16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 718

Registration Code: S-A1461552200

IDENTIFICATION OF SEISMICALLY-INDUCED LOCAL DAMAGE IN STEEL MOMENT-RESISTING FRAMES USING WIRELESS STRAIN SENSING

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Abstract

Steel moment-resisting frames subjected to severe ground shaking may suffer local damage (e.g., fractures) at beam-column connections, which likely impair the safety for occupancy. Post-quake safety evaluation and decision-making for steel moment-resisting frames mainly depends on the inspection results of local damage to beam-column connections. This paper presents the comprehensive overview on the recent developments at DPRI, Kyoto University on a local damage evaluation method for identifying seismically-induced beam fractures in steel moment-resisting frames. A local damage estimator that consists of damage index, decoupling algorithm, and damage curve is developed for quantitatively identifying beam fractures. A damage index is formulated from a comparative study of the strain responses before and after earthquakes. Decoupling algorithm is necessary to estimate the damage index for each individual fracture under multiple damage condition where the damage indices for neighboring fractures correlate each other. The derivation of a damage curve enables to translate the decoupled damage index into the physical properties of fracture, i.e. the reduction of its bending stiffness. Finally, the effectiveness of the local damage evaluation method was experimentally examined using vibrational tests on a large-scale five-story steel frame testbed that was designed to accommodate seismic fractures at beam-ends.

Keywords: damage quantification; seismic damage; steel moment-resisting frame; dynamic strain; wireless sensing

1. Introduction

Steel moment-resisting frame buildings have been widely adopted in earthquake-prone areas since the early 1970s, due to their excellent features such as ease construction, architectural and functional versatility, and high plastic deformation capacity. The seismic design of these buildings allowed that structural elements deform in-elastically in large earthquakes, which is useful for reducing design strength and dissipating energy. Nevertheless, welds at beam-column connections are not sufficiently ductile without careful detailing, especially at on-site conditions. Thus, steel buildings excited by severe and repeated ground shaking may suffer major damage (e.g., local buckling, brittle fractures and/or bolt slippage) at beam-column connections, which possibly classify them to be unsafe for occupancy. In recent devastating earthquakes (e.g., the 2011 Tohoku earthquake in Japan), in the absence of certified information on seismic damages, the authorities and building owners faced difficult decisions on restarting normal operations in earthquake-affected steel buildings.

Post-quake safety assessment of earthquake-affected steel buildings relies on the inspection of seismic damage to beam-column connections. Conventionally, non-destructive evaluation (NDE) techniques, such as visual examination and ultrasonic testing, are used to detect seismic local damage. Nonetheless, these methods require extensive costs and labors because steel members are covered with fire-proofing and architectural finishes. Moreover, in the surveys of steel moment-resisting frames affected by the 1994 Northridge earthquake and 1995 Kobe earthquake, while many damaged connections were discovered, numerous connections that remained intact had to be inspected because of obvious damage in concrete slabs or nonstructural elements around these connections.

A structural health monitoring (SHM) system is acknowledged as one of promising tools to support near real-time damage assessment following earthquakes as it enables structural engineers or owners to evaluate damage in structures in a prompt and objective manner [1]. Currently, a few important steel buildings located at metropolitan areas with high seismicity are instrumented with SHM systems, where the floor responses (e.g., acceleration and velocity) are primarily measured [2-6]. For SHM applications, damage identification methods utilizing the global characteristics of buildings (e.g., acceleration responses, modal frequency and mode shape, and inter-story drift ratio), such as modal-parameter based method [7], inter-story-drift based method [8], seismic-wave propagation method [9], and time-series analysis method [10], have been developed over the past few decades. However, experimental investigations into these methods demonstrated that they estimated the health conditions of buildings to some extent, but encountered serious challenges to give accurate information of local damage on structural members.

