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# Local Damage Indices of Frames Consisting of Composite Beams and RC Columns

Wei Li[1,*], Linzhu Sun[1], Kejia Yang[1], Lei Wang[2], Dongyan Wu[1] and XiaoQiang Ye[1]

[1] *College of Civil Engineering and Architecture, Wenzhou University, Wenzhou 325035, P.R. China*
[2] *College of Civil Engineering and Architecture, Guilin University of Technology, Guilin, P.R. China*

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**Abstract:** The study is to propose the local damage indices of composite frame structures consisting of high-strength concrete columns confined by continuous compound spiral ties and steel beams (CCSTRCS), the local damage indices would lay a foundation for the study of the overall damage indices for composite CCSTRCS frame. The Mehanny damage model has been modified to predict the local damage behavior of composite CCSTRCS frames, it enlarges the scope of application for the composite structures compared with the previous work. The proposed model is validated by comparing with the present references. The study results suggest the different components corresponding to the extent of the damage and its damage index.

**Keywords:** Composite beams, composite frame, joints, RC columns, seismic damage.

## 1. INTRODUCTION

The study of the seismic damage performance has been conducted during past decades, and a number of damage models have been suggested based on different conceptual assumptions. From the definition of the damage parameter and the process that described the seismic damage model of the structural elements, they were divided into the following categories:

(1) Non-cumulative damage model [1]; (2) deformation based on damage model [2]; (3) energy based on damage model [3, 4]; (4) combining deformation and energy based on damage model [5, 6]. Non cumulative damage model is to calculate component damage index using the outsourcing line of maximum response ductility or carrying capacity as a basic damage, but it does not reflect the effects of cyclic loading. Therefore, it can't accurately describe seismic performance. The energy based damage model accounts the influence on the force and deformation, but its calculation is very complicated. The combination of deformation and energy based on damage model is based on the largest deformation and energy dissipation. The most representative model is proposed by Park and Ang *et al.* [5], it is the linear combination of two parameter seismic damage model regarding maximum deformation and cumulative hysteretic energy. However, the model has deficiencies and its calculation is also complicated. The cumulative ductility-based damage model uses the cumulative plastic deformation or hysteretic energy to calculate the damage of structural members. The model is much easier than the energy based damage model. Although it ignored the force and deformation, it might be useful and more practical in damage assessment as its straightforward application with less complicated calculation. It gives good results when compared with experimental data.

On this basis, this present work proposed a modified Mehanny damage model to predict the damage performance of composite steel and concrete structural members. The analytical model provides plastic rotation capacity calculation method for the high strength concrete columns confined with high strength stirrups, composite steel and concrete beams as well as composite beams-concrete column joints. The study results will provide a basis for performance based

* Address correspondence to this author at the Chashan University Town, Wenzhou City, Zhejiang Province, 325035, P.R. China, College of Architecture and Civil Engineering, Wenzhou University People's Republic of China; Tel: +86-13506514071; E-mail: liweiwoaini521@hotmail.com

seismic design of composite steel and concrete frame.

## 2. MATERIAL AND METHODS

### 2.1. Structural Damage Index

The cumulative ductility based damage index proposed by Mehanny [2] was given as follows:

$$D_\theta^+ = \frac{\left(\theta_p^+\big|_{currentPHC}\right)^\alpha + \left(\sum_{i=1}^{n^+} \theta_p^+\big|_{FHC,i}\right)^\beta}{\left(\left(\theta_f - \theta_y\right)^+\right)^\alpha + \left(\sum_{i=1}^{n^+} \theta_p^+\big|_{FHC,i}\right)^\beta} \quad \textbf{(1)}$$

(for positive deformations)

$$D_\theta^- = \frac{\left(\theta_p^-\big|_{currentPHC}\right)^\alpha + \left(\sum_{i=1}^{n^-} \theta_p^-\big|_{FHC,i}\right)^\beta}{\left(\left(\theta_f - \theta_y\right)^-\right)^\alpha + \left(\sum_{i=1}^{n^-} \theta_p^-\big|_{FHC,i}\right)^\beta} \quad \textbf{(2)}$$

(for negative deformations)

$$D_\theta = \sqrt[\gamma]{\left(D_\theta^+\right)^\gamma + \left(D_\theta^-\right)^\gamma} \quad \textbf{(3)}$$

Where,. $\theta_p^+\big|_{currentPHC}$ is the current maximum positive plastic ration corresponding to latest Primary Half Cycles (PHC); once a new PHC is established, this term takes the new value, otherwise it keeps its old value.

$\theta_p^+\big|_{FHC,i}$ is maximum positive plastic rotation corresponding to Follower Half Cycles (FHC) number i;

$(\theta_f\text{-}\theta_y)^+$ is the plastic rotation capacity of the member that reach to the failure under monotonic loading in the positive deformation direction(method of calculation will be discussed later);

α,β and γ are calibration parameters.

