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A new damage index for reinforced concrete structures

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Abstract. Reinforced concrete (RC) structures are likely to experience damage when subjected to earthquakes. Damage index (DI) has been recognised as an advanced tool of quantitatively expressing the extent of damage in such structures. Last 30 years have seen many concepts for DI proposed in order to calibrate the observed levels of damage. The current research briefly reviews all available concepts and investigates their relative merits and limitations with a view to proposing a new concept based on residual deformation. Currently available DIs are classified into two broad categories – non-cumulative DI and cumulative DI. Non-cumulative DIs do not include the effects of cyclic loading, whilst the cumulative concepts produce more rational indication of the level of damage in case of earthquake excitations. Ideally, a DI should vary within a scale of 0 to 1 with 0 representing the state of elastic response, and 1 referring to the state of total collapse. Some of the available DIs do not satisfy these criteria. A new DI based on energy is proposed herein and its performances, both for static and for cyclic loadings, are compared with those obtained using the most widely accepted DI in literature. The proposed DI demonstrates a rational way to predict the extent of damage for a number of case studies. More research is encouraged to address some identified issues.

Keywords: cyclic loading, damage analysis, damage index, earthquake, hysteretic energy, reinforced concrete

1. Introduction

Earthquake excitations often cause damage to structures, the extent of which can be quantitatively described by an advanced tool called damage index (DI). Many proposals are currently available to calibrate DI based on a number of parameters such as deformation, stiffness, energy absorption etc. To date, there is no universally accepted range for the magnitude of DI, although a scale varying between 0 (no damage) to 1 (total collapse) would be rational for this indicator (Kappos 1997). Damage analysis has increasingly attracted many researchers to come up with both empirical and theoretical approaches in order to propose appropriate DIs. Currently

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available DIs have their own merits and limitations, and in some cases fail to reflect the state of damage appropriately. The most common drawbacks identified as part of the current research are as follows (a) the DI is not 0 when a structure operates within elastic range (b) the magnitude of DI often exceeds 1; i.e., there is no specific upper limit to define the state of collapse and (c) non-cumulative DIs do not include the effects of cyclic loading.

This paper briefly reviews currently available damage models, identifies their relative merits and limitations with a view to proposing a new concept based on residual deformation. Based on this concept, a new damage model is developed using a single parameter of energy which implicitly takes into account a number of parameters such as force, deformation and number of cycles. It is worth noting that the energy parameter has been increasingly demonstrated as a potential indicator for design and assessment rather than the concept of the maximum lateral deformation in the life time of structures (Surahman 2007). The proposed model satisfies all basic requirements as discussed in Section 3.1 and a damage classification based on the performance levels as defined in FEMA 356 (ASCE 2000) is made herein. It has demonstrated rational performances for a number of static and dynamic loading studied cases. More research is needed to address some of the issues identified in the model.

2. Literature review

Currently available concepts regarding DIs can be divided into two broad categories - non-cumulative DI and cumulative DI. Non-cumulative DIs are generally simple but often do not reflect the state of damage accurately due to the non-inclusion of the effects of cyclic loading. On the other hand, cumulative DIs are more rational but relatively more complicated than the non-cumulative DIs as they include the effects of cyclic loading. The following sections will review available DIs to date and will briefly address their significance.

2.1 Non-cumulative damage indices

The simplest available DI is the ductility ratio, which is expressed as the ratio of the maximum deformation u_m in loading time history to the yield displacement u_y. This concept produces damage indices varying from 0 to 1 when a structure works in the region before yielding and exceeds 1 when the structure goes into the plastic range after yield; i.e., there is no upper limit to define the state of collapse.

Lateral displacement is one of the most common parameters which can be used to define the extent of damage in a structure. This concept expresses DI as the ratio of the maximum relative lateral displacement Δu of a storey or a building to the height of that particular storey or building h. This ratio is called drift, which always produces damage indices with magnitudes much smaller than 1.