With the rapid and remarkable advances of microprocessor and wireless communication technologies, wireless sensing became a cost-effective alternative to wire monitoring and has the potential to fundamentally change health monitoring technology [11-12]. Wireless sensing is a spatially distributed autonomous sensor network. Its features are wireless communication, on-board computation, small size and low cost. Wireless sensing allows largely increasing the density of sensors installed in large-scale civil structures with reasonable investments. Moreover, as strain responses directly reflect the local damage information of the monitored structural members, piezoelectric strain sensors (e.g., lead zirocondate titanate (PZT) and polyvinylidene fluoride (PVDF)) which have high sensitivity, wide frequency range, and long-term durability, open up another new opportunity to improve conventional health monitoring [13-15]. Thus, by combining wireless sensing with piezoelectric strain sensors, one can form dense-array wireless strain sensing systems for localized damage detection in steel buildings.

This paper presents the comprehensive overview on the recent developments at DPRI, Kyoto University on a local damage evaluation method and an associated sensing system for monitoring seismically-induced beam fractures in steel moment-resisting frames [16-19]. First, a dense-array wireless strain sensing system that consisted of a wireless sensor network and high-sensitivity PVDF strain sensors was constructed for measuring elastic strain responses under small amplitude loadings including ambient vibrations. Second, a local damage estimator that consisted of damage index, decoupling algorithm, and damage curve was developed for quantitatively identifying beam fractures. Finally, the effectiveness and accuracy of the local damage evaluation method and associated measurement system were experimentally examined using a series of vibrational tests on a large-scale five-story steel frame specimen that was designed to accommodate seismic fractures at beam-ends.

2. Local damage evaluation method

In steel moment-resisting frames, local damage such as seismically-induced fractures on steel beams changes the distribution of bending moments sustained by members. For the frames behaving linearly, the moment distribution evaluated at a natural vibrational mode is independent of external loadings. In practice, the modal bending moments can be estimated by measuring strain responses of the members under ambient vibrations, e.g. such induced by wind, human activities or traffic. This section introduces a methodology of seismically-induced local damage evaluation using wireless piezoelectric strain sensing, as shown in Fig. 1. The location and extent of local damage are identified by a local damage estimator that is comprised of damage index, decoupling algorithm, and damage curve.

2.1 Wireless piezoelectric strain sensing

Wireless piezoelectric strain sensing system that consists of a dense array of PVDF sensors (DT1-028k, Measurement Specialties, VA, USA) [20] interfaced with *Narada* wireless sensing units (Civionics, LLC, CO, USA) [21] is developed for measuring the strain responses of beams at the states intact and after earthquakes. The sensing system, including detecting sensors and a reference sensor, is deployed to beams (see Fig. 1(a)). The detecting sensors are to monitor damage-prone beams pre-identified by structural calculations. The reference sensor is used to normalize the responses measured by the detecting sensors and thereby eliminate the effects of the excitations. A floor with small deformation where the concrete slabs and beams remain undamaged (e.g., the roof) is recommended for the location of the reference sensor. In steel frames, the probability of sustaining damage to beam-column connections increases as inter-story drift increases. Thus, several floors likely sustaining large inter-story drift (usually at the lower stories) have higher priority in the monitoring strategy.

2.2 Strain-based damage index

The strain-based damage index (*DI*) is defined as Eq. (1) [17], which is formulated from a comparative study of the strain responses at the intact state and a damage probable state after a major earthquake.

$$DI = \frac{R_j^d - R_j}{R_j} \times 100\% , \tag{1}$$

where R_j and R_j^d are the ratios of strain responses associated with a natural mode—the *j*th mode at the detecting and reference sensors under the undamaged condition and after an earthquake, respectively. In practice, the strain responses associated with the *j*th mode are extracted using a band-pass filter on strain time histories and the ratios R_j and R_j^d are evaluated as the root mean square (RMS) of the filtered strain time histories. The damage index thus obtained is proven to be independent of external excitations and vibrational modes thoretically. Here, the damage index of less than 0 indicates the existence of damage on the monitored beam end and the damage index of −100% means complete fracture. If the damage index is not less than 0, there is no damage on the monitored beam end, and the damage index indicates the changes in the strain responses measured at the beam end induced by neighboring damages. The strain sensors shall be located in the region unaffected by the local strain redistribution so that the damage index equals to the changes in the bending moments at the sensor location.