Similar definitions apply to Equation (2) for negative deformations. Note that values of variables corresponding to negative deformation are taken as absolute values. Another important purpose of the main cycles and subsequent cycles is to eliminate unimportant cycles; therefore, the PHC and FHC are introduced.

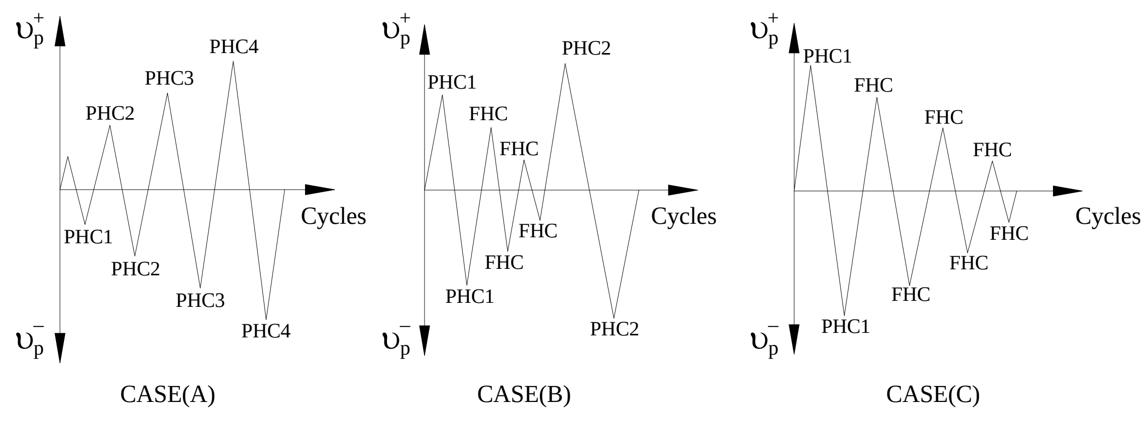



**Fig. (1).** Definition of PHCs and FHCs and load sequence effects by Mehanny and Deierlein (2001).

PHC corresponding to the first occurrence of the maximum displacement amplitude cycles, FHC corresponding to the smaller displacement amplitude cycles, as shown in (Fig. **1**). From case A to case C, which has different load history, but in the end will get the same result.

The calibration parameters of equations (1), (2) and (3) are respectively, α, β, and γ. They can be obtained through the test results or calibrated by specific members. When D≥1, the member is defined as failure. Reinforced concrete columns, composite steel-concrete beam and composite joints respectively corresponding to calibration parameters α, β, and γ values are listed in (Table **1**).

**Table 1. Damage index calibration parameters.**

| Parameters | Reinforced Concrete Columns | Composite Beams | Composite Joints |
|---|---|---|---|
| α | 1.00 | 1.00 | 0.75 |
| β | 1.50 | 1.50 | 3.00 |
| γ | 6.00 | 6.00 | 5.00 |

## 2.2. Inelastic Component Deformation Capacities

In this section, the criteria and procedures for computing failure values are discussed. As this study mainly focuses on seismic behavior of composite beams continuous compound spiral hoop concrete column frames, the failure criteria is only presented for reinforced concrete columns, composite steel and concrete beam as well as composite joints.

### *2.2.1. Reinforced Concrete Columns*

A number of failure criteria for reinforced concrete columns are as follows:

- In the curve of the load-deformation or moment-rotation, the strength drop (ranging from 10% to 30%) of the curve is observed. This method has considerable discrete, it is inappropriate in some cases.
- Failure of confinement corresponding to fracture of one hoop or stirrups that cause the onset of cyclic strength degradation as to progressive failure.
- Attainment of an ultimate tensile strain, $\varepsilon_{su}$ of longitudinal reinforcement, it is a measure of the likelihood of reinforcing bar rupture.
- The longitudinal reinforcement buckling for reinforced concrete columns between two consecutive layers of stirrups or series stirrups. The results will lead to a fracture of the longitudinal reinforcement, and its strength will rapidly degrade.
- Attainment of an ultimate compressive strain, $\varepsilon_{cu}$ of confined core concrete, it will cause crushing of concrete and loss of bearing capacity.

At first, these criteria should be used to define available capacities (failure points) as plastic rotation, ($\theta_f$-$\theta_y$). For reinforced concrete columns, the most promising variable can be used to quantify the limiting values describing failure, it is found to be the attainment of an ultimate compressive strain, $\varepsilon_{cu}$ , of confined core concrete. This is likely to happen in the columns before reaching an ultimate compressive strain. Thus, a limiting value for $\varepsilon_{cu}$ was adopted to complete available capacity followed by Paulay and Priestley in 1992 [7].