Drift can be divided into two types - transient drift and permanent drift. Both types of drift are closely related to the state of damage of a structure and hence are often used to evaluate the damage levels of a structure. Following guidelines are available in FEMA 356 (ASCE, 2000) to identify the damage state of a structure.

- Very light (operational): No permanent drift is observed. The original stiffness and strength of the structure are retained although individual elements may show minor cracking.

• Light (immediate occupancy): Transient drift < 1% and no or negligible permanent drift.
• Moderate (life safety): Transient drift < 2% and permanent drift < 1%. Residual strength and stiffness remain in the structure but the building may be economically un-repairable.
• Severe (collapse prevention): Transient or permanent drift < 4%.

In addition to the drift, FEMA 356 (ASCE 2000) defines different performance levels – Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) based on the use of plastic hinge capacity.

Roufaiel and Meyer (1981) analysed damage of concrete frame buildings and proposed a DI to be expressed as the ratio of the initial stiffness to the reduced secant stiffness at the maximum displacement. This model ignores tension cracks and produces a value of 0 at yielding, whilst generates a DI of 1 as the structure reaches its maximum displacement.

Banon *et al*. (1981) investigated the seismic damage in RC members and proposed a DI based on the flexibility of a structure, which was later modified by Roufaiel and Meyer (1987) as given in Eq. (1).

$$DI = \frac{f_m - f_o}{f_u - f_o} \tag{1}$$

where, f_o is the pre-yield flexibility, f_m is the secant flexibility at a given load, f_u is the secant flexibility at ultimate load. However, this model has the same limitations as that proposed by Roufaiel and Meyer (1981).

In recognition of the changing fundamental period (T) as structures experience different states of damage due to seismic excitation, DiPasquale *et al*. (1990) proposed an index called "final softening", which was later exploited by Kim *et al*. (2005) to define a DI as shown in Eq. (2). The changing fundamental period was later employed in Massumi and Moshtagh's (2010) damage model.

$$DI = 1 - \left(\frac{T_{initial}}{T_{final}} \right)^2 \tag{2}$$

where, T_{initia} is the fundamental period of the first step and T_{final} is the fundamental period of the last step.

Ghobarah *et al*. (1999) adopted a technique similar to DiPasquale *et al*. (1990) and Kim *et al*. (2005) but replaced the fundamental period terms by the stiffness parameters of the structure to assess the extent of damage. Eq. (3) shows formulation for the *i*-th storey, whilst Eq. (4) gives DI for the whole frame.

$$DI = 1 - K^i_{final} / K^i_{initial} \tag{3}$$

$$DI = 1 - K_{final} / K_{initial} \tag{4}$$

Powell and Allahabadi (1988) predicted seismic damage by deterministic approachs and used deformations to propose Eq. (5) to calculate DI, where u_m is the maximum deformation, u_y is the yield deformation and u_u is the ultimate deformation under a monotonic load. It is worth noting that the process of collapse can be clarified as: onset of collapse, near collapse (progressing towards collapse), and total collapse. The ultimate deformation u_u is defined as the deformation at the onset of collapse; therefore, u_m is larger than u_u when a structure is at the near collapse and total collapse situations. The limitation of Eq. (5) is that DI becomes negative when the structure works in the region before yielding and DI exceeds 1 when the structure is beyond the onset of collapse.

$$DI = \frac{u_m - u_y}{u_u - u_y} \tag{5}$$

Mergos and Kappos (2009) recently proposed a concept for DI combining the flexural DI (D_{fl}) and shear DI (D_{sh}) of a structure as shown in Eq. (6).

$$DI_{tot} = 1 - \left(1 - D_{fl}\right)^{\alpha} . \left(1 - D_{sh}\right)^{\beta} \tag{6}$$

where, α and β are exponents related to the relative importance of D_{fl} defined in Eq. (7) and D_{sh} defined in Eq. (8) to the total damage index DI_{tot}. Mergos and Kappos (2009) also proposed to take $\alpha = \beta = 1$. Eq. (9) shows a modified version of the total DI including the individual effects of flexure and shear.