2.3 Decoupling algorithm

When a beam end sustains severe damage, some portion of the bending moment sustained originally by the damaged beam is released and redistributed to the neighboring beams. Accordingly, the strain responses measured at neighboring beam ends increase slightly and, as a result, the accuracy of damage index in identifying small damages deteriorates with the presence of severe neighboring damages. For removing this issue of damage interaction, a decoupling algorithm is derived. The decoupling algorithm is formulated for beam damages on an individual floor as expressed in Eq. (2) [19]:

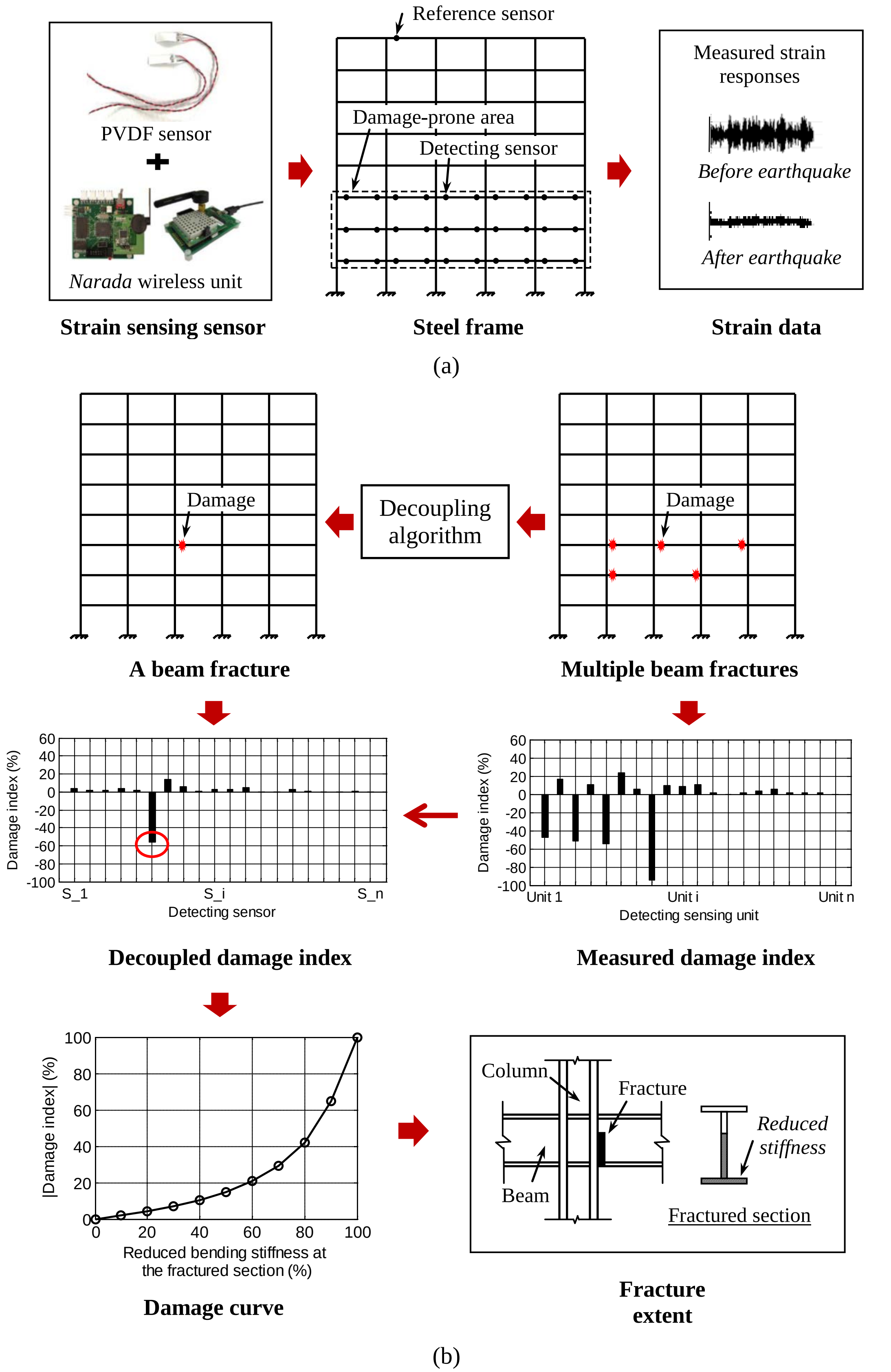


Fig. 1 – Overview of local damage evaluation method: (a) wireless strain sensing system on a steel frame; (b) quantification of a beam fracture using the local damage estimator (modified from [19]).

$$\overline{\boldsymbol{DI}} = \boldsymbol{\Delta}^{-1}[\boldsymbol{DI} - (\boldsymbol{DI})'], \tag{2}$$

