

**Evaluation of Earthquake-Induced Local Damage in
Steel Moment-Resisting Frames Using Wireless
Piezoelectric Strain Sensing**

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Xiaohua LI

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CHAPTER 1

Introduction

1.1 Background and motivation

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 dissipating energy. Nevertheless, welds and/or bolts at beam-to-column connections are not sufficiently ductile for high stress states. After a great earthquake, these buildings excited by severe ground shaking may suffer brittle fractures and/or bolt slippage at beam-to-column connections, which possibly classify them to be unsafe for occupancy. As witnessed in the 1994 Northridge earthquake and 1995 Kobe earthquake, a large number of steel moment-resisting frames suffered brittle fractures at steel beam-to-column connections [1-3]. The most commonly observed fracture damage initiated at the weld toe on bottom flange near the weld access hole. In some cases, severe brittle fractures extended to the web and the whole moment resisting connections failed.

Post-quake safety evaluation and decision-making on re-occupancy for the earthquake-affected steel buildings depends on the results of the inspection of damage to beam-to-column connections in frames. Conventionally, non-destructive evaluation (NDE) techniques such as visual examination and ultrasonic testing were 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.

Structural health monitoring (SHM), which enables structural engineers or owners to evaluate damage in structures in a prompt and objective manner, is acknowledged as one of promising tools to support rapid damage assessment and decision-making following earthquakes [4]. At present, a few important steel buildings located at metropolitan areas with high seismicity have installed SHM systems as an extension of strong ground motion monitoring systems, where the floor responses (e.g., acceleration and velocity) are primarily measured [5-9]. For SHM applications, damage identification methods, such as modal parameter-based method [10], inter-story drift ratio-based method [11], seismic wave propagation method [12], and time series analysis method [13], which utilize the global characteristics of buildings (e.g., acceleration responses, modal frequency and mode shape, and inter-story drift ratio) have been extensively studied over the past few decades.

Experimental investigations into these methods demonstrated that they estimated the health conditions of buildings to some extent, but encountered serious challenges to give reliable information of localized damage on structural members. For example, through a series of shaking table testing in which various levels of realistic seismic damage were reproduced for a high-rise steel building specimen at the E-Defense facility in Japan, Ji *et al.* [14] demonstrated that the natural frequencies of the specimen decreased by 4.1%, 5.4%, and 11.9% on average for three damage levels respectively, while the mode shapes changed very little. The changes in the modal properties were largely influenced by cracks in concrete slabs and barely provided the accurate location and extent of seismic damage on individual steel members. Besides, through the same testing, Chung [15] reported large variations in seismic damage at beam-column connections at the same floor level that experienced nearly identical deformation due to large uncertainties in materials and hysteresis behaviors of members and connections.

With the development of microprocessor and wireless communication technologies and declining in cost, wireless sensing as an alternative to wire monitoring has the potential to fundamentally change health monitoring technology [16-18]. 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 [19-22], piezoelectric strain sensors (e.g., lead zirconate 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 [23-26]. Thus, by combining wireless sensing with piezoelectric strain sensors, one can form dense-array wireless strain sensing systems for localized damage detection in steel moment-resisting frame buildings.

1.2 Objectives

The target of this dissertation is to develop a localized damage evaluation method specifically designed for detecting and quantifying seismically induced fractures to beam-to-column connections in steel moment-resisting frame buildings, which enables to rapidly and reliably estimate the remaining capacity of the earthquake-affected buildings and thus support post-quake decision-making on re-occupancy. In this dissertation, the following topics are studied.

(1) A high-sensitivity wireless strain sensing system is developed to measure strain responses under small-amplitude dynamic loads (e.g., ambient vibration and minor earthquake ground shaking).

(2) A strain-based damage index is presented from a comparative study of strain responses measured on steel beams at the intact conditions and after major earthquakes for evaluating seismically induced beam fractures.

(3) The sensitivity of the strain-based damage index to measurement environments and various structural parameters is investigated. It includes the independency of the damage index on input excitations and vibrational modes, the influence of sensor location on the damage index, the effect of interaction between multiple damages on the damage index, and the general applicability of the damage index.

(4) A closed-form expression of damage curve for single damage condition is derived to quantify the damage extent of beam fractures. The damage curve expresses the damage index as a function of reduction in bending stiffness of the fractured section.

(5) A decoupling method of estimating the damage index for multiple damages is presented for damage quantification of multiple beam fractures using the presented damage curve.

1.3 Organization

The dissertation comprises seven chapters. Chapter 1 introduces the background and motivation, and aims of the research. Chapters 2 to 6 contain the main contents of the thesis: (1) scheme of localized damage evaluation with wireless piezoelectric strain sensing; (2) general formulation of strain-based damage index; (3) sensitivity study of the damage index; (4) derivation of damage curve for single damage condition; (5) decoupling method of estimating damage index for multiple damages. Chapter 7 summarizes the main findings of the dissertation.

Chapter 2 introduces the conceptual scheme of localized damage evaluation using wireless piezoelectric strain sensing. Local damage such as brittle fracture at beam ends changes the

distribution of bending moments in steel moment-resisting frames. When frames behave linearly, the bending moments can be estimated by measuring strain responses. Thus, local damage can be identified from strain information. In the scheme, a wireless strain sensing system formed of a wireless sensor network and polyvinylidene fluoride strain sensors is designed to monitor strain responses. Moreover, the pre-identification of damage-prone region in steel frames using demand prediction methods is discussed.

Chapter 3 presents a strain-based damage index that is capable of evaluating seismically induced beam fractures in steel moment-resisting frames. The damage index is formulated from a comparative study of dynamic strain responses of steel beams monitored under ambient vibration before and after earthquakes. Then, a step-by-step signal processing procedure for extracting the damage index is presented. Finally, the effectiveness of the damage index and the associated wireless strain sensing system are examined with a series of vibration tests using a five-story steel frame testbed.

Chapter 4 further investigates the sensitivity of the presented damage index to measurement environments and various structural parameters. The sensitivity of the damage index is examined through numerical studies with a nine-story steel moment-resisting frame and experimental studies using the five-story steel frame testbed.

Chapter 5 presents a closed-form expression of damage curve where strain-based damage index is a function of reduction in beam bending stiffness induced by fracture. The damage curve allows quantitative evaluation on earthquake-induced fractures on beams. In addition, this chapter demonstrates that the presented damage curve is generally applicable for common multi-story multi-bay steel moment-resisting frames. The effectiveness of the damage curve is verified numerically using a nine-story steel moment-resisting frame model and experimentally using the one-quarter-scale five-story steel frame testbed.

Chapter 6 presents a decoupling method of estimating damage index for multiple beam fractures in order to quantify their damage extents using the damage curve presented in Chapter 5. Firstly, the mechanism of moment release and influence is demonstrated with a simple sub-frame. Then, the framework and algorithm of the decoupling method is presented. Finally, the effectiveness of the decoupling method is verified numerically through a nine-story steel moment-resisting frame and experimentally using the five-story steel frame testbed.

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CHAPTER 2

Scheme of local damage evaluation using wireless piezoelectric strain sensing

2.1 Overview

In steel moment-resisting frames, local damage such as seismically-induced fractures on steel beams changes the distribution of bending moments sustained by members. The moment distribution is independent of external loadings when it is evaluated at a natural vibrational mode. In practice, for the frames behaving linearly, the bending moments can be estimated by measuring strain responses in the members. Thus, local damage can be evaluated by monitoring strain responses. This chapter introduces a scheme of seismically-induced local damage evaluation using wireless piezoelectric strain sensing. Firstly, the influence of local damage on moment distributions in steel frames is illustrated using a simple frame. Then, the scheme of local damage evaluation is presented.

2.2 Influence of local damage on moment distribution

Inclusion of local damage on steel beams reduces the bending moments resisted by the damaged beams, which attributes mainly to the decreases of the bending stiffness of the beams. The following analytical study for a simple frame demonstrates a quantitative relationship between the reduction in the modal bending moment and a beam fracture.

A two-story one-bay frame shown in Fig. 2.1 subjected to earthquake excitation $\ddot{u}_g(t)$ is considered, where m_i ($i=1,2$) denotes the lumped mass for each floor; I_b and I_c are the second moment of inertia of the beams and columns, respectively; E is Young's modulus; h denotes the height of each story; and L denotes the width of the frame. The bending moments due to a natural

mode are obtained using the equivalent static forces method [1]. At any instant of time t , the equivalent static forces $\mathbf{f}_s(t) = [f_1(t), f_2(t)]$ associated with a natural mode are the external forces that act on the frame as illustrated in Fig. 2.1.

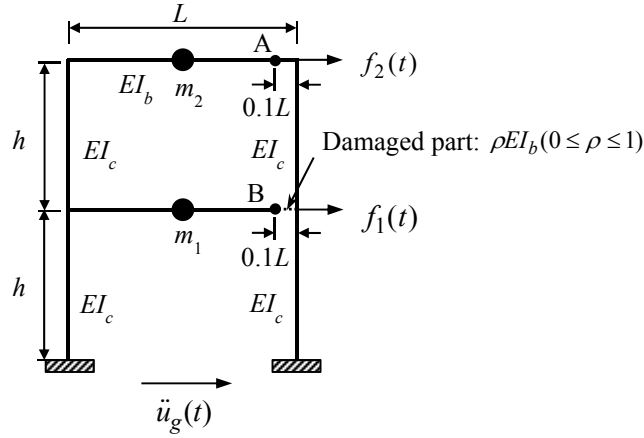


Fig. 2.1 Two-story frame model

To simplify the formulation, we note that

$$L = a \cdot h \quad (a > 1), \quad (2.1a)$$

$$I_b = k \cdot I_c \quad (0 < k < 1). \quad (2.1b)$$

where a denotes the aspect ratio of the frame and k denotes the stiffness ratio between the beams and columns. Assuming that the frame behaves linearly under small amplitude excitations, the bending moments at points A and B at any instant of time t are calculated by the force method of structural analysis as

$$M_A(t) = A_1 f_1(t) + A_2 f_2(t), \quad (2.2a)$$

$$M_B(t) = A_3 f_1(t) + A_4 f_2(t). \quad (2.2b)$$

where A_1 , A_2 , A_3 , and A_4 are:

$$A_1 = \frac{6ahk}{180k^2 + 90ak + 5a^2}, \quad (2.3a)$$

$$A_2 = \frac{36hk^2 + 24ahk}{180k^2 + 90ak + 5a^2}, \quad (2.3b)$$

$$A_3 = \frac{36hk^2 + 6ahk}{180k^2 + 90ak + 5a^2}, \quad (2.3c)$$

$$A_4 = \frac{72hk^2 + 18ahk}{180k^2 + 90ak + 5a^2}. \quad (2.3d)$$

In modal analysis, the equivalent static forces $\mathbf{f}_s(t)$ associated with a natural mode (e.g., the i th mode, $i = 1, 2$) can be expressed as

$$\mathbf{f}_s(t) = \omega_i^2 \mathbf{m} \Phi_i q^i(t), \text{ or } \begin{bmatrix} f_1(t) \\ f_2(t) \end{bmatrix} = \omega_i^2 \begin{bmatrix} m_1 & \\ & m_2 \end{bmatrix} \begin{bmatrix} \varphi_1^i \\ \varphi_2^i \end{bmatrix} q^i(t) \quad (2.4)$$

where ω_i and Φ_i are the i th modal frequency and mode shape, respectively, and $q^i(t)$ is the modal coordinate for the i th mode. Equation (2.4) indicates that the relationship between equivalent static forces corresponding to the first and second floor masses is constant for a linear vibrating system where the mode shape is invariant:

$$f_2(t) = u^i \cdot f_1(t) \quad (2.5)$$

where $u^i = m_2 \varphi_2^i / m_1 \varphi_1^i$.

The bending moments at points A and B due to the i th mode at any instant of time t are expressed as follows.

$$M_A(t) = -\frac{(24a + 36k)hku^i + 6ahk}{180k^2 + 90ak + 5a^2} \omega_i^2 m_1 \varphi_1^i q^i(t), \quad (2.6a)$$

$$M_B(t) = -\frac{6hk(3a + 12k)u^i + 6hk(a + 6k)}{180k^2 + 90ak + 5a^2} \omega_i^2 m_1 \varphi_1^i q^i(t). \quad (2.6b)$$

Equations (2.6) imply that the bending moments due to the i th mode are proportional to the i th modal coordinate $q^i(t)$, and thus the normalization of the bending moments with a reference point can remove the effects of external excitations. Thus, the ratio of the bending moments between points A and B (point A as a reference point), in other words, the normalized bending moment at point B, is used to analyze the influence of local damage.

$$R = \frac{M_B(t)}{M_A(t)} = \frac{(3a+12k)u^i + (a+6k)}{(4a+6k)u^i + a} \quad (2.7)$$

where R relates only to structural properties.

Now consider the same frame with a beam that has sustained damage (see Fig. 2.1). The damaged part near point B is simulated by the reduction of the second moment of inertia with a reduction factor of ρ . The length of damaged part is the 1/10th of the beam length. The normalized bending moments at point B associated with the i th mode is reformulated as,

$$R_d = \frac{C_1 u_d^i + C_2}{C_3 u_d^i + C_4} \quad (2.8)$$

where C_j ($j = 1, \dots, 4$) are coefficients given by $\alpha_j \rho^2 + \beta_j \rho + \lambda_j$ (α_j , β_j , and λ_j are functions of structural parameters a and k). u_d^i is the relationship between the equivalent static forces corresponding to the first and second floor masses in the damaged model. Note that R_d also has no relationship with the loadings.

The normalized bending moment at point B associated with the i th mode decreases by the damage at beam end as,

$$\Delta R = \frac{R_d - R}{R} \times 100\% \quad (2.9)$$

The relationship between the reduction of the normalized bending moment at point B ΔR and the extent of the damage ρ is illustrated using an example. The properties of the beam and column sections are H-100 $\times$ 60 $\times$ 6 $\times$ 8 ($I_x = 2.33 \times 10^6 \text{ mm}^4$) and H-100 $\times$ 100 $\times$ 6 $\times$ 8 ($I_x = 3.69 \times 10^6 \text{ mm}^4$), respectively. The width of the frame is 2 m and the height of each story is 1 m. The floor mass $m_1 = m_2 = 1.4$ ton. The relationship between ΔR and ρ associated with the first mode is shown

in Fig. 2.2. In the plot, the horizontal axis is the decrease of the second moment of inertia, and the vertical axis is the reduction of the normalized bending moment at point B. As the second moment of inertia of the damaged part decreases, the reduction of the normalized bending moment at point B drops from 0 to -100%. This monotonic relation indicates that local damage can be detected and quantified by the reduction of normalized bending moment measured around the damage.

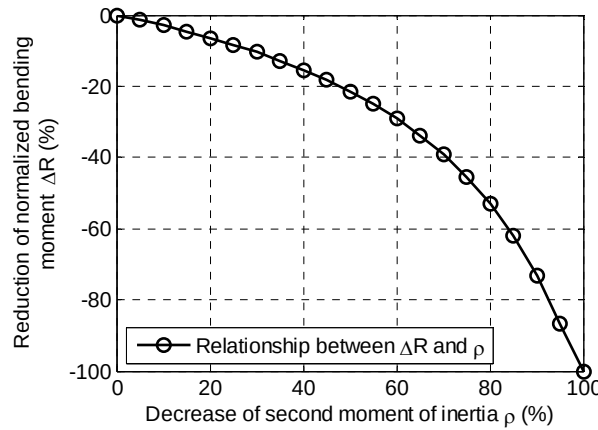


Fig. 2.2 Relationship between ΔR and ρ

2.3 Scheme of local damage evaluation

2.3.1 Concept

As illustrated in the preceding section, observation on the redistribution of bending moments provides quantitative information on the location and extent of damage, but it is not practical to measure bending moments in real buildings. Instead, this study considers the dynamic strain responses of members as sources for estimating the bending moments, assuming that the amplitude of the strain at a particular location in a member is proportional to the amplitude of the bending moment carried by the member. The amplitude of the strain is measured at an elastic part of the member with a reasonable distance away from the damaged nonlinear part.

Fig. 2.3 illustrates the conceptual scheme of seismic local damage evaluation by monitoring strain responses before and after earthquakes. As shown in the schematics, a wireless piezoelectric strain sensing system that consists of a dense array of polyvinylidene fluoride (PVDF) sensors (DT1-028k, Measurement Specialties, VA, USA) [2] interfaced with *Narada* wireless sensing units (Civionics, LLC, CO, USA) [3] is developed for measuring the strain responses of beams at the intact states and after earthquakes. The sensing system, including a reference sensor and detecting sensors, is deployed to monitor pre-identified damage-prone beams. The reference sensor for

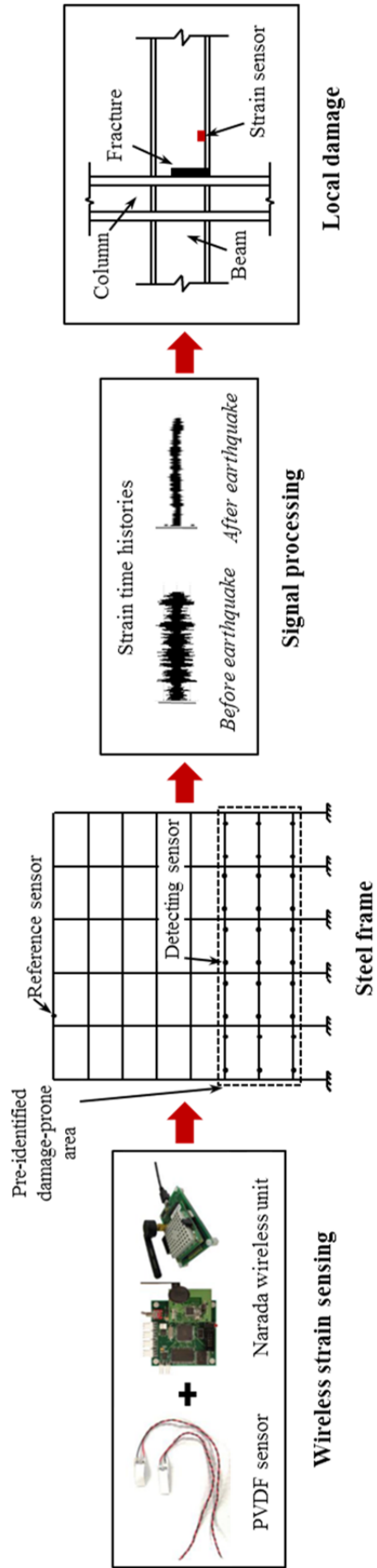


Fig. 2.3 Conceptual scheme of local damage evaluation

normalization is used to eliminate the effects of the excitations. The detecting sensor near probable damage is used to detect and quantify the local damage. The re-distribution of the bending moment associated with a natural mode is estimated from the dynamic strain response using signal processing techniques.

2.3.2 Wireless piezoelectric strain measurement

With the emergence of wireless sensing and piezoelectric materials technologies, one can form dense-array wireless strain sensing systems for large-scale civil structures with reasonable investments. Thus, this study develop a wireless piezoelectric strain measurement system particularly designed for monitoring steel moment-resisting frames using PVDF sensors and customized wireless sensing units.

The PVDF sensor comprises of a flexible film with silver ink screen printed electrodes covered by insulating urethane coating. While conventional foil strain gauges are resistive sensors and require a switch box to convert strain into voltage, the PVDF sensors generate electric charge and produce voltage in proportion to compressive or tensile mechanical stress or strain, making it an ideal dynamic strain gauge. The biggest advantage of the PVDF film is its high sensitivity about 60 dB higher than the voltage output of a foil strain gage. This enables the PVDF sensor to measure the strain time history in steel beams under small dynamic loadings such as ambient vibrations. By removing a static load from a fixed-supported single beam, the calibration factor of PVDF sensor attached to a steel beam was estimated approximately as 12mV per micro strain. The other notable advantages of the PVDF sensor include 1) flexibility for easy deployment, 2) a broad-band operating frequency throughout the high audio (>1 kHz) and ultrasonic (up to 100 MHz) range, 3) long term durability with an operating temperature range of -40 to $+60^{\circ}\text{C}$.

Due to these advantages, PVDF films have been utilized in applications of local damage detection of civil and mechanical structures in recent decades [4-8]. Wang et al. (1999) proposed an in-situ method for monitoring the tension of stayed cables of cable-stayed bridges through embedding PVDF sensors into the cables [4]. Yu et al. (2011) presented a wireless measurement system with PVDF sensors for monitoring the structural impact responses and the detection of damage in a bridge model [7].

An emerging wireless technology has great potential for reducing the cost and effort associated with the installation of the sensing system. The *Narada* wireless unit is designed with an on-board analog-to-digital converter (ADC) supporting high-speed data collection (up to 100 kHz) on four sensor channels [3]. The resolution of the ADC is 16-bits which is often considered a minimum

resolution for ambient response monitoring. The *Narada* communicates on the 2.4 GHz IEEE 802.15.4 radio standard (IEEE 2006) using a Texas Instruments CC2420 transceiver. The output power of the CC2420 transceiver can be varied from 0 to -25 dB with the highest power setting (0 dB) achieving a line-of-sight communication range of approximately 100 m when a 2.2 dBi swivel antenna (Titanis 2.4 GHz Swivel SMA Antenna) is equipped. The communication range can be further extended with the use of a high gain antenna.

2.3.3 Pre-identified damage-prone region and reference point

Steel moment-resisting frames have been popular in many regions of high seismicity. Most codes and design guidelines adopt the strong-column and weak-beam philosophy in design of steel moment-resisting frames. This design philosophy enhances overall seismic resistance and prevents development of a soft-story mechanism in a multistory frame. Thus, properly designed steel frames are prone to suffer seismic damage at beams rather than at columns. This allows the local damage evaluation system deployed only to beams.

Seismic damage to beam-to-column connections in the steel moment-resisting frames relates to story deformation demands, i.e., maximum inter-story drift. Thus, damage-prone beams can be pre-identified using the demand prediction methods, such as inelastic time history analysis and pushover analysis. This will greatly reduce the density of the wireless strain sensing system. Several floors likely sustaining large deformation are particularly monitored.

Reference point needs to be located in the undamaged floor for evaluating damage on beams. A floor with small deformation (e.g., the roof) is recommended for the reference point where the concrete slabs and beams at the floor remain undamaged. A twenty-story steel moment-resisting frame was studied to illustrate the procedure to identify damage-prone floors and to select undamaged floors for a reference point. The twenty-story steel moment-resisting frame designed according to the pre-Northridge design practice in Los Angeles, California was used. This frame was intensively studied in the SAC steel project and whose details were in FEMA-355C [9]. Inelastic time history analyses were conducted to predict the maximum story drifts under earthquake ground motions. The analysis was conducted using the finite element analysis software, SAP2000. The frame subjected to the 2/50 set of SAC Los Angeles ground motion records was studied [9]. Fig. 2.4 illustrates the maximum story drifts under twenty ground motions, and the median and 84th percentile values. The maximum story drifts had a very large dispersion in the lower six stories, and a smaller dispersion in the upper stories. The median and 84th percentile values of the maximum story drifts were larger at the lower six stories than at the upper stories. This

indicates that the beams at the lower six stories were prone to suffer damage and the upper floors with small story drift such as the roof were appropriate for a reference point.

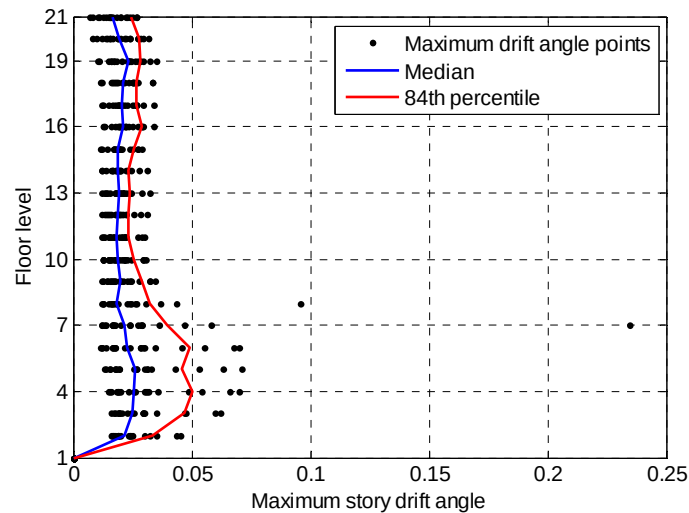


Fig. 2.4 Maximum story drift demands for LA 20-story Pre-Northridge steel moment-resisting frame under 2/50 set of ground motions

2.4 Summary

In this chapter, the influence of local damage on the moment distribution in a steel frame was analyzed using a simple frame example. The normalized bending moment associated with a natural mode relates only to structural properties and has no relationship with external excitations. The analytical study demonstrated that local damage can be detected and quantified by the reduction of normalized bending moment associated with a natural mode measured around the damage. Base on the finding, a conceptual scheme of local damage evaluation using wireless piezoelectric strain sensors was presented.

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CHAPTER 3

Strain-based damage index for evaluating seismically induced beam fracture

3.1 Overview

The primary objective of this chapter is to present and formulate a damage index from strain responses that is capable of evaluating seismically induced beam fractures in steel moment-resisting frames. In this chapter, first a novel damage index based on the monitoring of dynamic strain responses of steel beams under ambient vibration before and after earthquakes is formulated. Then, a step-by-step signal processing procedure for extracting the damage index is presented. Finally, the effectiveness of the damage index and the associated wireless strain sensing system are examined with a series of vibration tests using a five-story steel frame testbed.

3.2 General formulation of damage index

This section formulates a damage index directly from the bending strain responses of steel beams. It is assumed that the amplitude of the bending strain at elastic part of the beams (i.e., outside of the beam-end region that may sustain plastic deformation) is proportional to the amplitude of the bending moment carried by the beam. The damage index is defined as the ratio of the bending strain responses of beams in undamaged and damaged frames. The strain responses are obtained under small dynamic loads (e.g., ambient vibrations and minor earthquake ground motions).

When an n -story steel moment-resisting frame is subject to lateral dynamic loads such as ground motions, at any instant of time t , the equivalent static forces $F(t)=[f_1(t), f_2(t), \dots, f_i(t), \dots, f_{n-1}(t), f_n(t)]^T$ act on the frame as external forces, as illustrated in

Fig. 3.1. Suppose the frame vibrates linearly under small-amplitude excitations, at instant of time t , a bending strain response measured at any beam can be formulated as

$$\varepsilon(t) = \alpha_1 f_1(t) + \alpha_2 f_2(t) + \cdots + \alpha_i f_i(t) + \cdots + \alpha_{n-1} f_{n-1}(t) + \alpha_n f_n(t) = \sum_{i=1}^n \alpha_i f_i(t), \quad (3.1)$$

where α_i ($i = 1, \dots, n$) is an influence factor of the equivalent static force $f_i(t)$, which relates only to the structural properties (i.e., material and geometric properties) and is unaffected by the characteristics of external excitations. Since the equivalent static forces associated with the j th mode vibration are

$$F_j(t) = \omega_j^2 \mathbf{M} \Phi_j q_j(t), \quad (3.2)$$

the bending strain response of the beam associated with the j th mode is expressed as

$$\varepsilon_j(t) = \omega_j^2 q_j(t) \sum_{i=1}^n \alpha_i m_i \phi_{ij}, \quad (3.3)$$

where ω_j and $\Phi_j = [\phi_{1j}, \phi_{2j}, \dots, \phi_{ij}, \dots, \phi_{n-1j}, \phi_{nj}]^T$ are the j th modal frequency and mode shape; $\mathbf{M} = \text{diag}(m_1, m_2, \dots, m_i, \dots, m_{n-1}, m_n)$ is the mass matrix for the frame in which m_i ($i = 1, \dots, n$) is the floor mass; and $q_j(t)$ is the modal coordinate for the j th mode.

