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Utilization of wavelet-based damage-sensitive features for structural damage assessment of steel braced frames

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ABSTRACT: Rapid structural damage assessment methodologies for engineered facilities are essential in order to properly allocate and ensure efficient use of emergency rescue forces, minimize business interruption and perform effective repairs in damaged infrastructure in the aftermath of an earthquake. This paper assesses the efficiency of a recently developed "nonmodel-based" damage-sensitive feature based on wavelet analysis that can be used as a structural damage indicator in steel concentrically braced frames. The implementation of the wavelet-based damage-sensitive feature for structural damage detection is validated through the utilization of large-scale shake table tests of a single-story concentrically braced frame tested at E-Defense in Japan. The wavelet-based damage sensitive feature is further assessed through the utilization of numerical simulations of a multi-story concentrically braced frame. It is shown that key engineering demand parameters such as peak story drift ratios, peak floor absolute accelerations and residual story drift ratios are well correlated with the wavelet-based damage-sensitive feature.

1 INTRODUCTION

During and after an earthquake disaster, the primary objectives of our society are to achieve the life safety and to preserve a high quality of our built environment. In this regard, the concept of resilient cities is evolving in recent years. In the context of earthquake risk management, these cities should have the ability to "respond" fast in the aftermath of an earthquake to facilitate emergency responses and to minimize community disruption (Bruneau et al. 2003). Conventional damage assessment techniques typically require the explicit utilization of sophisticated nonlinear building models, as well as detailed engineering inspection by professionals (Tremblay et al. 1996; Uang et al. 1997; de la Llera et al. 2001). Therefore, a considerable time investment is necessary. Detailed knowledge of the geometric configuration and material properties of the structure and its components is also necessary. To this end, the utilization and further development of nonmodel-based damage diagnosis approaches is emerging.

Extensive research has been conducted on structural health monitoring (SHM) (Lynch et al. 2016). The utilization of SHM allows for a rapid assessment of the structural damage. This involves the observation of a structure over time using monitored earthquake response data. The structural parameters extracted from the recorded earthquake response data are utilized as damage-sensitive features (DSFs) that change over time with the progression of structural damage.

A number of SHM techniques have been historically validated with idealized scale structural models. In this case, the structural damage was introduced with a sudden loss of stiffness or strength [e.g., the International Association for Structural Control (IASC)-ASCE benchmark study (Johnson et al. 2004)]. However, in code-conforming structures that employ commonly used lateral load-resisting systems designed with capacity design principles, the damage progression follows a certain failure mode hierarchy that does not typically involve premature failures. In that respect, the role of landmark data from large- or full-scale shake table tests becomes important to further challenge the validity of the SHM techniques and highlight limitations and future challenges in their further implementation (Nakashima et al. 2010).

In this paper, a nonmodel-based damage assessment based on SHM is performed based on a large-scale shake table test of a single-story concentrically braced frame (CBF). The test was conducted at E-Defense in Japan (Okazaki et al. 2013). This is complemented with computer simulations. The employed DSFs that are utilized are derived from the vibration response data recorded during earthquakes through a wavelet transformation (Mallat 1999). Finally, the derived wavelet-based DSFs are related to various engineering demand parameters (EDPs) [i.e., peak story drift ratios (SDRs), peak floor absolute accelerations (PFAs) and residual SDRs]. These EDP parameters typically facilitate the earthquake-induced risk assessment of frame structures in the context of performance-based earthquake engineering (FEMA 2012).

2 WAVELET-BASED DAMAGE SENSITIVE FEATURES

In this paper, a nonmodel-based approach is employed to detect the structural damage of steel frame buildings with CBFs. In particular, wavelet-based DSFs are utilized as proposed in Noh et al. (2011, 2012). This section briefly describes the theoretical background of the wavelet-based DSF. Given a scale parameter $a > 0$, and a time shift parameter b, the continuous wavelet transform can be mathematically described as follows:

$$C(a,b)=\int_{-\infty}^{\infty} f(t)\frac{1}{\sqrt{a}}\psi^{*}\left(\frac{t-b}{a}\right)\mathrm{d}t \tag{1}$$

in which $f(t)$ is the recorded response history data; $\psi(t)$ is the mother wavelet function (in this paper, Morlet wavelet basis function (Morlet et al. 1982) is used as a mother wavelet); and * is the complex conjugate. A set of basis functions, which are termed as daughter wavelets, is established by continuously dilating and translating the mother wavelet function, $\psi(t)$. The continuous wavelet transform coefficients, $C(a, b)$ are then obtained by convoluting the basis functions [i.e., Morlet wavelet basis functions (Morlet et al. 1982)] and the response history data, $f(t)$ (e.g., recorded absolute acceleration response history at the building roof).

