

Decoupling algorithm for evaluating multiple beam damages in steel moment-resisting frames

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SUMMARY

Post-earthquake safety evaluation of steel moment-resisting frames mainly relies on the inspection of seismic damage to beam-column connections. Recently, in order to evaluate seismic damage of steel connections in a prompt and precise manner, a local damage evaluation method based on dynamic strain responses has been proposed and receives attention. In the evaluation method where strain responses are measured by piezoelectric strain sensors, a strain-based damage index has been developed for evaluating individual seismic beam damage in a steel frame. However, for a steel frame suffering multiple beam damages, the damage index deteriorates its performance in identifying small damages with the presence of neighboring severe damages due to the moment redistributions induced by larger damages. This paper presents a decoupling algorithm that removes the issue of damage interaction and improves the performance of the damage index. The decoupling algorithm was derived on the basis of damage-induced moment release and redistribution mechanism. The effectiveness of the decoupling algorithm was numerically and experimentally investigated using a nine-story steel frame model and a large scale five-story steel frame testbed that can simulate multiple fractures at beam ends.

KEY WORDS: damage quantification; seismic damage; steel moment-resisting frame; damage interaction; structural health monitoring; dynamic strain

1. INTRODUCTION

High-rise steel buildings that are subjected to long period ground motions likely suffer severe damage on steel beam-column connections, such as fracture, owing to strength deterioration under many cycles of inelastic deformation. This vulnerability was observed in the full scale shaking table tests of a high-rise steel building specimen conducted at the E-Defense facility in Japan [1, 2]. In addition, various numerical and experimental studies (e.g., Luco and Cornell [3], Rodgers and Mahin [4], Nakashima et al. [5], and Lignos et al. [6]) demonstrated that the occurrence of severe damage on beam-column connections may deteriorate the seismic capacity of steel buildings. Thus, in recent devastating earthquakes (e.g., the 2011 Tohoku earthquake in Japan), the strong sway of high-rise steel buildings excited by long period ground motions, raised considerable concerns about the post-earthquake safety of the buildings. In some cases, the lack of rapid and reliable information regarding the safety of buildings caused much disorder in the evacuation and re-occupancy.

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 safety evaluation and decision-making for earthquake-

50 affected buildings [7]. At present, a few important buildings located at metropolitan areas
51 with high seismicity have installed SHM systems, where the global characteristics of
52 buildings (e.g., acceleration responses, modal frequency and mode shape, and inter-story drift
53 ratio) are primarily used for damage assessment [8-10]. Experimental investigations into the
54 damage estimation using global characteristics demonstrated that they estimated the health
55 conditions of buildings to some extent, but encountered serious challenges to give reliable
56 information of seismic local damage on structural members that are critical for post-
57 earthquake safety evaluation. Accordingly, detection of local damage on structural members
58 using a dense-array sensing system has received attention in recent years [11, 12].

59 In case of steel moment-resisting frames, they are prone to suffer fracture damage at
60 welded beam ends when the strong-column and weak-beam philosophy is adopted in their
61 design. In the 1994 Northridge and 1995 Kobe earthquakes, a large number of steel moment-
62 resisting frames suffered fractures at welded beam ends [13-15]. After the earthquakes, the
63 inspection of fracture damage required extensive labor and costs involved in the removal of
64 fireproofing and architectural finishes [16]. In this context, Kurata et al. [17] and Li et al. [18,
65 19] proposed a local damage evaluation method for steel moment-resisting frames using
66 dynamic strain responses. In the method, the extent of beam fracture is quantified using a
67 dynamic-strain-based damage index and an associated damage curve in which the reduction
68 of bending stiffness at the fractured section is a function of the damage index. The method is
69 reported to be very effective in identifying single damage but when a steel frame sustains
70 multiple beam damages, the accuracy of damage estimation deteriorates due to the moment
71 redistributions triggered by neighboring damages.

72 During the past few decades, several methods have been developed for identifying multiple
73 damages in building structures. Sohn and Law [20] proposed a Bayesian probabilistic
74 approach for detecting the most likely locations and extents of damages in multi-story frame
75 structures. The approach was verified through numerical studies on several simple frame
76 models where the damages were simulated as the deterioration of substructures. Shi et al. [21]
77 developed a damage detection method based on modal strain energy change, which was
78 experimentally investigated using a two-story and single-bay portal steel frame. Results
79 indicated that the method was able to localize multiple damages, but the quantification of
80 damages was only successful in low-level noise environments. Cha and Buyukozturk [22]
81 proposed a multiple damage identification method based on modal strain energy and hybrid
82 multiobjective optimization, which was examined using numerical studies of three
83 complicated steel frame structures. The investigations indicated that the method was effective
84 in detecting multiple damages but the performance deteriorated for small damage with
85 incomplete and noise-contaminated mode shapes. The identification of multiple damages in
86 building structures is still challenged, especially using experimental data.

87 This paper presents a decoupling algorithm for improving the accuracy of the dynamic-
88 strain-based damage index proposed in [17-19] in the identification of multiple damages by
89 removing the influence of neighboring damage interaction in moment redistributions. The
90 decoupling algorithm was derived on the basis of the mechanism of damage-induced moment
91 release and redistribution in frames. In the derivation, an analytical study on a simple sub-
92 frame illustrated that the moment released by a beam fracture mainly distributes on the same
93 floor levels and the influence to neighboring floors are small. The effectiveness of the
94 decoupling algorithm was numerically studied using a nine-story steel moment-resisting
95 frame model and experimentally examined using a large scale five-story steel frame testbed
96 that can simulate multiple beam fractures.

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2. LOCAL DAMAGE EVALUATION METHOD

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In steel moment-resisting frames subject to earthquake loading, the bending moments sustained by a structural member decrease with a local damage of the member that reduces the member's stiffness. In practice, such changes in bending moments can be estimated using strain responses under ambient vibrations, assuming that the amplitude of the strain at a particular location of a member is proportional to the amplitude of the bending moment and structural members behave linearly. Thus, local damage on a structural member can be evaluated through a comparative study of strain responses of the member between the intact state and the damaged condition.

Figure 1 illustrates the schema of the local damage evaluation method presented in [17-19] for quantifying the damage extent of a beam seismic fracture that initiates at the toe of the weld access hole at beam-end in steel frames. As shown in Figure 1(a), a wireless strain sensing system that consists of a dense array of polyvinylidene fluoride (PVDF) sensors (DT1-028k, Measurement Specialties, VA, USA) [23] interfaced with *Narada* wireless sensing units (Civionics, LLC, CO, USA) [24] is deployed on a steel frame. Dynamic strain responses are measured under ambient vibrations before and after an earthquake. The sensing system includes a reference sensor and detecting sensors. The reference sensor is used to eliminate the effects of external excitations. A floor with small deformation where the concrete slabs and beams remain undamaged (e.g., the roof) is recommended for the location of the reference sensor. The detecting sensors are used to detect and quantify local damages on the damage-prone beams which are pre-identified using structural analysis. Detecting sensors are attached on both sides of beam bottom flanges at recommended locations where unaffected by the local redistribution of strains induced by damages. Li et al. [18] suggests the location as 1.5 beam depths away from column surfaces. In steel frames, the probability of sustaining fracture damage to beam-column connections increases as inter-story drift increases. Thus, several floors likely sustaining large inter-story drift (usually at the lower stories) have higher priority in the monitoring strategy.

The damage index (*DI*) is defined as Equation (1) [18], which is formulated from a comparison of strain responses measured before and after an earthquake.