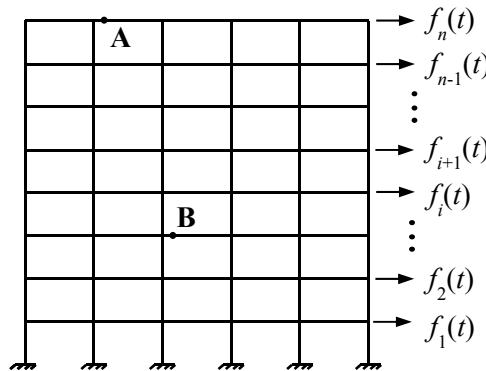


Fig. 3.1 n -story steel moment-resisting frame under equivalent static forces

Now consider the ratio of the bending strain responses of beams associated with the j th mode at any two different positions A and B (position A as a reference point) at any instant time t :

$$\frac{\varepsilon_j^B(t)}{\varepsilon_j^A(t)} = \frac{\omega_j^2 q_j(t) \sum_{i=1}^n \alpha_i^B m_i \phi_{ij}}{\omega_j^2 q_j(t) \sum_{i=1}^n \alpha_i^A m_i \phi_{ij}} = \frac{\sum_{i=1}^n \alpha_i^B m_i \phi_{ij}}{\sum_{i=1}^n \alpha_i^A m_i \phi_{ij}}. \quad (3.4)$$

The obtained ratio of the bending strain responses only relates to the structural properties of the frame, and has no relationship with external excitations.

In practice, errors or uncertainties in data measurement and signal processing (e.g., time-synchronization errors, outliers, and distortion with filters) affect the instantaneous bending strain responses associated with the j th mode vibration, which are estimated as a peak in the frequency domain response, especially when the signal-to-noise (S/N) ratio is not large with small-amplitude excitations. Therefore, given the bending strain time histories with a time interval of Δt (each including k points) at two positions A and B, the ratio of the root mean square (RMS) of these two time histories under the j th mode vibration is considered as

$$\frac{(\varepsilon_j^B)_{RMS}}{(\varepsilon_j^A)_{RMS}} = \frac{\sqrt{\frac{1}{k} \sum_{p=0}^{p=k-1} [\varepsilon_j^B(p\Delta t)]^2}}{\sqrt{\frac{1}{k} \sum_{p=0}^{p=k-1} [\varepsilon_j^A(p\Delta t)]^2}} = \frac{\sqrt{\frac{1}{k} \left(\omega_j^2 \sum_{i=1}^n \alpha_i^B m_i \phi_{ij} \right)^2 \sum_{p=0}^{p=k-1} [q_j(p\Delta t)]^2}}{\sqrt{\frac{1}{k} \left(\omega_j^2 \sum_{i=1}^n \alpha_i^A m_i \phi_{ij} \right)^2 \sum_{p=0}^{p=k-1} [q_j(p\Delta t)]^2}} = \frac{\sum_{i=1}^n \alpha_i^B m_i \phi_{ij}}{\sum_{i=1}^n \alpha_i^A m_i \phi_{ij}}. \quad (3.5)$$

The RMS ratio for the two bending strain time histories in Equation (3.5) equals the instantaneous relative bending strain in Equation (3.4) if there are no errors or uncertainties.

Two strain sensors S1 and S2 are placed on the bottom flanges of beams at positions A and B in Fig. 3.1, respectively, to detect seismic damage at the beam-end near position B. S1 at position A is used as a reference sensor, which is assumed to be far away from damaged beams in the frame to guarantee that the bending strain at S1 is hardly affected by the seismic damage. S2 is near the damage as a detecting sensor. In the undamaged condition, the relative RMS value of the bending strain time histories at the two sensors S1 and S2 associated with the j th mode is expressed as

$$R_j = \frac{(\varepsilon_j^{S2})_{RMS}}{(\varepsilon_j^{S1})_{RMS}} = \frac{\sum_{i=1}^n \alpha_i^{S2} m_i \phi_{ij}}{\sum_{i=1}^n \alpha_i^{S1} m_i \phi_{ij}}, \quad (3.6)$$

while under the damaged condition, it is expressed as

$$R_j^d = \frac{(\bar{\varepsilon}_j^{S2})_{RMS}}{(\bar{\varepsilon}_j^{S1})_{RMS}} = \frac{\sum_{i=1}^n \bar{\alpha}_i^{S2} m_i \bar{\phi}_{ij}}{\sum_{i=1}^n \bar{\alpha}_i^{S1} m_i \bar{\phi}_{ij}}, \quad (3.7)$$

where the variables with top bars are for the damaged condition. Finally, the damage index DI based on the bending strain responses of beams for detecting seismic damage on beams in steel moment-resisting frames can be defined as follows

$$DI = \frac{R_j^d - R_j}{R_j} \times 100\%. \quad (3.8)$$

Note that fracture at beam-ends has two influential factors on the bending strain responses measured by S2: (1) the bending strain decreases because of the reduction in the bending moment resisted by the damaged beam; and (2) the bending strain is affected by local strain redistribution around the fractured section. If sensor S2 is located in the region unaffected by the local strain redistribution, DI is proportional to the reduction of the bending moment.

3.3 Signal processing for extracting damage index

In signal processing, the strain time histories associated with a vibration mode is obtained by applying a narrow-band-pass filter at the frequency of interest as the transient strain responses of the structural members in the frame are a combination of responses associated with the various vibration modes of the frame. Fig. 3.2 shows the flowchart of the step-by-step procedure for calculating the damage index DI . First, raw dynamic strain data of steel beams is preprocessed with data cleaning techniques (e.g., the removal of drifts and false points). Second, one mode of the steel moment-resisting frame is selected and the strain responses associated with the selected mode are extracted using band-pass filters. Third, the RMS values of the filtered strain data are calculated and then normalized by the RMS value of a reference position. Finally, damage information (existence, location, and extent) is extracted from the damage index DI calculated in Equation (3.8) at each detecting sensor.

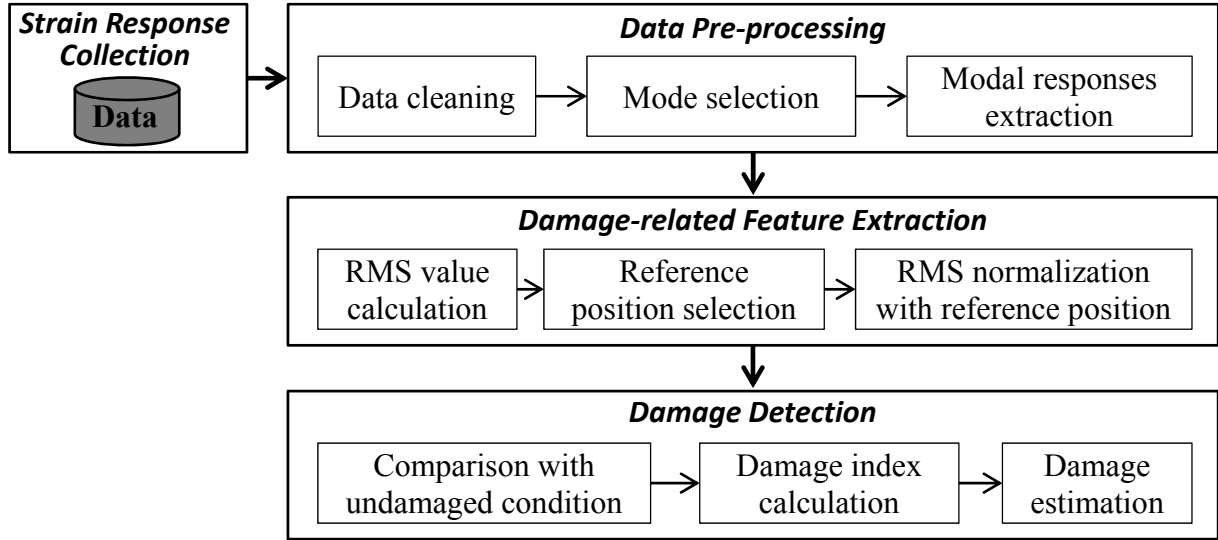


Fig. 3.2 Step-by-step procedure to extract damage index

3.4 Five-story steel frame testbed

The performance of the developed local damage evaluation strategy was verified with a steel frame testbed. The study using a sensing system deployed on a real building is ideal but it is very rare to acquire an opportunity to simulate damage in buildings. Therefore, a five-story steel frame testbed that accommodates earthquake-induced fracture at beam ends was constructed at the Disaster Prevention Research Institute (DPRI), Kyoto University, for promoting structural health monitoring related studies.

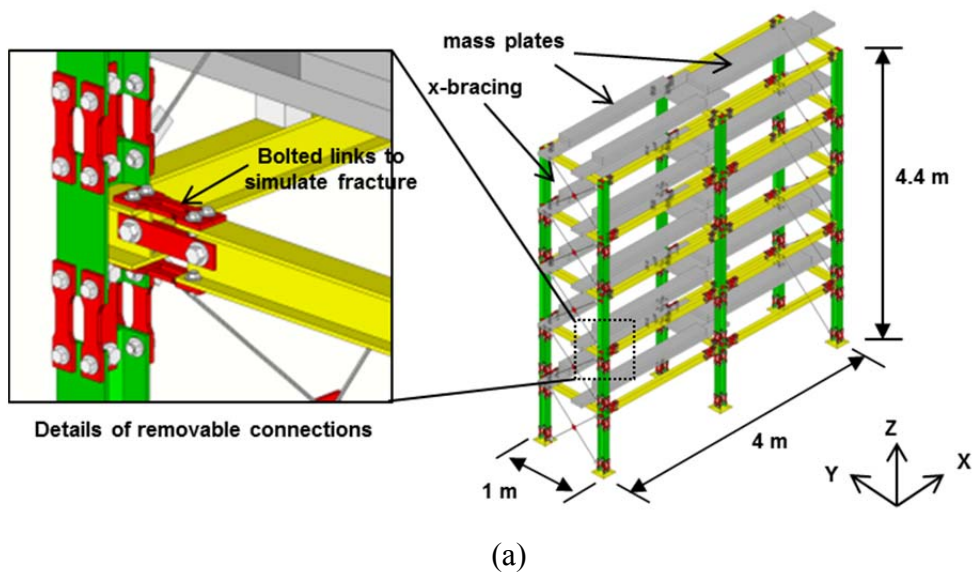
3.4.1 Design of testbed

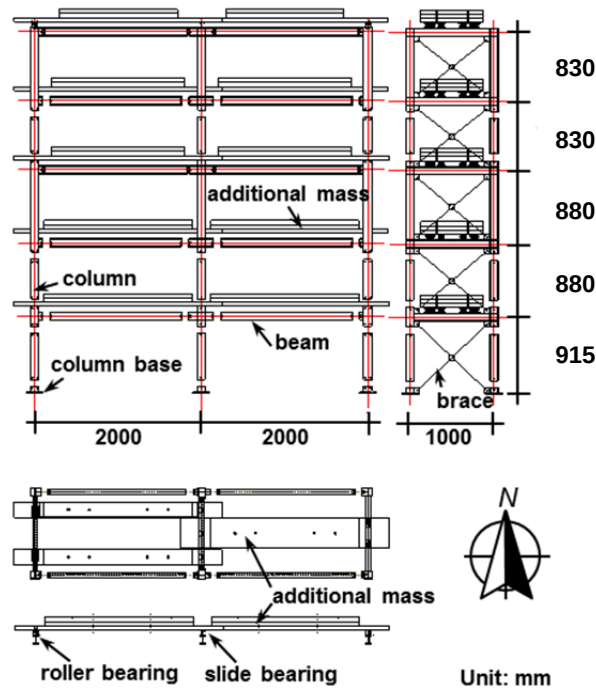
A scaled steel frame (Fig. 3.3(a)) was constructed to simulate typical seismic damage around beam-to-column connections. The overall dimensions of the steel frame were $1.0 \times 4.0 \times 4.4$ m. The plan of the frame was one bay by two bays. The design of the steel frame referred to the full-scale test frame standing at DPRI, Kyoto University [1]. The dimension scaling factor for the testbed frame was chosen as 4 considering the height limitation in the structural laboratory. Based on the law of similitude, all the dimensions in the prototype were scaled down according to the scaling factor. Since the acceleration due to gravity g was common to both the original and scaled structures, the scaling factor of the horizontal acceleration was chosen as unity. The member sizes in the steel frame are summarized in Table 3.1. All members were made of the SS400 steel.

To satisfy the scaling law, additional steel masses were attached to the transverse beams with simply-supported boundary conditions (Fig. 3.3(b)). One end of the additional mass was pin-connected to a transverse beam using a roller bearing and the other end of the mass was sliding-

supported by a Teflon plate so that the high stiffness of the additional mass did not constrain the deformation of the longitudinal beams. With the additional mass, the natural frequencies of the testbed frame became those of the original frame multiplied by the square root of the dimension scaling factor.

Seismic damage was simulated at the removable steel connections located at the second, third, and fifth floors. As seen in the enlarged drawing in Fig. 3.3(a), the longitudinal beams in the x direction and column were connected to a joint using removable steel links and structural bolts. The dimensions of the steel links were defined so that the second moments of inertia at the removable connections were equal to those of the connected beams or columns.





(b)

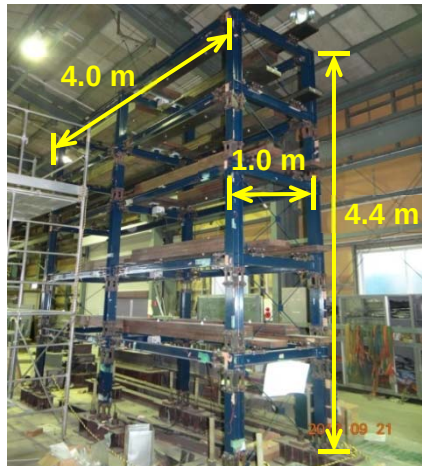
Fig. 3.3 Steel frame testbed: (a) isometric view; (b) plan and elevations

Table 3.1 Member sizes of the steel frame testbed (unit: mm)

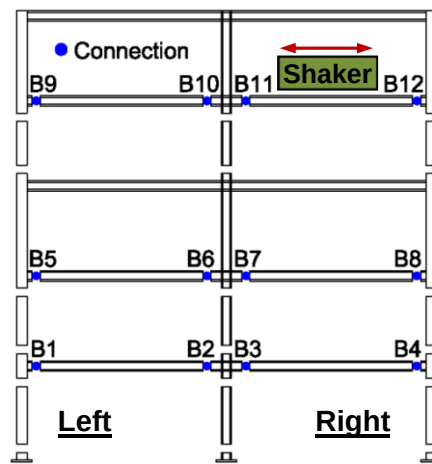
Member	Location	Size
Beam	2nd to 6th floors	H-100 × 60 × 6 × 8
Column	2nd to 6th floors	H-100 × 100 × 6 × 8
Brace	1st to 5th stories	M10 steel rod, x-bracing

3.4.2 Experiment views

The overview of the steel frame testbed is shown in Fig. 3.4(a). There were twelve removable connections at beam ends, i.e., connections B1 to B12 (see Fig. 3.4(b)), in each longitudinal frame. The removable steel connection consisted of four links at the flanges and one pair of links at the web (Fig. 3.4(c and d)). By removing or changing the links, fracture damage was simulated (see Fig. 3.4(d)). Fig. 3.4(e) illustrates the cross-section of the removable steel connection. In vibrational testing, the steel frame was excited by a modal shaker (APS-113, APS Dynamics) that was firmly fixed to the steel mass plate at the fifth floor (Fig. 3.4(f)). Fig. 3.5 shows the wireless measurement system deployed on the testbed to measure the dynamic strain responses of beams.



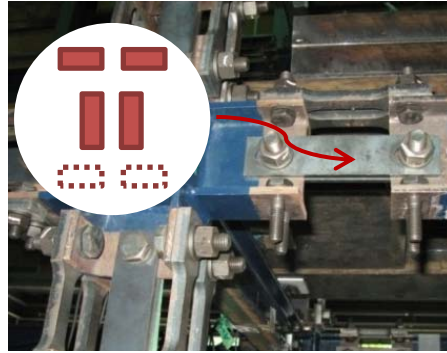
(a)



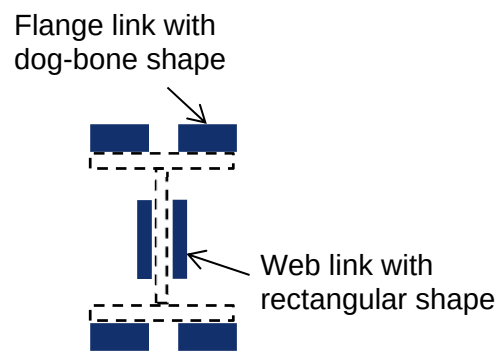
(b)



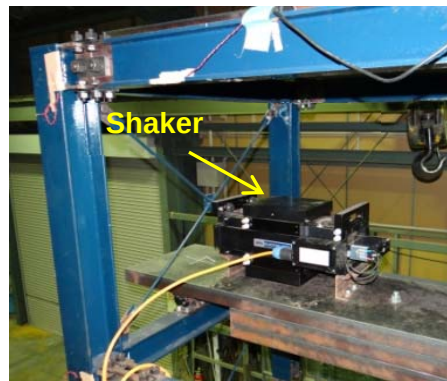
(c)



(d)

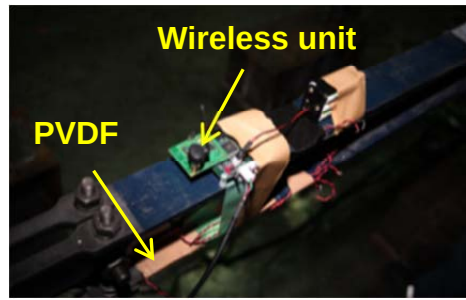


(e)

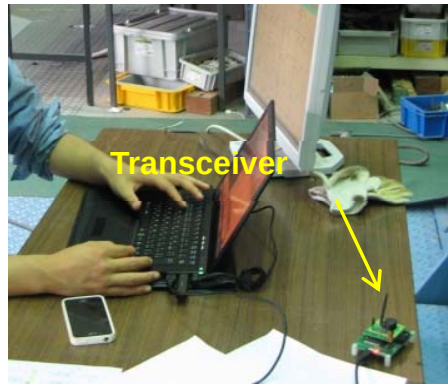


(f)

Fig. 3.4 Experiment views: (a) overview; (b) beam removable connections; (c) beam-column connection; (d) simulated damage; (e) cross-section of removable connection; (f) modal shaker



(a)



(b)

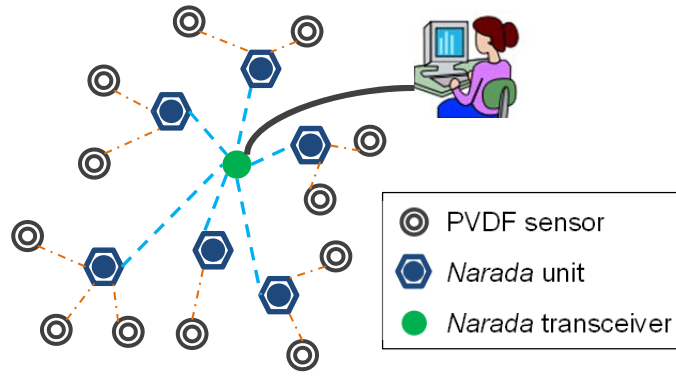
Fig. 3.5 Wireless measurement system deployed on the testbed: (a) PVDF sensor and wireless unit; (b) transceiver

3.5 Preliminary verifications

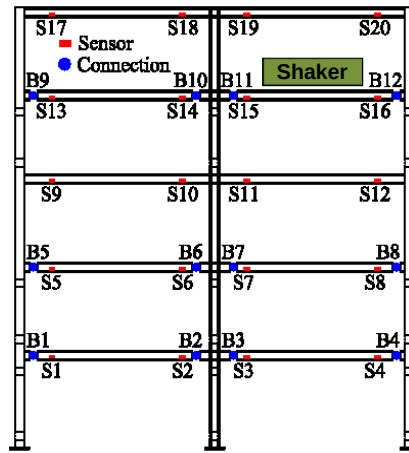
The effectiveness of the damage index and the associated wireless strain sensing system were investigated with a series of small-amplitude vibrational tests on the five-story steel frame testbed.

3.5.1 Measurement system

The wireless strain sensing system that consisted of twenty PVDF sensors (i.e., S1 to S20) interfaced with *Narada* wireless sensing units was deployed on one longitudinal frame of the five-story frame testbed (Fig. 3.6). Fig. 3.6(a) illustrates the structure of the sensing network. Strain signals measured by PVDF sensors were acquired with wireless units, and then were transmitted to a wireless transceiver. Fig. 3.6(b) shows the locations of the PVDF sensors. All sensors were attached with strong adhesive on one side of the bottom flange of beams at 330 mm away from the center line of columns.



(a)



(b)

Fig. 3.6 Wireless strain sensing system deployed on the testbed: (a) sensing network; (b) sensor location

3.5.2 Excitations

The steel testbed frame was excited at the fifth floor using a modal shaker (APS-113, APS Dynamics) that was fixed to the steel mass plate by four machine bolts (Fig. 3.4(f)). For each structural condition, the steel frame was excited in the longitudinal direction using three loadings: (1) ambient excitation (AmbE); (2) small amplitude white noise (WN1); and (3) relatively large amplitude white noise (WN2). In the structural laboratory where the testbed frame located, the level of ambient excitation mainly caused by ground microtremor was around 0.49 cm/s^2 in RMS at the top floor. When the two white noise excitations with the frequency range of 1-50 Hz were input for the undamaged condition, the acceleration responses of the top floor were 3.32 and 8.45 cm/s^2 for WN1 and WN2 in RMS, respectively.

3.5.3 Damage patterns

Two levels of seismic fracture damage, i.e., entire bottom flange fracture, and entire bottom flange and web fracture were simulated to investigate the effectiveness of the damage index. Fig. 3.7 illustrates the simulated damage, and the geometric relationship of the defined damage categories to the position of PVDF sensors. In the damage level 1 (L1), all two links of the bottom flange of the connections were removed. In the damage level 2 (L2), a web link in addition to bottom flange links were removed. Table 3.2 summarizes these damage categories and their reduction in the second moment of inertia about the strong axis of the beam section.

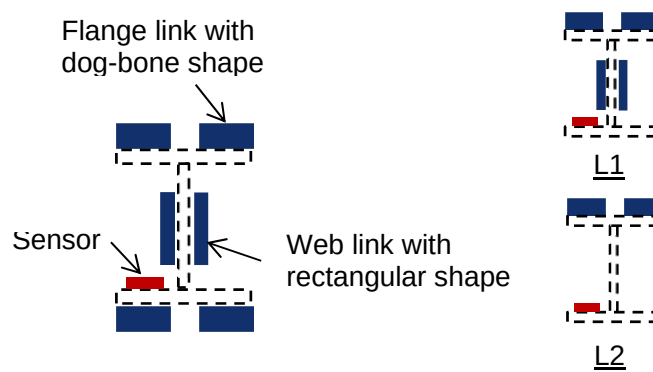


Fig. 3.7 Two levels of fracture damage

Table 3.2 Damage patterns and their descriptions

Damage patterns	Descriptions	Reduction of EI (%)
L1	All links of bottom flange are removed	68.5
L2	All links of bottom flange and web are removed	99.8

3.5.4 Damage cases

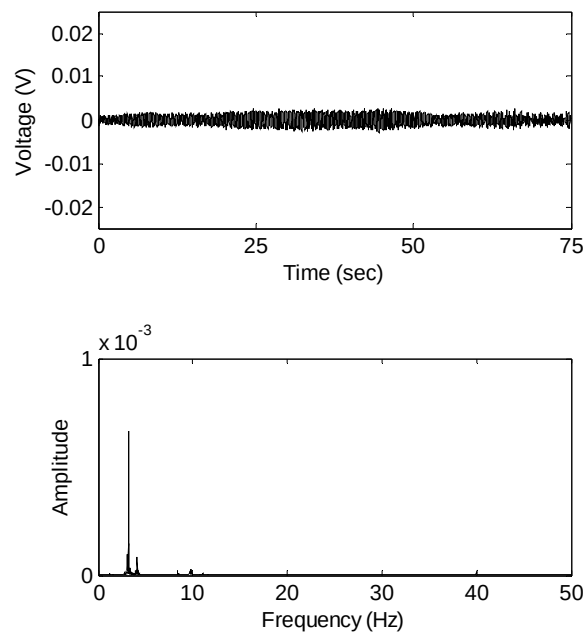
Six damage cases were considered for evaluating the performance of the damage index (Table 3.3). Case 1 denotes the undamaged state of the frame. In Case 2 and 3, damage L1 and L2 were simulated at B2. Damage L1 and L2 were simulated at B10 in Case 4 and 5. In these cases, only individual damage was studied. Case 6 included multiple fractures at the second floor. Damage L2 was at both connections B2 and B4.

Table 3.3 Damage cases

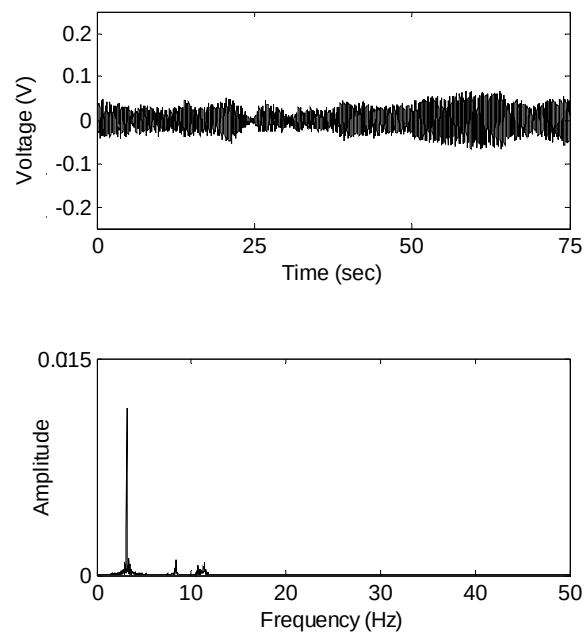
Damage cases	Locations of removable connections and associated damage patterns
Case 1	Undamaged state
Case 2	B2 (L1)
Case 3	B2 (L2)
Case 4	B10 (L1)
Case 5	B10 (L2)
Case 6	B2 (L2), B4 (L2)

3.5.5 Test results

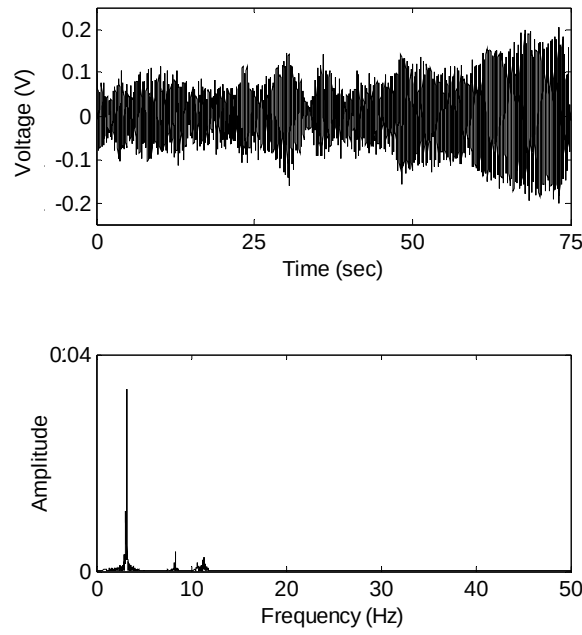
In each measurement, a strain time history was measured for 75 sec with a sampling rate of 100 Hz. Fig. 3.8 shows the dynamic strain responses at the undamaged condition in voltage and their amplitude spectra at the beam of the second floor (S2 in Fig. 3.6(b)) under three excitations. The dynamic characteristics of the testbed frame were evaluated from the floor acceleration responses under large amplitude white noise excitation (WN2) 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 even under ambient vibrations.