Noh et al. (2011) introduced the DSFs based on a continuous wavelet transform algorithm. The mathematical form of the wavelet-based DSFs is defined as follows:

$$DSF=1-\frac{E_{\text{scale}(\hat{a})}}{E_{\text{tot}}} \tag{2}$$

in which $E_{\text{scale}(\hat{a})}$ is the wavelet energy at scale $\hat{a}$ over time as defined in Nair and Kiremidjian (2007). This energy can be computed as follows:

$$E_{\text{scale}(\hat{a})}=\sum_{b=1}^{K}\left|C\left(\hat{a},b\times\Delta t\right)\right|^{2} \tag{3}$$

The total wavelet energy, E_{tot} of the acceleration response data is the sum of the wavelet energies over time at the pre-defined scales. In this paper, E_{tot} is the sum of the wavelet energies over time at scales $\hat{a}$ and $2\hat{a}$ that correspond to the first and half of the first natural frequency of the building under consideration, respectively (i.e., $\hat{a}$ is the scale when pseudo-frequency of the daughter wavelet is equivalent to the first natural frequency of undamaged state). Wavelet-based DSF values computed based on Eq. (2) range between 0 (representing no structural damage) and 1 (representing severe structural damage).

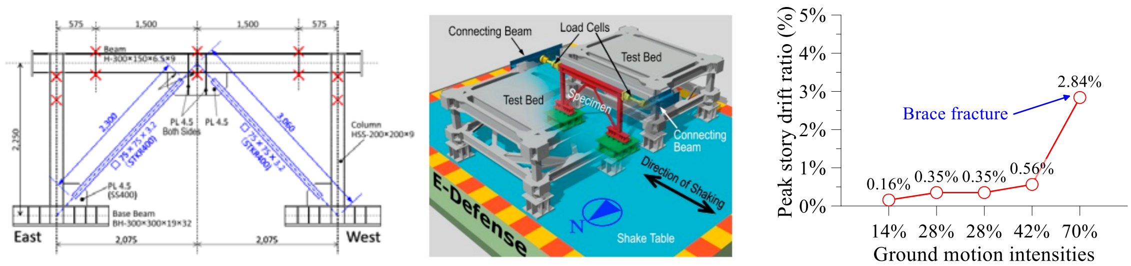
3 BRIEF DESCRIPTION OF CASE-STUDY BUILDINGS

To validate the efficiency of the wavelet-based DSFs discussed in Section 2, experimental data from a full-scale shake table experiment are utilized. The test structure employs a code-compliant steel CBF (Okazaki et al. 2013). The progression of structural damage observed in the test structure is well documented during the tests (e.g., formation and extent of plastic hinging in steel beams and columns, steel brace flexural buckling and fracture). Furthermore, the progression of structural component damage is well correlated with story-based EDPs such as peak SDRs, PFAs and residual SDRs.

In order to investigate the efficiency of the wavelet-based DSFs discussed in Section 2 in predicting the redistribution of forces after the occurrence of structural damage, the shake table test data are complemented with nonlinear simulations of a three-story steel frame building with perimeter CBFs. The subsections below provide a brief description of the case-study buildings.

3.1 Single-story, chevron concentrically braced frame tested at E-Defense

In order to investigate the applicability of wavelet-based DSFs for assessing the structural damage in steel frame buildings with CBFs, we utilize the test data from a single-story, single-span, chevron CBF (Okazaki et al. 2013). The test structure represents a 2/3 scale of the lower story of a three- to five-story steel frame building with CBFs commonly used in Japan. Figures 1(a) and (b) show the CBF geometry and test bed, respectively. The test structure was subjected to a range of seismic intensities (14%, 2×28%, 42% and 70%) of the east west component of the JR Takatori motion recorded during the 1995 Kobe earthquake. Figure 1(c) shows the structural damage progression in terms of peak SDRs with the respective seismic intensity. During the lower seismic intensities of the JR Takatori record, the steel braces buckled globally as intended (i.e., brace buckling occurred during the second 28% scaled ground motion). During the 70% of the unscaled JR Takatori intensity, the two steel braces fractured near the brace center due to low-cycle fatigue. This caused a significant lateral strength and stiffness loss to the CBF. During this motion, the beam yielded near the column face as the peak SDR exceeded 1%. Even though this was a single-story CBF, its damage progression reflects the typical one observed in code-conforming steel CBFs based on past reconnaissance reports and recent experimental studies related to the seismic performance of multi-story CBFs (Lai and Mahin 2014). More details regarding the performance of the test structure as well as its design specifics can be found in Okazaki et al. (2011, 2013).