According to Paulay and Priestl y [2], the strain at peak stress, $\varepsilon_{cc}$, shown in Fig. (**2**), it does not represent the maximum effective strain, as high compression stresses can be maintained at several times larger strains. The effective limit strain occurs when stirrups confine steel fractures, thus ultimate compressive strain of confined core concrete, $\varepsilon_{cu}$ is obtained through stirrups at the strain energy of fracture as follows:

$$\varepsilon_{cu} = \varepsilon_c + \frac{1.4\rho_v f_{yv}\varepsilon_{sm}}{f'_{cc}} \quad \textbf{(4)}$$

Where, $\varepsilon_c$ is the maximum compressive strain of unconfined concrete (generally assumed to be 0.004); $\rho_v$ is the volumetric ratio of stirrup. For rectangular sections $\rho_v$=$\rho_x$+$\rho_y$, $\rho_x$=$A_{sh,x}$/$sh_c$, $\rho_y$=$A_{sh,y}$/$sb_c$, $s$ is the stirrup spacing, $A_{sh,x}$ and $A_{sh,y}$ are the stirrup-sectional area parallel to the x-direction and y-direction, respectively, $h_c$ and $b_c$ are width and depth of confined concrete (centerline to centerline of stirrups) respectively; $f_{yv}$ is the yield strength of the stirrups; $\varepsilon_{sm}$ is the reinforcement strain corresponding to the maximum tensile stress. $f'_{cc}$ is the compressive strength of confined concrete. The typical maximum compressive strain $\varepsilon_{cu}$ is from 0.01 to 0.06.

The compressive strength of the confined concrete is directly related to the effective confined stress $P_e$, the compressive strength of the concrete, stirrups strength and volumetric ratio of stirrup, was given by:

$$p_e = k_e \rho_v f_{yv} \quad \textbf{(5)}$$

Where, $k_e$ is the confinement effectiveness coefficient; $\rho_v$ is the volumetric ratio of stirrup; $f_{yv}$ is the yield strength of stirrups.

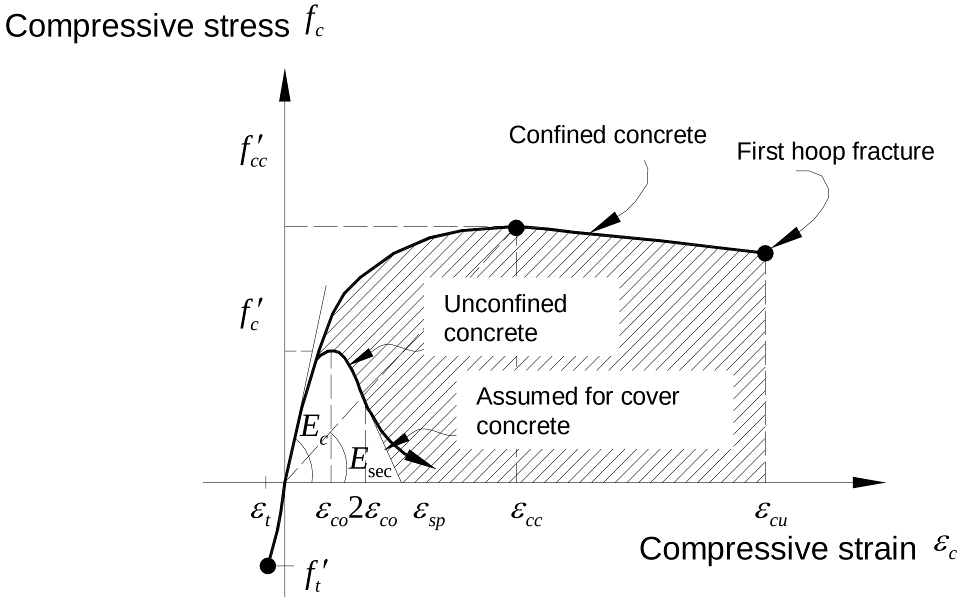



**Fig. (2).** Stress-strain model for monotonic loading of confined and unconfined concrete in compression (Paulay and Priestley, 1992).

The confinement effectiveness coefficient $k_e$ was given by Mander in 1988 [8]:

$$k_e = \frac{\left(1 - \frac{\sum_{i=1}^{n} w_i^2}{6 b_c h_c}\right)\left(1 - \frac{s'}{2 b_c}\right)\left(1 - \frac{s'}{2 h_c}\right)}{1 - \rho_{cc}} \quad \textbf{(6)}$$

Where, $\rho_{cc}$ is the ratio of longitudinal reinforcement area to core section area; $w_i$ is the *i*th clear stirrup spacing between adjacent longitudinal reinforcement; *n* is the number of longitudinal reinforcement; $s'$ is the clear stirrup spacing.

The establishment of the equation (6) is based on ordinary strength of concrete and stirrups. To increase the range of applications, Akiyama *et al.* [9] have conducted the test of the large size high strength rectangular concrete columns confined with high-strength stirrups. Thus, the equation (6) was modified as follows:

$$k_e = \frac{\left(1 - \frac{\sum_{i=1}^{n} w_i^2}{6 b_c h_c}\right)\left[2s'^2 - 5\left(b_c + h_c\right)s' + 15 b_c h_c\right]}{15 b_c h_c \left(1 - \rho_{cc}\right)} \quad \textbf{(7)}$$

Where, the notation of the equation (7) is the same with equation (6).