(a)



(b)



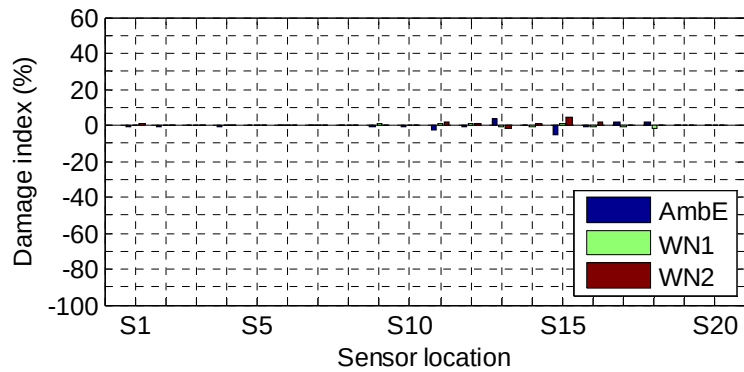
(c)

Fig. 3.8 Measured signals at S2: (a) AmbE; (b) WN1; (c) WN2

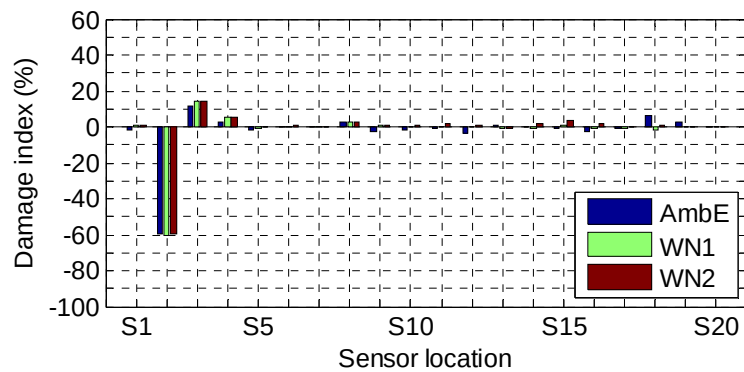
The strain response associated with the first mode was used for computing the damage index. Considering the change in the first mode frequency with relation to the extent of damage, a band-pass filter of 2.7–3.3 Hz was selected to obtain the dynamic strain associated with the first mode. Then, the RMS values of each of the filtered strain responses was normalized by the RMS values of the reference strain data measured at the beam of the top floor (i.e., S20 in Fig. 3.6(b)).

The damage index at all the sensor locations was evaluated for the undamaged condition and the other five damaged cases described in Table 3.3. Fig. 3.9 shows the damage index results for the undamaged and damaged cases under three different excitations. At the undamaged condition, the variation of the damage index was less than 7% for all the input excitations (Fig. 3.9(a)). Fig. 3.9(b) shows the results for Case 2 where the entire bottom flange links were removed at B2 near S2. The damage index of –60% at S2 clearly indicates the existence of severe damage at B2. In addition, the damage index of +18% at S3 also indicates damage at nearby connections. Fig. 3.9(c) shows the results for Case 3 where all the links of the bottom flange and web were removed at B2. With the removal of both the web and the flange links, the damage index at S2 decreased by 90%, indicating severe damage at B2. As in Case 2, the damage index at the nearby sensors increased (by 35% at S3). Similarly, the damage index of about –60% and –90% in Case 4 and 5 corresponded to the damage L1 and L2 at the connection B10 (Fig. 3.9(d and e)). When all bottom flange links as well

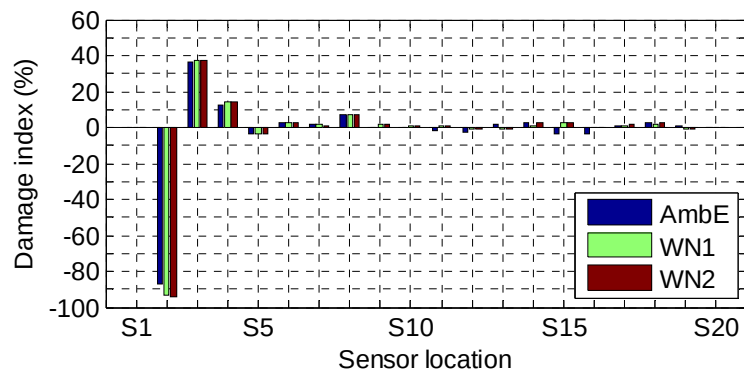
as web links were removed at B2 and B4 in Case 6, the damage index was about -90% for these damages (Fig. 3.9(f)). In summary, the distribution of damage index clearly indicates the damaged connections in all the considered damage cases. In addition, the damage index values were able to apparently separate the two different damage extent, i.e., about -60% for damage L1, and -90% for damage L2.



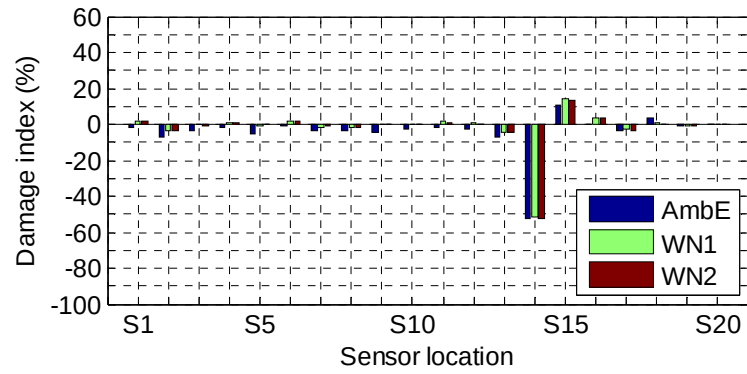
(a)



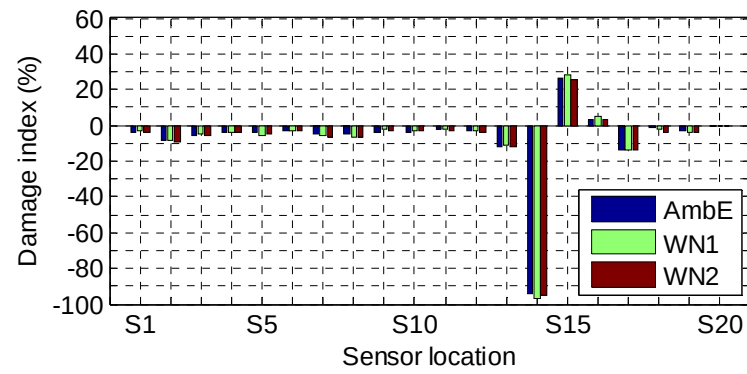
(b)



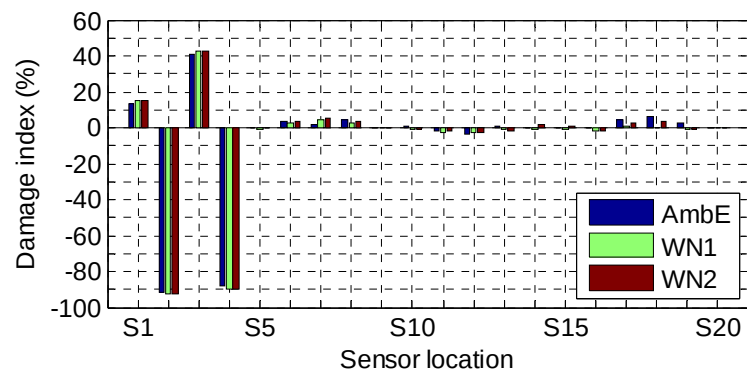
(c)



(d)



(e)



(f)

Fig. 3.9 Damage index: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6

The difference in damage index values was very small for the three excitation inputs, implying a weak dependency of damage index on the characteristics of the external excitations as mentioned in the preceding formulation of damage index. For example, in Case 5 (Fig. 3.9(e)) the maximum difference at S14 for the three excitation inputs was only 2.3%. For reference, Table 3.4 summarizes the associated changes in the natural frequencies that were estimated from the floor acceleration responses under large white noise excitation (WN2) using the FDD technique. The frequency change was less than 0.5% for Cases 2 to 5 and at most 5.4% for the severe damage in Case 6, from which the damage was not easily detected. The presented damage index derived from the strain responses has a much higher sensitivity than the index derived from the changes of the modal frequencies of the frame. This fact strongly indicates the advantages of the proposed damage evaluation method compared to the traditional methods.

Table 3.4 Change in frequencies for damaged cases

Modes	Undamaged	Case 2		Case 3		Case 4		Case 5		Case 6	
	f	f	diff.	f	diff.	f	diff.	f	diff.	f	diff.
	(Hz)	(Hz)	(%)	(Hz)	(%)	(Hz)	(%)	(Hz)	(%)	(Hz)	(%)
1st	3.16	3.15	-0.39	3.15	-0.39	3.15	-0.39	3.15	-0.39	2.99	-5.41
2nd	8.33	8.33	0.00	8.33	0.00	8.25	-0.88	8.25	-0.88	8.25	-0.88

3.6 Summary

This chapter presented the development of a damage index for evaluation of seismic damage to steel moment-resisting frames using dynamic strain responses. The effectiveness of the damage index and the performance of the wireless strain sensing system were verified using a quarter-scale steel frame testbed, which was designed to simulate fracture damage at member ends. The significant findings are summarized as follows:

(1) The developed wireless strain sensing system, comprised of PVDF sensors and *Narada* wireless units, showed excellent performance for monitoring the dynamic strain in the steel structures under small amplitude vibrations and even ambient excitations.

(2) In the experimental results for five-story steel frame testbed, the variation in the damage index under the undamaged conditions for different excitations was less than 7% and the weak dependency of damage index on the characteristics of the external excitations was preliminarily verified.

(3) The damaged locations were successfully identified in the tests using the distribution of damage index values. Moreover, the damage index values for the various severity levels showed clear discrete values that would enable the quantification of seismic fracture damage.

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- [1] Iemura. H., Igarashi, A., Fujiwara, T., and Toyooka, A. (2000). Full-scale Verification Test of Dynamic Response Control Techniques for Strong Earthquakes, *the Proceedings of 12th World Conference of Earthquake Engineering*, 1795.

CHAPTER 4

Sensitivity investigation of strain-based damage index

4.1 Overview

This chapter further investigates the sensitivity of the presented damage index to measurement environments and various structural parameters. The sensitivity of the damage index is examined through numerical studies with a nine-story steel moment-resisting frame and experimental studies using the five-story steel frame testbed.

4.2 Numerical studies with a nine-story steel moment-resisting frame

4.2.1 Nine stories building model

The sensitivity of the presented damage index was examined through a numerical case study using the LA pre-Northridge nine-story building intensively studied in the SAC steel project [1]. The nine-story building represents typical medium-rise buildings designed according to the pre-Northridge design practice in Los Angeles, California. The building is 45.73 m by 45.73 m in plan, and 37.19 m in elevation (see Fig. 4.1). Each bay spans 9.15 m in both the N-S and E-W directions. The lateral load-resisting system of the building comprises four perimeter steel moment-resisting frames. The interior bays of the structure contain gravity frames with composite floors. The wide flange columns of the moment-resisting frames are made from 345 MPa steel. The column bases are modeled as pin connections. The horizontal displacement of the structure at ground level is assumed to be restrained. The floor system consists of wide flange beams made of 248 MPa steel acting compositely with floor slabs. The typical beam sizes are W36x160 (with I_x of $4.062 \times 10^9 \text{ mm}^4$) from the ground to the third floors, W36x135 (with I_x of $3.247 \times 10^9 \text{ mm}^4$) from the fourth to seventh floors, and smaller beam sizes for the upper levels. The inertial forces at each floor are

assumed to be evenly carried by each perimeter moment-resisting frame through the floor system. Hence, each frame resists one half of the seismic mass. The seismic mass of the ground level is 9.65×10^5 kg, for the second floor is 1.01×10^6 kg, for the third through ninth floors is 9.89×10^5 kg, and for the tenth floor is 1.07×10^6 kg.

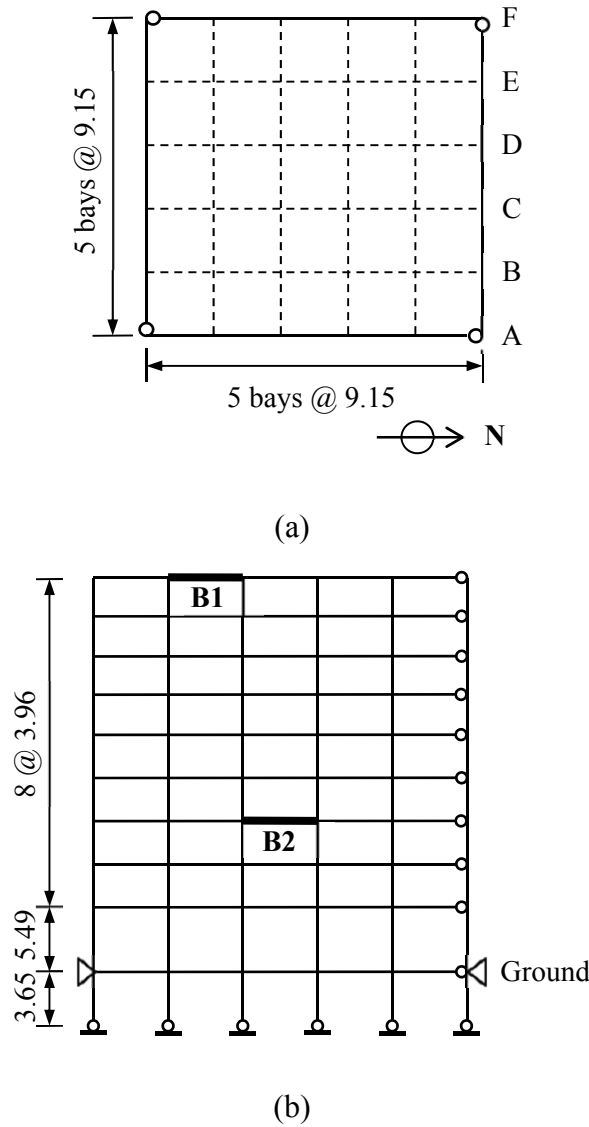


Fig. 4.1 SAC nine-story building (unit: m): (a) building plan; (b) frame A elevation

4.2.2 Analysis model

The analysis model was built using the finite element (FE) analysis software, Marc [2]. As most seismic-induced beam fractures begin at the toe of the weld access hole and extend to the web, the beam fracture was simulated by cutting the bottom flange and/or web near the column surface at the left end of beam B2 (Fig. 4.2). The length of the cut was one percent of the beam length. There

were seven damage patterns for beam seismic fracture simulation, as listed in Table 4.1. DP1 to DP3 simulated fracture at one side of the bottom flange, where the decreases of the bending stiffness EI_x at the cut section were smaller than 22%. DP4 simulated the entire bottom flange fracture, in which the bending stiffness EI_x at the cut section decreased by 49%. Severe fracture damage extending from the bottom flange to the web was simulated in DP5 to DP7 with the reduction of more than 75% in the bending stiffness EI_x at the cut section. In the finite element model, two beams B1 and B2 were modeled with shell elements, and other beams and columns were modeled with beam elements (Fig. 4.3). The nodes of shell elements at the beam-ends were connected to the nodes of beam elements with rigid links.

The measurement locations of the bending strain responses of beams are shown in Fig. 4.4. S_{ref} as a reference sensor was set on the left end of beam B1 at the top floor where was considered to be far from the damage location (Fig. 4.4(a)). In practice, several beams with the least damage probability may be selected to set reference sensors. S1 to S8 as detecting sensors were on one side of the bottom flange of beam B2 at intervals of l or $2l$ ($l = 0.2d_2$, where d_2 is the depth of beam B2) from the column surface (Fig. 4.4(b)). The frame was excited with two excitations (Fig. 4.5): (1) a white noise (WN); and (2) an earthquake ground motion (EM).

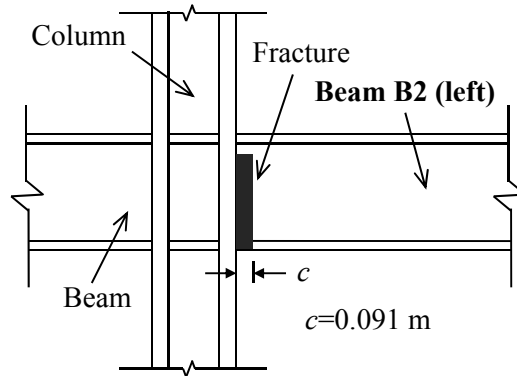


Fig. 4.2 Simulated fracture

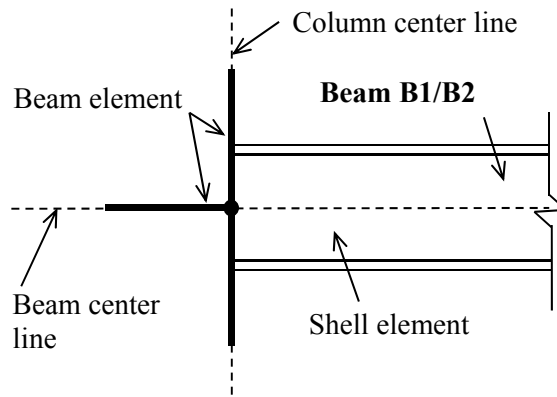
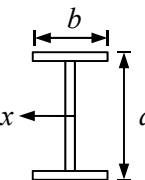
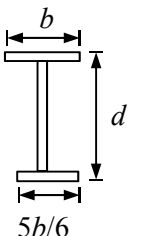
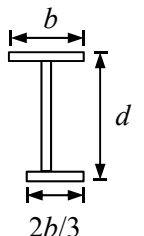
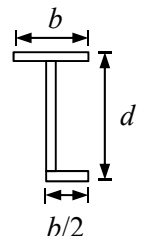
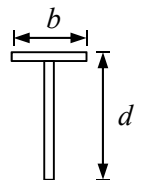
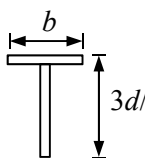
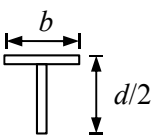
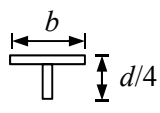


Fig. 4.3 Connection between beam elements and shell elements

Table 4.1 Damage patterns for fracture simulation

Damage pattern	Undamaged	DP1	DP2	DP3	DP4
Cross-section					
EI_x reduction (%)	0	6.5	13.5	21.2	49.1
Damage pattern	DP5	DP6	DP7		
Cross-section					
EI_x reduction (%)	76.1	91.8	98.7		

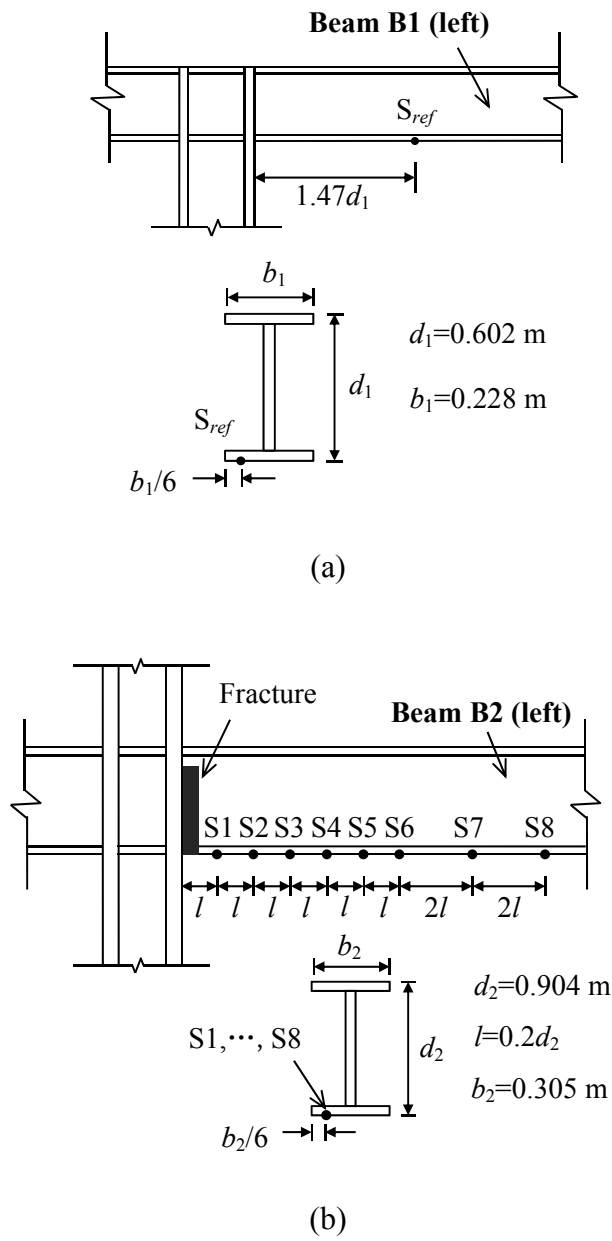
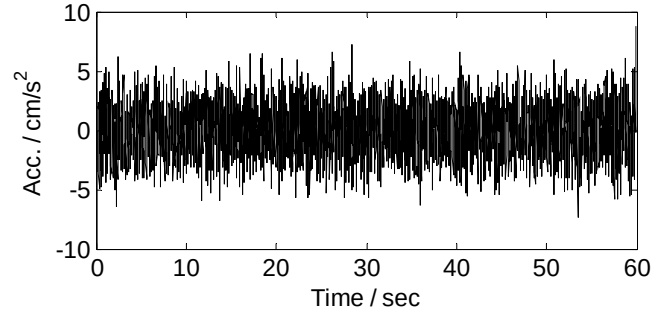
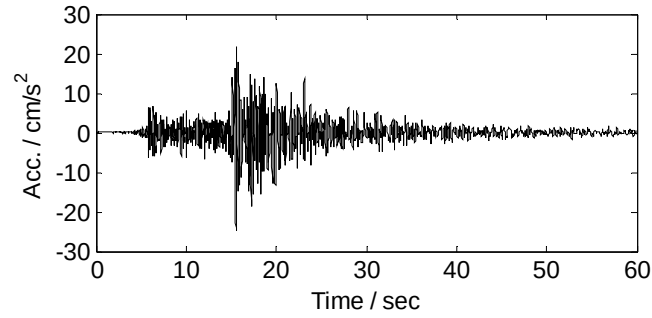


Fig. 4.4 Strain output location: (a) reference sensor; (b) detecting sensors



(a)



(b)

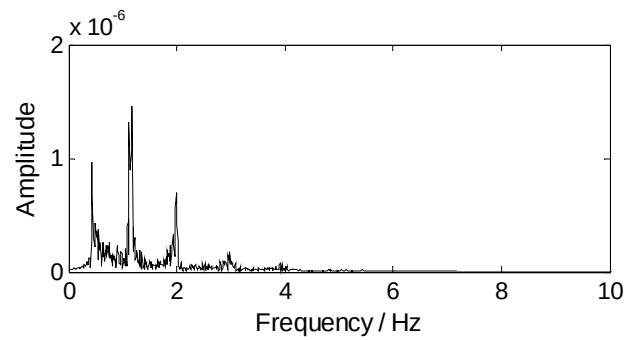
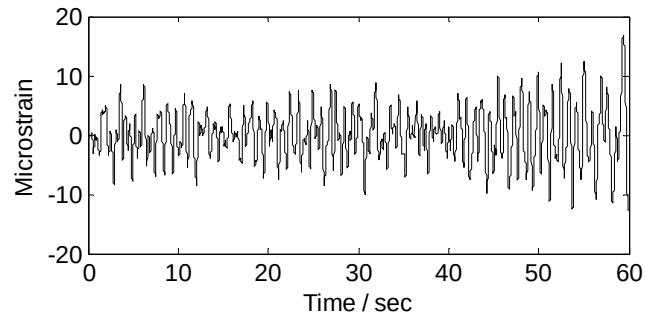
Fig. 4.5 Input excitations: (a) white noise; (b) earthquake ground motion

4.2.3 Data preprocessing

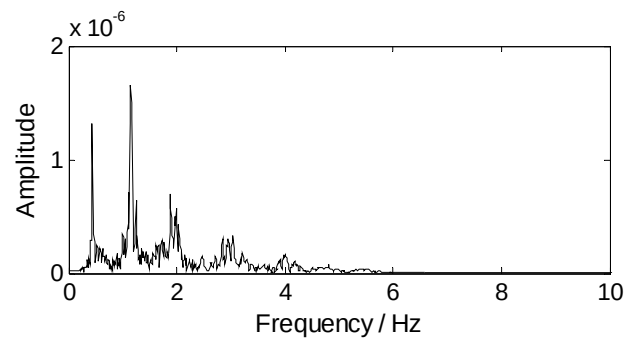
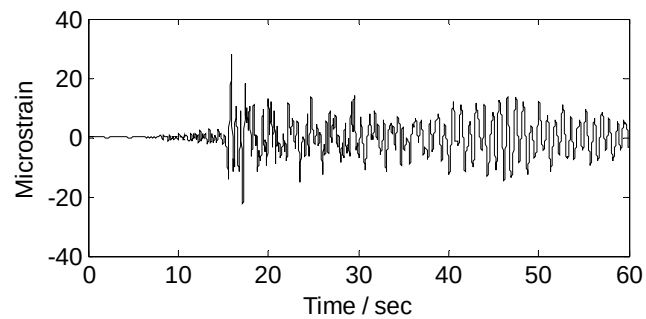
The first four natural frequencies of the undamaged model of the nine-story frame were 0.432, 1.150, 1.987, and 2.988 Hz, which were consistent with those reported previously [3]. For reference, the inclusion of the severest damage condition at Beam B2 (DP7 with a reduction of 99% in the bending stiffness EI_x at the cut section) reduced the first four natural frequencies to 0.429, 1.150, 1.980, and 2.963 Hz, where the largest change in these frequencies was only 0.9%. Note that damage to a critical member that assures the overall stability of the frame, such as a column, can lead to a more significant change in the natural frequency.

Fig. 4.6 shows the bending strain responses and their amplitude spectra of the reference sensor S_{ref} at the undamaged condition. The amplitude spectra for both excitations indicate that the responses of the frame were mainly dominated by the first three modes. Therefore, the bending strain responses associated with the first three modes were respectively used to calculate the damage index DI . The strain responses associated with each mode were obtained using band-pass filters on raw strain responses. Considering the slight changes in the natural frequencies with the inclusion of damage, the bandwidth of the band-pass filter was set to include $\pm 10\%$ of each natural

frequency. Thus, the band-pass filters were 0.38-0.48, 1.04-1.27, and 1.79-2.19 Hz for the first three modes.



(a)



(b)

Fig. 4.6 Bending strain responses at reference sensor: (a) white noise; (b) earthquake ground motion

4.2.4 Simulation results

The sensitivity of the presented damage index to input excitation, vibrational mode, the selection of reference data, and sensor location were studied using the constructed analysis model.