(a) (b) (c)

Figure 1. Two-third single-story chevron concentrically braced frame: (a) elevation view of test structure [adopted from Okazaki et al. (2013)]; (b) test-bed system [adopted from Okazaki et al. (2013)]; and (c) peak story drift ratios at various ground motion intensities.

3.2 Nonlinear building model of steel CBF building

In an effort to highlight potential issues with damage identification techniques in low-rise steel frame buildings with CBFs, the experimental data discussed in the previous section are complemented with numerical simulations of a three-story steel frame CBF building. This building has been deigned in accordance with AISC (2005) as a standard office building (i.e., occupancy category II). The building is assumed to be located in downtown Los Angeles. More details regarding the building design can be found in NIST (2010) and Hwang and Lignos (2017).

A 2-dimensional (2-D) numerical model of the steel frame building is developed in the Open System for Earthquake Engineering Simulation (OpenSees) Platform (Mckenna 1997). The steel braces are modeled based on the computational framework proposed by Karamanci and Lignos (2014). The steel beams and columns in the CBF are modeled as elastic elements with concentrated plasticity flexural hinges at their ends as shown in Figure 2(a). The phenomenological deterioration model discussed in Ibarra et al. (2005) and Lignos and Krawinkler (2011) is employed for this purpose. In brief, the nonlinear models explicitly simulate brace cyclic buckling and fracture initiation due to low cycle fatigue, and beam and column cyclic deterioration in flexural strength as well as geometric nonlinearities with the corotational transformation. Figure 2(b) illustrates a comparison of the measured and simulated hysteretic axial force-axial displacement relation of a rectangular HSS steel brace based on the modeling recommendations discussed in Karamanci and Lignos (2014). From this figure, the employed modeling approach reflects the experimental results.

The dynamic response of the three-story steel frame building is investigated from the onset of structural damage through the occurrence of dynamic instability based on multiple nonlinear response history analyses. For this purpose, the numerical model of the three-story CBF building is subjected to the fault normal component of the Canoga Park record from the 1994 Northridge earthquake. The EDPs of interest (i.e., peak SDRs, PFAs, residual SDRs) of the frame building are obtained for the ground motion over the entire range of structural response from elastic behavior through structural collapse. Figure 2(c) illustrates the pseudo spectral acceleration, $S_a(T_1, 5\%)$ at the first mode period of the three-story CBF. In the same figure, additional plots illustrate the seismic intensities corresponding to the pseudo spectral acceleration at a service-level (SLE) and a design-basis earthquake (DBE) hazard levels. Once this curve becomes flat, dynamic instability occurs as discussed in Lignos et al. (2009). Figure 2(d) shows the distribution of peak SDRs along the height of the steel frame building at selected seismic intensities. Referring to this figure, it is evident that the three-story CBF building tends to form a weak story due to concentration of plastic deformations in the first story. This is deemed to be reasonable based on prior studies associated with the seismic response of steel CBFs through collapse (NIST 2010; Hwang et al. 2015; Hwang and Lignos 2017).

4 WAVELET-BASED DSF AS A STRUCTURAL DAMAGE INDICATOR

The structural damage identification for the case-study buildings is evaluated based on the wavelet-based DSF discussed in Section 2. For this purpose, the first natural frequency, f_1 of the undamaged state of each case-study building is needed. This frequency is the scale $\hat{a}$ at which the wavelet energy is computed over time as discussed in Section 2. The f_1 of each case-study building is obtained based on the results from a white noise excitation and/or eigenvalue analysis. In case of the single-story CBF test structure, the natural frequency is identified based on the autoregressive with exogenous term method (Pakzad and Fenves 2009). The natural frequency and the damping ratio of the undamaged buildings is tabulated in Table 1.