Although the $f_{yv}$ in the equation (5), the yield strength of stirrups is generally adopted. The stirrup strength corresponding to the peak stress of confined concrete does not necessarily reach the yield strength, especially when using high-strength concrete confined with stirrups. To this end, Akiyama *et al.* [10] proposed stirrups at peak stress $f_{s,c}$:

$$f_{s,c} = E_s\left[0.45\varepsilon_{co} + 6.39\left(\frac{k_e\rho_v}{f'_c}\right)^{0.881}\right] \le f_{yv} \tag{8}$$

Where,

$$\varepsilon_{co} = 0.0028 - 0.0008k_b \tag{9}$$

$$k_b = \frac{40}{f'_c} \le 1.0 \tag{10}$$

$$f'_c = 0.85f'_{co} \tag{11}$$

Where, $\varepsilon_{co}$ is the strain at the peak stress of unconfined concrete; $\rho_w$ is the volumetric ratio of stirrup; $k_e$ is the confinement effectiveness coefficient; $f'_c$is the compressive strength of unconfined concrete; $k_b$ is the effective coefficient of unconfined concrete; $f'_{co}$ is the cylinder compressive strength of ordinary concrete (cylinder diameter of 100mm, and a height of 200mm).

Suzuki *et al.* [9] used the regression analysis for test data of high-strength concrete columns confined with stirrups. The relationship of the peak stress and its corresponding peak strain was given by:

$$f'_{cc} = f'_c\left[1 + 2.28\left(\frac{p_e}{f'_c}\right)^{0.647}\right] \tag{12}$$

$$\varepsilon_{cc} = \varepsilon_{co} + 0.0766\left(\frac{p_e}{f'_c}\right) \tag{13}$$

Where, the definition of notation for equations (12) and (13) are the same with that of equations (5), (8) and (11); $\varepsilon_{co}$ is the unconfined concrete strain corresponding to the peak stress; $f'_c$ is the unconfined concrete compressive strength. Thus, ultimate compressive strain of confined core concrete, $\varepsilon_{cu}$ can be determined by equations (4), (5), (7), (8), (9), (10), (11), (12). After $\varepsilon_{cu}$ is determined, according to a strain-compatible moment-curvature analysis, the plastic curvature capacity of the column section is defined as ($Ø_f$-$Ø_y$). Where $Ø_f$ is the curvature corresponding to the attainment of; $\varepsilon_{cu}Ø_y$ is the yield curvature; the available plastic rotation capacity,$\theta_p$=$\theta_f$-$\theta_y$, could be obtained by plastic curvature ability and multiplied by the length of the plastic hinge:

$$\theta_p = \theta_f - \theta_y = \left(\phi_f - \phi_y\right)l_p \tag{14}$$

Where, $l_p$is plastic hinge length; $l_p$ was given by Paulay and Priestly [7]:

$$l_p = 0.08l + 0.022df_y \tag{15}$$

*l* is the ratio of column shear span; *d* is the diameter of the column longitudinal reinforcement in mm; $f_y$ is the yield strength of column longitudinal reinforcement.

### *2.2.2. Steel and Composite Beams*

The main criteria that define failure of steel and composite beams, which involves the interaction of local and lateral buckling, unloading (or strain-weakening) mechanisms, the crushing of concrete slab, separation between concrete slab and steel cross-section that is a type of loss of composite action.

Inelastic rotation capacity of steel and composite beams $\theta_p$ can reflect its ductile failure capacity. Many researchers

(1969 Lukey and Adams, [11] in 1986, Kemp [12], the 1987 Roik and Kuhlmann [13]) have conducted experimental studies. These studies have shown that the main factors affecting the inelastic rotation capacity are as follows: (1) the lateral slenderness ratio of steel beam flange under the negative moment area, (2) the width to thickness ratio of compressive flange and web. Based on the above results, Kemp and Dekker 1991 [14] established a simplified model, the model accounts for the distinction between positive moment and negative moment area of composite beams.

For lateral unbraced steel beams, the effective lateral slenderness ratio was given as follows:

$$\lambda_e = K_f K_w K_d \left(L_i / i_c\right)\gamma_f, \quad 25<\lambda_e<140 \tag{16}$$

Where, $L_i$ is the length of steel beam from the section of maximum moment to the point of inflection; $i_c$ is the radius gyration about the minor axis for the part of the cross-section in compression; $\varepsilon_f$ is, $\sqrt{235/f_{yf}}$ where $f_{yf}$ is the yield strength of flange in MPa; $K_f$ and $K_w$ are empirical factors to allow for actual flange and web slenderness, respectively. They were given as follows:

$$K_f = \frac{\left(b / t_f \varepsilon_f\right)}{20} \tag{17}$$

$$K_w = \frac{\alpha d_w}{33 t_w \varepsilon_w}, \quad 33<\frac{\alpha d_w}{t_w \varepsilon_w}<40 \tag{18}$$

$$K_w = \frac{\left[460 - \left(L_i / i_c \varepsilon_w\right)\right]\sqrt{\frac{\alpha d_w}{33 t_w \varepsilon_w}}}{400}, \quad \frac{\alpha d_w}{t_w \varepsilon_w} \leq 33 \tag{19}$$

Where, $b$ and $t_f$ are respectively the flange width and thickness of steel beams; $\alpha=h_c/d_w$ is the proportion of web depth in compression, $d_w$ and $t_w$ are the web depth and thickness, and $\varepsilon_w$ is, where, $\sqrt{235/f_{yw}}$ is the yield strength of web in Mpa.