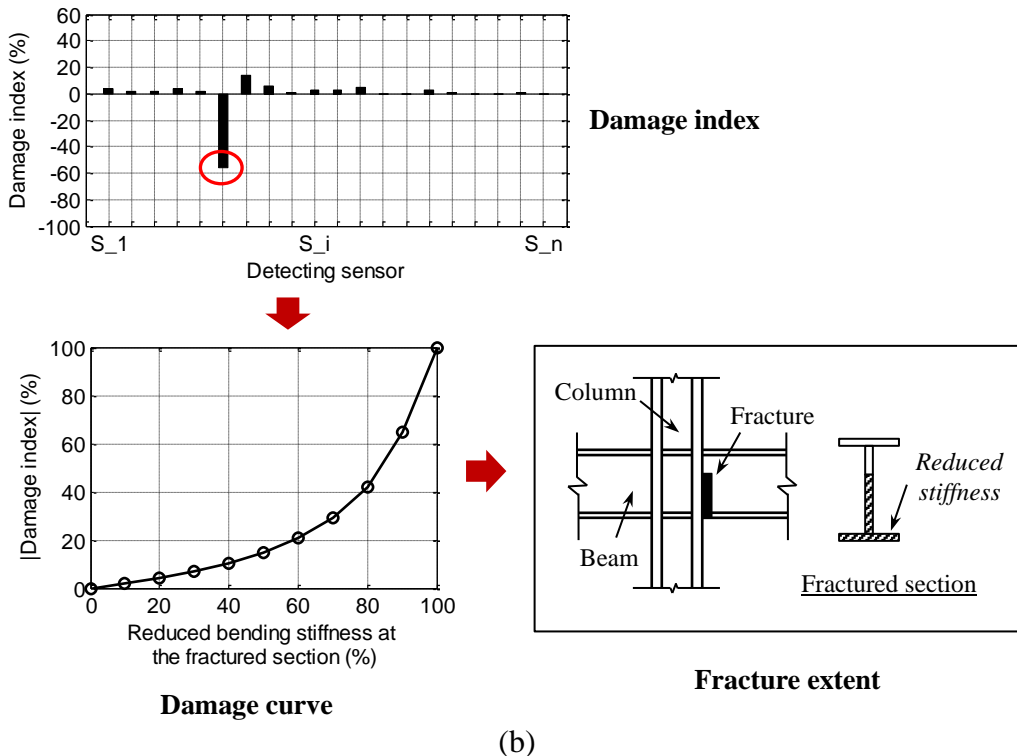
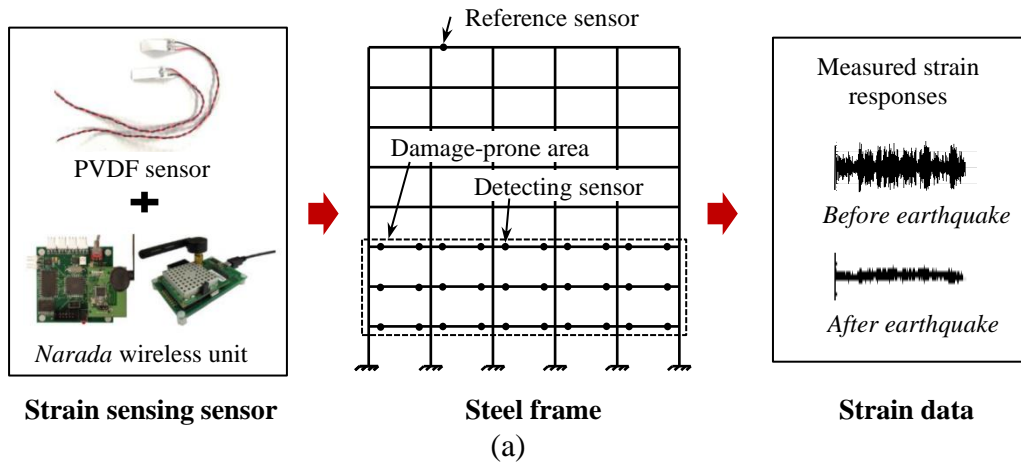
$$DI = \frac{R_j^d - R_j}{R_j} \times 100\% , \quad (1)$$

where R_j and R_j^d are the ratios of strain responses associated with a natural mode—the j th mode at the detecting and reference sensors under the undamaged condition and after an earthquake, respectively. In practice, the strain responses associated with the j th mode are extracted using band-pass filters on strain time histories; ratios R_j and R_j^d are evaluated by the root mean square (RMS) of the filtered strain time histories. The damage index is proven to be independent of external excitations and vibrational modes. The damage index of less than 0 indicates the existence of damage on the monitored beam end. The damage index of -100% means complete fracture. If the damage index is not less than 0, there is no damage on the monitored beam end, and the damage index indicates the changes in the strain responses measured at the beam end induced by neighboring damages (see Figure 1(b)). When strain sensor is located in the region unaffected by the local strain redistribution, the damage index equals to the changes in the bending moments at the sensor location.

The damage extent of a seismic fracture at beam end is evaluated using the damage curve (see Figure 1(b)) expressed as Equation (2) that is presented in [19],

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

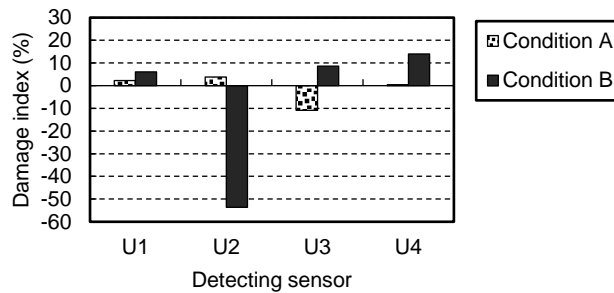
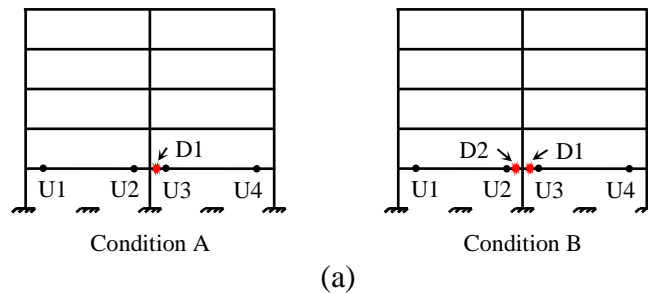
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 149 where ρ is the reduction of the bending stiffness at the fractured section; $A_1, A_2, A_3, B_1, B_2,$ and
 150 B_3 are coefficients that are functions of structural parameters. Note that the absolute value of
 151 the damage index is adopted alternatively in the damage curve. Using the expression of the
 152 damage curve, the reduction of bending stiffness of the damaged beam can be directly
 153 estimated from the damage index. The damage curve is limited for a single beam fracture in
 154 steel frames. This is how the local damage evaluation method estimates the damage extent of
 155 a seismic beam fracture.
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 161 Figure 1. Local damage evaluation method: (a) wireless strain sensing system on a steel frame;
 162 (b) quantification of a beam fracture using the damage index and the damage curve.
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164 When a steel frame sustains multiple beam fractures, the strain responses measured at a
 165 damaged beam end increases by the neighboring damage-induced moment releases. Thus, the

166 damage index for damage at a beam end increases with the existence of neighboring damages
 167 in the frame. Figure 2 illustrates a comparison of the damage indices for beam ends at a floor
 168 of a frame suffering two damage conditions. In the damage condition A, the right beam end of
 169 the interior beam-column connection at the second floor sustains the fracture damage D1, i.e.,
 170 a 30% decrease in the bending stiffness. In the damage condition B, besides the damage D1 at
 171 the right beam end of the connection, the left beam end of the same connection sustains the
 172 fracture damage D2, i.e., an 80% decrease in the bending stiffness. In Figure 2(b), compared
 173 with the damage index of -10.8% at sensor U3 for the single damage D1 in the condition A,
 174 the damage index increases by 19.5% for the same damage D1 in the condition B because of
 175 the influence of neighboring damage. As a result, the damage index is inaccurate in detecting
 176 the damage D1 in the condition B. In order to identify multiple beam fractures in steel frames,
 177 the influence of neighboring damages needs to be removed and thus the damage indices are
 178 identical to those associated with single damage conditions.
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 184 Figure 2. Comparison of the damage index for the same damage between single and multiple
 185 damage conditions: (a) two damage conditions; (b) damage indices.

188 3. DECOUPLING ALGORITHM

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 190 This section presents a decoupling algorithm for removing the influence of neighboring
 191 damage interaction on the damage index. First, the mechanism of damage-induced moment
 192 release and redistribution in frames is analytically studied using a simple sub-frame. Then, a
 193 decoupling algorithm of estimating the damage index for multiple beam damages is
 194 formulated.

195 3.1 Influence of damage-induced moment release

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 198 Inclusion of beam damages in a steel moment-resisting frame results in the releases of the
 199 bending moments sustained by the beams and thus the released bending moments are
 200 redistributed in the frame [25]. The following analytical study on a simple sub-frame, which

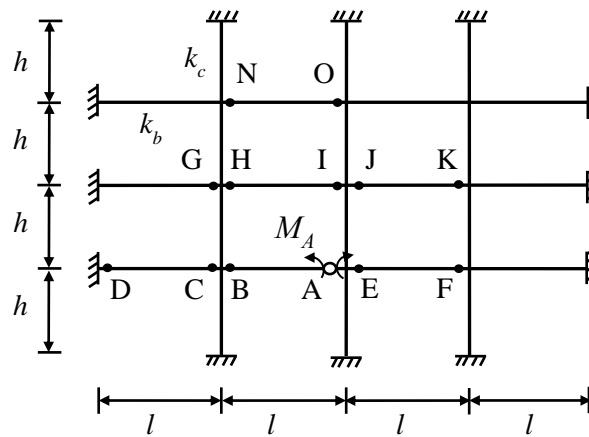
201 is extracted from a multi-story multi-bay frame, demonstrates the redistribution of released
 202 moments induced by a beam fracture.