$$D_{fl} = 1 - \left(1 - \frac{\varphi_{\max} - \varphi_o}{\varphi_u - \varphi_o}\right)^{\xi} \tag{7}$$

$$D_{sh} = 1 - \left(1 - \frac{\gamma_{\max} - \gamma_o}{\gamma_u - \gamma_o}\right)^{\rho} \tag{8}$$

$$D_{tot} = 1 - \left(1 - \frac{\varphi_{\max} - \varphi_o}{\varphi_u - \varphi_o}\right)^{\xi} . \left(1 - \frac{\gamma_{\max} - \gamma_o}{\gamma_u - \gamma_o}\right)^{\rho} \tag{9}$$

where, $\varphi_{\max}$ is the maximum curvature, φ_u is the curvature capacity and φ_o is the threshold value of curvature whilst, $\gamma_{\max}$ is the maximum shear distortion, γ_u is the shear distortion capacity and γ_o is the threshold value for shear distortion. ξ and ρ are parameters to incorporate the flexural deformation ratio and shear deformation ratio respectively. Assuming that they are equally important i.e. $\xi = \rho$, their proposed DI may be expressed as shown in Eq. (10).

$$D_{tot} = 1 - \left(1 - \frac{\varphi_{\max} - \varphi_o}{\varphi_u - \varphi_o}\right)^{\xi} . \left(1 - \frac{\gamma_{\max} - \gamma_o}{\gamma_u - \gamma_o}\right)^{\xi} \tag{10}$$

In their study, the assumption of $\varphi_o = \gamma_o = 0$ was used. This results in damage indices larger than 0 for any small elastic deformation. If φ_o and γ_o are corresponding to yielding values, the curvature ratio of Eq. (7) and the shear distortion ratio of Eq. (8) are very similar to the deformation ratio of Eq. (5), in which the deformation is separated into curvature and shear distortion. Hence, the limitation of this proposal is the same as that of Powell and Allahabadi (1988).

2.2 Cumulative damage indices

Cumulative damage models are more rational to evaluate damage states of structures, which experience cyclic loading or earthquake excitation. A trend to address the issue is to use a parameter which relates to damage and is cumulative during the loading time. In a simple way, Banon and Veneziano (1982) used normalised cumulative rotation as a DI which is expressed by the ratio of the sum of inelastic rotations during half cycles to the yield rotation.

Park and Ang (1985) proposed a DI based on deformation and hysteretic energy due to an earthquake as shown in Eq. (11). This is the best known and the most widely used DI (Kim *et al*. 2005), largely due to its general applicability and the clear definition of different damage states provided in terms of DI.

$$DI = \frac{u_m}{u_u} + \beta \frac{E_h}{F_y u_u} \tag{11}$$

where, u_m is the maximum displacement of a single-degree-of-freedom (SDOF) system subjected to earthquake, u_u is the ultimate displacement under monotonic loading, E_h is the hysteretic energy dissipated by the SDOF system, F_y is the yield force and β is a parameter to include the effect of repeated loading.

However, the following limitations are worth noting – DI > 0 when a structure works within elastic range and DI > 1 when the structure collapses with no specified upper limit for DI.

Park and Ang (1985) classified damage states into the following five levels.

DI < 0.1: No damage or localized minor cracking.
0.1 ≤ DI < 0.25: Minor damage: light cracking throughout.
0.25 ≤ DI < 0.40: Moderate damage: severe cracking, localized spalling.
0.4 ≤ DI < 1.00: Severe damage: concrete crushing, reinforcement exposed.
DI ≥ 1.00: Collapse.