Kemp1996 [14] introduced torsion confined coefficient $K_d$, thus equation (16) was modified:

$$\lambda_e = K_f K_w K_d \left(L_i / i_c\right)\gamma_f, \quad 25<\lambda_e<140 \tag{20}$$

$$K_f = \frac{\left(b / t_f\right)\gamma_f}{9}, \quad 0.7<K_f<1.5 \tag{21}$$

$$K_w = \frac{\left(h_c / t_w\right)\gamma_w}{70}, \quad 0.7<K_w<1.5 \tag{22}$$

$$K_d = \begin{cases} 1, \text{simplified beams} \\ 0.71, \text{continuous beams} \end{cases} \tag{23}$$

Where, $\gamma = \sqrt{f/250}$; $f$ is the flange or web yield strength in Mpa; $h_c$ is the composite beam web depth in compression.

By adjusting the inelastic rotation capacity of composite beam, it is obtained [15, 16]:

$$\theta_p = \frac{3.015\left(60/\lambda_e\right)^{1.5}}{\alpha}\theta_y \tag{24}$$

Assuming linear moment gradients, the elastic yield rotational capability is given as follows:

$$\theta_y = \frac{0.5M_p L_i}{EI} \tag{25}$$

Where, $M_p$ is the plastic moment strength of steel beams; *EI* is the elastic flexural stiffness of the steel beams.

### *2.2.3. Steel Beams-Reinforced Concrete Columns Joints*

The composite frames presented in this paper consisting of steel beams and high-strength concrete columns confined with high-strength stirrups. The steel beams continuously pass through the concrete columns at the beam-column joint. The typical beam-column joint is shown in (Fig. **3**). Such joint test results have revealed that it has two failure modes, one for shear failure of web yield and concrete cracking or spalling, another is the bearing failure of high-pressure concrete crushing as well as up and down to the flange yield. Shear failure also exist in the bearing failure, but the bearing failure does not produce shear deformation. The bearing failure should be avoided for seismic design, which was also reflected in the ASCE design guidelines 1994.

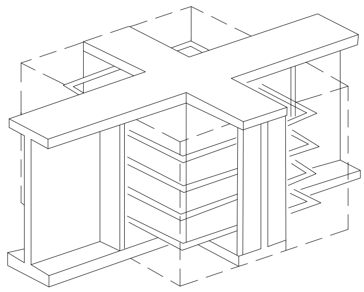



**Fig. (3).** The typical beam-column joint.

At present, no models are available to predict the ultimate deformation capacity of composite joints. Therefore, the selection of suitable values for the damage parameter is based on the results of experimental work by Kanno 1993 [17] and Bugeja 1999 [18], Parra-Montesinos *et al.* 2000 [19], Liang 2003 [20], Fargier-Gabaldón 2005 [21], the predetermined joint rotation capacity at the point where the joint resistance drops to below 0.8 time its maximum strength. Based on linear regression of test data for eleven specimens failing in joint shear and four specimens in bearing, as shown in Fig. (**4**), the joint rotation capacity is given as follows:

$$\theta_p = 0.075 - 0.033\left(M_{ps}/M_{vb}\right) \quad 0.3<M_{ps}/M_{vb}<1.7 \tag{26}$$

Where, $M_{ps}$ and $M_{vb}$ are the nominal shear and bearing moment capacity of the joint, respectively. The expression of $M_{ps}$ and $M_{vb}$ follows ASCE design guidelines 1994. Regarding the equation (26) established is based on cyclic loading, under monotonic loading, it should be multiplied by 1.2 amplification factors, which is obtained as:

$$\theta_p = 0.09 - 0.04\left(M_{ps}/M_{vb}\right) \quad 0.3<M_{ps}/M_{vb}<1.7 \tag{27}$$

## 3. RESULTS

### 3.1. Validity of Damage Modeling

The local damage indexes have been given in previous section, and the rotation equation for high-strength concrete columns confined with high-strength stirrups, steel and composite beams, beam-column joint is derived based on the published test results. The proposed local damage index and rotation capacity are calibrated based on different test results.