4.2.4.1 Independency on excitations and modes

First, the dependency of the damage index on input excitations was examined. The variations in the ratio of RMS values of the bending strain responses were studied for the undamaged condition. Fig. 4.7 shows the ratios of the first mode for each detecting sensor (i.e., S1 to S8) relative to the reference sensor S_{ref} . The values of the ratio were the largest at S1 and the smallest at S8, and proportional to the bending moments sustained at each beam section. When two excitations were compared using modal analysis (i.e., no need to extract the modal strain responses from the time histories), the difference was up to 3.8% for the white noise, and 0.05% for the earthquake ground motion, which confirmed the independence of the extracted ratio of RMS values on external excitation as indicated by Equation (3.5) of the preceding theoretical formulation. Note that the differences arise from errors in the extraction of the modal strain responses with band-pass filters. Compared to the white noise, the earthquake ground motion that generated a relatively large-amplitude strain response (see Fig. 4.6) had a small discrepancy.

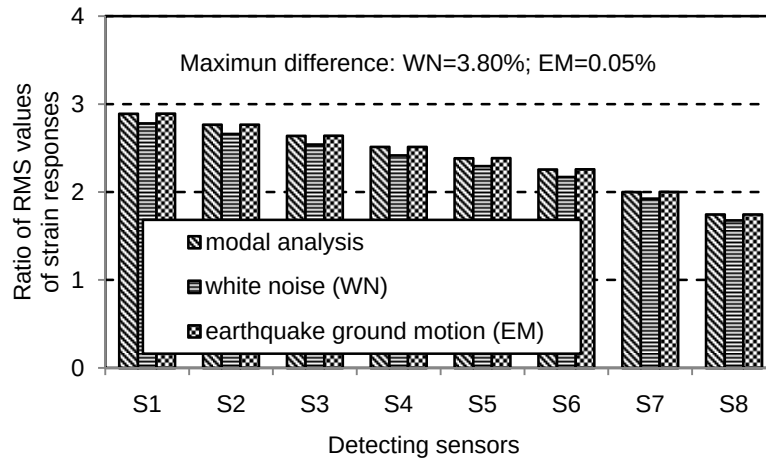


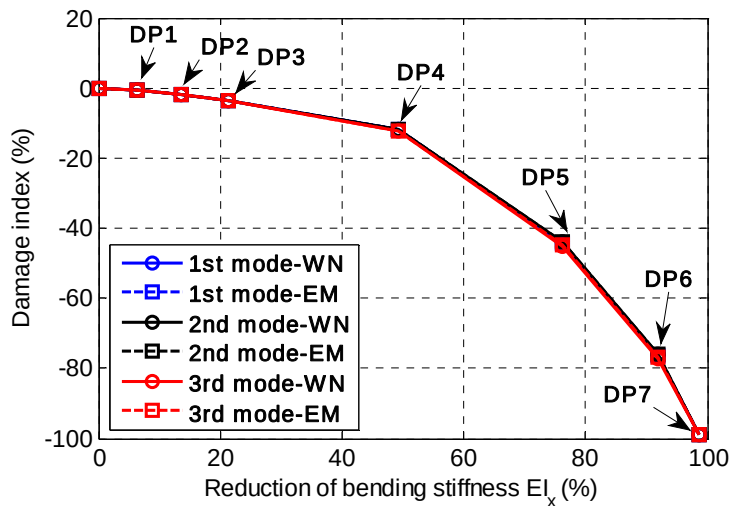
Fig. 4.7 Ratio of RMS values for different detecting sensors and excitations

Next, the selection of reference values and modes were studied. Fig. 4.8 shows the damage index DI at sensor S6 for two different selections of the reference values under the undamaged condition.

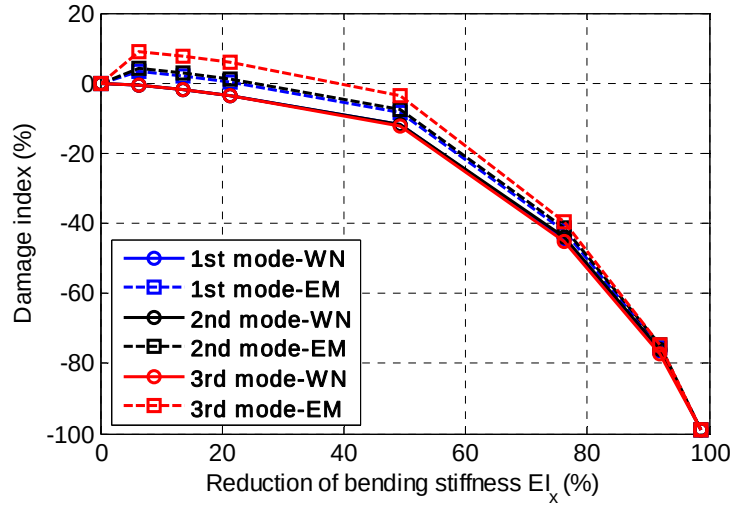
Reference 1: Ideal case where the same excitation was used for undamaged and damaged conditions.

Reference 2: Practical case where ambient vibration assumed to be white noise was used to prepare the reference values under the undamaged condition.

The horizontal axes of the plots are the reduction of the bending stiffness at the fractured section and the vertical axes are the damage index DI calculated with Equation (3.8). As the bending stiffness EI_x decreases, the damage index drops from 0 to -100% . When reference 1 was applied, the damage indices were identical for different excitations and selected modes. In contrast, when reference 2 was applied, while the damage indices extracted from strain responses under white noise were identical for the first three modes, the damage indices extracted from the strain responses under earthquake ground motion contained errors (as the errors significantly exceeded the real damage index at DP1 to DP3, the damage index takes positive values that are false-negative). This is because the errors in the extraction of modal responses with band-pass filters under the undamaged and damaged conditions were not identically offset. The maximum error of the damage indices extracted from the first two modes was not greater than 4%, while that for the third mode without a clear fundamental peak (see Fig. 4.6(b)) exceeded 9%. In Fig. 4.6(b), the power ratio of the fundamental peak to the irrelevant noise (i.e., responses not related to the natural vibration modes) in the filter bandwidth is 64.1 dB for the first mode, 2.1 dB for the second mode, and 0.3 dB for the third mode. In summary, the dominant modes with a higher peak in the amplitude spectrum are more suitable for computing the damage index.



(a)



(b)

Fig. 4.8 Damage index DI at detecting sensor S6: (a) with the first selection of reference values; (b) with the second selection of reference values

4.2.4.2 Influence of sensor location

As mentioned in the Chapter 3, strain responses near beam-ends are influenced by local strain redistributions around fractures. Thus, the transition of the damage index along the beam axis was studied. According to the finding in the preceding section, the damage index was extracted from the first mode vibration under white noise excitation and with reference 1. Fig. 4.9 shows the damage index for all detecting sensors S1 to S8. The damage index was affected by the local strain redistribution at S1 to S5 (i.e., the region that is less than $1.2d$ from the column surface). In contrast, the damage index was almost identical at S6 to S8 (i.e., the region that is more than $1.2d$ from the column surface), which indicates that the influence of the local strain redistribution is negligible and the values of the damage index are related primarily to the extent of moment redistribution induced by the damage.

Practically speaking, as the beam-end region within one beam depth from column surfaces may sustain large plastic deformation during strong earthquake events, detecting sensors had better be placed outside that region to be fully functional after the events. Thereby, it is recommended to place damage-detecting sensors at a distance of larger than $1.2d$ from the column surface, and to estimate the reduction in the bending stiffness at the fractured section.

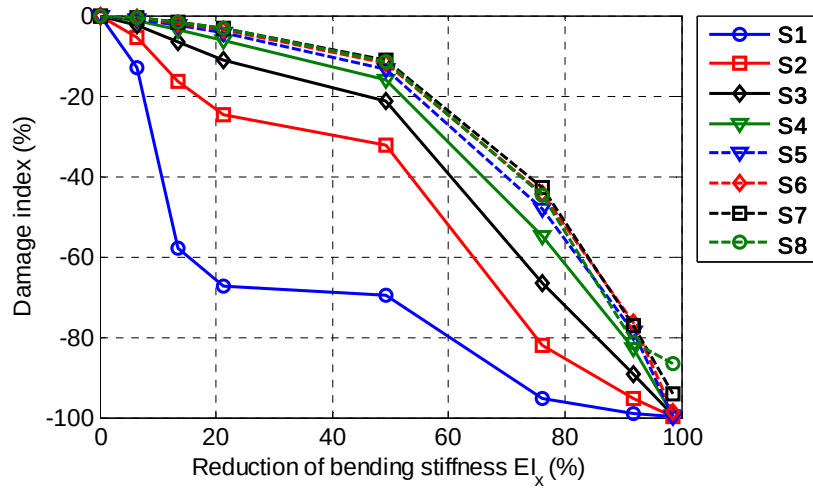


Fig. 4.9 Damage index at all detecting sensors S1 to S8

4.3 Sensitivity study using the five-story steel frame testbed

Four different types of vibration test (i.e., Test 1 to Test 4) including a total of seventeen damage cases were conducted on the five-story steel frame testbed to experimentally investigate the sensitivity of the presented damage index. The tests are summarized in Table 4.2, where damage patterns (i.e., damage L1 and L2) were illustrated in Fig. 3.7 and Table 3.2 and removable connections were shown in Fig. 3.4(b).

Test 1: Independency of the damage index on external excitations and vibration modes was verified with a shaking table at the DPRI as excitation source. Damage L1 and L2 were simulated at connection B2 near the inner joint of the second floor.

Test 2: Influence of sensor location on the damage index was examined with a modal shaker as excitation source. Damage L1 and L2 were simulated at connection B1 near the exterior joint of the second floor.

Test 3: General applicability of the damage index was examined with a modal shaker as excitation source. Two levels of fracture damage, Damage L1 and L2, were simulated at three different connections B2, B6, and B10.

Test 4: Influence of neighboring damage on the damage index was studied with a modal shaker as excitation source. As beam seismic damage changes the moment distribution rather locally, only the influence of fracture damage at the closest beam-ends on the same floor level was considered.

Test 4 was conducted to obtain preliminary data for multiple damage condition; at this moment, the presented damage index does not explicitly consider the influence of neighboring damage and further study is required. Note that all the tests considered fracture damage only in one longitudinal

frame, while another longitudinal frame remained intact. The inclusion of asymmetric damage may induce torsional vibrations of the frame but the influence on the lateral mode vibrations was found negligible.

Table 4.2 Damage cases

Test	Damage Case	Damage				Targets	Loading system
		As detected		Influence sources			
		Location	Category	Location	Category		
Test 1	Undamaged	-	-	-	-	Independency on excitations and modes	Shaking table
	Case 1	B2	L1	-	-		
	Case 2	B2	L2	-	-		
Test 2	Case 3	B1	L1	-	-	Influence of sensor location	Modal shaker
	Case 4	B1	L2	-	-		
Test 3	Case 5	B2	L1	-	-	General applicability	Modal shaker
	Case 6	B6	L1	-	-		
	Case 7	B10	L1	-	-		
	Case 8	B2	L2	-	-		
	Case 9	B6	L2	-	-		
	Case 10	B10	L2	-	-		
Test 4	Case 11	B3	L1	-	-	Influence of neighboring damage	Modal shaker
	Case 12	B3	L1	B2	L1		
	Case 13	B3	L1	B2	L2		
	Case 14	B3	L1	B4	L2		
	Case 15	B3	L2	-	-		
	Case 16	B3	L2	B2	L2		
	Case 17	B3	L2	B4	L2		

4.3.1 Excitations for vibration tests

In Test 1, the steel frame was excited in the longitudinal direction by the shaking table at the DPRI, Kyoto University, with two small-amplitude excitations (Fig. 4.10): (1) a white noise (WN) with a frequency range of 1 to 50 Hz and RMS of 2 cm/s²; and (2) an small-amplitude earthquake ground motion (EM) with the maximum acceleration of 18 cm/s². In the undamaged frame, these

excitations induced the top floor acceleration responses of 4.38 and 12.32 cm/s^2 in RMS, respectively. In Tests 2, 3 and 4, the steel frame testbed was excited at the fifth floor using a modal shaker (APS-113, APS Dynamics) firmly fixed to the steel mass plate (Fig. 3.4(f)). The steel frame was excited in the longitudinal direction using three excitations: (1) ambient excitation (AmbE); (2) small-amplitude white noise with a frequency range of 1 to 50 Hz (WN1); and (3) relatively large-amplitude white noise with a frequency range of 1 to 50 Hz (WN2). In the structural laboratory where the testbed frame was located, ambient vibrations mainly caused by ground microtremor was around 0.49 cm/s^2 in RMS at the top floor. When the undamaged frame was excited with two white noise excitations, the roof acceleration responses were 3.32 and 8.45 cm/s^2 in RMS for WN1 and WN2, respectively.

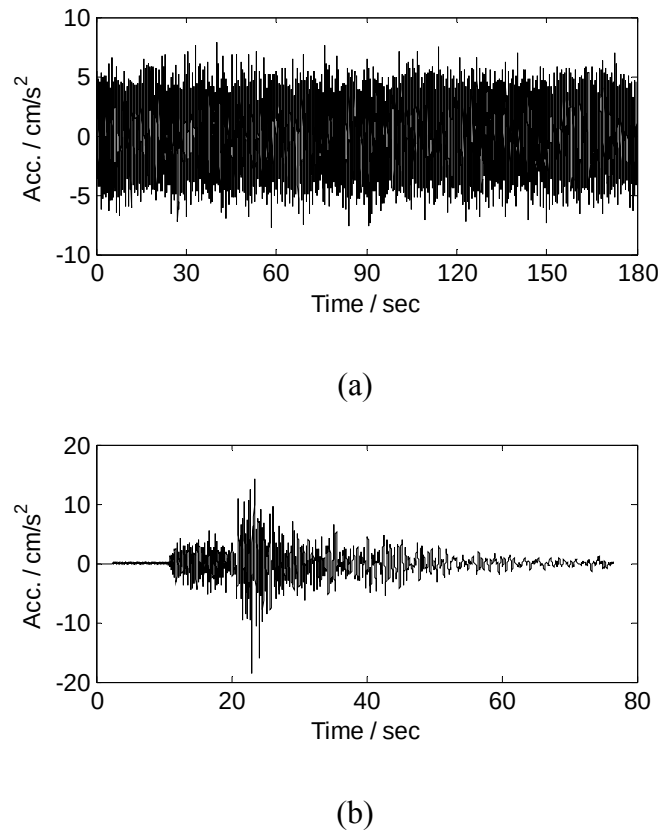
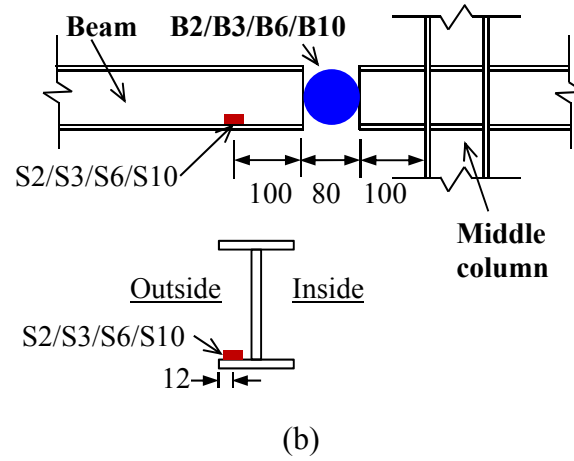
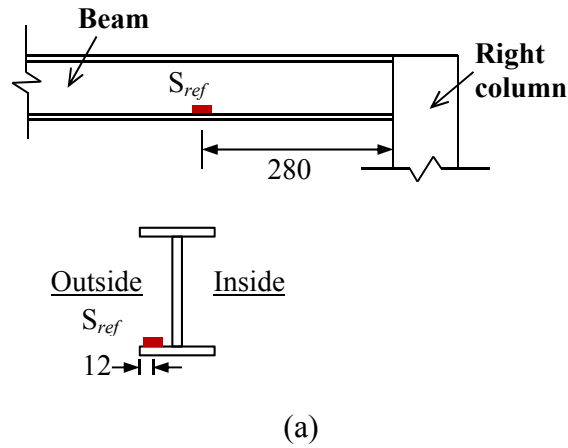


Fig. 4.10 Input excitations for the shaking table: (a) white noise; (b) earthquake ground motion

4.3.2 Sensor location

In all tests, the reference sensor S_{ref} (Fig. 4.11(a)) was placed at the top floor. In Tests 1, 3 and 4, detecting sensors were placed on one side of the beam bottom flange at $1.0d$ (the beam depth d is 100 mm) away from the edge of the fracture, as illustrated in Fig. 4.11(b). Sensors S2, S3, S6, and S10 were used to detect the simulated damage at connections B2, B3, B6, and B10 respectively. In

Test 2, detecting sensors were attached on both sides of the beam bottom flange at $1.0d$, $1.5d$, and $2.0d$ away from the edge of the fracture to examine the influence of sensor location. Six sensors S11 to S16 used to detect the damage at connection B1 are shown in Fig. 4.11(c). While not included in this paper, when the fracture progressed from the tail of the weld access hole asymmetrically about the beam axis (e.g., the fracture of half of the bottom flange), the amount of local strain redistribution differed at each side of the bottom flange. Nevertheless, the influence of local strain redistribution was expected to disappear at a sufficient distance from the fractured section.



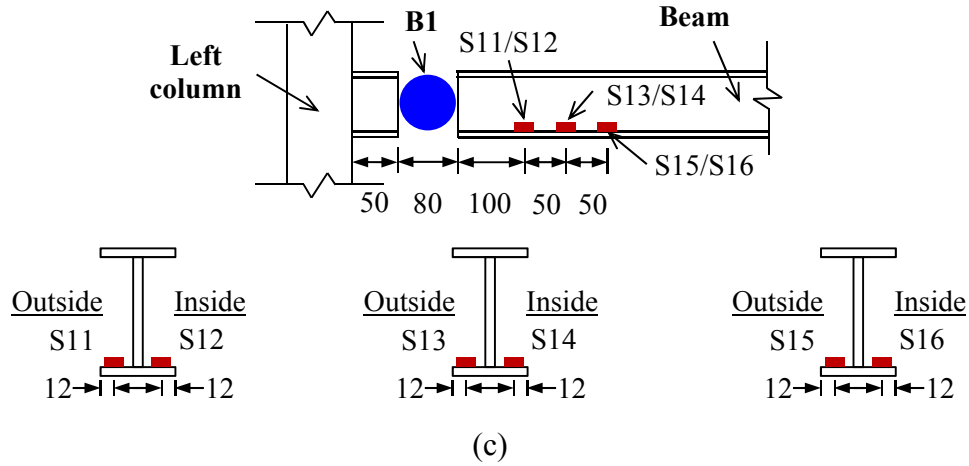
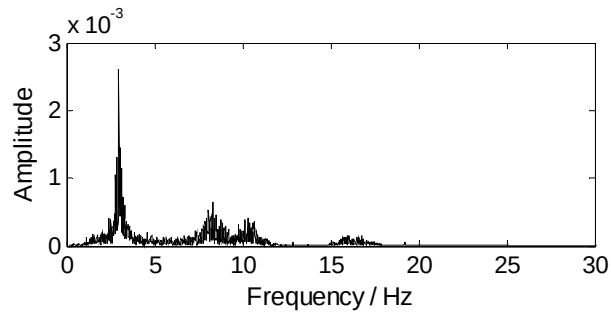
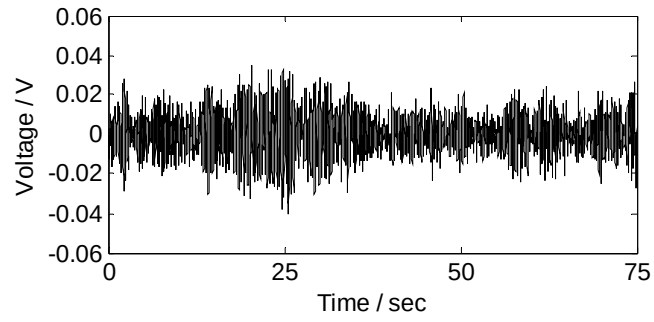


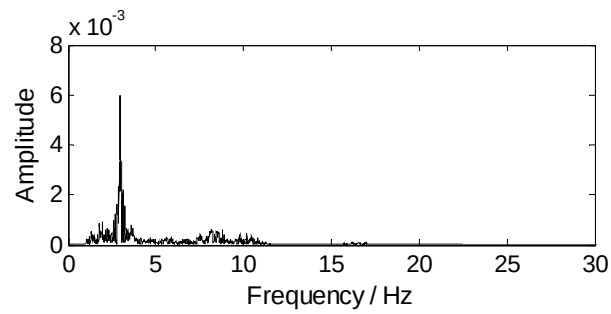
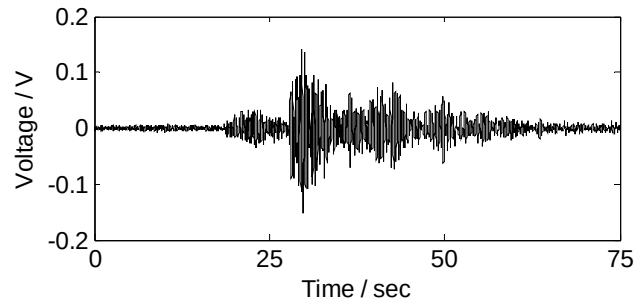
Fig. 4.11 PVDF sensor location (unit: mm): (a) reference sensor; (b) sensors in Tests 1, 3 and 4; (c) sensors in Test 2

4.3.3 Results and discussions

In all tests, bending strain responses were recorded for 75 seconds with the sampling rate of 100 Hz. Fig. 4.12 shows the strain responses in voltage units (one microstrain corresponds approximately to 12 mV) and their amplitude spectra at the reference sensor S_{ref} for two excitations, which were measured from the undamaged condition in the shaking table tests of Test 1. The amplitude spectra indicated that the structural vibration was mainly dominated by the first mode. The first two natural frequencies of the testbed frame were 3.16 and 8.33 Hz for the undamaged condition, 3.11 and 8.25 Hz for Case 2, and 3.05 and 8.31 Hz for Case 17. Note that Case 17 was one of serious damage cases among all considered damage cases. The band-pass filter of 2.70-3.30 and 7.40-9.20 Hz ($\pm 10\%$ of the natural frequencies at the undamaged condition as the band width) were used to obtain the modal strain responses of the first two modes. The averaged ratio of RMS values for different excitations at the undamaged condition were used as the reference values.



(a)



(b)

Fig. 4.12 Measured signals at S_{ref} in Test 1: (a) white noise; (b) earthquake ground motion

4.3.3.1 Test 1

Fig. 4.13 shows the ratios of RMS values of strain responses between sensor S2 and reference sensor S_{ref} for the first mode. The largest difference in the ratio values between the two different excitations was 0.78%, which verified independency of the extracted ratio on external excitations as observed in the preceding theoretical formulation and numerical simulations.

The damage indices of sensor S2 for detecting damage L1 and L2 at connection B2 are summarized in Table 4.3. In Case 1 with damage L1, i.e., entire bottom flange fracture with the reduction of 68.5% in the bending stiffness, the damage indices were about -60% for both excitations with the use of the first mode vibrations but changed to -70% with the use of the second mode vibrations. Compared to the damage index extracted from the first mode, the damage index of the second mode had larger discrepancy as the modal strain responses were weak and unclear (see Fig. 4.12). In Case 2 with damage L2, i.e., entire bottom flange and web fracture with the decrease of 99.8% in the bending stiffness, the damage indices were smaller than -90% for two excitations with the first mode vibrations and slightly decreased with the second mode vibrations. As a result, the dominant modes with clear modal responses and high S/N ratios are highly desirable to increase the accuracy of the calculation of damage index.

4.3.3.2 Test 2

Table 4.4 summarizes the results of Test 2 for different sensor locations. When damage L1 was considered at connection B1 in Case 3, the damage indices at sensors S11 and S12, both placed at 100 mm (i.e., the beam depth) away from the edge of the fracture, were about -50%, whereas the damage indices at sensors S13 and S14, both at 150 mm (i.e., one and half beam depths) away from the edge, were around -35% to -37%. The damage indices at sensors S15 and S16, both attached at 200 mm (i.e., two beam depths) away from the edge, were -32% to -36% and consistent with those at S13 and S14. When damage L2 was considered at connection B1 in Case 4, the damage index at the six sensors S11 to S16 was less than -85%. Compared to the damage index of about -95% at sensors S11 and S12, the damage index at sensors S13 to S16 had slight changes of 10%, which was consistent with the findings in the preceding numerical simulations for severe damage DP7; strain sensors needed to be set within two beam depths to guarantee the monotonic relation between the damage index and the reduction of bending stiffness. Note that the values at the different sides of the flange (e.g., S13 and S14, and S15 and S16) varied by 5% with beam torsional vibrations observed when web links were removed. In conclusion, in order to obtain a stable relation between the damage index and the reduction of bending stiffness, like the damage index curves for sensors

S6 to S8 in Fig. 4.9, detecting sensors need to be placed with the distance of at least $1.5d$ but no farther than $2.0d$ from fracture damage as recommended by the previous simulations using the SAC nine-story frame.

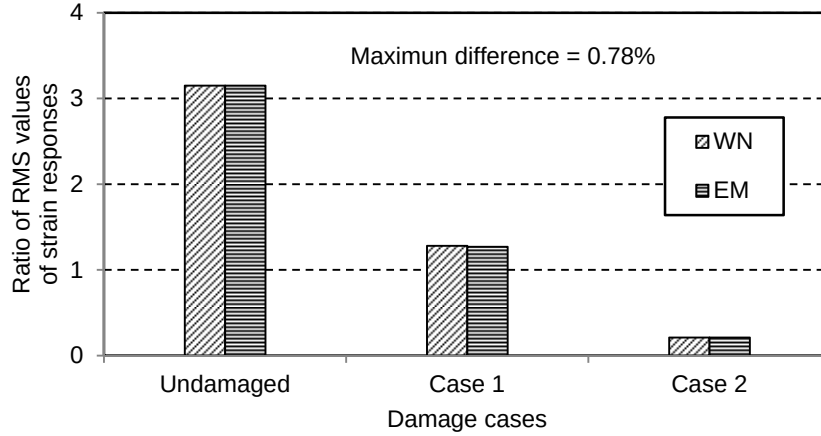


Fig. 4.13 Ratio of RMS of strain responses at S2 and S_{ref} for the first mode in Test 1

Table 4.3 Damage index for detecting damage at connection B2 in Test 1

Mode	Excitation	Damage index (%)		
		Undamaged	Case 1	Case 2
1st mode	WN	0.0	-59.5	-93.5
	EM	0.0	-59.7	-93.4
2nd mode	WN	-1.0	-68.9	-96.5
	EM	1.0	-68.4	-96.7

Table 4.4 Damage index for detecting damage at connection B1 in Test 2

Damage case	Excitation	Damage index (%)					
		S11	S12	S13	S14	S15	S16
Case 3	AmbE	-47.9	-51.6	-35.0	-36.3	-32.3	-35.5
	WN1	-47.8	-51.6	-35.3	-36.3	-34.0	-33.7
	WN2	-48.9	-52.1	-35.9	-37.4	-35.0	-35.4
Case 4	AmbE	-96.5	-94.9	-93.2	-89.6	-90.0	-85.1
	WN1	-96.9	-95.0	-93.7	-90.2	-90.2	-85.1
	WN2	-97.0	-95.0	-93.8	-90.2	-90.3	-84.9

4.3.3.3 Test 3

The stability of the damage index was examined by changing the location of damage in the testbed frame. As given in Table 4.5, the mean values of the damage index at three different connections B2, B6, and B10 were -59% , -55% , and -52% for damage L1 and -91% , -92% , and -95% for damage L2. The standard deviations in the damage indices for three excitations were less than 0.7% for damage L1 and 3.9% for damage L2. The variation was larger for the severer damage condition. The damage index slightly varied for different damage locations but the observed variation was at most 7.8% for damage L1 and 3.8% for damage L2. This indicated the general applicability of the damage evaluation based on the proposed damage index for the presented level of damage.