203 A four-story four-bay sub-frame is considered as shown in Figure 3, where k_b and k_c are
 204 the bending stiffness of beams and columns, respectively; h denotes the height of each story; l
 205 is the width of each span. The bending moment M_A is the release of the moment caused by the
 206 fracture damage at the beam end A. Assuming that the frame behaves linearly, the bending
 207 moments at the beam ends B, C, D, E, F, G, H, I, J, K, N, and O generated by the released
 208 moment M_A are calculated by the displacement method for the analysis of indeterminate
 209 structures as follows,

$$211 \quad M_i = \delta_{iA} M_A = f_i(a) M_A, \quad (i = B, C, D, E, F, G, H, I, J, K, N, \text{ and } O) \quad (3)$$

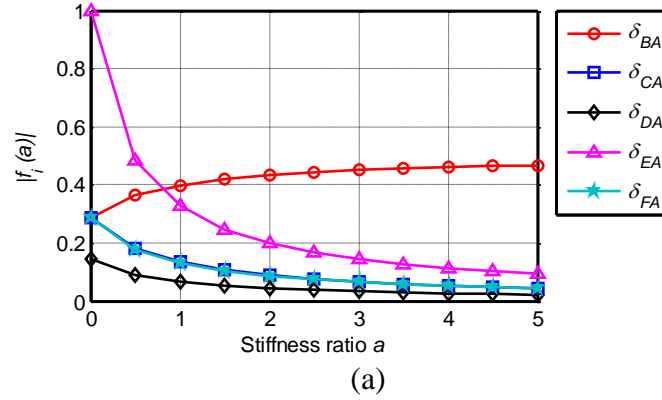
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 213 where δ_{iA} ($i = B, C, D, E, F, G, H, I, J, K, N, \text{ and } O$) are influence coefficients. The influence
 214 coefficients are constants, which indicate that the redistributed moments are proportional to
 215 the released moment. In addition, the influence coefficients only relate to the column-to-beam
 216 stiffness ratio a ($= k_c/k_b$). Figure 4 illustrates the relationships between the influence
 217 coefficients δ_{iA} and the column-to-beam stiffness ratio a . The column-to-beam stiffness ratio
 218 ranges from 0 to 5 for common steel moment-resisting frames in this study. The influence
 219 coefficient δ_{EA} for the neighboring beam end E decreases from 1 to 0.1 as the stiffness ratio
 220 increases from 0 to 5. The influence coefficient δ_{BA} is more than 0.3 for the beam end B on the
 221 same beam. The influence coefficients δ_{CA} , δ_{DA} and δ_{FA} for the beam ends C, D and F on the
 222 neighboring beams at the damaged floor are less than 0.3. On the neighboring floor, the
 223 influence coefficients δ_{HA} , δ_{IA} , and δ_{JA} are at most 0.05 for the nearby beam ends H, I, and J,
 224 while the influence coefficients δ_{GA} and δ_{KA} are at most 0.01 for the farther beam ends G and
 225 K. On the non-adjacent floor, the influence coefficients δ_{NA} and δ_{OA} for the beam ends N and
 226 O are at most 0.01 (see Figure 4(b)). These findings imply that the released moment M_A
 227 mainly distributes to the neighboring beam ends at the same floor level. The influence is at
 228 most 5% for the nearby beam ends on the neighboring floors, and negligible influence to other
 229 beam ends on the neighboring floors and to all beam ends on the non-adjacent floors.

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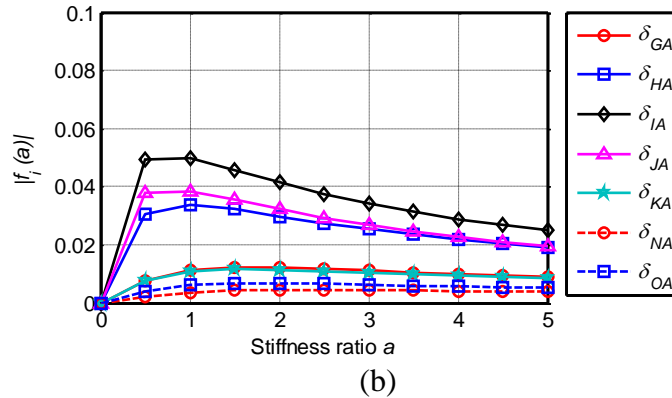


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 232 Figure 3. A sub-frame for studying moment release and influence.
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Figure 4. Relation between influence coefficient and column-to-beam stiffness ratio: (a) damaged floor; (b) upper floors.

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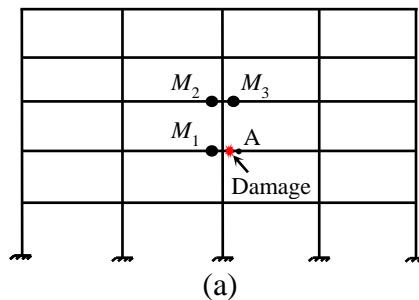
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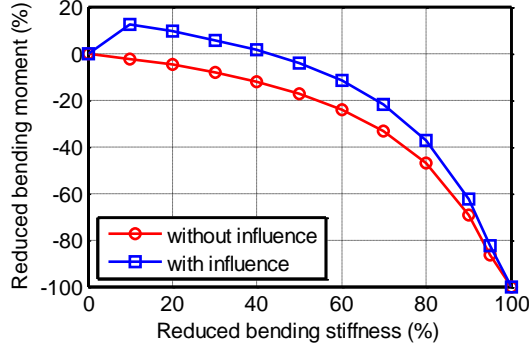
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In steel moment-resisting frames, when multiple beams suffer damages, damage-induced moment releases complicate the estimates on the bending moments of beams reduced by damages. Figure 5 shows the influence of moment releases on the reduced bending moments of a damaged beam in a frame. The released moments M_1 , M_2 , and M_3 are caused by three serious fracture damages nearby the damaged beam. All fractures reduce the stiffness of beam ends by 90%. Without the influence of moment releases, the reduced bending moment at point A drops from 0 to -100% as the reduced bending stiffness of the damage increases from 0 to 100%. With the influence of the three moment releases M_1 , M_2 , and M_3 , the reduced bending moments at point A change with increases from 4.2% to 14.4% (see Figure 5(b)). The increases for small damage are relatively larger than those for severe damage. Beam suffering small damage has less decrease in its stiffness and thus it sustains larger forces in the redistributions of moment releases. This indicates that the neighboring damage interaction largely affects the damage index of small damage, and has slight influence on the damage index for serious damage.

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(b)

Figure 5. Influence of moment releases on reduced bending moments of a damaged beam: (a) a damaged frame; (b) reduced bending moments at point A.

3.2 Decoupling algorithm

As observed in the preceding analytical study, the damage-induced moment releases in steel moment-resisting frames mainly distributes at the same floor levels and the influence to other floors is very small (at the most 5% to neighboring floors and negligible influence to non-adjacent floors). Thus, the interactions between beam damages located at two non-adjacent floors are assumed to be negligible. The damage interactions at the same floors are primarily considered to formulate a decoupling algorithm aiming to remove the influence of damage-induced moment releases on the presented damage index. Three consecutive floors of an n -span frame are modeled as shown in Figure 6(a). Each floor is sensed with $2n$ strain sensors, e.g., sensors $S_{1,r}$ to $S_{2n,r}$ placing for the r th floor. The decoupling algorithm is formulated for beam damages on the r th floor. In local damage evaluation, the damage index is identical to the changes of bending moments at a sensor location caused by damage of beam. Thus, according to the superposition principle, the damage indices measured on the r th floor, which are coupled with each other, are equally expressed as a combination of the damage indices associated with individual beam damages on the r th floor in addition to the influence from the moment releases of beam damages at two neighboring floors, i.e., the $(r-1)$ th and $(r+1)$ th floors, as follows,

$$\mathbf{DI} = \mathbf{A} \overline{\mathbf{DI}} + (\mathbf{DI})', \quad (4a)$$

$$\mathbf{DI} = \begin{bmatrix} (DI)_1 \\ \vdots \\ (DI)_i \\ \vdots \\ (DI)_{2n} \end{bmatrix} \quad \mathbf{A} = \begin{bmatrix} \delta_{1,1} & \cdots & \delta_{1,j} & \cdots & \delta_{1,2n} \\ \vdots & & \vdots & & \vdots \\ \delta_{i,1} & \cdots & \delta_{i,j} & & \delta_{i,2n} \\ \vdots & & \vdots & & \vdots \\ \delta_{2n,1} & \cdots & \delta_{2n,j} & \cdots & \delta_{2n,2n} \end{bmatrix} \quad \overline{\mathbf{DI}} = \begin{bmatrix} (\overline{DI})_1 \\ \vdots \\ (\overline{DI})_j \\ \vdots \\ (\overline{DI})_{2n} \end{bmatrix} \quad (4b)$$

where \mathbf{DI} is a vector of measured damage indices at all sensors on the r th floor; $\overline{\mathbf{DI}}$ is a vector of the damage indices associated with individual beam damages, named as decoupled damage indices; \mathbf{A} is an influence coefficient matrix, and $\delta_{i,j}$ ($i = 1, \dots, 2n, j = 1, \dots, 2n$) denotes the influence coefficients from $S_{j,r}$ to $S_{i,r}$ due to the moment release of the beam damage monitored by $S_{j,r}$ (Figure 6(b)); $(\mathbf{DI})'$ denotes the influence from the moment releases of beam damages at the $(r-1)$ th and $(r+1)$ th floors.