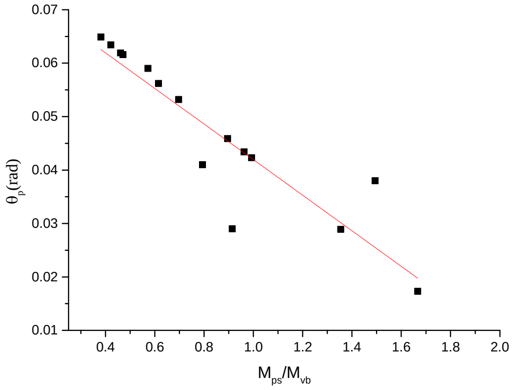



**Fig. (4).** The relation of joint distortion failure and ratio of moment under cyclic loading.

#### *3.1.1. High-Strength Concrete Columns Confined with High-Strength Stirrups*

In section 3.1, the model of damage index and deformation ability is verified by the test data available in the literature. The columns are as follows: one specimen (V5.5-66) by Woods *et al.* 2007 [22]; two specimens (HHL91 and UHL61) by YAN *et al.* 2006 [23]; three specimens (HSC-S1-1, HSC-S2-1 and HSSC-S3-1) by Chen *et al.* 2009 [24]. All specimens were conducted under cyclic loading reached failure, and these specimens were computed in terms of damage index which are listed in (Table **2**). The damage index is taken as 1, which agrees well with the test results.

**Table 2. Calculation of damage index parameters for RC columns.**

| The Number of Specimens | Limited Compressive Strain™$_{\varepsilon cu}$ | $\theta_p=\theta_f-\theta_y$/rad | Damage Index$D_\theta$ |
|---|---|---|---|
| V5.5-66 | 0.0154 | 0.0231 | 0.9611 |
| HHL91 | 0.0203 | 0.0313 | 1.0606 |
| UHL61 | 0.0253 | 0.0328 | 1.0327 |
| HSC-S1-1 | 0.0153 | 0.0261 | 0.9913 |
| HSC-S2-1 | 0.0157 | 0.0254 | 0.9956 |
| HSSC-S3-1 | 0.0158 | 0.0268 | 1.0044 |

#### *3.1.2. Steel and Composite Beams*

To verify the proposed damage index and deformation capacity for steel and composite beams, the published experimental data are adopted to assess. The test data include ordinary steel beams and steel and composite beams. These tests are considered as follows: two specimens for ordinary steel beams tested by Kanno 1993 [17]. The tested results of the two specimens (OB1-1 and OBJS1-1) have shown that they occur beam failure rather than joint failure. For the steel and composite beams, four specimens are considered, respectively, the test specimen(G2) by Tagawa 1989 [25], the specimen(sy-1) by Xue *et al.* 2002 [26] , two specimens by lightweight 2005 [27], one is simplified beam(SCB-14), another is continuous beam(CCB-13). The computed values of damage index parameters are listed in (Table **3**). From statistical measures, as shown in Table **3**, the overall damage status agrees well with damage index for the specimen, the failure mode of almost all specimens is the compression buckling of steel beams causing instability, steel beams under flange and web buckling, finally the crushing of concrete slab.

**Table 3. Calculation of damage index parameters for composite beams.**

| The Number of Specimens | $\theta^+_p=(\theta_f-\theta_y)^+$/rad | $\theta^-_p=(\theta_f-\theta_y)^-$/rad | Damage Index$D_\theta$ |
|---|---|---|---|
| OB1-1 | 0.0630 | 0.0630 | 1.03 |

*(Table 3) contd.....*

| The Number of Specimens | $\theta^+_p=(\theta_f-\theta_y)^+$/rad | $\theta^-_p=(\theta_f-\theta_y)^-$/rad | Damage Index$D_\theta$ |
|---|---|---|---|
| OBJS1-1 | 0.0624 | 0.0624 | 1.04 |
| G2 | 0.0867 | 0.0612 | 1.02 |
| sy-1 | 0.0548 | 0.0766 | 1.01 |
| SCB-14 | 0.0557 | 0.0864 | 0.98 |
| CCB-13 | 0.0550 | 0.0919 | 0.99 |

### 3.1.3. Steel Beams-Reinforced Concrete Columns Joints

In this study, the proposed damage index and the deformation capacity model in section 3.3 are verified based on the published tested results. These tests are conducted by Kanno 1995 [17], Bugeja 1999 [18], Parra-Montesinos *et al.* 2000 [19], Liang 2003 [20], and Fargier-Gabaldón 2005 [21]; they comprise of ten specimens, failing mainly in joint shear failure and five specimens failing in joint bearing failure. The calculation of the damage indexes are listed in Table **4**, these values are computed based on the procedure presented in section 3.3. From statistical measures, it is shown in Table **4**, the overall damage status agree well with that of the tested specimen. The failure modes are the crushing of concrete, concrete spalling and the yielding of stirrups and steel beam web, finally core concrete is exposed in the joint. It is worth pointing out that the prediction of failure through the proposed damage index is much better for joints with predominately shear failure mode than for joints with bearing failure mode. Inelastic rotation capacity of shear failure joints is significantly larger than that of bearing failure joints. It has revealed that shear failure joints have a better energy dissipation capacity and ductility. Therefore, the design of the joint should be designed to shear failure as much as possible.