Table 4.5 Damage index for detecting damage L1 and L2 in Test 3

Damage category	Damage case	Damage index (%)				
		AmbE	WN1	WN2	Mean	Standard deviation
L1	Case 5	-59.6	-60.4	-59.6	-59.9	0.5
	Case 6	-55.2	-55.8	-55.2	-55.4	0.3
	Case 7	-52.0	-51.4	-52.8	-52.1	0.7
L2	Case 8	-87.1	-93.8	-94.0	-91.6	3.9
	Case 9	-89.5	-93.2	-93.6	-92.1	2.3
	Case 10	-94.3	-96.5	-95.4	-95.4	1.1

4.3.3.4 Test 4

Another important influential factor for the damage index is the increases of bending moment sustained at damage-neighboring connections in the moment redistributions. The existence of severe damage nearby in particular affects the damage index for detecting small damage. Thus, a systematic approach to identify the extent of damage at multiple locations is needed. As this issue will be a focus of further developments of the presented method, in Test 4, preliminary test data for the multiple damage condition was obtained (Table 4.6). In Case 12, damage L1 at the left and right sides of a beam-column connection was considered. The damage index at the right side (i.e., connection B3) increased approximately by 5% with the existence of the left side damage compared to those for the single damage condition in Case 11 (i.e., from -55.5% to -49.3% in mean). The damage index further increased by 15% (i.e., from -49.3% to -34.1% in mean) with the existence

of damage L2 in Case 13. In contrast, when damage L2 existed nearby beam-column connections in Case 14, the increment was only around 5%. In Cases 16 and 17, damage L2 was considered at two locations. The results indicate that the influence was negligible at this severity of damage compared to that for the single damage condition in Case 15.

Table 4.6 Damage index for detecting damage L1 and L2 at connection B3 in Test 4

Damage category	Damage case	Damage index (%)				
		AmbE	WN1	WN2	Mean	Standard deviation
L1	Case 11	-55.9	-55.1	-55.5	-55.5	0.4
	Case 12	-49.7	-49.1	-49.1	-49.3	0.3
	Case 13	-34.8	-34.1	-33.5	-34.1	0.7
	Case 14	-50.3	-50.9	-50.4	-50.5	0.3
L2	Case 15	-92.0	-93.2	-93.1	-92.8	0.7
	Case 16	-90.4	-91.6	-91.3	-91.1	0.6
	Case 17	-92.4	-99.1	-99.0	-96.8	3.8

4.4 Summary

In this chapter, the sensitivity investigations of the damage index were numerically and experimentally conducted using an SAC nine-story steel frame and a five-story steel frame testbed. The notable findings are summarized as follows.

(1) The independency of the presented damage index on the characteristics of external excitations and the selection of vibration modes was verified in numerical simulations and shaking table tests. As the extraction of modal responses required preset band-pass filters, the use of dominant vibration modes with clear responses and high power was highly desirable.

(2) Both in the numerical simulations and experiments, the damage index extracted within a distance of $1.2d$ (d is beam depth) from a fracture was largely affected by local strain redistributions induced by the fracture. A distance between $1.2d$ and $2.0d$ from the fracture was recommended for evaluating the moment redistributions in steel moment-resisting frames and the reduction in bending stiffness at fractured sections.

(3) Consistency of the damage index in the evaluation of damage at different locations was verified in experimental studies using the five-story steel testbed frame. The level of variation was at most 7.8% for damage L1 with fracture of the bottom flange and 3.8% for damage L2 with fracture of the bottom flange and web.

(4) The increases of the damage index at damage-neighboring connections were verified using a preliminary study considering multiple damage condition. The interaction between neighboring damage at the same beam-column connections was much larger than at different connections. The explicit effects will be further studied for the damage quantification of multiple beam fractures.

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- [3] Ohtori Y, Christenson RE, Spencer BF, Dyke SJ. (2004). Benchmark control problems for seismically excited nonlinear buildings. *Journal of Engineering Mechanics*, 130(4): 366-385.

CHAPTER 5

Simplified derivation of a damage curve for seismic beam fracture

5.1 Overview

Damage curve is a relationship showing a strain-based damage index as a function of reduction in beam bending stiffness induced by fracture, from which one can estimate the amount of earthquake-induced fractures on beams in a steel moment-resisting frame. However, the construction of the damage curve requires a laborious parametric study on simulation of various fracture damage in a numerical frame model. This chapter presents a simplified method of deriving a closed-form expression for damage curves generally applicable for common multi-story multi-bay steel moment-resisting frames. The effectiveness of the closed-form expression is verified numerically using a nine-story steel moment-resisting frame model and experimentally using the one-quarter-scale five-story steel frame testbed.

5.2 Damage curve

In steel moment-resisting frames, inclusion of seismically-induced fractures on steel beams reduces the bending moments resisted by the damaged beams, which attributes mainly to the decreases of the bending stiffness at the fractured sections. When the frames behave linearly, the bending moments of beams can be estimated by measuring beam strain responses. Thus, a comparative study of strains on steel beams under small dynamic loads (e.g., ambient vibrations and minor earthquake ground motions) at intact state and after a major earthquake allows evaluation and quantification of fractures on the steel beams.

Fig. 5.1 illustrates damage evaluation of seismically-induced fractures on steel beams with measured strain data and the damage curve. A dense array of strain sensors, including a reference

sensor and detecting sensors, is deployed to monitor damage-prone beams pre-identified by a structural analysis. The reference sensor is placed at floors (e.g., roof) where relatively small deformation is expected and is used to eliminate the effects of input excitations, which are essentially different at each measurement. The detecting sensors near probable damage are used to detect and quantify fractures. Strain data under ambient vibrations is acquired before and after earthquakes and damage index is calculated from a comparison of the measured strain responses. Using the damage curve, the strain-based damage index is converted into reduction in bending stiffness at the fractured section. This is how the proposed local damage evaluation method provides the damage information (i.e., existence, location, and extent) of the monitored beams quantitatively that potentially support a rapid post-earthquake damage assessment and decision-making. Note that the damage index is a negative value in the original definition in Equation (3.8). In this chapter, the absolute value of the damage index is adopted alternatively.

5.3 Simplified method

5.3.1 Simplified frame

A beam fracture changes moment distribution rather locally in steel moment-resisting frames [1], which likely allows use of a simple frame to derive damage curves for complex frames. For the three frames in Fig. 5.2(a), reduction in the bending moment associated with the first mode induced by a fracture is studied. The stiffness of beams and columns in the box with a dashed line are the same for Frame 1 and Frame 2, while the stiffness of other beams and columns of Frame 1 are 1.25 and 1.32 times that of Frame 2 respectively. A lumped mass at each beam-column joint is identical for two frames.

A frame analysis whose details are explained later in this chapter shows that the amount of beam bending moments reduced at point A is nearly identical for Frame 1 and Frame 2 (Fig. 5.2(b)). This implies that influence of fracture on beam bending moment is primarily limited to the structural properties of neighboring members, and thus a two-story two-bay subframe boxed by a dotted line is likely sufficient to study the influence. The validity of this observation may be reasoned by the classical Saint-Venant's principle. Thus, a two-story two-bay frame with fixed supports in Fig. 5.2(a) is considered as a simplified frame. The simplified frame has the stiffness of members and mass of each beam-column joint identical to those in the box with a dashed line in Frame 1 and Frame 2. The results for this frame are also shown in Fig. 5.2(b). The discrepancy of the reduced bending moments at point A between the simplified and two original frames is at most 2.4%. Thus,

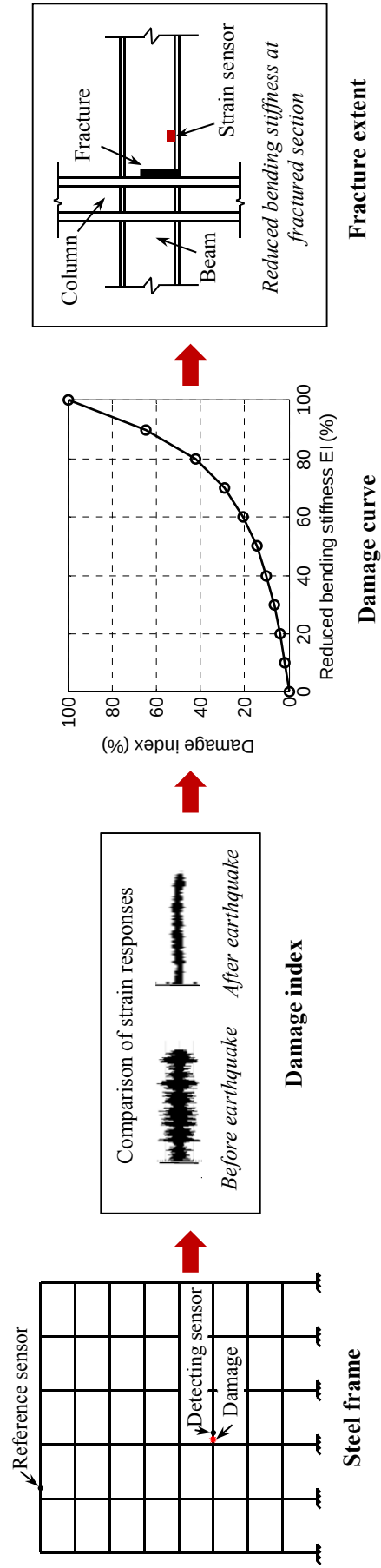


Fig. 5.1 Damage evaluation with strain responses and damage curve

this implies that two-story two-bay frame with fixed supports can be used to conduct an analytical study for deriving a closed-form expression of the damage curve with reasonable accuracy.

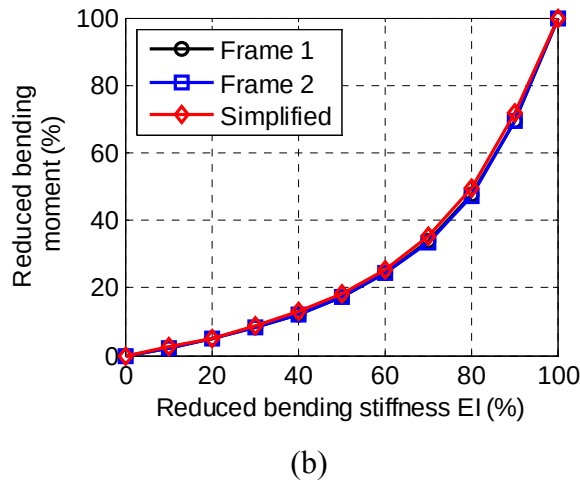
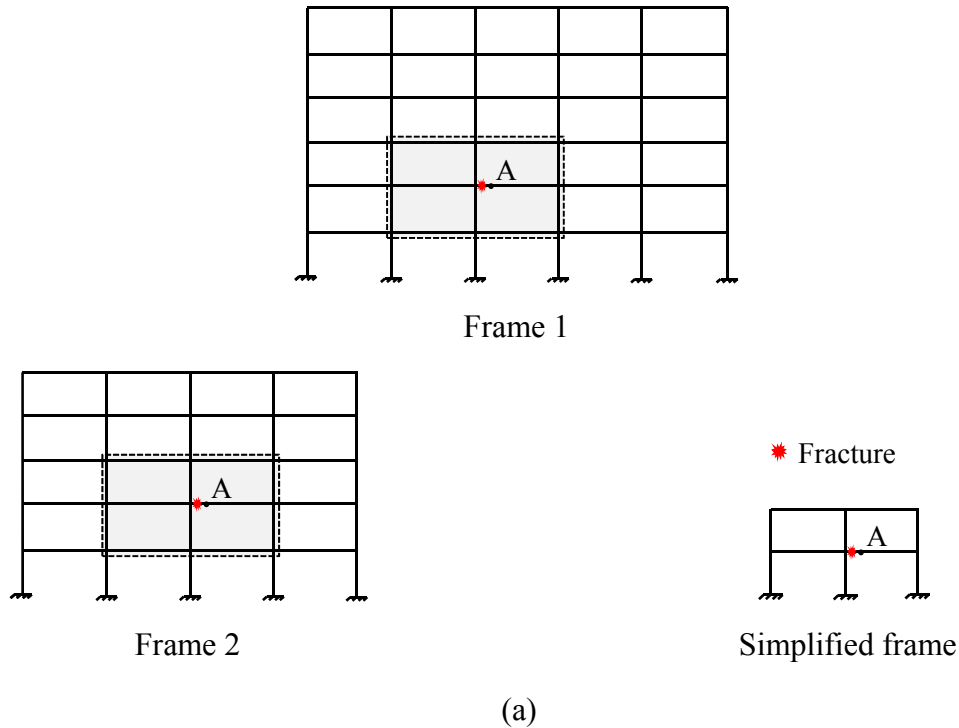
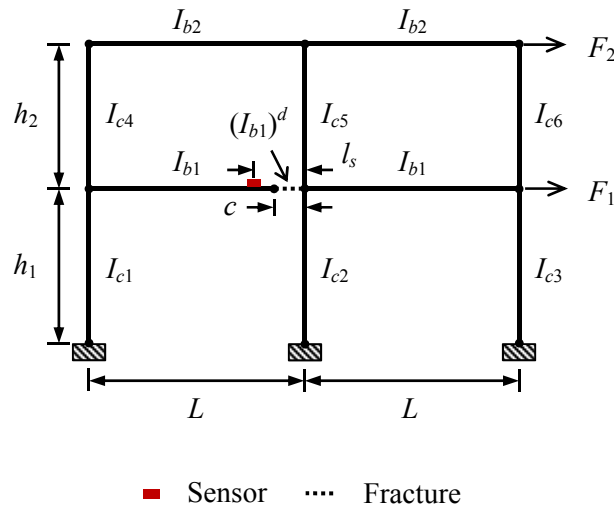


Fig. 5.2 Comparison of reduced bending moment induced by fracture: (a) two frames and a simplified frame; (b) reduced bending moment at point A

5.3.2 Analytical model

A two-story two-bay frame with fixed supports is used to formulate a closed-form expression of damage curve for beam seismic fractures. The frame model in Fig. 5.3(a) considers damage at interior beam-column connections, while the model in Fig. 5.3(b) is for damage on exterior

connections. In the figure, the variables I_{bi} ($i = 1, 2$) and I_{ci} ($i = 1, \dots, 6$) are the second moment of inertia of the beams and columns, respectively; $(I_{b1})^d$ is the second moment of inertia of the fractured section; h_i ($i = 1, 2$) are the height of story; L is the width of each bay; F_i ($i = 1, 2$) are equivalent lateral forces for a fundamental vibrational mode. The distribution of the equivalent lateral forces F_i is assumed to remain the same for the undamaged and damaged conditions because changes in lower mode shapes induced by a limited number of beam fractures are sufficiently small in steel moment-resisting frames [2]. A strain sensor is placed at 1.5 beam depth away from the fractured section near the column surface, i.e., $l_s = 1.5d$ (d is beam depth): the region that is more than 1.2 beam depth away from the fractured section is unaffected by local strain redistribution as demonstrated in the Chapter 4. A beam fracture is modeled by referring to crack model proposed in [3], where the fracture is simulated by a segment of beam with reduced stiffness that is equivalent to the fractured section; the length of the beam segment c is determined as $0.75d$ for wide flange beams.



(a)

where η is the height ratio between two stories; δ is the span-depth ratio of the fractured beam; ν is the stiffness ratio between the beams at two floors; γ_i ($i=1, \dots, 6$) are the stiffness ratios of the column to the beam; ρ is the reduction in the second moment of inertia at the fractured section (normally expressed as a percentage), i.e., the reduction of bending stiffness at fractured section; μ is the lateral force ratio between two floors.

The damage index DI , which is equal to the changes of the bending moments at sensor location between the undamaged and damaged conditions, is a function of reduction in the bending stiffness at the fractured section ρ as follows:

$$DI = \frac{A_1\rho^2 + A_2\rho + A_3}{B_1\rho^2 + B_2\rho + B_3} \times 100\%, \quad (5.2)$$

where A_1, A_2, A_3, B_1, B_2 , and B_3 are coefficients that are a function of structural parameters $\eta, \delta, \nu, \gamma_i$ ($i=1, \dots, 6$), and μ .

5.3.3 Parametric analysis

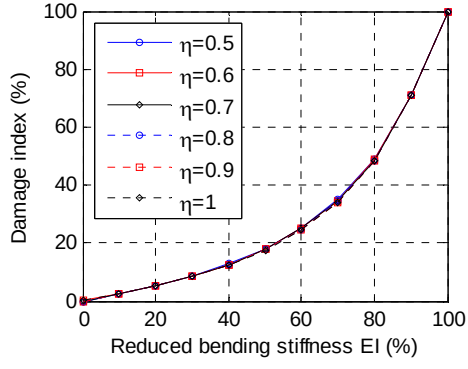
The sensitivity of the damage index to the structural parameters $\eta, \delta, \nu, \gamma_i$ ($i=1, \dots, 6$), and μ are investigated to simplify Equation (5.2). Considering that the structural parameters for common steel moment-resisting frames vary: $0.5 \leq \eta \leq 1$; $8 \leq \delta \leq 28$; $0.2 \leq \nu \leq 5$; $0.5 \leq \gamma_i \leq 2.5$ for exterior columns, $0.5 \leq \gamma_i \leq 5$ for inner columns; $0.2 \leq \mu \leq 10$, two sets of the simplified frames, i.e., Set A for damage on interior connection; Set B for damage on exterior connection, are considered in parametric analysis (Table 5.1).

Fig. 5.4 shows the damage curves for all the considered frames. The horizontal axes of the plots are the reduced bending stiffness at the fractured section and the vertical axes are the damage index computed with Equation (5.2). In the plots of Subset 1, Subset 3 to Subset 10, Subset 11, and Subset 13 to Subset 20, the damage curves are almost the same for the parameters with different values. In the plots of Subset 2 and Subset 12, the damage curve changes notably with different span-depth ratios. The results indicate that the damage curve is primarily affected by the span-depth ratio, and is insensitive to the other parameters. This is because these parameters that do not affect the damage curve influence the bending moment distribution at undamaged and damaged conditions in a same manner, i.e., the damage curve is normalized to these parameters. For the span-depth ratio, as the beam depth closely relates to the equivalent beam length affected by fracture damage, its influence

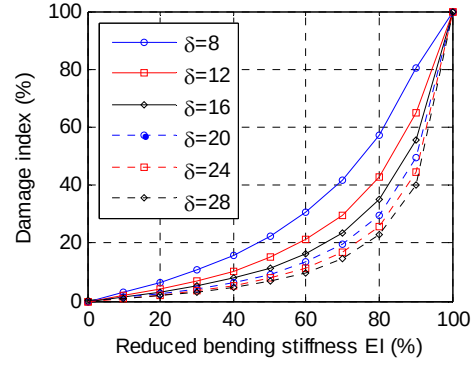
is larger to the bending moment distribution at the damaged conditions than that at the undamaged conditions, i.e., the damage curve is not normalized to the span-depth ratio.

Table 5.1 Two sets of the simplified frames in parametric analysis

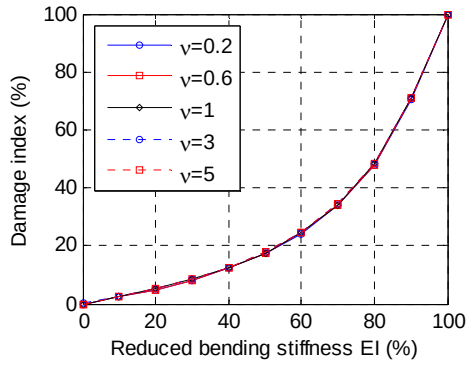
Set A
Default values: $\eta = 1$; $\delta = 10.11$; $\nu = 1$; $\gamma_1 = \gamma_2 = \gamma_3 = \gamma_4 = \gamma_5 = \gamma_6 = 2.13$; $\mu = 1$.
<i>Subset 1</i> : $\eta = \{0.5, 0.6, 0.7, 0.8, 0.9, 1\}$;
<i>Subset 2</i> : $\delta = \{8, 12, 16, 20, 24, 28\}$;
<i>Subset 3</i> : $\nu = \{0.2, 0.6, 1, 3, 5\}$;
<i>Subset 4</i> : $\gamma_1 = \{0.5, 1, 3, 5\}$;
<i>Subset 5</i> : $\gamma_2 = \{0.5, 1, 3, 5\}$;
<i>Subset 6</i> : $\gamma_3 = \{0.5, 1, 3, 5\}$;
<i>Subset 7</i> : $\gamma_4 = \{0.5, 1, 3, 5\}$;
<i>Subset 8</i> : $\gamma_5 = \{0.5, 1, 3, 5\}$;
<i>Subset 9</i> : $\gamma_6 = \{0.5, 1, 3, 5\}$;
<i>Subset 10</i> : $\mu = \{0.2, 0.6, 1, 3, 5, 10\}$.
Set B
Default values: $\eta = 1$; $\delta = 10.11$; $\nu = 1$; $\gamma_1 = \gamma_4 = 1.61$; $\gamma_2 = \gamma_3 = \gamma_5 = \gamma_6 = 2.13$; $\mu = 1$.
<i>Subset 11</i> : $\eta = \{0.5, 0.6, 0.7, 0.8, 0.9, 1\}$;
<i>Subset 12</i> : $\delta = \{8, 12, 16, 20, 24, 28\}$;
<i>Subset 13</i> : $\nu = \{0.2, 0.6, 1, 3, 5\}$;
<i>Subset 14</i> : $\gamma_1 = \{0.5, 1, 1.5, 2, 2.5\}$;
<i>Subset 15</i> : $\gamma_2 = \{0.5, 1, 3, 5\}$;
<i>Subset 16</i> : $\gamma_3 = \{0.5, 1, 3, 5\}$;
<i>Subset 17</i> : $\gamma_4 = \{0.5, 1, 1.5, 2, 2.5\}$;
<i>Subset 18</i> : $\gamma_5 = \{0.5, 1, 3, 5\}$;
<i>Subset 19</i> : $\gamma_6 = \{0.5, 1, 3, 5\}$;
<i>Subset 20</i> : $\mu = \{0.2, 0.6, 1, 3, 5, 10\}$.