292 The influence $(DI)'$ is expressed as,

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$$294 \quad (DI)' = \sum_{q=1}^m \delta_q (\overline{DI})_q = \sum_{q=1}^m \begin{bmatrix} \delta_{1,q} \\ \vdots \\ \delta_{i,q} \\ \vdots \\ \delta_{2n,q} \end{bmatrix} (\overline{DI})_q, \quad (5)$$

295

296 where δ_q denotes the influence coefficient vector for the influence from the damage on the $(r-$
 297 $1)$ th or $(r+1)$ th floors, in which $\delta_{i,q}$ ($i = 1, \dots, 2n$) denotes the influence coefficient from
 298 sensor S_q to $S_{i,r}$ due to the moment release of the beam damage monitored by S_q (Figure
 299 6(c)); $(\overline{DI})_q$ denotes the damage index corresponding to individual damage on the $(r-1)$ th or
 300 $(r+1)$ th floors. The m denotes the number of beam damages on the $(r-1)$ th and $(r+1)$ th floors.

301 As mentioned before, the influence coefficients of moment releases from neighboring
 302 floors are at most 0.05. In addition, as illustrated in Figure 5(b), the neighboring moment
 303 releases cause the reduced bending moments of a damaged beam to increase by at most 15%.
 304 This implies that the measured damage index of beam damage in multiple damage state is not
 305 largely different from the damage index corresponding to individual damage state. In
 306 Equation (5), thus, the measured damage index $(DI)_q$ of damages on the $(r-1)$ th or $(r+1)$ th
 307 floors can be used to compute the influence $(DI)'$ instead of the damage index $(\overline{DI})_q$ as
 308 follows.

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$$310 \quad (DI)' = \sum_{q=1}^m \delta_q (\overline{DI})_q \approx \sum_{q=1}^{m'} \delta_q (DI)_q, \quad (6)$$

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312 In some cases, the measured damage index of small damage is not less than 0 with the
 313 presence of neighboring damages and thus the damage is undetectable from the measured
 314 values. Therefore, the detectable damages on the $(r-1)$ th and $(r+1)$ th floors, i.e., the damage
 315 with the measured damage index of less than 0, are considered only in Equation (6). The m'
 316 denotes the number of detectable damages from measured damage indices on the $(r-1)$ th and
 317 $(r+1)$ th floors.

318 Given the measured damage indices of all sensors on the three consecutive floors, the
 319 decoupled damage indices associated with individual damages are expressed as,

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$$321 \quad \overline{DI} = \Delta^{-1} [DI - \sum_{q=1}^{m'} \delta_q (DI)_q]. \quad (7)$$

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323 The influence coefficients $\delta_{i,j}$ ($i = 1, \dots, 2n, j = 1, \dots, 2n$) are the ratio of the damage
 324 indices of sensors $S_{i,r}$ and $S_{j,r}$ when the frame only suffers a damage at the beam end
 325 monitored by the sensor $S_{j,r}$. These coefficients can be estimated using the moment release
 326 method with numerical models as the procedure below.

327 (1) Build a numerical model for a monitored steel moment-resisting frame.

328 (2) Set the releases of bending moments at the beam end monitored by the sensor $S_{j,r}$ as
 329 unity.

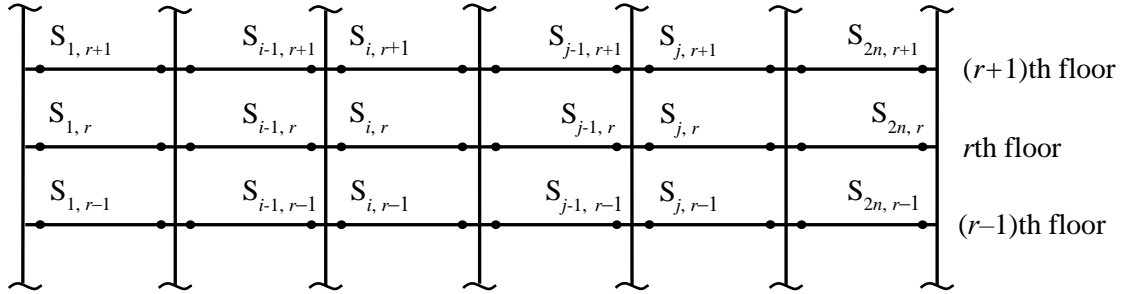
330 (3) Compute the bending moments at the positions of sensors $S_{i,r}$ and $S_{j,r}$ induced by the
 331 moment releases.

332 (4) Normalize the bending moment at the position of sensor $S_{i,r}$ using that at the position
 333 of sensor $S_{j,r}$ as influence coefficient $\delta_{i,j}$ (see Figure 6(b)).

334 When strain sensors are located around the beam ends, the influence coefficients can be
 335 estimated using the beam end moments instead of the moments sustained at the position of
 336 sensors.

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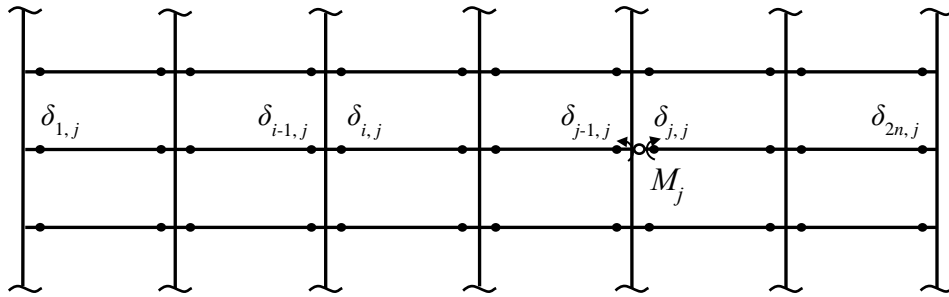


(a)

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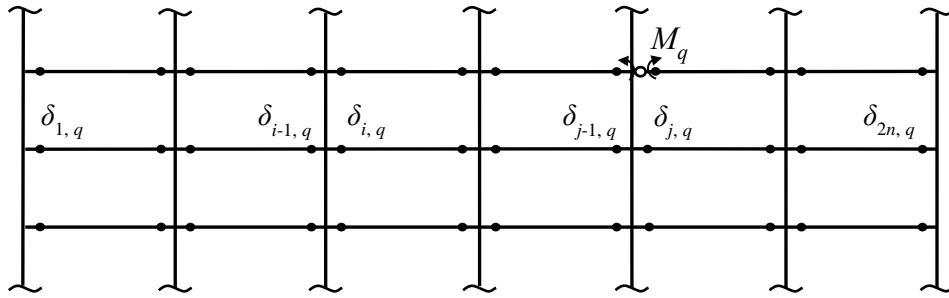


(b)

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(c)

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347 Figure 6. Three consecutive floors of an n -span frame: (a) sensors; (b) influence coefficients

348 on the r th floor; (c) influence coefficients for moment releases on the $(r+1)$ th floor.

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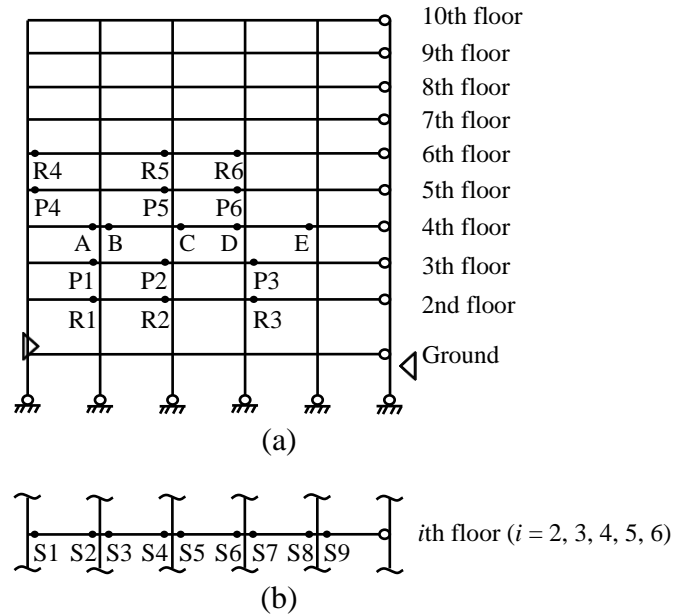
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4. NUMERICAL VERIFICATIONS

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353 The effectiveness of the decoupling algorithm was examined through a numerical study of a
 354 nine-story steel moment-resisting frame (see Figure 7) designed for the SAC project and
 355 whose details were in FEMA-355C [26]. The numerical analysis was conducted using the
 356 SAP2000 software. In the numerical model, all members were modeled using beam elements.
 357 Beam fractures were simulated at beam ends by referring to the crack model proposed by
 358 Sinha et al. [27], where the fracture was modeled by a segment of beam whose stiffness was

359 reduced to that of the fractured section; the length of the beam segment was determined as
 360 0.75 beam depths for I-shaped beams. The damage index was extracted from the bending
 361 moment responses of beams using the extraction procedure reported in [17, 18].
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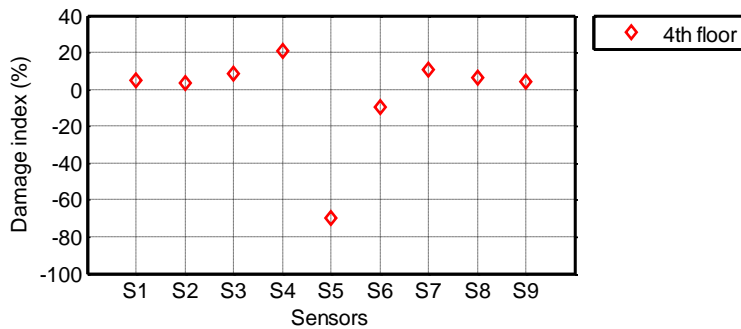
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368 Figure 7. Nine-story steel moment-resisting frame: (a) frame; (b) sensor location.