where $\boldsymbol{DI}$ is a vector of measured damage indices under multiple damage conditions; $\overline{\boldsymbol{DI}}$ is a vector of the damage indices associated with individual beam damages, named as decoupled damage indices (see Fig. 1(b)); $\boldsymbol{\Delta}$ is an influence coefficient matrix; $(\boldsymbol{DI})'$ denotes the influence from the moment releases of beam damages at neighboring floors.

2.4 Damage curve

The damage extents of seismic beam fractures are calculated from the damage curve shown in Fig. 1(b). This curve relates the absolute value of the damage idex with the reduced bending stiffness at the fracture section. This relationship is expressed as Eq. (3) whose theoretical derivation is presented in [18].

$$\rho = \frac{-(B_2(DI) - A_2) - \sqrt{(B_2(DI) - A_2)^2 - 4(B_1(DI) - A_1)(B_3(DI) - A_3)}}{2(B_1(DI) - A_1)}, \tag{3}$$

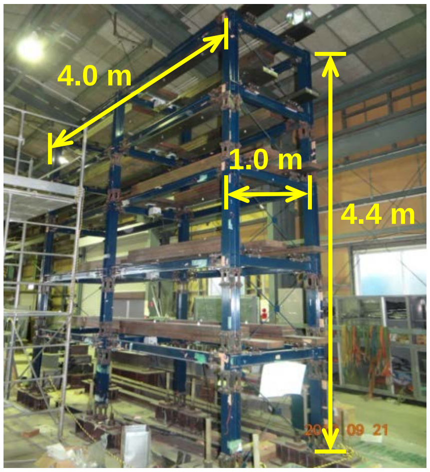
where ρ is the reduction of the bending stiffness at the fractured section; DI is the decoupled damage index for a single beam end; A_1, A_2, A_3, B_1, B_2, and B_3 are coefficients that are functions of structural parameters. Using the expression of the damage curve, the reduction of bending stiffness of the damaged beam can be directly estimated from the decoupled damage index.