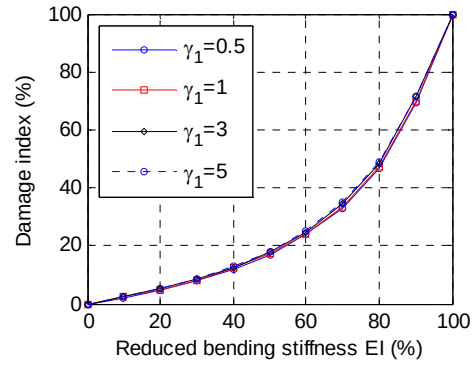
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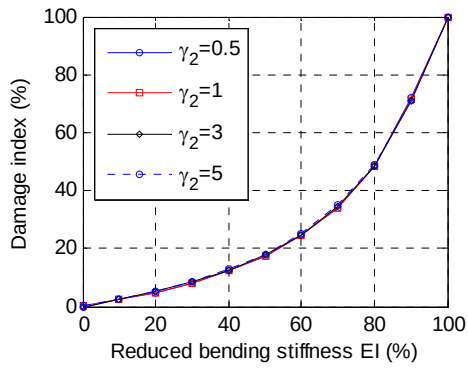
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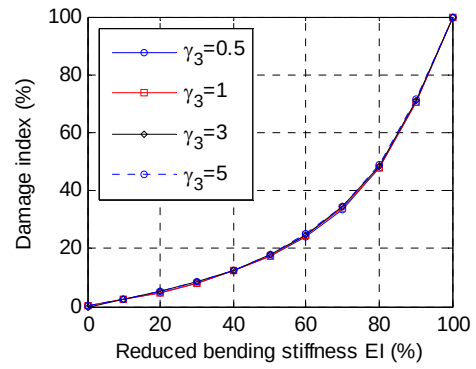
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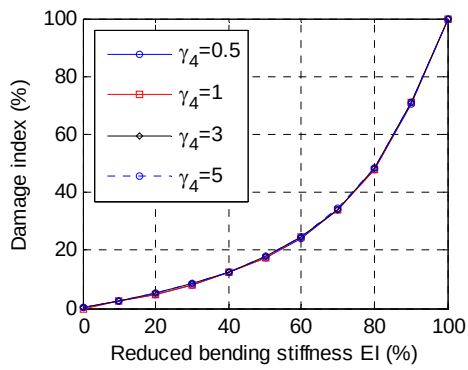
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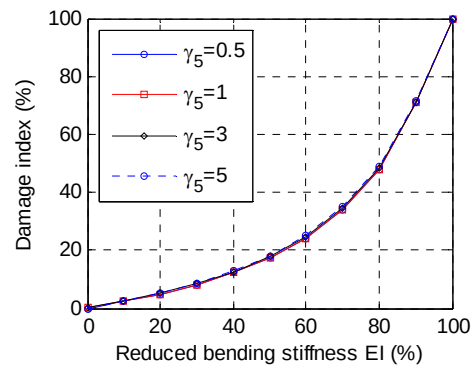
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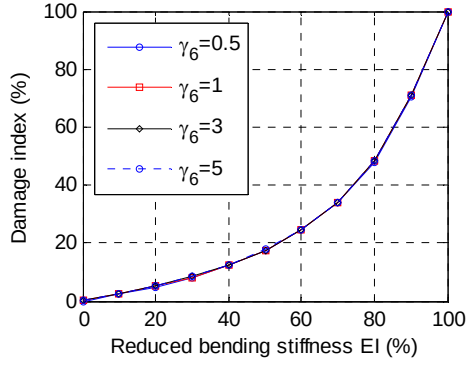
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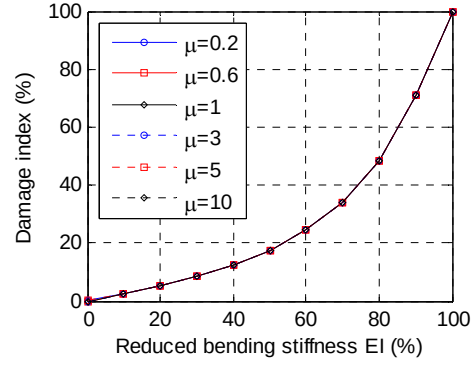
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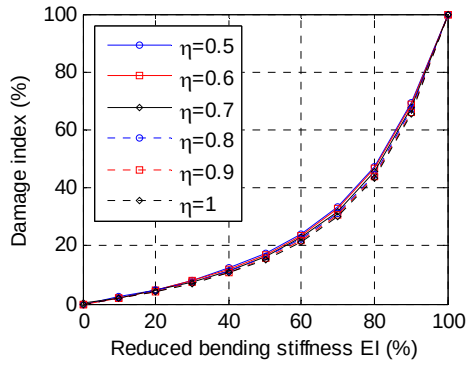
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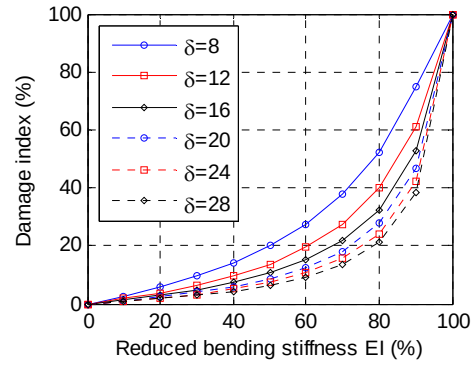
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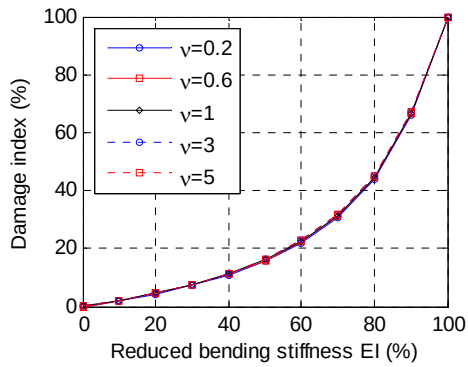
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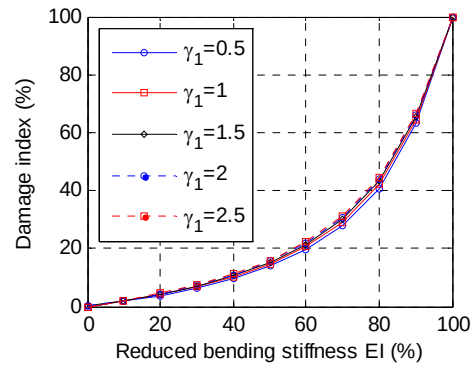
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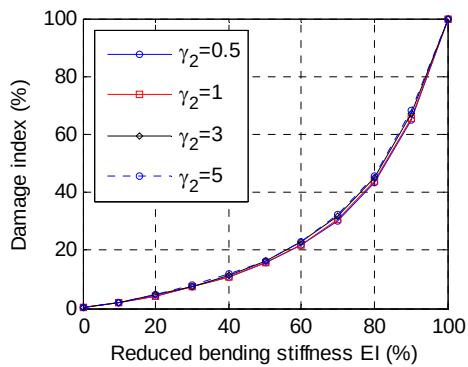
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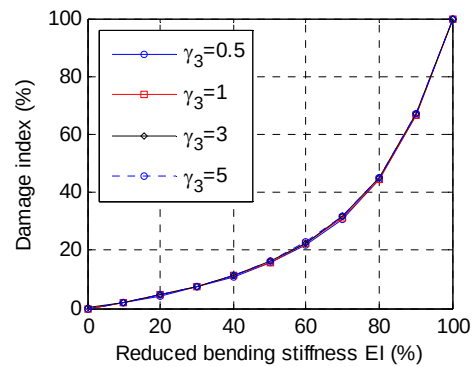
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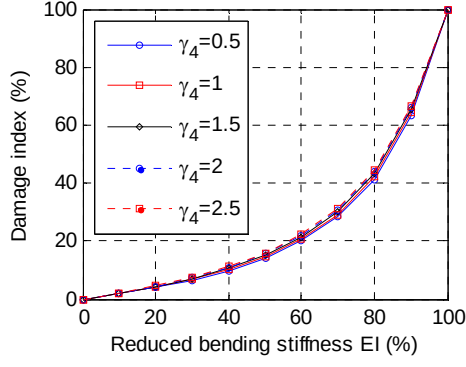
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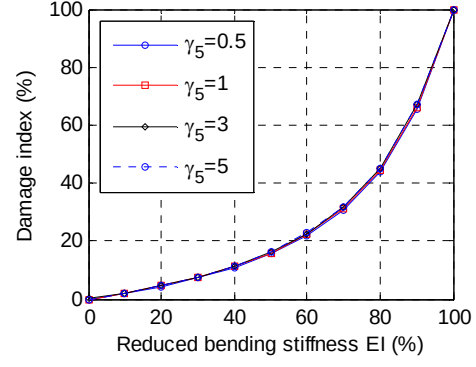
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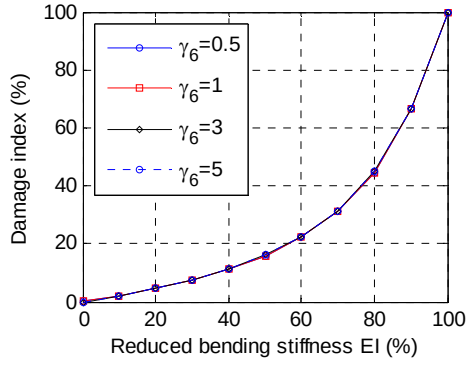
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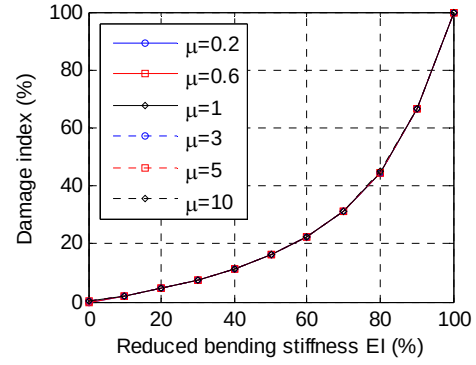
Subset 17



Subset 18



Subset 19



Subset 20

Fig. 5.4 Damage curves for the set of frames

5.3.4 Simplified closed-form expression

Now the closed-form expression of the damage index is formulated with δ as a single variable and the others as constants, i.e., $\eta = 1$; $\nu = 1$; $\gamma_1 = \gamma_2 = \gamma_3 = \gamma_4 = \gamma_5 = \gamma_6 = 1$; $\mu = 1$. The coefficients A_1 , A_2 , A_3 , B_1 , B_2 , and B_3 in Equation (5.2) become as follows:

for damage on interior connection

$$A_1 = -1.803\delta^4 + 5.657\delta^3 - 5.989\delta^2 + 2.671\delta - 0.387, \quad (5.3a)$$

$$A_2 = 1.803\delta^4 - 5.657\delta^3 + 5.989\delta^2 - 2.552\delta, \quad (5.3b)$$

$$A_3 = 0, \quad (5.3c)$$

$$B_1 = \delta^5 - 5.074\delta^4 + 7.570\delta^3 - 6.253\delta^2 + 2.671\delta - 0.387, \quad (5.3c)$$

$$B_2 = -2.000\delta^5 + 8.345\delta^4 - 7.570\delta^3 + 6.253\delta^2 - 2.552\delta, \quad (5.3d)$$

$$B_3 = \delta^5 - 3.271\delta^4, \quad (5.3e)$$

for damage on exterior connection

$$A_1 = -1.502\delta^4 + 4.659\delta^3 - 5.014\delta^2 + 2.280\delta - 0.328, \quad (5.3f)$$

$$A_2 = 1.502\delta^4 - 4.659\delta^3 + 5.014\delta^2 - 2.161\delta, \quad (5.3g)$$

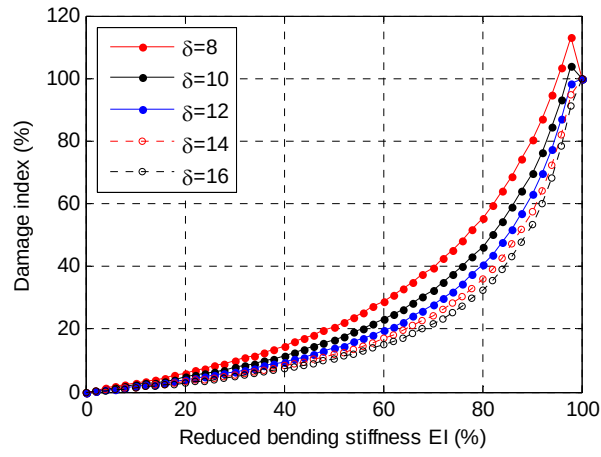
$$A_3 = 0, \quad (5.3h)$$

$$B_1 = \delta^5 - 4.272\delta^4 + 5.608\delta^3 - 4.791\delta^2 + 2.280\delta - 0.328, \quad (5.3i)$$

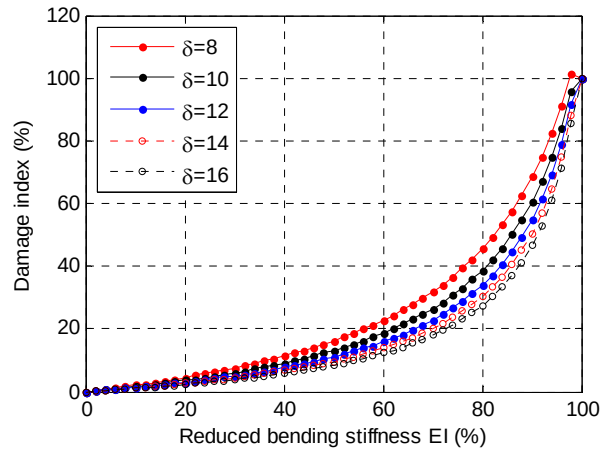
$$B_2 = -2.000\delta^5 + 7.042\delta^4 - 5.608\delta^3 + 4.791\delta^2 - 2.161\delta, \quad (5.3j)$$

$$B_3 = \delta^5 - 2.770\delta^4, \quad (5.3k)$$

Fig. 5.5 illustrates the damage curves derived from the simplified expression including numerical error. When the reduced bending stiffness approaches 100%, the damage index exceeds 100% in the cases where the span-depth ratio equals 8 and 10 for damage on interior connection and the span-depth ratio equals 8 for damage on exterior connection. This is because when the reduced bending stiffness is close to 100%, the values of numerator and denominator of Equation (5.2) are about zero (Fig. 5.6) and thus bring some numerical error in those three cases. Practically, when the reduced bending stiffness is more than 90%, i.e., almost complete fracture, the damage index is about 100%. Hence, the damage index of more than 100% can be reasonably rectified as 100%.

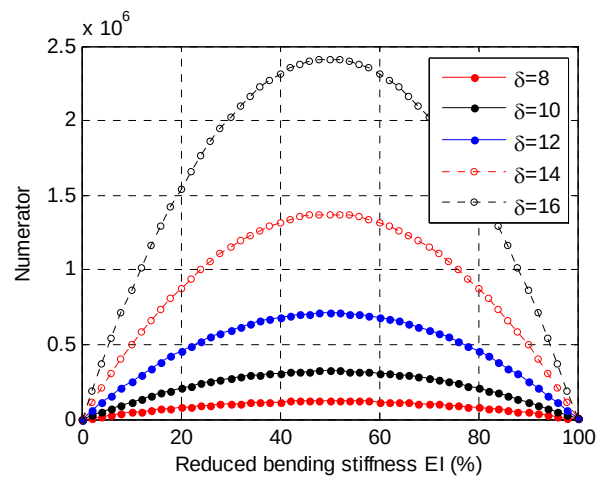


(a)

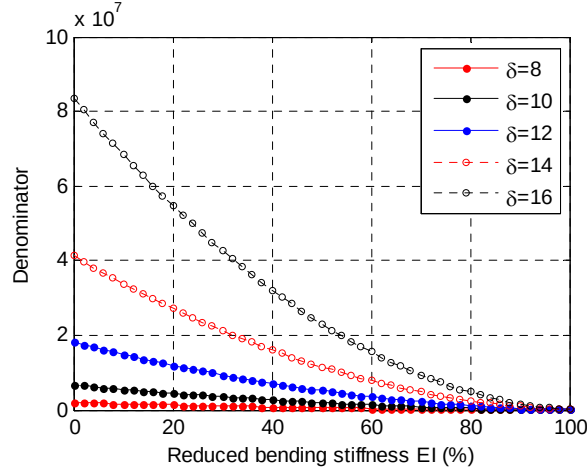


(b)

Fig. 5.5 Damage curve derived from simplified expression: (a) damage on interior connection; (b) damage on exterior connection



(a)



(b)

Fig. 5.6 Numerator and denominator of the simplified expression for damage on interior connection:
(a) numerator; (b) denominator.

Given the damage index DI , reduction of the bending stiffness at a fractured section is expressed as:

$$\rho = \frac{-(B_2(DI) - A_2) - \sqrt{(B_2(DI) - A_2)^2 - 4(B_1(DI) - A_1)(B_3(DI) - A_3)}}{2(B_1(DI) - A_1)} \quad (5.4)$$

5.4 Verifications

5.4.1 SAC nine-story steel moment-resisting frame

The derived expression of the damage curve was verified through numerical studies of a multi-story multi-bay frame with fractures (Fig. 5.7). The frame was the nine-story steel moment-resisting frame at a Los Angeles site with pre-Northridge design, which was intensively studied in the SAC steel project and whose details were in FEMA-355C [4]. The numerical studies were conducted using the finite element analysis software, Marc [5]. In numerical model, the fractured beams and the beam with reference point (see Fig. 5.7) were modeled using shell elements, and other beams and columns were modeled using beam elements. The nodes of shell elements at the beam-ends were connected to the nodes of beam elements with rigid links. Fractures were simulated by removing shell elements on beams.

A numerical damage curve was constructed by introducing various levels of fracture at one beam end, and it was compared with the closed-form expression. Nine damage cases with different fracture locations were considered (see Fig. 5.7). In Case (a) to Case (d), fracture damage was

simulated at exterior beam-column connections, while fractures in Case (e) to Case (i) were at interior connections. The sections of the fractured beam were W36×160 in Case (a) and Case (f), W36×135 in Case (b), Case (c), Case (e), Case (g) and Case (h), and W30×99 in Case (d) and Case (i). The structural parameters of the analytical model for each damage case are listed in Table 5.2.

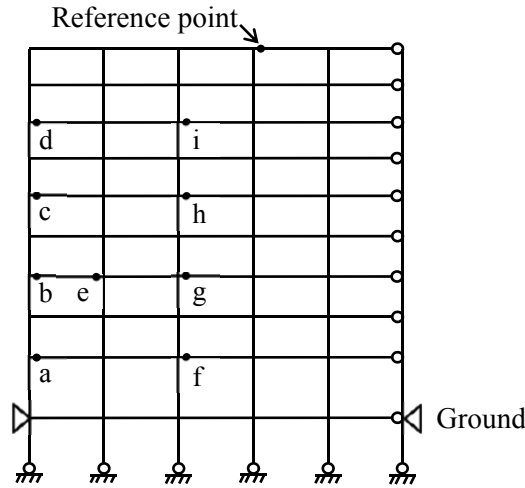


Fig. 5.7 Nine-story steel moment-resisting frame with different fracture location

Table 5.2 Damage cases in numerical studies

Damage cases	Beam section	Structural parameters of analytical model								
		η	δ	ν	γ_1	γ_2	γ_3	γ_4	γ_5	γ_6
Case (a)	W36×160	0.72	10.00	1.00	0.93	1.40	1.40	1.29	1.94	1.94
Case (b)	W36×135	1.00	10.11	1.00	1.61	2.13	2.13	1.61	2.13	2.13
Case (c)	W36×135	1.00	10.11	1.00	1.14	1.61	1.61	1.14	1.61	1.61
Case (d)	W30×99	1.00	12.12	0.71	1.97	2.22	2.22	1.97	2.22	2.22
Case (e)	W36×135	1.00	10.11	1.00	1.61	2.13	2.13	1.61	2.13	2.13
Case (f)	W36×160	0.72	10.00	1.00	1.40	1.40	1.40	1.94	1.94	1.94
Case (g)	W36×135	1.00	10.11	1.00	2.13	2.13	2.13	2.13	2.13	2.13
Case (h)	W36×135	1.00	10.11	1.00	1.61	1.61	1.61	1.61	1.61	1.61
Case (i)	W30×99	1.00	12.12	0.71	2.22	2.22	2.22	2.22	2.22	2.22

As most beam fractures initiate at the toe of the weld access hole and then extend to the web, beam fractures were simulated by cutting the bottom flange and/or web near the column surface. The cut width was one percent of the beam length. Seven damage patterns were considered at one beam end, as summarized in Table 5.3. DP1 to DP3 simulated fracture at one side of the bottom

flange, where the decreases of the bending stiffness EI_x at the cut sections were smaller than 23% for the three beam sections. DP4 simulated the entire bottom flange fracture, in which the bending stiffness EI_x at the cut sections decreased by around 50%. Severe fracture damage extending from the bottom flange to the web was simulated in DP5 to DP7 with the reductions of more than 75% in the bending stiffness. The damage index was computed from the bending strain responses of beams measured at 1.5 beam depths away from the column surface using the extraction procedure reported in the Chapter 3.

Table 5.3 Damage patterns for fracture simulation

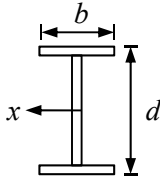
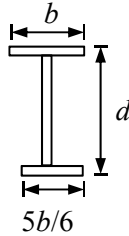
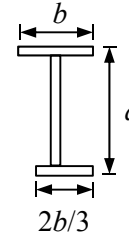
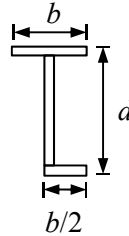
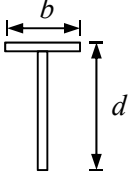
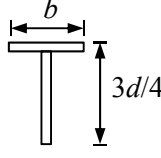
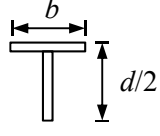
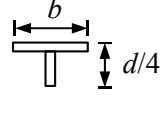
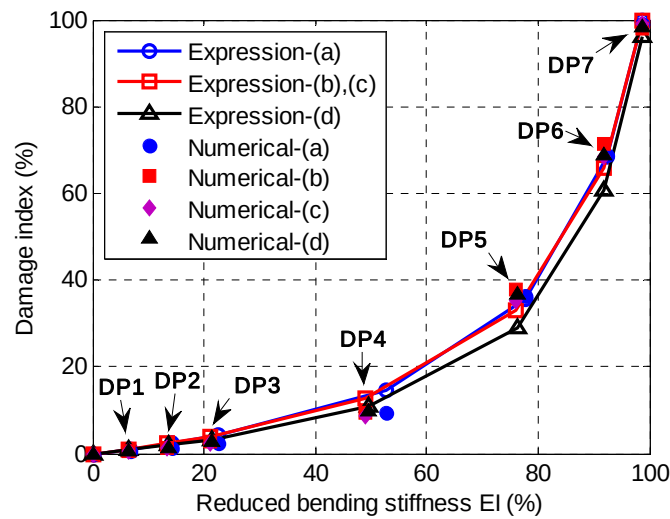
Damage pattern		Undamaged	DP1	DP2	DP3
Cross-section					
EI_x reduction (%)	W36×160	0	6.8	14.2	22.4
	W36×135	0	6.5	13.5	21.2
	W30×99	0	6.5	13.6	21.4
Damage pattern		DP4	DP5	DP6	DP7
Cross-section					
EI_x reduction (%)	W36×160	52.8	77.8	92.3	98.7
	W36×135	49.1	76.1	91.8	98.7
	W30×99	49.6	76.4	91.9	98.7

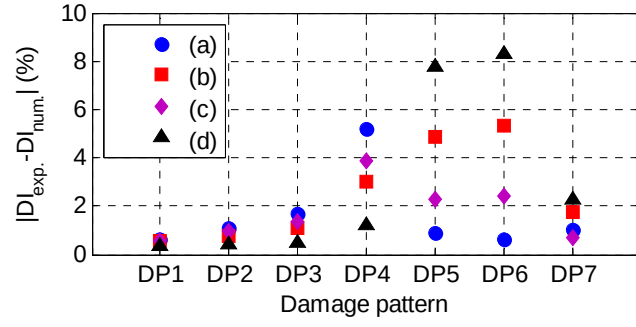
Fig. 5.8 compares the damage index derived from numerical analyses and that computed from the closed-form expression with assigned reduced bending stiffness. Compared to the numerical results, the damage index derived from the expression had the absolute difference of at most 8.2% for damage on exterior connections, i.e., in Case (a) to Case (d), and 6.6% for damage on interior connections, i.e., in Case (e) to Case (i). The consistency between the presented expression and numerical analysis implies that the expression was effective in deriving damage curve for quantifying the amount of beam fractures in steel moment-resisting frames. Case (b), Case (c), Case

(e), Case (g), and Case (h) had the same span-depth ratio and different stiffness ratio of the column to the beam (see Table 5.2). The largest absolute discrepancy between numerical damage indices in Case (b) and Case (c) was 2.9% and that among Case (e), Case (g), and Case (h) was 2.1%. This indicates that the damage curves for the fractured beams with the same span-depth ratio are nearly identical and the damage curve is mainly dominated by the span-depth ratio as mentioned in the preceding parametric analysis.

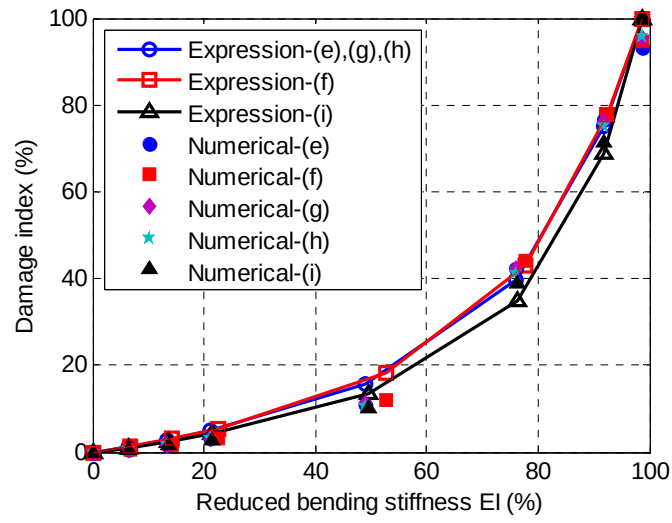
Fig. 5.9 illustrates the reduced bending stiffness computed with Equation (5.4) and numerically-estimated damage indices. Compared to the exact values (i.e., computed from the sectional properties), the estimates had the absolute discrepancy of at most 12.2% for beam W36×160, 10.9% for beam W36×135, and 7.3% for beam W30×99. The discrepancy at DP1 to DP4 was larger than at DP5 to DP7 where it was about 5% for DP1 to DP4, and 1% for DP5 to DP7 in average. This means that Equation (5.4) slightly underestimated the reduced bending stiffness at DP1 to DP4 by about 5% in average. This is because of the approximation in the adopted crack model where the equivalent length of $0.75d$ fixed independently to beam depth was slightly long for bottom flange fracture.



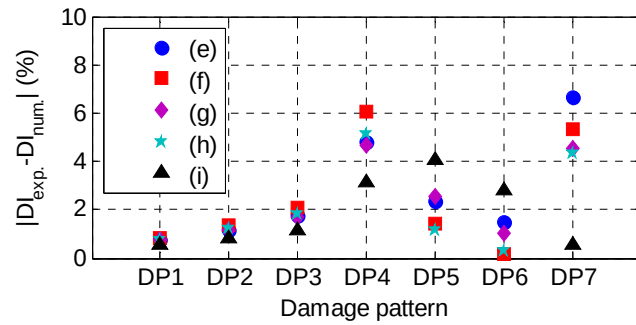
(a)



(b)

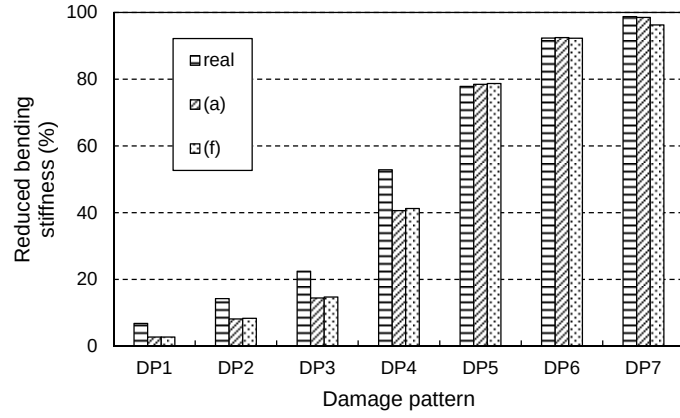


(c)

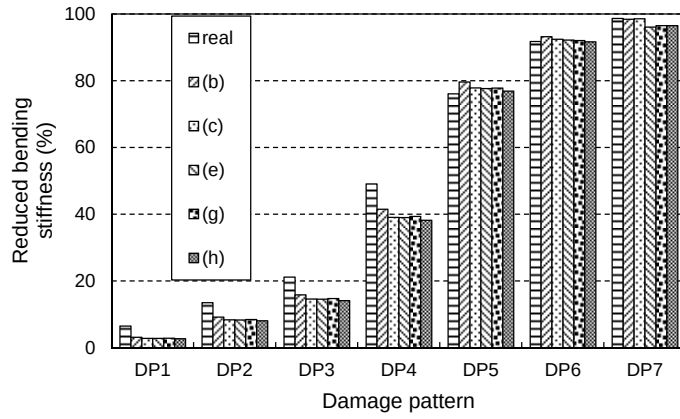


(d)

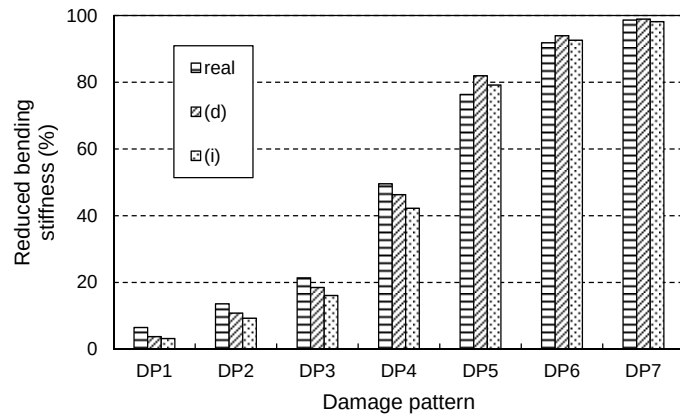
Fig. 5.8 Comparison of damage index obtained from the presented expression and numerical analysis: (a) damage curve and (b) absolute difference for damage on exterior connection; (c) damage curve and (d) absolute difference for damage on interior connection



(a)



(b)



(c)

Fig. 5.9 Reduced bending stiffness computed from Equation (5.4) using numerical damage index: (a) beam W36×160; (b) beam W36×135; (c) beam W33×99

5.4.2 Five-story steel frame testbed

The presented simplified expression of the damage curve was further verified with experimental studies of the five-story steel frame testbed. Fig. 5.10 illustrates the cross-section of the removable steel connection and four levels of simulated fracture damage. Damage level 1 to level 4 (L1 to L4) corresponded to fracture of half bottom flange, fracture of whole bottom flange, fracture of bottom flange and one-quarter web, and fracture of bottom flange and half web, respectively. As summarized in Table 5.4, the reduction in the bending stiffness about the strong axis of the beam section was 21.9% for damage L1, 53.4% for damage L2, 79.4% for damage L3, and 93.6% for damage L4. The length of each fracture was $0.8d$ (the beam depth d was 100 mm).

Four damage cases with different fracture position were considered as summarized in Table 5.5. In Case (a) and Case (b), fractures were at the removable connections B1 and B5 (see Fig. 3.4(b)) near exterior beam-column connections, while in Case (c) and Case (d) fractures were at the removable connections B2 and B6 (see Fig. 3.4(b)) near inner beam-column connections. The fractured beam in four cases had the same span-depth ratio.