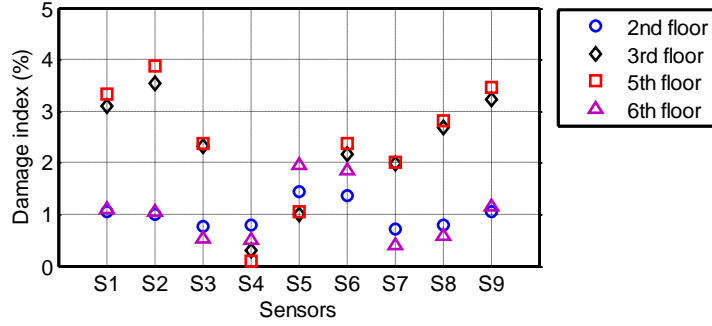
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370 The beam end C on the fourth floor (see Figure 7(a)) was damaged for studying the
 371 mechanism of damage-induced moment release. From the second to sixth floors, sensors S1
 372 to S9 were placed on beams at 1.5 beam depths from the columns, as shown in Figure 7(b).
 373 Figure 8 shows the damage indices on the second to sixth floors when the damage reduced the
 374 bending stiffness by 90% at the beam end C. With the existence of a severe damage at the
 375 beam end C, the damage index of sensor S5 near the damage was about -70%, while that of
 376 sensor S6 at another end of the same beam was about -10%. Moreover, the damage index of
 377 sensor S4 on the neighboring beam at the same floor was 21%, and those of other sensors at
 378 the fourth floor varied from 5% to 10%. By contrast, the damage index was less than 4% on
 379 the third and fifth floors, and less than 2% on the second and sixth floors (see Figure 8(b)).
 380 This indicates that the releases of the moment induced by the damage were primarily
 381 distributed at the neighboring beam ends on the same floor and the influence to the
 382 neighboring and farther floors was small.

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(b)

Figure 8. Distribution of the damage index on the second to six floors: (a) the fourth floor; (b) other floors.

Figure 9 shows the influence coefficients on the fourth floor for the damage at the beam end C, in which the influence coefficients were obtained through the normalization of the damage indices and the calculation procedure, respectively. The damage indices on the fourth floor were normalized by that of sensor S5. Two levels of the damage, i.e., the decreases of 10% and 90% in the bending stiffness, were considered in the normalization of the damage indices. The influence coefficients were identical for two levels of the damage, which indicates that the influence coefficients do not relate to the damage extent. This also verifies that the damage-induced moment releases are linearly redistributed in frames as mentioned in the section 3.1. In addition, the influence coefficients calculated by the presented procedure were consistent with the values normalized from the damage indices, which imply that the calculation procedure was capable of calculating the influence coefficients.

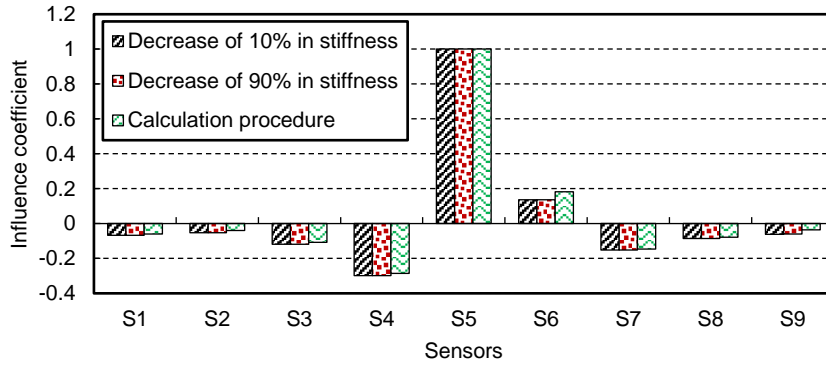


Figure 9. Influence coefficients on the fourth floor.

Seven damage cases in Table 1 were studied to examine the effectiveness of the decoupling algorithm. Seven damage cases can be sorted into four groups. Group 1 was used to verify the decoupling algorithm for multiple damages at an individual floor. Group 2 considered the influence of damages at the neighboring floors. Group 3 considered the influence of damages at the nonadjacent floors. Group 4 considered extremely damaged conditions. In all cases, five damages were simulated at the beam ends A, B, C, D, and E on the fourth floor (see Figure 7(a)). These damages reduced the bending stiffness of the beam ends by 50%, 70%, 30%, 30%, and 80%, respectively.

Group 1 —There is no damage on other floors.

Group 2 —Six damages are simulated at the beam ends P1 to P6 on the neighboring floors, i.e., the third and fifth floors (see Figure 7(a)). The damage reduces the bending stiffness of all beam ends by 50% in Case 2, and 80% in Case 3.

418 Group 3 —Six damages are simulated at the beam ends R1 to R6 on the nonadjacent floors,
 419 i.e., the second and sixth floors. The damages reduce the bending stiffness of all beam ends by
 420 50% in Case 4, and by 80% in Case 5.

421 Group 4 —Twelve damages are simulated at the beam ends P1 to P6 on the third and fifth
 422 floors and R1 to R6 on the second and sixth floors. In Case 6, the damages reduce the bending
 423 stiffness of all beam ends by 50%. In Case 7, the damages reduce the bending stiffness by 50%
 424 at the beam ends P1 to P6, and by 80% at the beam ends R1 to R6.

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Table 1 Damage cases

Groups	Cases	Locations (reduction of bending stiffness)	
		Fourth floor	Other floors
Group 1	Case 1	A (50%), B (70%), C (30%), D (30%), E (80%)	No damage
Group 2	Case 2	Same as Case 1	P1 to P6 (50%)
	Case 3	Same as Case 1	P1 to P6 (80%)
Group 3	Case 4	Same as Case 1	R1 to R6 (50%)
	Case 5	Same as Case 1	R1 to R6 (80%)
Group 4	Case 6	Same as Case 1	P1 to P6 (50%), R1 to R6 (50%)
	Case 7	Same as Case 1	P1 to P6 (50%), R1 to R6 (80%)

427

428 Figure 10 illustrates the measured and decoupled damage indices of sensors on the fourth
 429 floor for all damage cases. In all damage cases, the measured damage indices hardly detected
 430 the damages at the beam ends C and D. For instance, in Case 1, the measured damage indices
 431 of -0.3% and 0.8% at sensor S5 and S6 encountered large challenges to identify the existence
 432 of the damages (see Figure 10(a)). By contrast, the decoupled damage indices clearly detected
 433 the damages at the beam ends C and D in all cases. In Case 1, the decoupled damage indices
 434 were -7.6% and -7.4% at sensor S5 and S6 for the damages at the beam ends C and D, which
 435 were almost identical to the expected values. In addition, in all cases, compared to the
 436 expected values, the measured damage indices had the difference of about 15% for the beam
 437 end A, 10% for the beam end B, and 5% for the beam end E. The decoupled damage indices
 438 well matched with the expected values at the beam ends A, B, and E.

439 More specifically, in the Case 1 of Group 1, the largest absolute difference between the
 440 expected and decoupled damage indices for five damages was about 2%. This indicates that
 441 the decoupling algorithm works very well for multiple damages on individual floors. In
 442 addition, the decoupled damage indices for the undamaged beam ends had the absolute
 443 difference of 3.5% on average compared with the expected values.

444 In Group 2, in which the neighboring floors, i.e., the third and fifth floors, sustained
 445 damages, when the decoupled damage indices for the damages were compared to the expected
 446 values, the largest absolute difference was 2.5% in Case 2, and 3.9% in Case 3 (Figure 10(b)).
 447 This indicates that the estimation method of the influence from damages of neighboring floors
 448 in the decoupling algorithm is effective. The measured damage index of damages on the
 449 neighboring floors can be used to compute the influence instead of the damage index
 450 corresponding to individual damage condition.