3. Experimental investigations

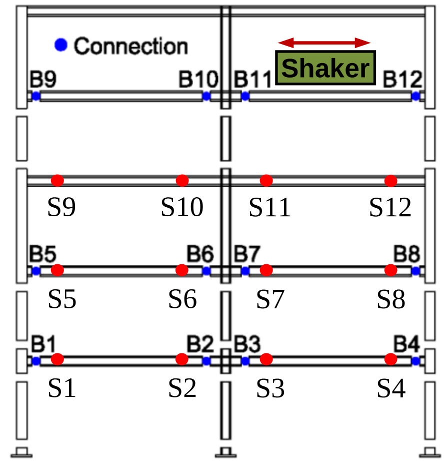
The local damage evaluation method was experimentally investigated using a five-story steel frame testbed (see Fig. 2(a)) constructed at the Disaster Prevention Research Institute (DPRI), Kyoto University. The dimensions of the testbed were 1.0 × 4.0 × 4.4 m. Its plan was one bay by two bays. In each longitudinal steel frame, there were twelve removable steel connections at beam ends (i.e., connections B1 to B12, see Fig. 2(b)), located at the second, third and fifth floors. Removable connection was made of four steel links at the flanges and one pair of steel links at the web (Fig. 2(c)). The detailed information of the testbed was reported in [16].

In vibrational testing, the testbed was excited using a modal shaker (APS-113, APS Dynamics) fixed to a steel mass plate on the fifth floor (Fig. 2(d)). The strain responses of steel beams were measured using the wireless strain sensing system. PVDF strain sensors were placed on both sides of the beam bottom flange at 1.5 beam depths away from the edge of the fracture; the location of 1.5-2.0 beam depths away from the edge of the fracture is recommended to avoid local strain redistribution by fracture [17]. The damage index was extracted from the strain responses measured under small-amplitude white noise excitations (i.e., when the undamaged frame was excited, the roof acceleration responses were 3.32 cm/s^2 in RMS). Two PVDF strain sensors at the same beam section were treated as one sensor location because the average of the damage indices at two sides of the bottom flange was used in experimental investigations. There were 12 sensor locations, i.e., S1 to S12, located in the second to fourth floors, as shown in Fig. 2(b).

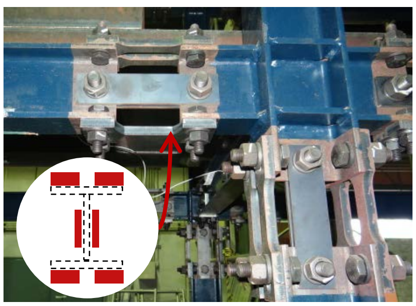
Strain time histories were measured for 75 sec with a sampling rate of 100 Hz. Fig. 3 shows the dynamic strain responses at the undamaged condition in voltage and their amplitude spectra at the beam of the second floor (S2 in Fig. 2(b)). The dynamic characteristics of the testbed frame were evaluated from the floor acceleration responses measured under the white noise excitations using the Frequency Domain Decomposition (FDD) method. The acceleration records were measured at a sampling rate of 100 Hz. The identified frequencies were 3.16 and 8.33 Hz for the first and second modes in the undamaged condition, respectively. Compared to the identified two frequencies from acceleration records, the frequencies of 3.15 and 8.33 Hz obtained from the peaks in the amplitude spectra of the measured strain responses have differences of less than 0.5%. This indicates that the wireless strain sensing system was effective and sufficiently sensitive for monitoring strain responses.



(a)



(b)



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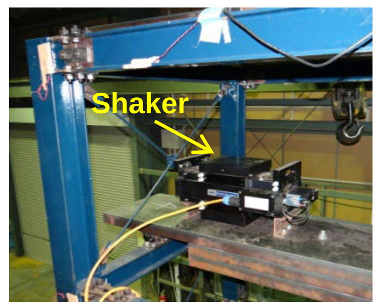


(d)

Fig. 2 – Five-story steel frame testbed: (a) overview; (b) beam connection and sensor location; (c) steel removable connection; (d) modal shaker [17, 18].