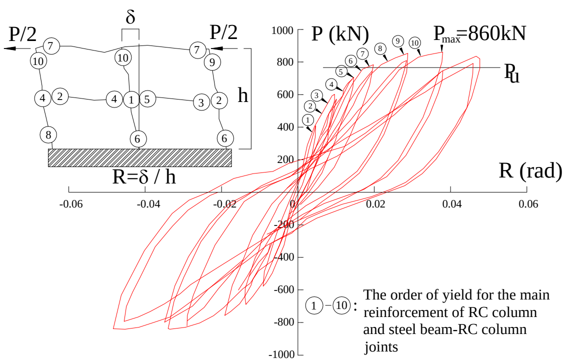
**Table 4. Calculation of damage index parameters for composite joints.**

| The Number of Specimens | $M_{ps}/M_{vb}$ | $\theta_p$/rad | Damage Index$D_\theta$ |
|---|---|---|---|
| **Shear failure** | | | |
| Kanno OJS7-0 | 0.7212 | 0.0523 | 0.9978 |
| Kanno HJS1-0 | 0.4711 | 0.0616 | 1.0084 |
| Kanno HJS2-0 | 0.4220 | 0.0634 | 0.9989 |
| Kanno HJS3-0 | 0.3815 | 0.0649 | 1.0211 |
| Bugeja #2 | 0.6149 | 0.0562 | 1.0322 |
| Bugeja #3 | 0.9625 | 0.0434 | 0.9826 |
| Parra 1 | 0.4613 | 0.0619 | 0.9959 |
| Parra 6 | 0.9929 | 0.0423 | 1.0152 |
| Liang 2 | 0.6973 | 0.0532 | 0.9756 |
| Fargier 1W | 0.8959 | 0.0459 | 1.0093 |
| **Bearing failure** | | | |
| Kanno OJB4-0 | 1.4963 | 0.0236 | 1.0220 |
| Kanno OJB5-0 | 1.2907 | 0.0312 | 1.0264 |
| Kanno OJB6-1 | 1.5465 | 0.0218 | 0.9848 |
| Bugeja #4 | 1.6667 | 0.0173 | 1.0469 |
| Bugeja #5 | 1.3547 | 0.0289 | 1.0319 |

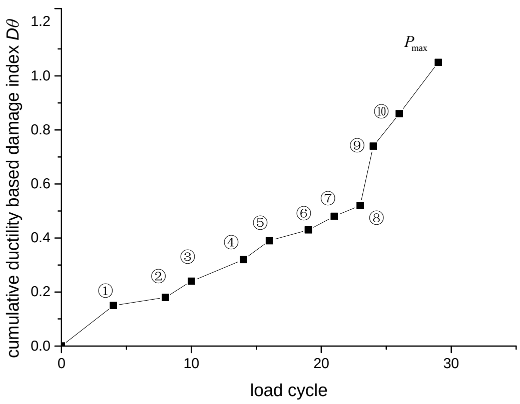
In the case of the progressively increasing seismic demands, the evolution of the damage index can predict the damage (necessary repairs and associated economic losses) process. The composite frame tested by Baba and Nishimura (1998) [28] is to illustrate that mentioned problems. Fig. (**5a**) shows the relationship of hysteretic load *versus* deformation, and Fig. (**5b**) shows the relationship of damage index *versus* load cycle. The two figures reflect the damage index evolution process under different damage level, which can provide a basis for designing or repairing.

**Table 5. The relations of structural damage index and structural performance level.**

| Performance level | Damage index (D) |
|---|---|
| Basic intact(intact) | 0~0.25 |
| Little damage | 0.25~0.40 |
| Life safety | 0.40~0.60 |
| Collapse prevention | 0.60~0.95 |
| Collapse | ~0.95 |




(a)




(b)

**Fig. (5).** The damage evolution for composite frame specimen: **(a)** hysteretic load *versus* deformation; **(b)** damage index *Du versus* load cycle.

## 4. DISCUSSION

When the damage indexes are calculated at the local level for various structural components, it is useful to relate them to the level of damage attained by the component. This information is quite important to assess the structural behavior according to the performance criteria often expressed at the following structural performance levels: basic intact (intact), little damage, moderate damage, significant damage and collapse. It describes high strength concrete columns confined with high strength stirrups. It is hard to draw some strong conclusions concerning relating damage index values to the actual damage due to the lack of observed damage information reported during testing. However, the ductility level and the damage index of each column are corresponding to structural performance level. The displacement ductility coefficient between 1.0 and 1.5, which is corresponding to a damage index between 0 to 0.25 is related to structural intact. The displacement ductility coefficient between 1.5 and 2.0, which is corresponding to a damage index between 0.25 to 0.40 is related to structural little damage. The displacement ductility coefficient between 2.0 and 2.5, is corresponding to a damage index between 0.40 to 0.60 which is related to structural life safety level. The displacement ductility coefficient between 3.0 and 4.0, is corresponding to a damage index between 0.60 to 0.95 that related to prevent structural collapse. When the damage index value is more than 0.95 which corresponding to structural collapse or overall damage. It is seen in (Table **5**). Proposed damage index is based on limited test data, it is estimated approximately. In addition, the damage index of the test for local damage index, it is not the actual response to the overall damage of the structure. However, it can provide a reference to evaluate structural damage. For steel-concrete composite beams and composite beams reinforced concrete column joint, it is recommended that the same range of the damage index with high-strength concrete columns confined with high-strength stirrups is recommended. However, the local damage index just only reflect the performance of the local members or structural frames, to better indicate the overall damage of frame structures, it is very important to quantify the relations of local damage and the overall damage of frame structures.