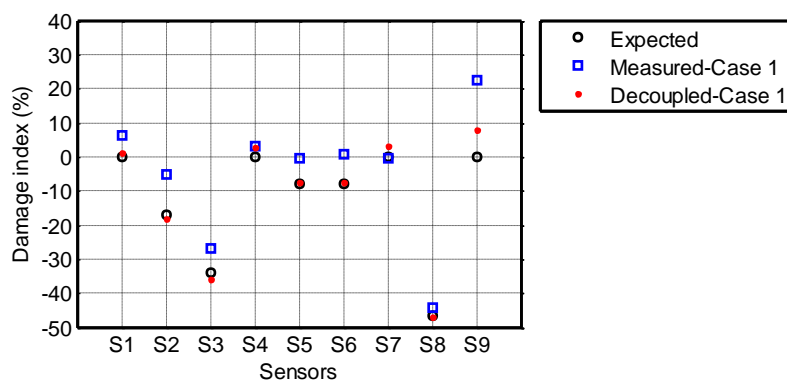
451 In Group 3, in which the nonadjacent floors, i.e., the second and sixth floors, sustained
 452 damages, the largest absolute difference between the expected and decoupled damage indices
 453 for five damages was 2.1% in Case 4, and 4.2% in Case 5 (Figure 10(c)), which verified that
 454 the influence from damages of nonadjacent floors is negligible in the decoupling algorithm.

455 In Group 4, the steel frame suffered a large number of damages at beam ends on the four
 456 neighboring floors. In Case 6, the decoupled damage indices for five damages on the fourth
 457 floor had the largest absolute difference of 4.4% at sensor S6 in comparison with the expected

458 values. This means the decoupling algorithm is capable of estimating the damage index in a
 459 complicated situation where many neighboring and farther floors are damaged. Nonetheless,
 460 in Case 7, in which the second and sixth floors were seriously damaged, the decoupled
 461 damage indices had large difference for small damages at the beam ends C and D compared to
 462 the expected values. The decoupled damage indices were -1.8% and -1.0% at sensor S5 and
 463 S6 for the small damages at the beam ends C and D, which was not easy for the damage
 464 identification.

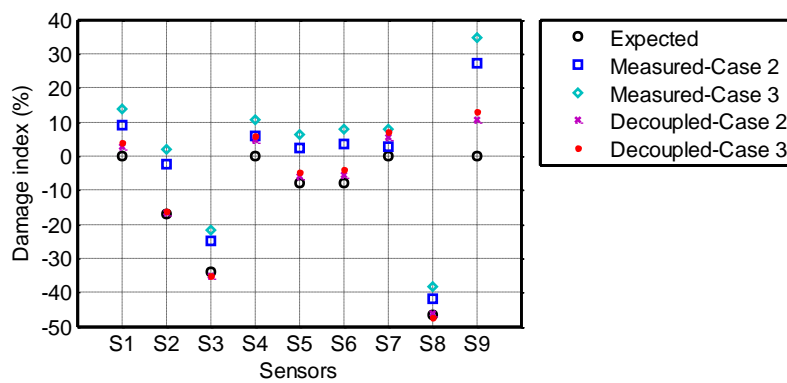
465 In summary, the suggested decoupling algorithm was effective in identifying multiple
 466 damages in steel frames but its performance slightly weakened for small damage in the
 467 extremely damaged conditions. Practically speaking, in the health monitoring of earthquake-
 468 affected steel buildings, damage detection has more focus on light or moderate damage
 469 conditions rather than serious damage states because steel buildings designed well hardly
 470 sustain a large number of severe damages close to collapse.

471



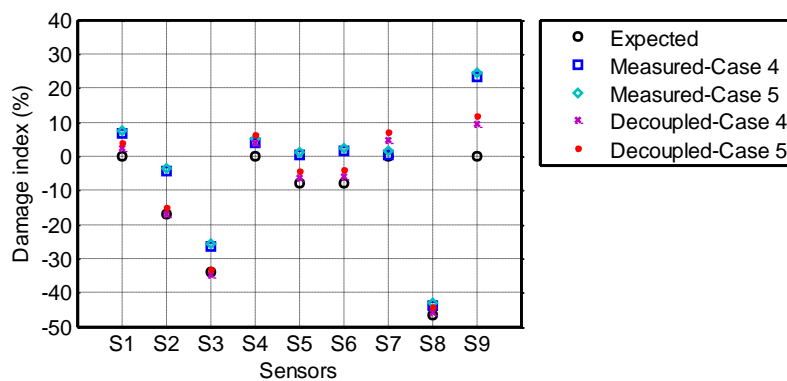
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(a)



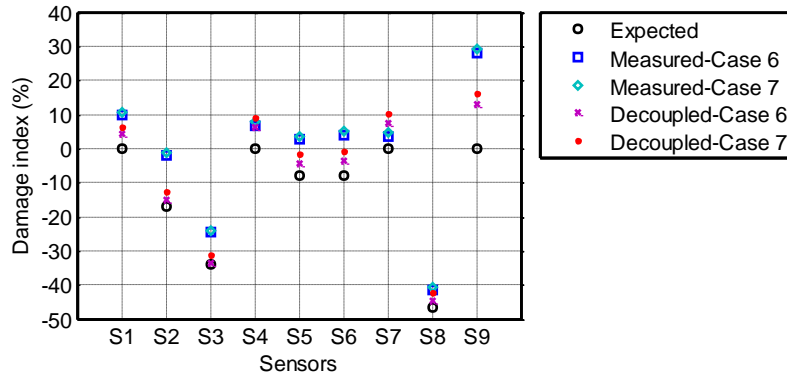
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(b)



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(c)



(d)

Figure 10. Measured and decoupled damage indices for multiple damages on the fourth floor: (a) Group 1; (b) Group 2; (c) Group 3; (d) Group 4.

5. EXPERIMENTAL INVESTIGATIONS

The decoupling algorithm of estimating damage indices for multiple damages was experimentally studied using a five-story steel frame testbed (see Figure 11(a)) constructed at the Disaster Prevention Research Institute (DPRI), Kyoto University. The dimensions of the testbed were $1.0 \times 4.0 \times 4.4$ m. Its plan was one bay by two bays. In each longitudinal steel frame, there were twelve steel removable connections at beam ends (i.e., connections B1 to B12, see Figure 11(b)), located at the second, third and fifth floors. Removable connection was made of four links at the flanges and one pair of links at the web (Figure 11(c)). The detailed information of the testbed was reported in Kurata et al. [17].

In vibrational testing, the testbed was excited using a modal shaker (APS-113, APS Dynamics) that was fixed to the steel mass plate on the fifth floor (Figure 11(d)). The strain responses of steel beams were measured using the wireless strain sensing system. PVDF strain sensors were placed on both sides of the beam bottom flange at 1.5 beam depths 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). Two PVDF strain sensors at the same beam section were treated as one sensor location as the average of the damage indices at two sides of the bottom flange was used in experimental investigations. There were 12 sensor locations, i.e., S1 to S12, located in the second to fourth floors, as shown in Figure 11(b).

By changing or removing the links, fracture damage was simulated. Figure 12 illustrates the undamaged state of the removable connection and three levels of fracture damage. Damage level 1 to level 3 (L1 to L3) simulated fracture of the whole bottom flange, fracture of the bottom flange and one-quarter of the web, and fracture of the bottom flange and half the web, respectively. As summarized in Table 2, the reduction in the bending stiffness about the major axis of the beam section was 53.4% for damage L1, 79.4% for damage L2, and 93.6% for damage L3.

Three tests including 9 damage cases were considered (Table 3) for the experimental investigation. In Test 1, damages L1 to L3 were simulated at the connection B1 for examining the mechanism of damage-induced moment release and redistribution. In Test 2, damage L3 was respectively simulated at four removable connections B1 to B4 to investigate the influence coefficients. In Test 3, two multiple damage cases were studied for the verification of the decoupling algorithm. Case 8 simulated two beam fractures at an individual floor; Case 9 simulated many fractures at two neighboring floors.