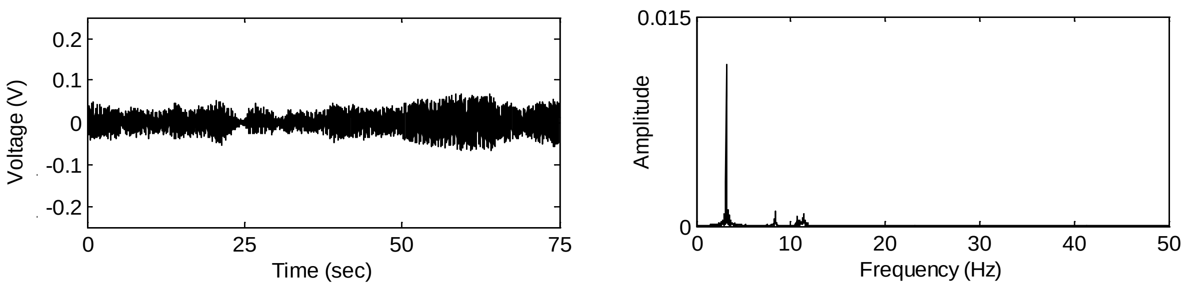


Fig. 3 – Measured strain responses.

By changing or removing the links, fracture damage was simulated. Fig. 4 illustrates the undamaged state of the removable connection and three levels of fracture damage. Damage level 1 to level 3 (L1 to L3) simulated fracture of the whole bottom flange, fracture of the bottom flange and one-quarter of the web, and fracture of the bottom flange and half the web, respectively. As summarized in Table 1, the reduction in the bending stiffness about the major axis of the beam section was 53.4% for damage L1, 79.4% for damage L2, and 93.6% for damage L3 [18].

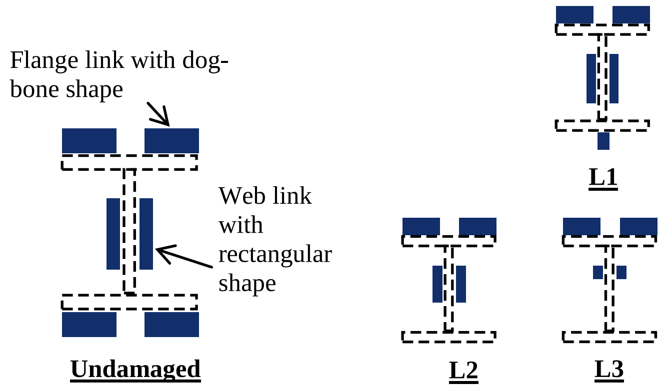


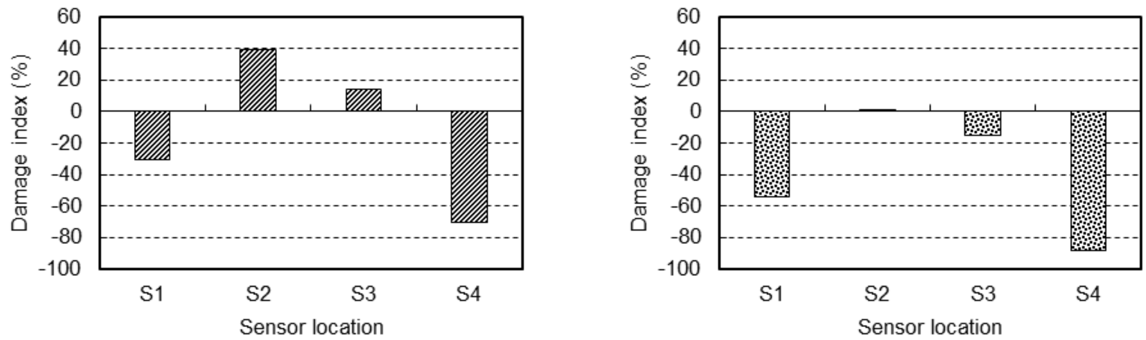
Fig. 4 – Undamaged state and damage patterns [18].

Table 1. Damage patterns [18].