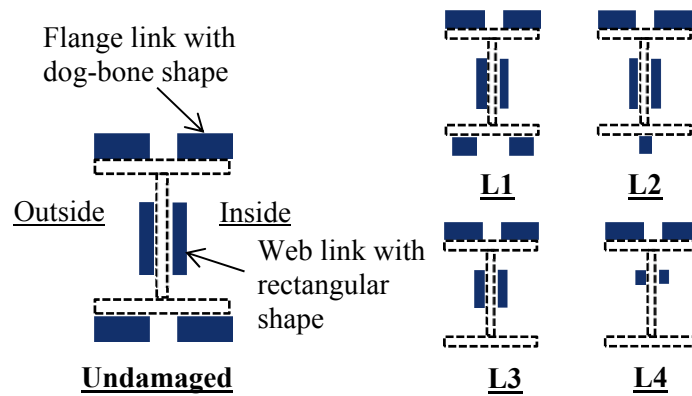


Fig. 5.10 Cross-section of connection and damage patterns

Table 5.4 Damage patterns

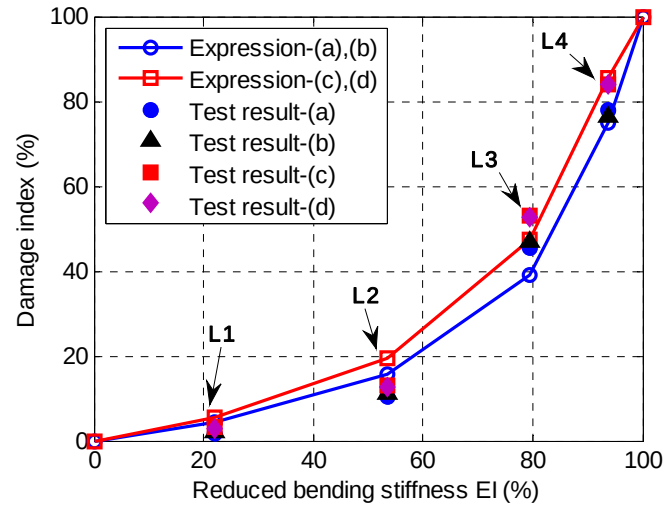
Damage pattern	Target of simulation	Reduction of EI_x (%)
L1	Fracture of half bottom flange	21.9
L2	Fracture of whole bottom flange	53.4
L3	Fracture of bottom flange and one-quarter web	79.4
L4	Fracture of bottom flange and half web	93.6

Table 5.5 Damage cases

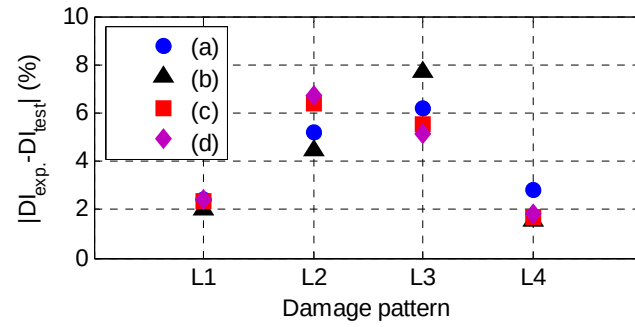
Damage cases	Location	Structural parameters of analytical model								
		η	δ	ν	γ_1	γ_2	γ_3	γ_4	γ_5	γ_6
Case (a)	B1	0.96	20.00	1.00	1.25	3.46	1.25	1.30	3.60	1.30
Case (b)	B5	1.00	20.00	1.00	1.30	3.60	1.30	1.30	3.60	1.30
Case (c)	B2	0.96	20.00	1.00	1.25	3.46	1.25	1.30	3.60	1.30
Case (d)	B6	1.00	20.00	1.00	1.30	3.60	1.30	1.30	3.60	1.30

PVDF strain sensors were attached on both sides of the beam bottom flange at $1.5d$ from the edge of the fracture. 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). The average of the damage index at two sides of the bottom flange was used to construct experimental damage curve. In analytical model, the distance between sensor location and column surface l_s was $3.3d$, and the equivalent length c was $1.55d$ (including the non-negligible length of fracture and beam-end part for connecting links with bolts). Note that the number coefficients in the expressions of A_1 , A_2 , A_3 , B_1 , B_2 , and B_3 changed compared to those in Equations (5.3).

Fig. 5.11 compares the damage index obtained from the simplified expression and experimental investigations. Compared to the experimental results, the analytical damage index had the absolute difference of about 2% at damage L1 and L4, and about 6% at damage L2 and L3 in each damage case. Small discrepancy between the analytical and experimental damage indices indicated that the simplified expression was effective in constructing the damage curves for beam fractures in steel moment-resisting frames. As fractured beams in Case (a) to Case (d) had the same span-depth ratio, the experimental damage indices of four damage patterns in Case (a) and Case (b) (i.e., damage on two different exterior connections) had slight difference of less than 1.5%, and those in Case (c) and Case (d) (i.e., damage on two different interior connections) had a difference of smaller than 0.4%, which implies that the damage curve was mainly affected by the span-depth ratio of fractured beam as indicated by the preceding parametric analysis and numerical studies.



(a)



(b)

Fig. 5.11 Comparison of damage index obtained from simplified expression and experimental investigation: (a) damage curve; (b) absolute difference

Fig. 5.12 shows reduced bending stiffness evaluated from Equation (5.4) using experimental damage index. When the evaluated values of reduced bending stiffness was compared with the exact values, the absolute difference was about 9% for damage L1 and L2, 3% for damage L3 and L4. In real measurement systems, data processing procedures, and construction of specimens, errors or uncertainties (e.g., outliers, distortion with filters, and uncertainties in connections) were inevitable and thus influenced the experimental damage index. Compared to the preceding numerical studies, the differences of the damage index and the evaluated reduced bending stiffness between analytical and experimental values nearly remained at the same level of less than 12%. This demonstrates the powerful capability of the simplified method of deriving damage curve, and the damage evaluation with strain-based damage index and damage curve in realistic applications. The simplified method can facilitate the application of the proposed damage evaluation method for

identifying the location and extent of localized damage and thus monitoring the health conditions of steel frames under earthquake loadings.

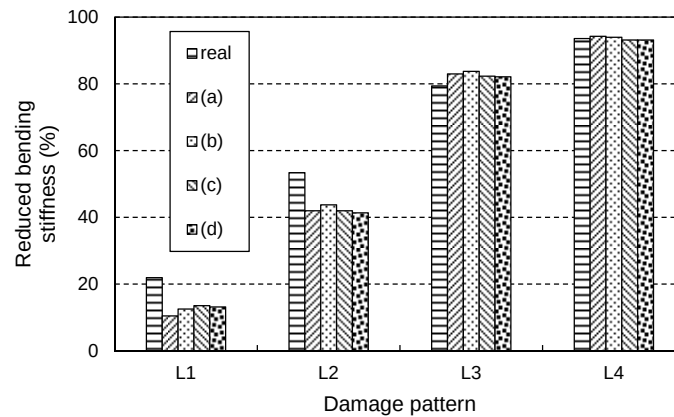


Fig. 5.12 Reduced bending stiffness evaluated from Equation (5.4) using experimental damage index

5.5 Summary

The conclusions of this chapter are as follows.

(1) This chapter presented a simplified method of deriving damage curve to quantify the damage extent of beam fractures in multi-story multi-bay steel moment-resisting frames under earthquake loading. A closed-form expression of damage curve was developed based on an analytical parametric study using a two-story two-bay frame.

(2) The closed-form expression was verified through the numerical studies of the nine-story steel moment-resisting frame and the vibration tests of a one-quarter-scale five-story steel frame. It was demonstrated that the damage curve was primarily dominated by the span-depth ratio of fractured beam and hardly affected by the other structural parameters, such as the height ratio between stories, the column-to-beam stiffness ratio, and the lateral force ratio between floors. Thus, the developed expression can be applicable to common steel moment-resisting frames.

(3) In this study, the damage curve was constructed for a single beam fracture. When damage curve is applied for a frame with multiple beam fractures, the damage index is needed to be uncoupled to remove marginal but non-negligible interaction between neighboring beam fractures. The method for the uncoupling is a topic of the next chapter.

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CHAPTER 6

Decoupling interaction between multiple beam fractures

6.1 Overview

The increase of the damage index with the existence of neighboring fractures, as mentioned in the Chapter 4, indicates that the damage indices influence each other when a steel moment-resisting frame suffers fractures at multiple beam ends. This chapter presents a decoupling method for considering the interaction between multiple damages in order to quantify the damage extent of each beam fracture accurately. First, the influence of moment release by fracture is studied with a simple sub-frame. Then, a decoupling method of estimating the damage index for multiple beam fractures is formulated. Finally, the effectiveness of the decoupling method is verified numerically through a nine-story steel moment-resisting frame and experimentally using the five-story steel frame testbed.

6.2 Influence of moment release

Inclusion of beam fracture in a steel moment-resisting frame results in the release of the bending moment sustained by the fractured section and thus the bending moments are re-distributed in the frame. The following analytical study on a simple sub-frame, which is extracted from a multi-story multi-bay frame, first demonstrates the influence of moment release by fracture.

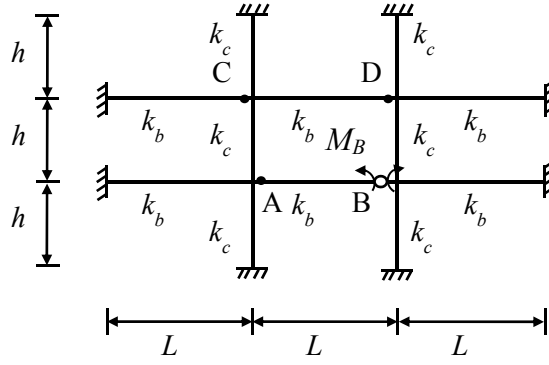


Fig. 6.1 A sub-frame for studying moment release and influence

A three-story three-bay sub-frame shown in Fig. 6.1 is considered, where k_b and k_c are the bending stiffness of beams and columns, respectively; h denotes the height of each story; L is the width of each span. The bending moment M_B is the release of the moment caused by fracture damage at the beam end B. Assuming that the frame behaves linearly, the bending moments at the beam ends A, C and D generated by the released moment M_B are calculated by the displacement method of structural analysis as,

$$M_A = C_{BA}M_B = f_1(a)M_B, \quad (6.1(a))$$

$$M_C = C_{BC}M_B = f_2(a)M_B, \quad (6.1(b))$$

$$M_D = C_{BD}M_B = f_3(a)M_B. \quad (6.1(c))$$

where C_{BA} , C_{BC} , and C_{BD} are influence coefficients, which only relate to the beam-to-column stiffness ratio a ($= k_c/k_b$). Fig. 6.2 illustrates the relationships between the influence coefficients and the beam-to-column stiffness ratio. The beam-to-column stiffness ratio ranges from 1 to 5 for common steel moment-resisting frames. The influence coefficient C_{BA} is more than 0.4 for the beam end A at the same floor level, while the influence coefficients C_{BC} and C_{BD} are less than 0.05 for the beam ends C and D at the neighboring floor. This implies the released moment M_B mainly distributes on the same floor level and the influence to neighboring floors are very small and negligible.

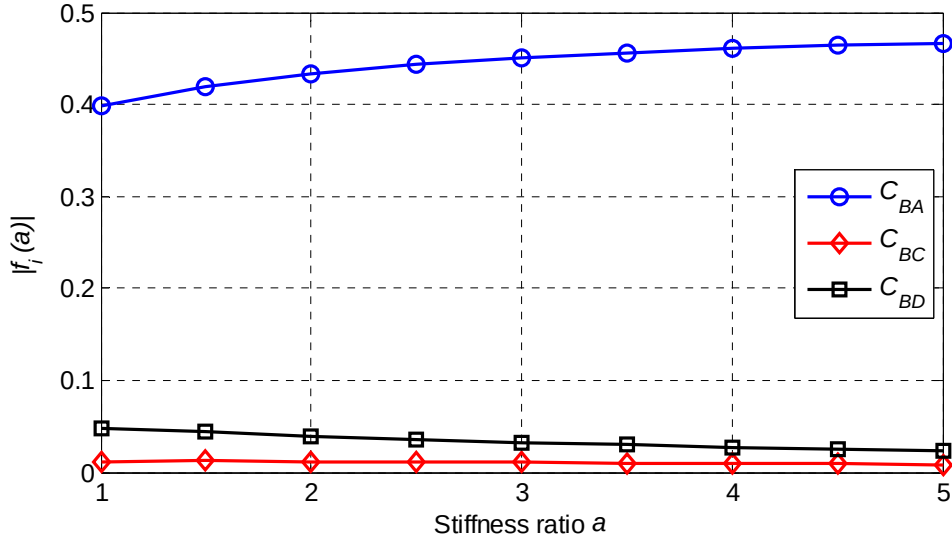


Fig. 6.2 Relation between influence coefficient and beam-to-column stiffness ratio

6.3 Decoupling method

In order to use the damage curve derived for single damage to evaluate the damage extents of multiple beam fractures, the interaction between multiple damages needs to be uncoupled. The interaction between beam fractures located at two different floors is assumed to be negligible as observed in the preceding analytical study. The influence coefficients between damage indices at different sensor locations can be computed using the moment release and influence method.

A floor of an n -span steel moment-resisting frame is considered (Fig. 6.3(a)). The total of $2n$ strain sensors (S_1 to S_{2n}) are used to monitor probable damage at all beam ends on the floor. The measured damage index— $(DI)_i$ ($i = 1, \dots, 2n$) at each sensor can be expressed as a combination of the damage index associated with each individual beam fracture— $(DI)'_j$ ($j = 1, \dots, 2n$) as follows,

$$\begin{bmatrix} (DI)_1 \\ \vdots \\ (DI)_i \\ \vdots \\ (DI)_{2n} \end{bmatrix} = \begin{bmatrix} C_{1,1} & \cdots & C_{1,j} & \cdots & C_{1,2n} \\ \vdots & & \vdots & & \vdots \\ C_{i,1} & \cdots & C_{i,j} & \cdots & C_{i,2n} \\ \vdots & & \vdots & & \vdots \\ C_{2n,1} & \cdots & C_{2n,j} & \cdots & C_{2n,2n} \end{bmatrix} \begin{bmatrix} (DI)'_1 \\ \vdots \\ (DI)'_j \\ \vdots \\ (DI)'_{2n} \end{bmatrix}, \text{ or } (\mathbf{DI}) = \mathbf{C} \cdot (\mathbf{DI})' \quad (6.2)$$

where $C_{i,j}$ ($i = 1, \dots, 2n, j = 1, \dots, 2n$) denotes the influence coefficient from S_j to S_i due to the beam fracture monitored by S_j (Fig. 6.3(b)), which is calculated using the below procedure.

(1) In the steel moment-resisting frame, set the release of the bending moment at the beam end monitored by S_j as unity.

(2) Compute the bending moment sustained at the position of sensor S_i .

(3) Normalize the bending moment sustained at the position of sensor S_i using that at the position of sensor S_j as influence coefficient $C_{i,j}$.

Note that the influence coefficients only relate to the beam-to-column stiffness ratios and the methods of structural inner forces analysis considering the bending stiffness of beams and columns in frames can be used in the calculation of influence coefficients.

Given the measured damage index— $(DI)_i$, the damage index associated with each single damage— $(DI)'_j$ is thus expressed as,

$$(DI)' = C^{-1} \cdot (DI) \quad (6.3)$$

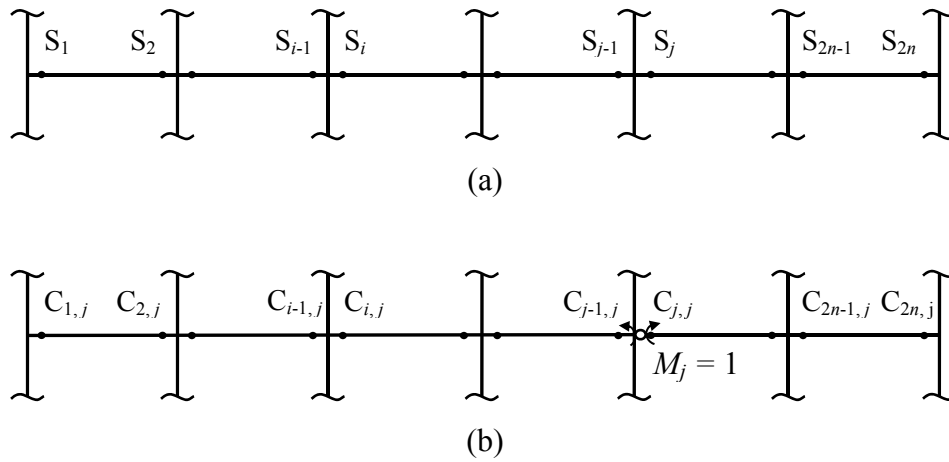


Fig. 6.3 An intermediate floor of an n -span frame: (a) sensors; (b) influence coefficients

6.4 Numerical studies

The effectiveness of the developed decoupling method is verified through numerical studies on the SAC nine-story steel moment-resisting frame (see Fig. 6.4) at a Los Angeles site with pre-Northridge design and whose details are in FEMA-355C [1]. The numerical studies are conducted using SAP2000 software. In the numerical model, all members are modeled using beam elements. Beam fractures are simulated at beam-ends by referring to crack model proposed by Sinha et al. [2], where the fracture is modeled by a segment of beam whose stiffness is reduced to that of the fractured section; the length of the beam segment is determined as 0.75 beam depths for wide flange beams. The fourth floor of the frame with probable beam fractures at the beam ends A to H is

considered (the span with pin connection that is used for connection with the moment-resisting frame at another direction is not included as the beams resist relatively small bending moment). All the sensors, i.e., S1 to S8 at the fourth floor, E1 to E8 at the fifth floor, and Z1 to Z8 at the sixth floor are at 1.5 beam depths from the columns. The bending moment associated with the first mode is used for damage evaluation.

Four damage cases are studied to verify the presented decoupling method, as summarized in Table 6.1. In Case 1 and 2, single beam fracture is simulated at the beam end E. Two beam fractures are at the beam ends B and C around a beam-column connection in Case 3. In Case 4, multiple beam fractures are simulated at the beam ends B, C, E, and F on the fourth floor, while the third floor with the same damage condition is considered to study the influence of neighboring floor with damage on the decoupling method in Case 5.

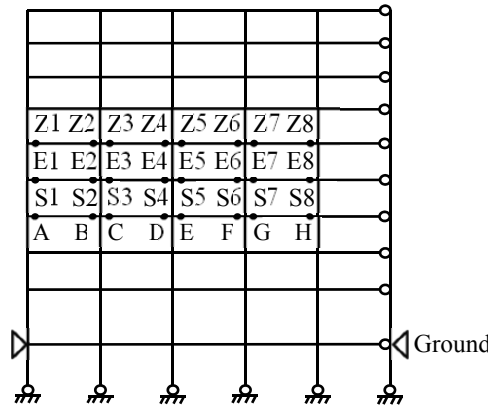


Fig. 6.4 Nine-story steel moment-resisting frame

Table 6.1 Damage cases

Damage cases	Locations (reduction of bending stiffness at fractured section)
Case 1	E (10%)
Case 2	E (90%)
Case 3	B (50%) and C (70%)
Case 4	B (50%), C (70%), E (30%), and F (30%)
Case 5	B (50%), C (70%), E (30%), and F (30%); the 3rd floor with the same damage condition

Fig. 6.5 illustrates the distribution of the damage index caused by the fracture at the beam end E on the fourth floor in Case 2. The horizontal axis denotes 24 sensor locations as shown in Fig. 6.4. The vertical axis denotes the damage index. When the bending stiffness of the beam end E reduced

by 90%, the damage indices of S5 and S6 on the damaged beam were about -70% and -10% respectively. Compared to that, the damage indices of the other sensors increased. The damage index of sensor S4 on the neighboring beam was 21% , and the other sensors at the same floor increased by from 5% to 10% . The damage indices of the sensors at the fifth and sixth floors were less than 4% . This indicates that the release of bending moment induced by the fracture was primarily redistributed in the same floor and the influence to the other floors was nearly negligible.

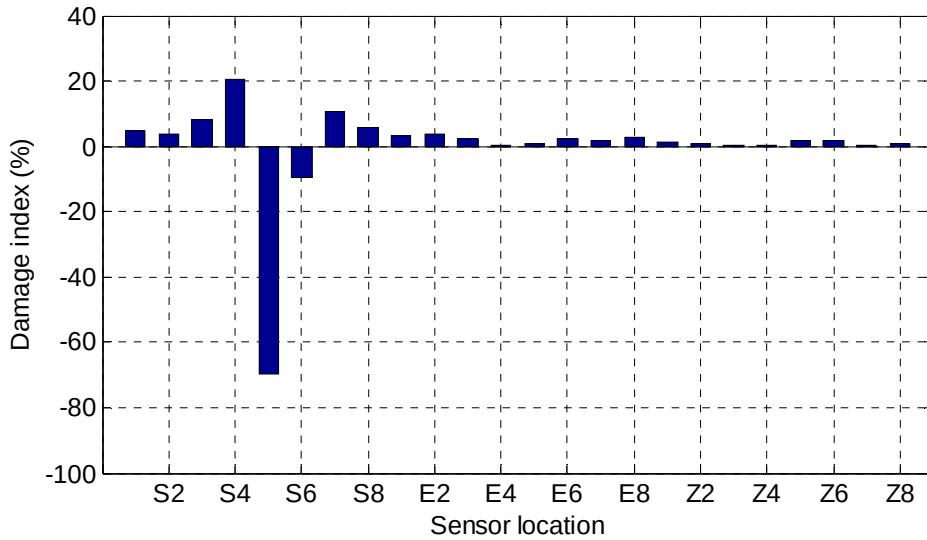


Fig. 6.5 Distribution of damage index caused by fracture at beam end E

Fig. 6.6 shows the distribution of the influence coefficients for the fracture damage at the beam end E. In Case 1 and 2, the influence coefficients were computed by normalizing all damage index values with the damage index of sensor S5. The influence coefficients were identical for two levels of the damage, which indicates that the influence coefficients are independent to the damage extent of fractures. Small discrepancy between the influence coefficients computed by the presented procedure and those in numerical simulations proved that the presented procedure is sufficiently accurate.

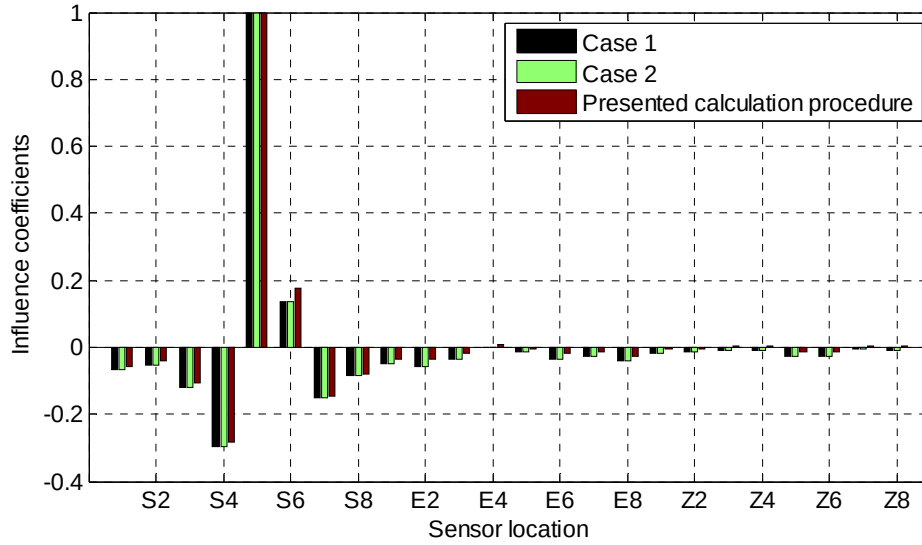
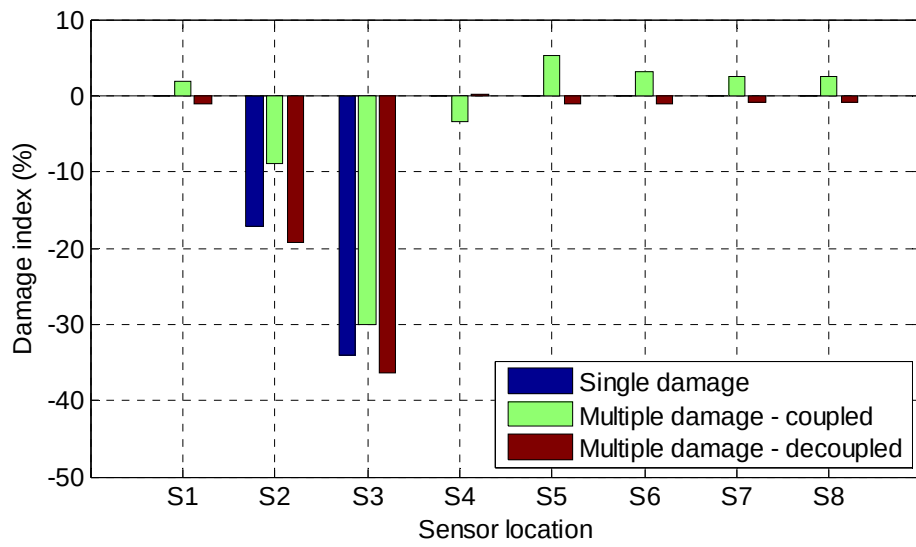


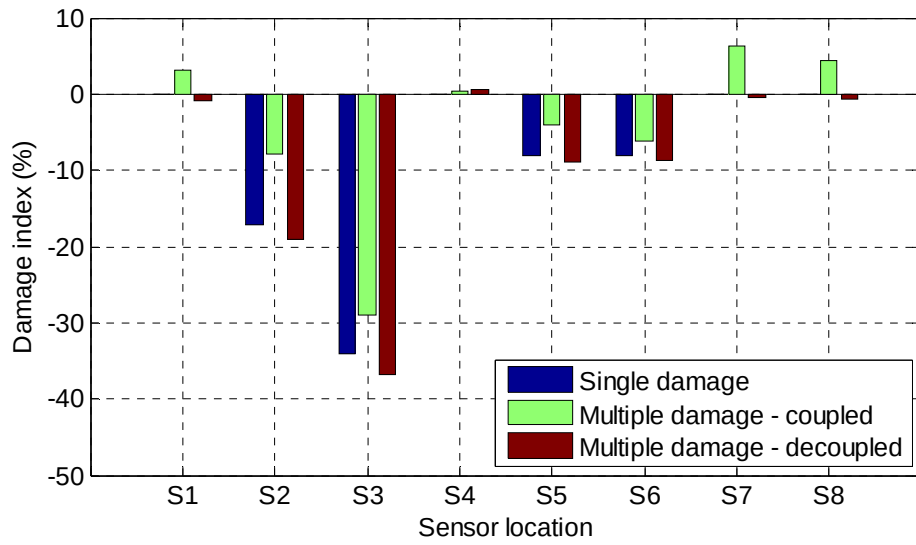
Fig. 6.6 Distribution of influence coefficients caused by fracture at beam end E

Fig. 6.7 compares the damage indices between single and multiple damage conditions in Case 3, 4 and 5. The damage index for multiple damage conditions includes coupled and decoupled values. In Case 3, the values of the damage index at S2 and S3 were about -17.1% and -34% for single damage at the beam ends B and C on the fourth floor, respectively (i.e., the reduction of bending stiffness was 50% for single fracture at the beam end B, and 70% for single fracture at the beam end C). When the frame suffered multiple damages with the same extent at the beam ends B and C, the coupled damage indices at S2 and S3 increased by 8.3% and 3.9% compared to those for the single damage. Using the decoupled method, the decoupled damage index for multiple damage condition had the discrepancy of less than 2.5% for two fractures by comparison with the damage index for single fracture. In addition, the false positive error of -3.3% at sensor S5 under the coupled condition increased to zero after the decoupling. This indicates that the decoupling method was effective in separating the interaction between multiple damages.