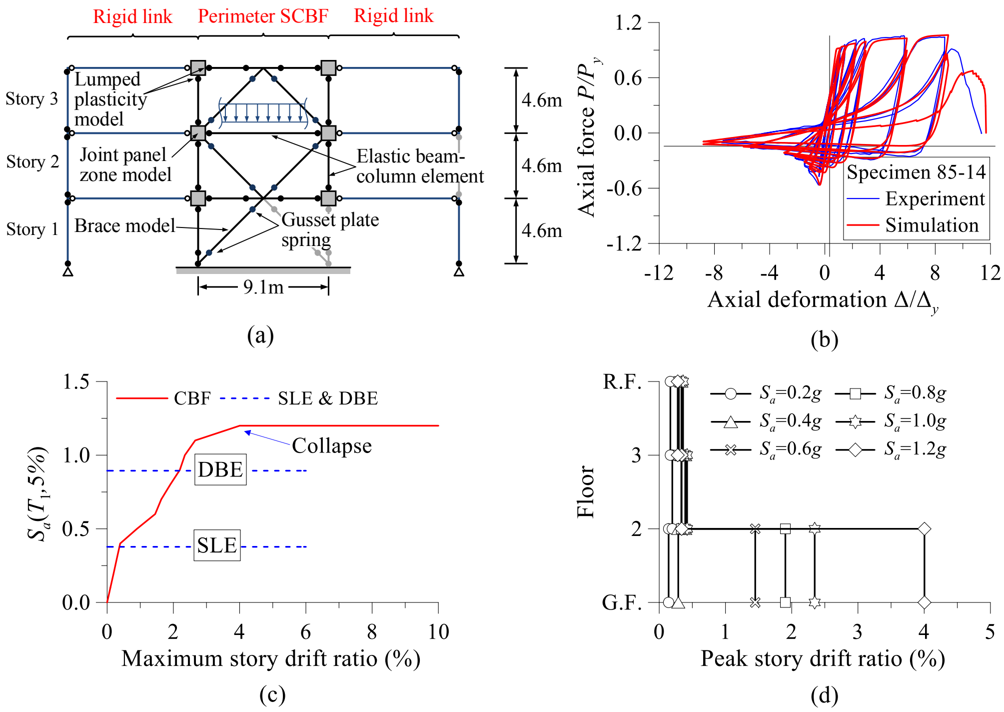


Figure 2. Nonlinear building model of steel frame building with CBF: (a) 2-D nonlinear model; and (b) axial force-deformation relation for rectangular hollow structural sections brace section [data from Han et al. (2007)].

Table 1. Identified natural frequency and damping ratio of case study buildings.

Test structure	Natural frequency f_1 (Hz)	Equivalent damping ratio ζ_1 (%)
Single-story CBF	4.86	5.29
3-story CBF	1.83	2.00

Figure 3 illustrates the wavelet-based DSFs as a function of the seismic intensity for the two case-study buildings considered in this paper. In the same figure, the corresponding peak SDRs per seismic intensity are shown in a dual plot for both case-study buildings. Referring to Figure 3, it is evident that the wavelet-based DSF is well correlated with the damage extent in both case-study buildings; the wavelet-based DSF increases from 0 to 1 while the seismic intensity of the input ground motion increases. This implies that the wavelet energy at scale $\hat{a}$ (i.e., the scale corresponding to the first natural frequency) decreases while the structural damage within the steel CBFs progresses. Referring to Figure 3(a), the wavelet-based DSF increase is practically negligible for low seismic intensities associated with a frequently occurring earthquake event (i.e., 14% to first 28% of the JR Takatori record). This is to be expected considering that the seismic response of the single-story CBF was essentially elastic in this range. Same observations hold true for the three-story CBF up to 0.4g [see Figure 3(b)]. However, a large increase in the wavelet-based DSFs of the steel CBFs is observed at higher seismic intensities. This is attributed to geometric instabilities associated with steel brace flexural buckling. This results into an appreciable loss of the CBF's lateral stiffness.

Referring to Figure 3, when low-rise steel CBFs are subjected to a DBE, a DSF value of 0.1 to 0.2 imply that steel brace flexural buckling is likely to occur with limited out-of-plane brace rotation (i.e., 3% out-of-plane rotation for both case-studies). This is important to note considering that at such rotations it is not easy to identify the steel brace damage due to flexural buckling because they are typically hidden behind partition walls or other non-structural elements.