## CONCLUSION

This paper is focused on modifying the Mehanny damage model, it is calibrated based on different test results in the literature. The results have shown that the damage index agrees well with damage extent of various specimens under the actual testing. Based on the experimental data of the literature, the extent of the damage for the different components corresponding to the damage index value is recommended. It is divided into: basic intact (intact), little damage, life safety, collapse prevention and collapse. The damage index values can provide a reference for the performance-based seismic design. As the proposed damage index is a local damage index, therefore, the overall damage of the structure should be determined considering the relationship of the damage index value of various members. Thus, to describe the overall damage of the structure, the next work requires to establish an overall damage index of the structure considering its overall damage behaviors.

## NOMENCLATURE

| | | |
|---|---|---|
| $A_s$ | = | Steel beam cross-sectional area |
| $A_{sh,x}$, $A_{sh,y}$ | = | Area of stirrup in x and y directions, respectively |
| $\alpha$, $\beta$ , $\gamma$ | = | Calibration parameters of damage index |
| $\varepsilon_c$ | = | The maximum compressive strain of unconfined concrete |
| $\rho_v$ | = | The volumetric ratio of stirrup |
| $s$ | = | The stirrup spacing |
| $h_c$, $b_c$ | = | Width and depth of confined concrete (centerline to centerline of stirrups) respectively |
| $f_{yv}$ | = | The yield strength of the stirrups |
| $\varepsilon_{sm}$ | = | The reinforcement strain corresponding to the maximum tensile stress |
| $f'_{cc}$ | = | The compressive strength of confined concrete |
| $f'_c$ | = | The compressive strength of unconfined concrete |
| $P_e$ | = | The compressive strength of the confined concrete is directly related with effective confined stress |
| $k_e$ | = | The confinement effectiveness coefficient |
| $\rho_{cc}$ | = | The ratio of longitudinal reinforcement area to core section area |
| $w_i$ | = | The *i*th clear stirrup spacing between adjacent longitudinal reinforcement |
| $n$ | = | The number of longitudinal reinforcement |
| $s'$ | = | The clear stirrup spacing |
| $f_{s,c}$ | = | Stirrups at peak stress |
| $\varepsilon_{cc}$, $\varepsilon_{co}$ | = | The strain at the peak stress of confined and unconfined concrete |
| $k_b$ | = | The effective coefficient of unconfined concrete |
| $f'_{co}$ | = | The cylinder compressive strength of ordinary concrete |
| $\varepsilon_{cu}$ | = | Ultimate compressive strain of confined core concrete |
| $f_f$ | = | The curvature corresponding to the attainment of $TM_{cu}$ |
| $f_y$ | = | The yield curvature |
| $u_p$ | = | The available plastic rotation capacity |
| $l_p$ | = | Plastic hinge length |
| $l$ | = | The ratio of column shear span |
| d | = | The diameter of the column longitudinal reinforcement in mm |
| $f_y$ | = | The yield strength of column longitudinal reinforcement |
| $L_i$ | = | The length of steel beam from the section of maximum moment to the point of inflection |
| $i_c$ | = | The radius gyration about the minor axis for the part of the cross-section in compression |
| $\varepsilon_f$ | = | $\sqrt{235 / f_{yf}}$ where $f_{yf}$ is the yield strength of flange in MPa |
| $K_f$ , $K_w$ | = | Empirical factors to allow for actual flange and web slenderness, respectively |

| | | |
|---|---|---|
| $b$, $t_f$ | = | The flange width and thickness |
| $\alpha = h_c/d_w$ | = | The proportion of web depth in compression |
| $d_w$, $t_w$ | = | The web depth and thickness, |
| $\varepsilon_w$ | = | $\sqrt{235 / f_{yw}}$ |
| $f_{yw}$ | = | The yield strength of web in Mpa. |
| $K_d$ | = | Torsion confined coefficient |
| $f$ | = | The flange or web yield strength in Mpa |
| $h_c$ | = | The composite beam web depth in compression |
| $M_p$ | = | The plastic moment strength of the steel beam |
| $EI$ | = | The elastic flexural stiffness of the steel beam. |
| $M_{ps}$ , $M_{vb}$ | = | The nominal shear and bearing moment capacity of the joint |
| $\Delta u_e$ | = | The elastic story drift |
| $\Delta u_p$ | = | The elasto-plastic story drift |
| $l_e$ | = | Effective lateral slenderness ratio |

## CONFLICT OF INTEREST

The authors confirm that this article content has no conflict of interest.

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