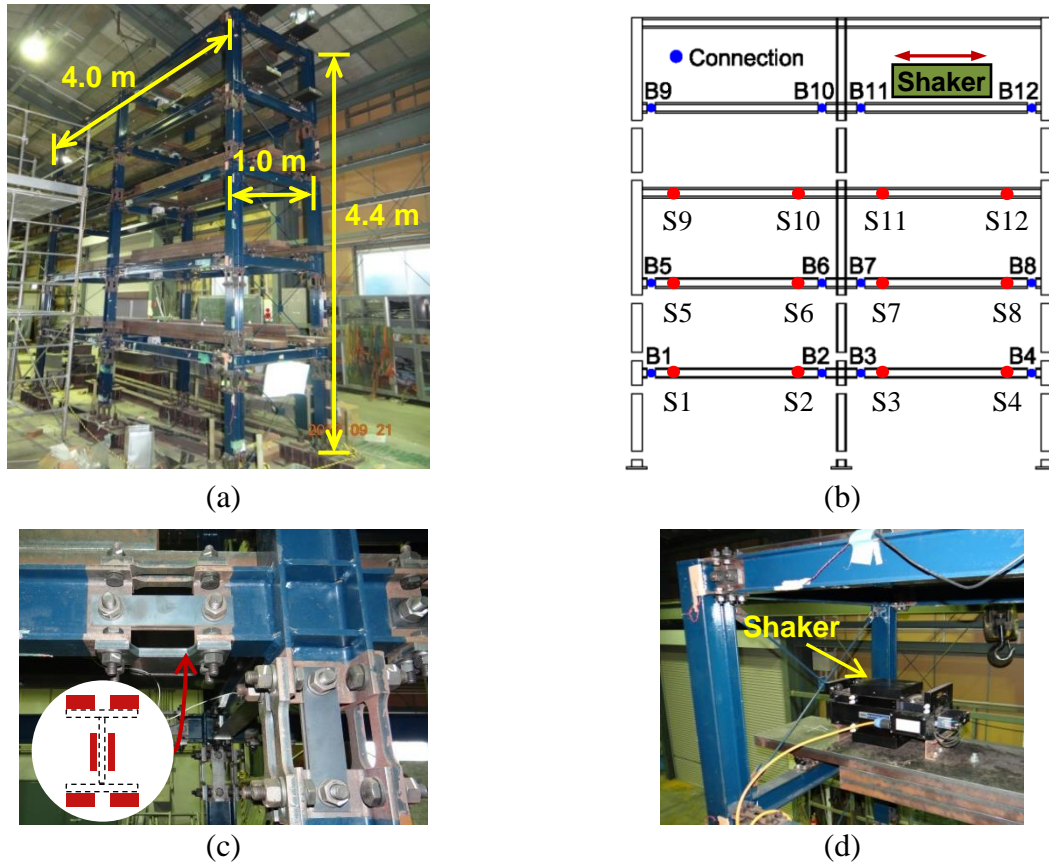


Figure 11. Five-story steel frame testbed: (a) overview; (b) beam connection and sensor location; (c) steel removable connection; (d) modal shaker [17, 18].

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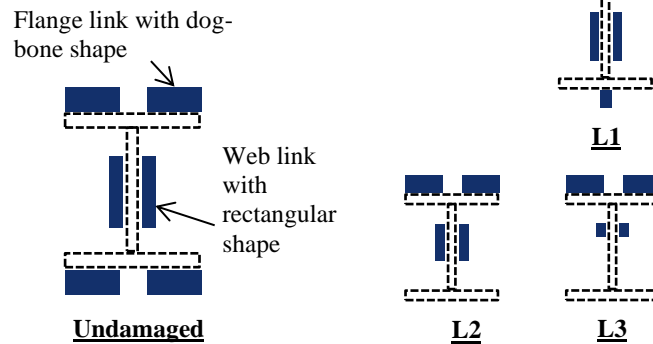


Figure 12. Undamaged state and damage patterns [19].

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Table 2. Damage patterns [19].

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

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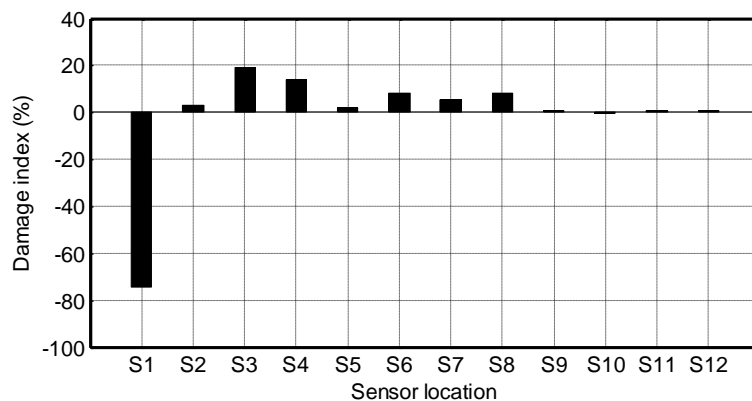
Table 3. Damage cases in experimental investigation.

Tests	Damage cases	Connections (Damage patterns)	Targets
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Test 1	Case 1	B1 (L1)	Investigation of moment release and influence
	Case 2	B1 (L2)	
	Case 3	B1 (L3)	
Test 2	Case 4	B1 (L3)	Investigation of influence coefficients
	Case 5	B2 (L3)	
	Case 6	B3 (L3)	
	Case 7	B4 (L3)	
Test 3	Case 8	B2 (L3), B3 (L1)	Verification of the decoupling algorithm
	Case 9	B1 (L2), B3 (L1), B4 (L3), B5 (L2), B8 (L3)	

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Figure 13 illustrates the damage indices of 12 sensor locations for the damage L3 at the connection B1 in Case 1. When the bending stiffness of connection B1 decreased by 93.6%, the damage index at sensor S1 was -74.2% . The damage index at other sensors on the same floor was at most 19.1% (see sensor 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.

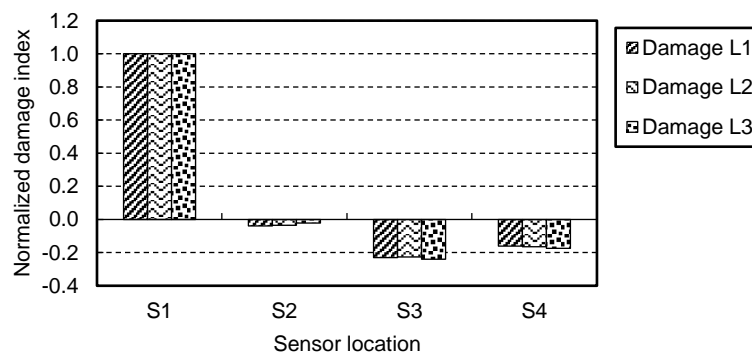


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Figure 13. Distribution of the damage index in Case 1.

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Figure 14 shows the normalized damage indices at four sensors S1 to S4 for three levels of the fracture damage (i.e., damages L1 to L3) simulated at the connection B1. The damage indices were normalized by the values of sensor S1. The normalized damage indices were nearly identical for three levels of the damage, which verifies the linear properties of the damage-induced moment release and redistribution.



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Figure 14. Normalized damage indices for three levels of fracture damage.

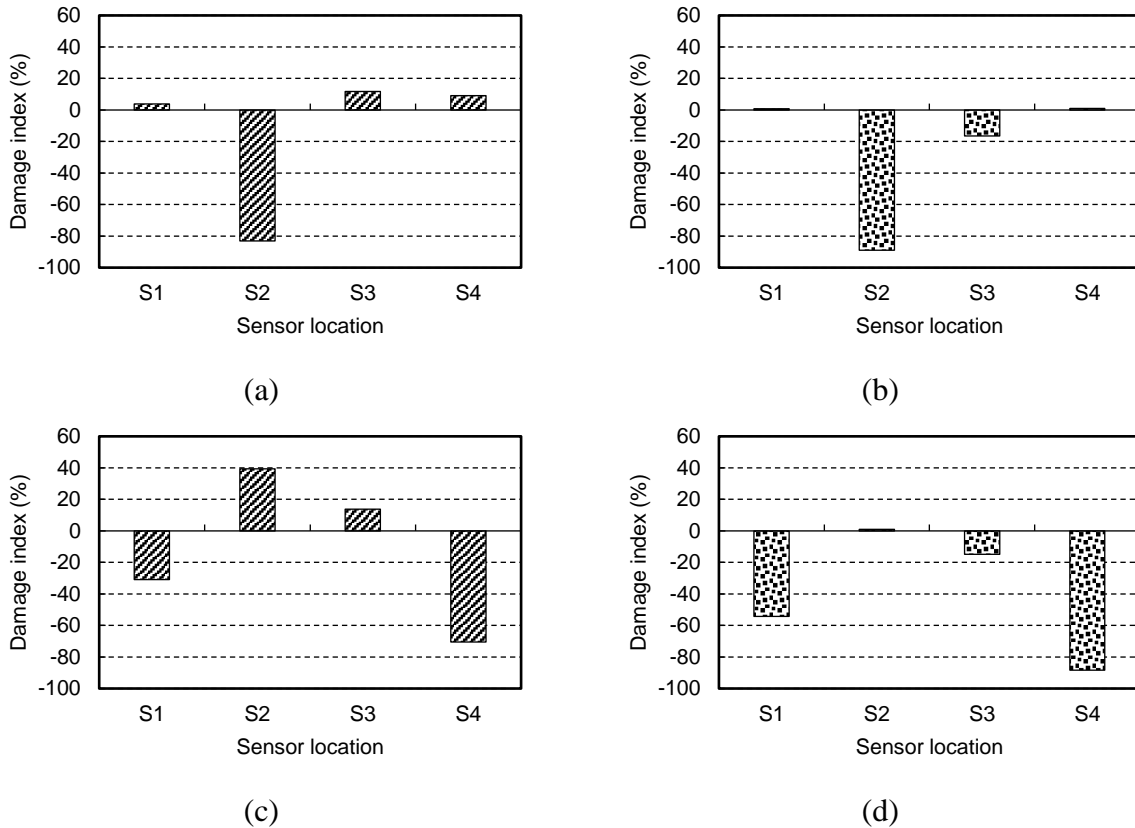
549 In order to verify the presented procedure for calculating the influence coefficients, an
550 experimental matrix C_e of the influence coefficients (see Equation 8(a)) was obtained through
551 the normalization of the damage indices in Test 2. The damage indices at four sensors S1 to
552 S4 were normalized by the damage index at the sensor near the simulated damages. For
553 example, the first column of C_e was calculated by normalizing the damage indices at sensors
554 S1 to S4 using the damage index at sensor S1 when the connection B1 sustained damage L3
555 in Case 4. The matrix C_p of the influence coefficients (see Equation 8(b)) was obtained using
556 the presented calculation procedure. When the matrix C_p was compared with the experimental
557 matrix C_e , only the influence coefficient $C_p(4, 2)$ had obvious differences.
558