In Case 4, two more beam fractures with relatively small damage extent, i.e., the decrease of 30% in the bending stiffness, were simulated at the beam ends E and F on the beam at the neighboring span compared to Case 3. The decoupled damage indices had the differences of less than 2.5% for four fractures compared to the damage index for individual damage, especially the difference of less than 0.8% for small fractures at E and F. In Case 5, when the third floor had the same damage condition, the difference between the decoupled damage index for multiple damage condition and the damage index for single damage condition was at most 2.7% at the beam end F. These results further verified the effectiveness of the presented decoupled method for complicated damage conditions.



(a)



(b)

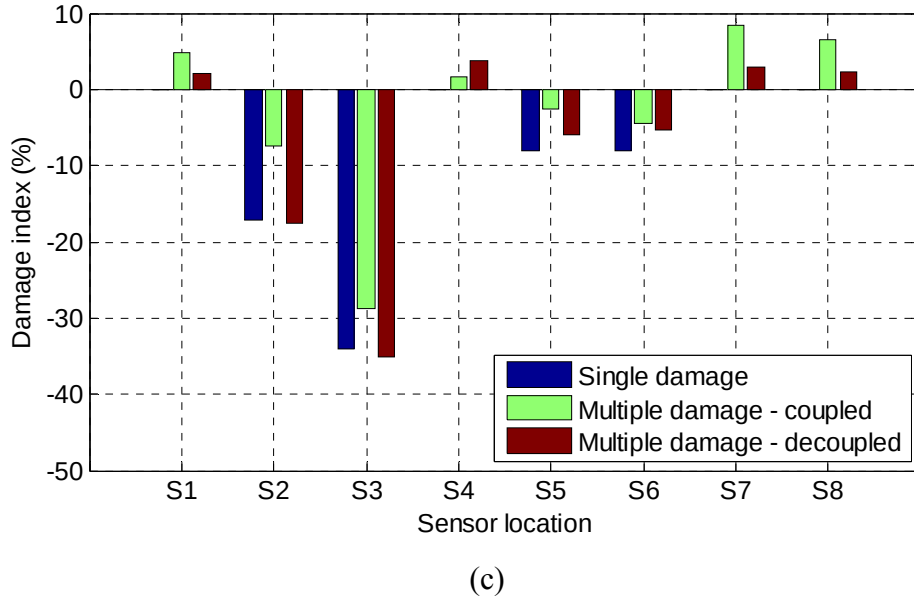


Fig. 6.7 Comparison of damage index between single and multiple damage conditions: (a) Case 3; (b) Case 4; (c) Case 5

Fig. 6.8 shows the reduction of bending stiffness estimated with the decoupled damage index for the multiple damage condition in Case 5. The estimation was obtained using the closed-form expression of damage curve formulated in the Chapter 5. The exact values were calculated from the sectional properties of beams. Compared to the exact values, the estimated values had the differences of at most 7% for the damage at the beam end F, and the presented decoupling method was competent in estimating the reduction of bending stiffness under multiple damage conditions.

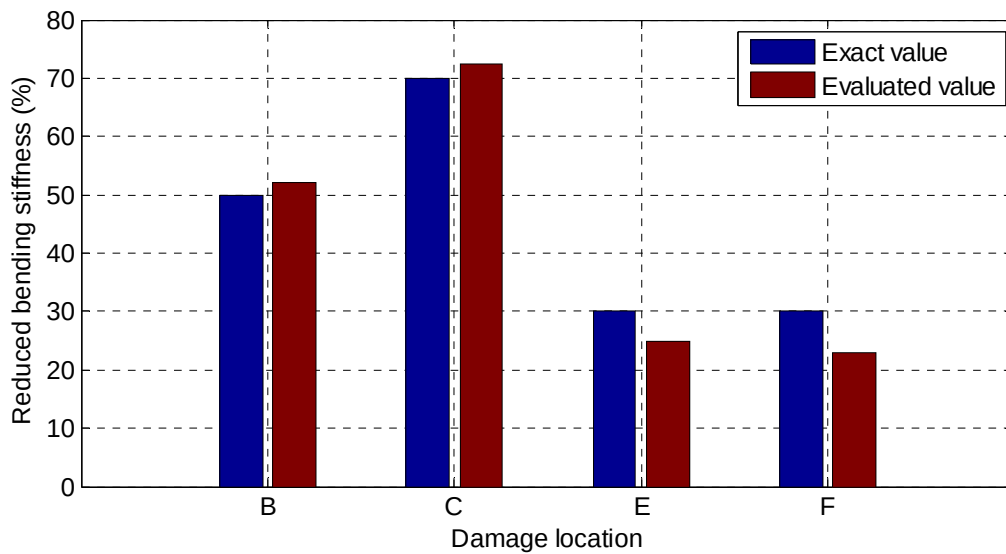


Fig. 6.8 Reduction of bending stiffness evaluated from the decoupled damage index in Case 5

6.5 Experimental investigations

The presented decoupling method was further verified with experimental studies using the five-story steel frame testbed. Two tests including seven damage cases were considered (Table 6.2), where the damage patterns (i.e., damage L2 to L4) were illustrated in Fig. 5.10 and Table 5.4 and the removable connections were shown in Fig. 3.4(b). In Test 1, individual damage L4 was simulated at four removable connections at the second floor in order to investigate the influence coefficients. In Test 2, multiple damage cases were studied for the verification of the decoupling method. For comparative studies of the damage index for single and multiple damage conditions, all damages in multiple damage conditions was also individually tested at the same locations in Test 2.

Table 6.2 Damage cases in experimental investigation

Tests	Damage cases	Removable connections (Damage patterns)	Targets
Test 1	Case 1	B1 (L4)	Investigation of influence coefficients
	Case 2	B2 (L4)	
	Case 3	B3 (L4)	
	Case 4	B4 (L4)	
Test 2	Case 5	B2 (L4) and B3 (L2)	Verification of the decoupling method
	Case 6	B2 (L3), B3 (L2), and B4 (L3)	
	Case 7	B1 (L3), B3 (L2), B4 (L4), and B5 (L3)	

PVDF strain sensors were attached on both sides of the beam bottom flange at 1.5 beam depths from the edge of the fracture. 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). The average of the damage index at two sides of the bottom flange was used in the investigation of the decoupling method. Two PVDF strain sensors at the same beam section were treated as one sensor location. There were 12 sensor locations, i.e., S1 to S12, located in the second to fourth floors, as shown in Fig. 6.9.

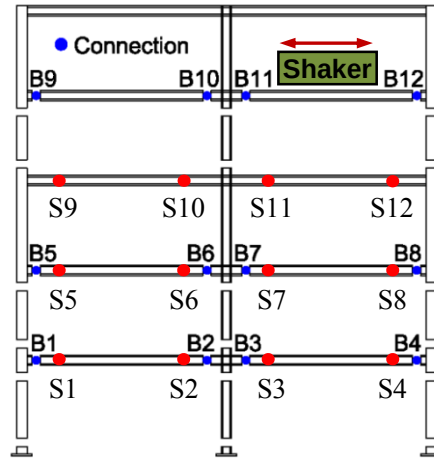


Fig. 6.9 Sensor locations

The region in the frame influenced by one beam damage was investigated with the distribution of damage index caused by the damage. Fig. 6.10 illustrates the distribution of damage index induced by damage L4 at the connection B1. When damage L4 was simulated at the connection B1, i.e., the reduction of 93.6% in the bending stiffness, the damage index of sensor S1 was -74.2% . The damage index at other location on the same floor was at most 19.1% at S3, while the largest values of the damage index for the third and fourth floors were 8.0% at S8 and 0.9% at S9. This verified that the release of moment caused by beam damage mainly distributed on the same floor as demonstrated in the previous analytical studies and numerical analysis.

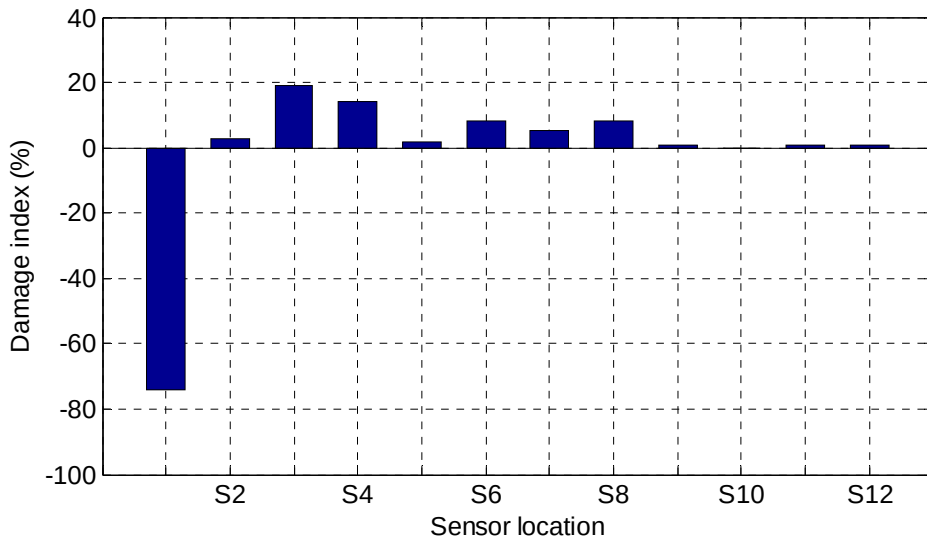


Fig. 6.10 Distribution of damage index in Case 1

The experimental matrix C_e of influence coefficient was extracted from the damage indices of all damage cases in Test 1 where individual damage L4 was simulated at the connections B1 to B4 in turn. For example, the first column of C_e was calculated by normalizing the damage indices of sensors S1 to S4 using the damage index of sensor S1 when damage L4 existed at the connection B1 in Case 1. The analytical matrix C_p was obtained using the presented procedure for calculating influence coefficients. The two matrices are shown in Equation (6.4). When the analytical matrix C_p was compared with the experimental matrix C_e , only two coefficients $C_p(2, 3)$ and $C_p(4, 2)$ had some non-negligible differences. This indicated that the presented procedure for influence coefficient was effective in the experimental applications.

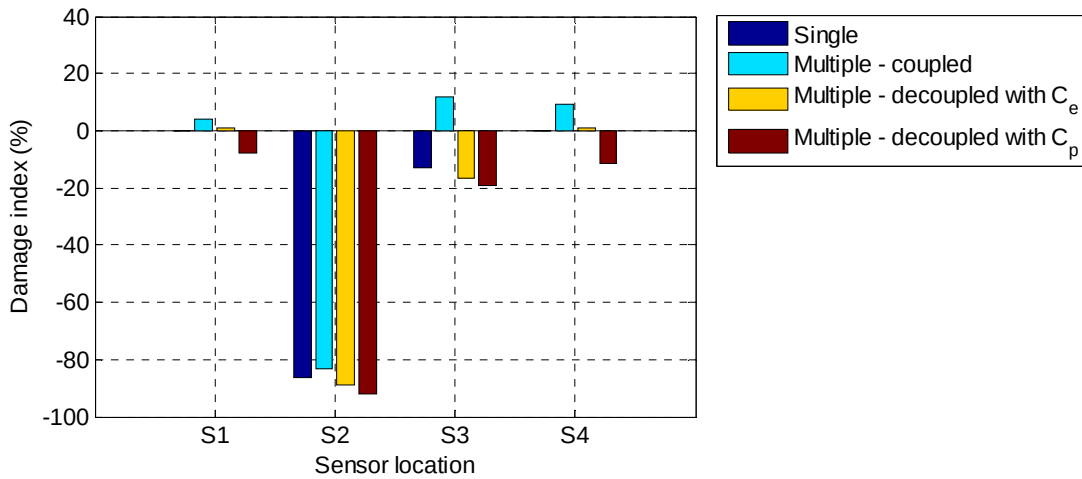
$$C_e = \begin{bmatrix} 1.00 & -0.01 & -0.14 & -0.19 \\ -0.04 & 1.00 & -0.38 & -0.26 \\ -0.26 & -0.32 & 1.00 & -0.04 \\ -0.19 & -0.08 & -0.07 & 1.00 \end{bmatrix} \quad (6.4(a))$$

$$C_p = \begin{bmatrix} 1.00 & -0.06 & -0.19 & -0.22 \\ -0.04 & 1.00 & -0.31 & -0.25 \\ -0.25 & -0.31 & 1.00 & -0.04 \\ -0.22 & -0.19 & -0.06 & 1.00 \end{bmatrix} \quad (6.4(b))$$

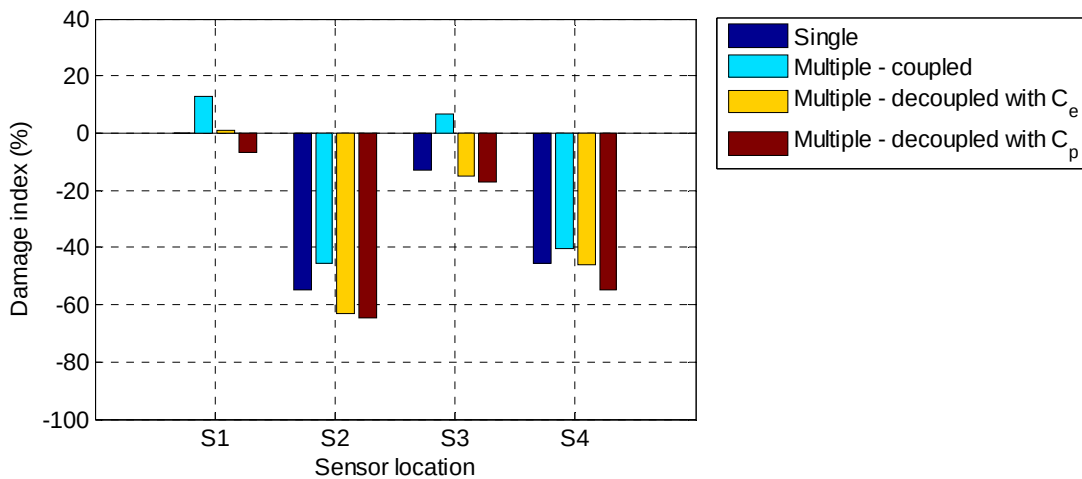
A comparative study of the damage index for single and multiple damage conditions was conducted for the experimental verification of the presented decoupling method. Fig. 6.11 shows the damage index for the multiple damage conditions in Case 5 to 7. In Case 5 where two fractures existed on two beams at the same beam-column connection, severe damage L4 was easily identified with the coupled damage index of -83.0% , while damage L2 could not be detected from the coupled damage index of 11.7% because of large influence of the neighboring damage L4. In comparison, the damage L2 could be identified by the decoupled damage index of -16.5% (decoupled with C_e) or -19.3% (decoupled with C_p). Moreover, compared to the damage index for single damage conditions, the damage indices for the two fractures decoupled with C_e and C_p had the largest differences of 3.4% and 6.2% , respectively. The damage indices in Case 5 decoupled with the analytical coefficient matrix C_p had the false positive error of 9.5% at sensors S1 and S4 in average.

Similarly, the decoupling method with experimental and analytical coefficient matrices was effective in estimating the damage indices for the multiple damage conditions in Case 6 and 7. In

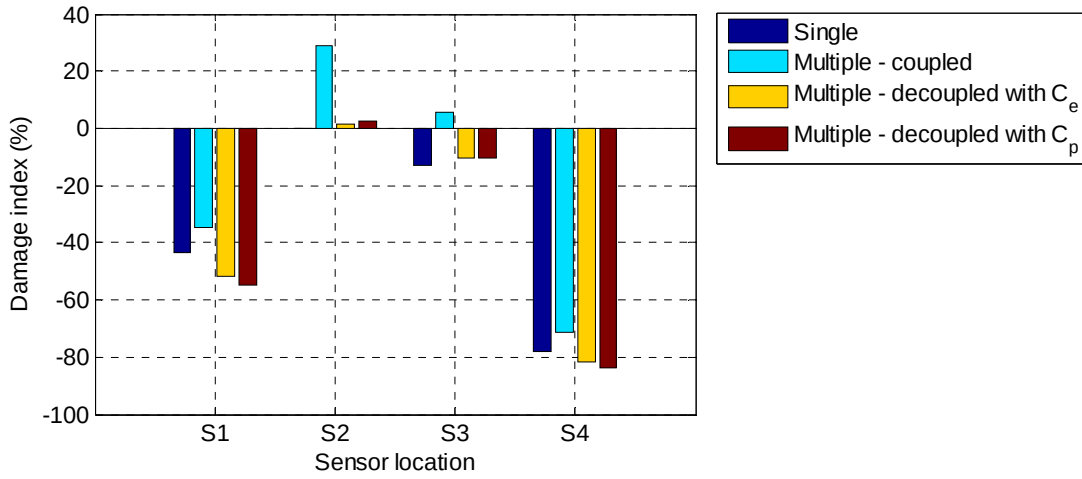
Case 7, when the third floor had damage L3 at the connection B5, compared with the damage index of single damage, the decoupled damage indices had the differences of at most 8.2% (decoupled with C_e) or 11.5% (decoupled with C_p) at the connection B1. This difference was slightly more than those in Case 5 and 6. In addition, Fig. 6.12 illustrates the reduced bending stiffness evaluated from the decoupled damage indices for the damage condition in Case 7. The evaluated reduction of bending stiffness estimated from the damage indices decoupled with C_e and C_p was nearly identical, and they had the largest difference of 6% compared with those obtained from the damage indices for single damage. Compared to the exact values, the estimated values obtained from the decoupled damage indices had the differences of 8% for the damage at B1, 17% at B3, and 2% at B4. The relatively large difference at B3 resulted from the closed-form expression of damage curve presented in the Chapter 5 which slightly underestimated fractures on bottom flanges.



(a)



(b)



(c)

Fig. 6.11 Comparison of damage index between single and multiple damage conditions: (a) Case 5; (b) Case 6; (c) Case 7

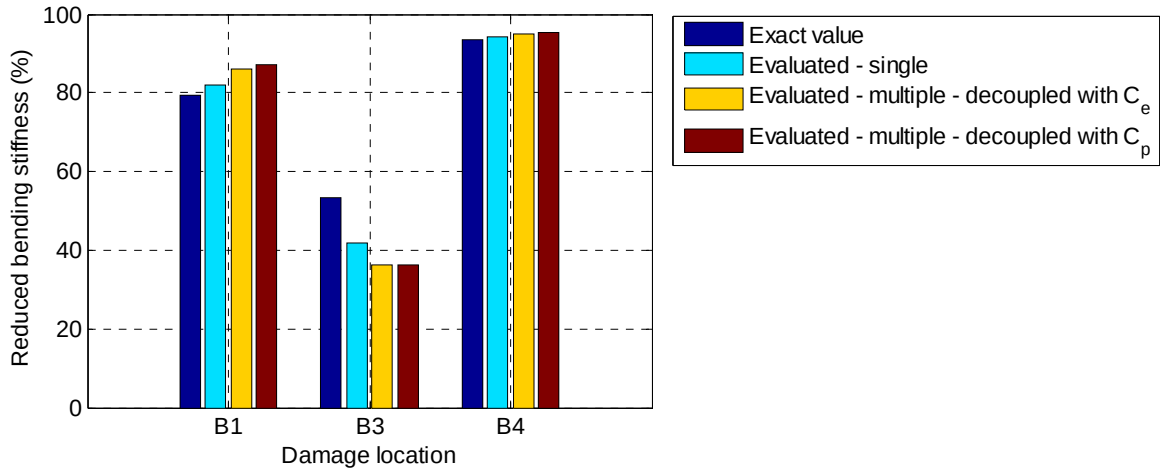


Fig. 6.12 Reduction of bending stiffness evaluated from the decoupled damage index in Case 7

6.6 Summary

This chapter presented a decoupling method of estimating the damage index for multiple beam fractures in steel moment-resisting frames based on the mechanism of moment release and influence. The notable findings are as follows.

(1) The analytical study of a sub-frame demonstrated that the releases of moment induced by beam fractures were primarily redistributed at the same floor and the influence to other floors was nearly negligible.

(2) The computing procedure of influence coefficients and the effectiveness of the decoupling method were verified numerically through a nine-story steel moment-resisting frame and experimentally using the five-story steel frame testbed.

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CHAPTER 7

Conclusions and future studies

7.1 Conclusions

This dissertation developed a localized damage evaluation method specifically designed for detecting and quantifying seismically-induced beam fractures to beam-to-column connections in steel moment-resisting frames. The proposed method would facilitate rapid and reliable estimation on the remaining capacity of the earthquake-affected steel buildings and thus support post-quake decision-making on re-occupancy. The effectiveness of the method was investigated through numerical studies with a nine-story steel moment-resisting frame and experimental studies using the five-story steel frame testbed. In this dissertation, the notable findings are summarized as follows.

Chapter 2: Concept of local damage evaluation

(1) In steel moment-resisting frames, local damage such as seismically-induced fractures on steel beams changes the distribution of bending moments sustained by members. In practice, when the frames behave linearly under small amplitude vibrations, the bending moments can be estimated by measuring strain responses on the members. Based on these physical mechanisms, a concept of seismically-induced local damage evaluation method using wireless piezoelectric strain sensing was presented.

Chapter 3: Strain-based damage index

(1) A novel damage index was formulated using a comparative study of the bending strain responses of beams associated with a natural mode between the undamaged and damaged frames. The damage index was proved to be independent of external excitations and vibrational modes.

(2) The developed wireless piezoelectric strain sensing system, comprised of PVDF sensors and *Narada* wireless units, showed excellent performance for monitoring the dynamic strain in the steel structures under small amplitude vibrations and even ambient excitations.

(3) In the experimental results for the five-story steel frame testbed, variation in the damage index under the undamaged conditions was less than 7% for different excitations, and weak dependency of damage index on the characteristics of the external excitations was preliminarily verified.

(4) Damaged locations were successfully identified in the tests using the distribution of damage index values. Moreover, the damage index values for various severity levels showed clear discrete values that would enable the quantification of seismic fracture damage.

Chapter 4: Sensitivity investigation of the damage index

(1) Independency of the presented damage index on the characteristics of external excitations and the selection of vibration modes was verified in numerical simulations and shaking table tests. As the extraction of modal responses required preset band-pass filters, the use of dominant vibration modes with clear responses and high power was highly desirable.

(2) Both in the numerical simulations and experiments, the damage index extracted from a distance not more than $1.2d$ (d is the beam depth) from a fracture was largely affected by local strain redistributions induced by the fracture. A distance between $1.2d$ and $2.0d$ from the fracture was recommended for evaluating the moment redistributions in steel moment-resisting frames and the reduction in bending stiffness at fractured sections.

(3) Consistency of the damage index in the evaluation of damage at different locations was verified in experimental studies using the five-story steel testbed frame. The level of variation was at most 7.8% for fracture on the bottom flange, and 3.8% for fracture of the bottom flange and web.

(4) The increases of the damage index at damage-neighboring connections were verified using a preliminary experimental study considering multiple damage conditions. The interaction between neighboring damage at the same beam-column connections was much more significant than that observed at different connections.

Chapter 5: Simplified derivation of damage curve

(1) A closed-form expression of damage curve was derived from an analytical parametric study using a two-story two-bay frame. The damage curve was a relationship that represents the strain-based damage index as a function of the reduction in beam bending stiffness induced by fracture.

(2) The presented damage curve was dominated primarily by the span-depth ratio of fractured beam and hardly affected by the other structural parameters, such as the height ratio between stories, the column-to-beam stiffness ratio, and the lateral force ratio between floors.

(3) The damage curve was demonstrated to be capable of evaluating the amount of earthquake-induced fractures on beams for common multi-story multi-bay steel moment-resisting frames through numerical studies conducted for a nine-story steel moment-resisting frame and experimental investigations using a one-quarter-scale five-story steel frame testbed.

Chapter 6: Decoupling interaction between multiple damage

(1) An analytical study of a sub-frame demonstrated that the releases of moment induced by beam fractures were primarily redistributed at the same floor and the influence to other floors was nearly negligible.

(2) A decoupling method of estimating the damage index for multiple beam fractures was presented based on the mechanism of moment release and influence.

(3) The effectiveness of the decoupling method was verified numerically through a nine-story steel moment-resisting frame and experimentally using the five-story steel frame testbed.

7.2 Future studies

The proposed method in this dissertation is a strategy for local damage evaluation in steel moment-resisting frames, which provides damage information (i.e., existence, location, and extent) of the monitored beams and thus potentially supports rapid post-earthquake damage assessment and decision-making on re-occupancy for the earthquake-affected steel buildings.

Future studies are needed to develop a prototype of rapid post-earthquake damage and safety evaluation program by integrating global characteristics with local damage information for steel moment-resisting frames. The notable features and functions of the program will be (1) instant inspection of local damage, (2) rapid assessment of damage state, (3) re-occupancy safety evaluation based on identified local damage. The program will be capable of providing reliable damage and safety information to support decision making on re-occupancy and recovery. Proposed topics are as follows.

(1) Damage state assessment using story drift-based fragility functions updated with local damage information

As illustrated in the CSMIP-3DV [1] software systems, the drift-based fragility functions derived from experimental data for structural components is widely used in the SHM-based post-earthquake damage condition assessment. Once the story drift is obtained from measured data, the probability of damage state for floor levels and entire buildings can be estimated through fragility functions. Nonetheless, in steel moment-resisting frames, because of the hysteresis behaviors of members and connections involving large uncertainties, the story drift-based fragility functions are not necessarily effective in damage assessment. For instance, a story experiencing an inter-story drift of 1.5% has a 5% probability of not having damage, 44% of having slight damage, 46% of experiencing moderate damage, and 5% of sustaining severe damage. This result would make decision makers indecisive. The local damage identified with the presented local damage evaluation method is evidence of damage in buildings. This valuable information can be used to update the developed fragility functions with Bayesian inference to reduce uncertainties. With this background, a method of updating story drift-based fragility function with identified local damage information for rapid damage condition assessment will be studied.

(2) Re-occupancy safety evaluation based on identified local damage

After a damaging earthquake, when an earthquake-affected building undergoes local damage on structural members that are identified through a health monitoring system, most important is to determine if the identified damage affects the structural safety of the buildings for re-occupancy. Except the buildings suffered obviously severe damage and near collapse, other buildings need to be assessed on their ability to resist future loadings. The commonly accepted evaluation approaches for seismic safety are based mainly on the residual strength and stiffness of the damaged buildings relative to the current code requirements and pre-earthquake conditions, and the probability of collapse of the damaged buildings induced by a given hazard (usually 50% chance of exceedance in 50 years). Thus, the safety evaluation methods based on residual capacity (i.e., strength and stiffness) and probability of collapse under a given hazard will be investigated and their feasibility for application to the proposed program will be studied.

(3) Design of rapid post-earthquake damage and safety evaluation program

Compared to existing programs where only damage condition information estimated from global characteristics is provided for post-earthquake decision-making, the proposed program will be designed to further include local damage-based damage condition assessment and safety estimation. This program will achieve two-stage assessments: rapid damage state assessment and re-occupancy safety evaluation. For instance, after an earthquake, the program immediately conducts the rapid

damage state assessment. If the building probably experienced unsafe damage state, the program warns the users in the building to evacuate in a short time. Then, the program continues the re-occupancy safety evaluation. If the building keeps 90% of the capacity compared to the pre-earthquake condition, the decision makers may make the judgement of re-occupancy. To realize these functions in the proposed program, a framework and procedure of the rapid post-earthquake damage and safety evaluation program and the design of an associated computer program will be studied.

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