishbone model was separately identified in the undamaged state and five damage cases using the obtained information of the natural frequencies and mode shapes. Then, the damage index of the column base was calculated from the identified rotational stiffness of column base. Fig. 6 shows the damage indices of the column base in Cases 1 to 5. Compared to the calculated values, the estimated values had differences of 8.7%, 5.5%, 1.0%, 1.6%, and 0.6% for Cases 1 to 5. These small discrepancies implied that the presented estimation method was effective in assessing seismic damage to column bases in steel frames.

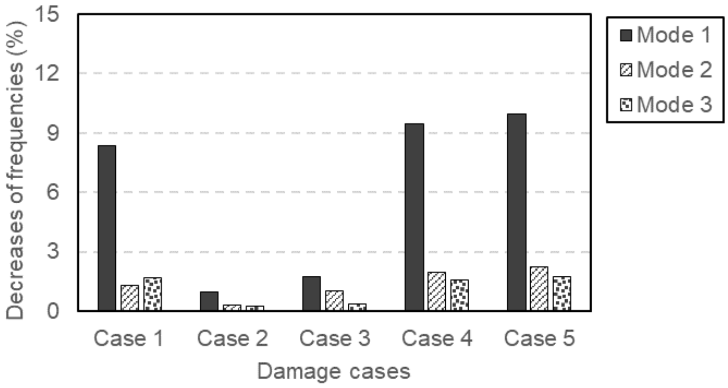


Fig. 4 – Decreases of identified natural frequencies

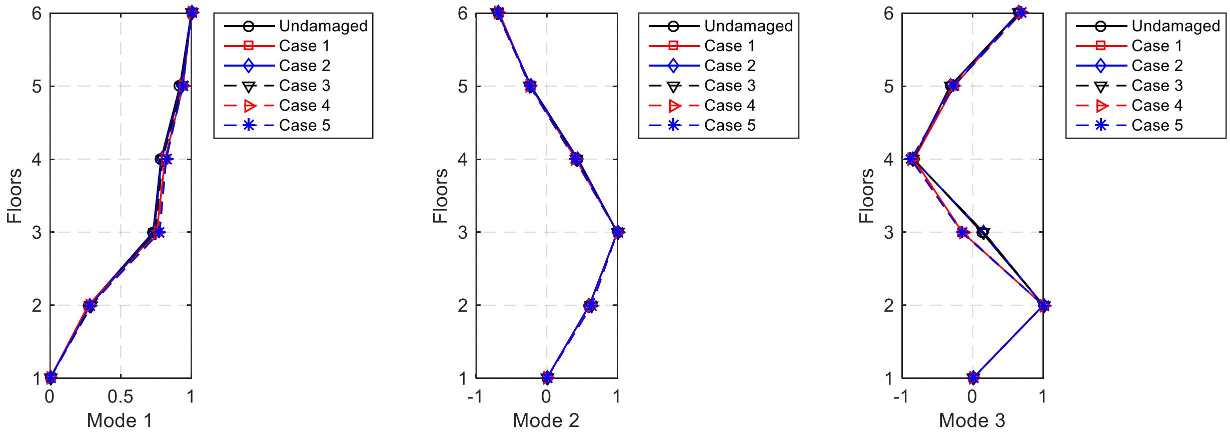


Fig. 5 – Identified mode shapes

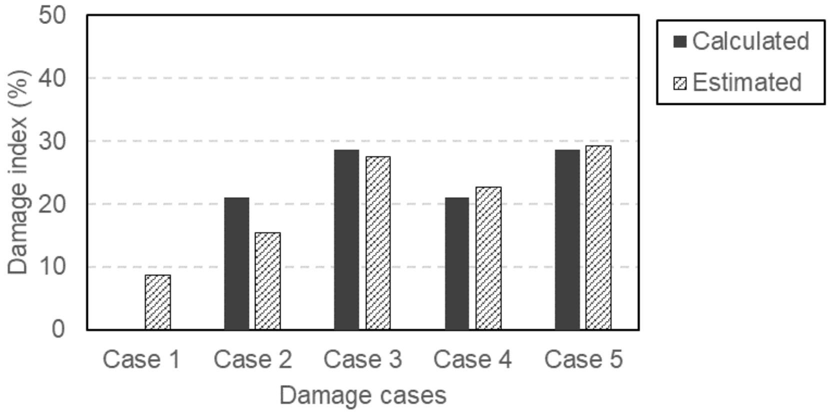


Fig. 6 – Damage index in five damage cases

4. Conclusions

This paper presented a method of estimating seismic damage of column bases in steel moment-resisting frames using modal data. The method was derived based on a fishbone model and Bayesian model updating approaches. The effectiveness of the method was experimentally investigated through the shaking table tests of a five-story steel frame testbed. The test results confirmed that the presented method was capable of quantifying the extent of seismic damage to column bases in steel frames. The proposed method would facilitate near real-time damage assessment of the earthquake-affected steel buildings and thus support rapid post-earthquake decision-making on re-occupancy.

5. Acknowledgements

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6. Copyrights

7. References

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