$$559 \quad C_e = \begin{bmatrix} 1.00 & -0.01 & -0.14 & -0.19 \\ -0.04 & 1.00 & -0.38 & -0.26 \\ -0.26 & -0.32 & 1.00 & -0.04 \\ -0.19 & -0.08 & -0.07 & 1.00 \end{bmatrix} \quad (8(a))$$

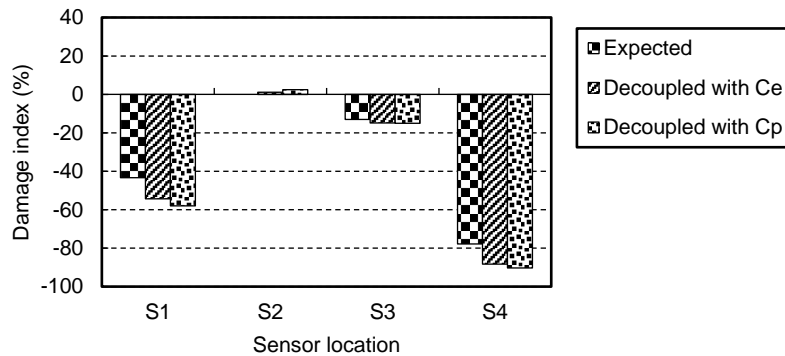
$$560 \quad C_p = \begin{bmatrix} 1.00 & -0.06 & -0.19 & -0.22 \\ -0.04 & 1.00 & -0.31 & -0.25 \\ -0.25 & -0.31 & 1.00 & -0.04 \\ -0.22 & -0.19 & -0.06 & 1.00 \end{bmatrix} \quad (8(b))$$

562
563 Figure 15 illustrates the measured and decoupled damage indices at four sensors S1 to S4
564 in Test 3. In Case 8, two fractures were simulated at neighboring connections B2 and B3. In
565 this case, the fracture L1 at connection B3 could not be detected from the measured damage
566 index of 11.7% (see Figure 15(a)), while the fracture was easily detected using the decoupled
567 damage index of -16.5% (Figure 15(b)). In Case 9, fracture damages were simulated at
568 connections B1, B3 and B4 on the second floor and connections B5 and B8 on the third floor.
569 Without the use of the decoupling algorithm, damage L2 and L3 at connections B1 and B4
570 were detected as the measured damage indices of -30.9% and -70.6% respectively, while
571 damage L1 at connection B3 was not identified from the measured damage index of 13.8%
572 (Figure 15(c)). In comparison, with the application of the decoupling algorithm, the damage
573 L1 at connection B3 was identified by the decoupled damage index of -14.9% (Figure 15(d)).

574 Figure 16 shows a comparison between the expected and decoupled damage indices at
575 sensors S1 to S4 in Case 9. The expected values were extracted from tests of individual
576 damage conditions. Compared with the expected damage indices, the damage indices
577 decoupled with experimental matrix C_e had the absolute differences of about 2% at sensor S2
578 and S3, and that of about 11% at sensors S1 and S4. This indicates that the decoupling
579 algorithm was effective in estimating the damage indices for multiple damage conditions. In
580 addition, the damage indices were nearly identical for the uses of the experimental matrix C_e
581 and analytical matrix C_p in the decoupling algorithm, which implies that the presented
582 procedure worked well in calculating the influence coefficients.
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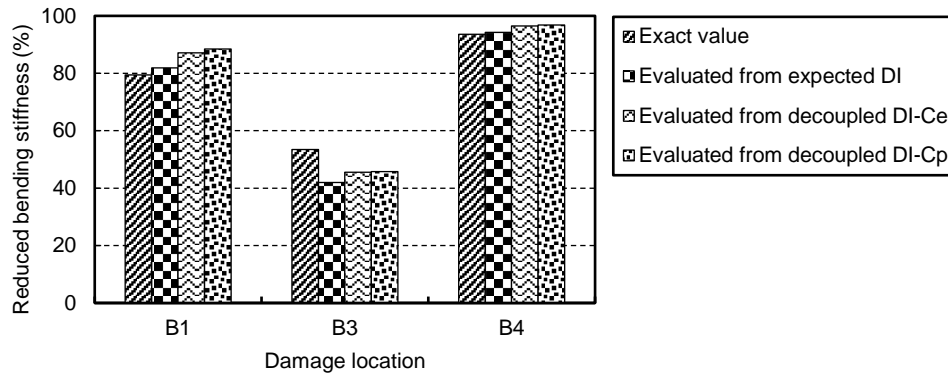
585 Figure 15. Measured and decoupled damage indices of sensors S1 to S4 on the second floor in
 586 Test 3: (a) measured damage indices in Case 8; (b) damage index decoupled with C_e in Case 8;
 587 (c) measured damage indices in Case 9; (d) damage index decoupled with C_e in Case 9.
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589 Figure 16. Expected and decoupled damage indices of sensors S1 to S4 on the second floor in
 590 Case 9.
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593 Figure 17 illustrates the estimated reduction of bending stiffness for fracture damages on
 594 the second floor in Case 9. The reduction of bending stiffness evaluated from the expression
 595 of the damage curve, i.e., Equation (2), using the expected and decoupled damage indices.
 596 Compared with the values estimated from the expected damage indices, the values estimated
 597 from the damage indices decoupled with C_e (or C_p) had the largest absolute difference of
 598 about 5%. When the values estimated from the damage indices decoupled with C_e were
 599 compared with the exact values calculated from the sectional properties, the differences were
 600 7.7% for damage L2 at connection B1, 7.9% for damage L1 at connection B3, and 2.9% for
 601 damage L3 at connection B4. The relatively large difference for damage L1 at connection B3

602 resulted from the expression of the damage curve which slightly underestimated fractures on
 603 bottom flanges as mentioned in [19].
 604



605
 606 Figure 17. Estimated reduction of bending stiffness for fracture damages on the second floor
 607 in Case 9.
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609 5. CONCLUSIONS

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 611
 612 This paper presented a decoupling algorithm of removing the influence of the neighboring
 613 damage interaction for accurately estimating the damage indices of multiple beam damages in
 614 steel moment-resisting frames. The decoupling algorithm was derived on the basis of the
 615 mechanism of damage-induced moment release and a model of three consecutive floors of a
 616 frame. The effectiveness of the decoupling algorithm was verified through numerical studies
 617 of a nine-story steel moment-resisting frame and vibrational tests of a large-scale five-story
 618 steel frame.

619 In the derivation, the analytical study of the four-story four-bay sub-frame illustrated that
 620 damage-induced moment releases in steel frames mainly distributes at the same floor levels,
 621 and the influence to other floors is at the most 5% to neighboring floors and negligible
 622 influence to non-adjacent floors. In addition, the moment releases largely affect the damage
 623 index of neighboring small damage, and has slight influence on the damage index for
 624 neighboring serious damage.

625 In numerical studies, the decoupling algorithm was very effective in identifying moderate
 626 and severe damages in all considered multiple damage conditions. For small damage which
 627 was hardly detected by the measured damage index, the decoupled damage index had
 628 powerful capability to identify it in most cases, but its performance slightly weakened in the
 629 extremely damaged states.

630 In experimental investigations, with the application of the decoupling algorithm, the
 631 accuracy of the damage indices for multiple beam damages was largely improved. The extent
 632 of the beam damage was successfully estimated using the decoupled damage index with the
 633 error of about 7%. Therefore, the decoupling algorithm facilitates the application of the
 634 proposed local damage evaluation method for monitoring the conditions of steel moment-
 635 resisting frames affected by earthquakes.

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639
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