Damage pattern	Target of simulation	Reduction of EI_x (%)
L1	Fracture of whole bottom flange	53.4
L2	Fracture of bottom flange and one-quarter web	79.4
L3	Fracture of bottom flange and half web	93.6

A multiple damage case was considered for the experimental investigation. At the second and third floors, five removable connections B1, B3, B4, B5, and B8 (see Fig. 2(b)) were simulated to damage as L2, L1, L3, L2, and L3, respectively. Fig. 5 illustrates the measured and decoupled damage indices at four sensors S1 to S4 on the second floor. From the measured damage indices, damage L2 and L3 at connections B1 and B4 were detected from the measured damage indices of –30.9% and –70.6% respectively, while damage L1 at connection B3 was not identified from the measured damage index of 13.8%. In comparison, the damage L1 at connection B3 was identified by the decoupled damage index of –14.9%. Fig. 6 shows the estimated reduction of bending stiffness for fracture damages on the second floor. The reduction of bending stiffness evaluated from the expression of the damage curve, i.e., Eq. (3), using the decoupled damage indices. Compared with the exact values calculated from the sectional properties, the estimated values had the largest absolute difference of about 7%. This indicates that the local damage evaluation method was effective in quantitatively identifying local damage in steel moment-resisting frames.

For reference, the associated changes in the modal properties (frequencies and mode shapes) that are commonly used for tranditional damage evaluation methods are studied. The first two natural frequencies and mode shapes are estimated from the floor acceleration responses using the FDD technique. The first two frequencies of the multiple damage case were 2.99 and 8.31 Hz. Compared with the undamaged state, the frequency changes were about 5.7% and 1.2% for the first and second frequencies. In addition, the Modal Assurance Criterion (MAC) values of the first two mode shapes derived from the intact state and the multiple damage condition are 0.9998 and 0.9993. From the changes in the modal properties, the damage is not easily detected. This fact strongly indicates the advantages of the proposed local damage evaluation method compared to the traditional methods. Another strong advantage of the proposed framework is to present the damage evaluation results explicitly in terms of a conventional structural property, i.e. bending stiffness at the fractured section, that is familiar to structural engineers.



(c) (d)

Fig. 5 – Damage indices of sensors S1 to S4 on the second floor: (a) measured; (b) decoupled [19].

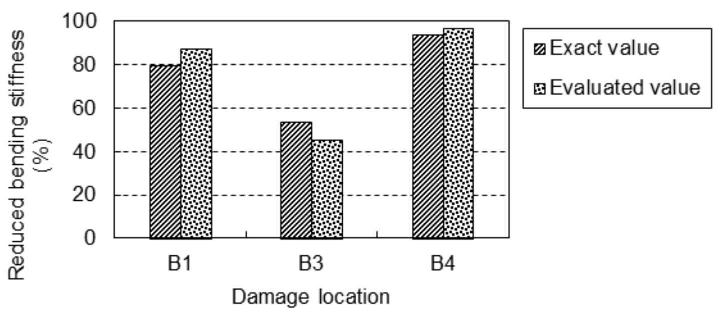


Fig. 6 – Estimated reduction of bending stiffness for fracture damages on the second floor (modified from [19]).

4. Conclusions

This paper presented comprehensively a local damage evaluation method specifically designed for detecting and quantifying seismically-induced beam fractures in steel moment-resisting frames. The effectiveness of the method was verified through experimental studies using the five-story steel frame testbed. The test results confirmed that the presented framework of the local damage estimator and the wireless strain sensing system are capable of quantifying the extent of seismic beam fractures in steel frames. The identified local damage estimator can be expressed in terms of a conventional structural propety, i.e. bending stiffness at the fractured section, through the developed damage curve. The proposed method would facilitate near real-time assessment on the remaining capacity of the earthquake-affected steel buildings and thus support rapid post-quake decision-making on re-occupancy.

4. Acknowledgements

This work was supported by JSPS KAKENHI Grant Number 26820230 and by the General Collaborative Research program of the Disaster Prevention Research Institute, Kyoto University. The authors want to express their sincere gratitude to Prof. Masayoshi Nakashima for his unlimited supports and insightful advice in the research.

5. Copyrights

5. References

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