From Figure 3(a), steel brace fracture is associated with a considerable loss of the lateral stiffness of the steel CBF, and thus causes a significant increase in the wavelet-based DSF of the single-story steel CBF. Referring to Figure 3(b), the wavelet-based DSF captures well the fact that the three-story CBF collapsed with a first story mechanism due to inelastic buckling and fracture of its first-story steel braces. From Figures 3(a) and (b), for seismic intensities associated with a low probability of occurrence earthquake, a DSF ≥ 0.6 seems appropriate. In this case, steel brace fracture is likely to occur.

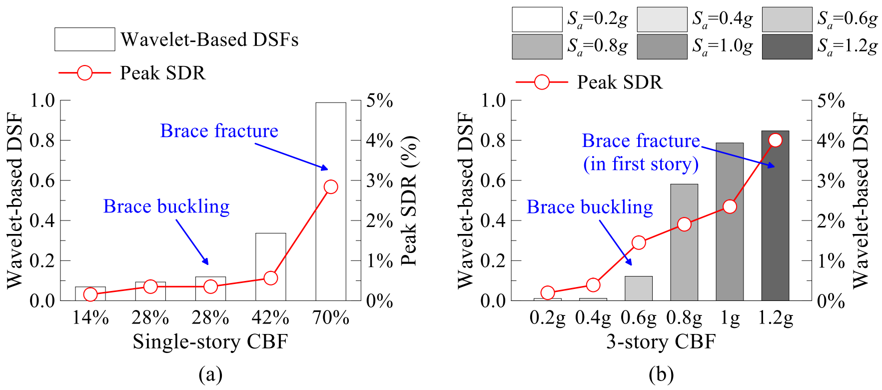


Figure 3. Wavelet-based damage-sensitive feature for the case-study buildings: (a) single-story CBF tested at E-Defense; and (c) three-story CBF.

Figure 4 illustrates a scatter plot of the wavelet-based DSF versus the maximum EDPs over the height of the two case-study buildings. The results are illustrated in a log-log scale. Superimposed in the same figure is a power fit as well as the correlation coefficient, ρ of the wavelet-based DSF with respect to the maximum recorded EDPs. Referring to Figure 4, the DSF is well correlated with the maximum and residual SDRs along the height of the case-study buildings; therefore the wavelet-based DSF has the potential to be used as a global damage indicator for predicting peak SDRs. Referring to Figure 4, the wavelet-based DSF may be utilized as a global damage indicator for the potential of building demolition in the aftermath of an earthquake. This is related to the expected residual SDRs along the height of a building (Ramirez and Miranda 2012).

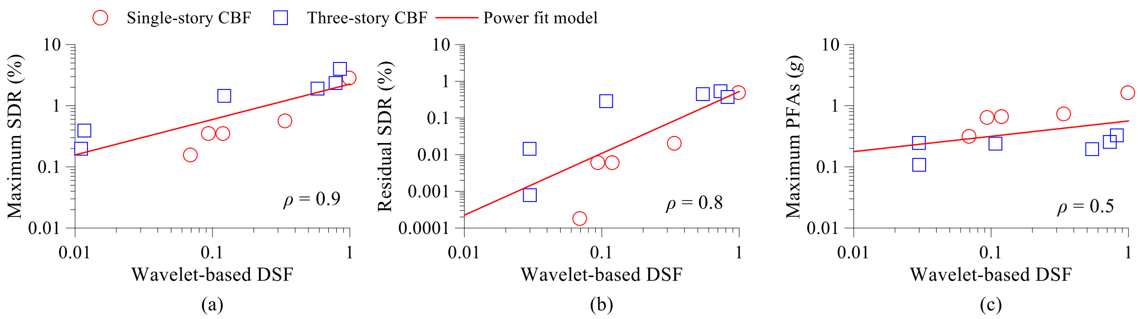


Figure 4. Scatter plots of wavelet-based DSF determined from the building roof versus maximum EDPs: (a) maximum SDRs; (b) residual SDRs; and (c) PFAs.

5 CONCLUSIONS

In this paper, a wavelet-based damage-sensitive feature (DSF) is utilized as a potential structural damage indicator in low-rise steel concentrically braced frames (CBFs). For this purpose, we utilized experimental data from a large-scale shake table test on a single-story CBF that were complemented with numerical simulations of the seismic response of a three-story steel CBF at various seismic intensities. The wavelet-based DSF is computed based on the absolute acceleration response recorded at the building's roof. The results suggest that the employed DSF is well correlated with various engineering demand parameters (EDPs) of steel frame buildings with CBFs. This is because the wavelet energies at the first natural frequency of the corresponding CBFs decrease while the structural damage progresses.

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