DI ≥ 0.8 has been suggested to represent collapse (Tabeshpour *et al*., 2004). Park and Ang (1985) also proposed DI for an individual storey and for an overall structure using the weighting factor based on the amount of hysteretic energy (E_i) absorbed by the element or the component as shown in Eqs. (12) – (15).

$$DI_{storey} = \sum_{i=1}^{n} \left(\lambda_{i,component} . DI_{i,component} \right) \tag{12}$$

$$\lambda_{i,component} = \left[\frac{E_i}{\sum_{i=1}^{n} E_i} \right]_{component} \tag{13}$$

$$DI_{overall} = \sum_{i=1}^{n} \left\{ \lambda_{i,storey} DI_{i,storey} \right\} \tag{14}$$

$$\lambda_{i,storey} = \left[\frac{E_i}{\sum_{i=1}^{n} E_i} \right]_{storey} \tag{15}$$

Park and Ang's (1985) concept has been widely adopted by researchers and a number of proposed modifications are briefly discussed herein. The most significant modification was made by Kunnath *et al*. (1992) who used the moment-rotation behaviour to replace the deformation terms used by Park and Ang (1985) and subtracted the recoverable rotation as shown in Eq. (16).

$$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \beta \frac{E_h}{M_y \theta_u} \tag{16}$$

where, θ_m is the maximum rotation in loading history, θ_u is the ultimate rotation capacity, θ_r is the recoverable rotation when unloading and M_y is the yield moment. The merit of this modification is that DI will be 0 when structures work within elastic range. The major limitation to this proposal is, however, that the DI > 1 when the structure fails.

Some other modified versions of Park and Ang's (1985) model were proposed by Fardis *et al.* (1993), Ghobarah and Aly (1998) and Bozorgnia and Bertero (2001) which are very similar to the original model. It is worth noting that Park and Ang model is widely used up to now by researchers (e.g. Bassam *et al*. 2011, Ghosh *et al*. 2011, Yüksel and Sürmeli 2010) although it was proposed in 1985.

Stephens (1985) proposed a DI based on the theory of low-cycle fatigue to analyse the damage of structures subjected to seismic loads. The calibration of the proposed DI is relatively complicated, which is related to the whole response history of structures but does not include the effects of plastic deformation (Ghobarah *et al*. 1999). Reinhorn and Valles (1995) proposed Eq. (17) considering a similar approach following the rules of low-cycle fatigue. The major limitation of this proposal is that the DI becomes negative when the structure works in the region before yield and DI > 1 when the structure fails.

$$DI = \frac{u_m - u_y}{u_u - u_y} \frac{1}{\left(1 - \frac{E_h}{4(u_u - u_y)F_y} \right)} \tag{17}$$

The amount of energy absorbed by a structure is closely related to its corresponding damage state. Hence DI may be expressed as the ratio of the hysteretic energy demand (E_h) to the absorbed energy capacity of a structure under monotonic loading ($E_{h,u}$) (Cosenza *et al*. 1993; Fajfar 1992; Rodriguez and Padilla 2009). However, this proposed DI has no specific upper limit to define the state of collapse.

The absorbed energy is also used by Teran-Gilmore and Jirsa (2005) to propose their damage index as shown in Eq. (18).

$$DI = (2-b)\frac{aNE_h}{2r(\mu_u - 1)} \tag{18}$$

in which, a is the structural parameter that accounts for the energy content of the ground motion; b is the structural parameter that characterises the stability of the hysteretic cycle; r is the reduction factor that characterises the cyclic deformation capacity of a system; $\mu_u = u_u / u_y$ is the ultimate ductility capacity; $NE_h = E_h / F_y u_y$ is the normalised hysteretic energy. The parameter b varies from 1.5 to 1.8 (Teran-Gilmore and Jirsa 2005). For seismic design of ductile structures, b = 1.5 was used (Teran-Gilmore *et al*. 2010). In this case, Eq. (18) can be re-write as Eq. (19).

$$DI = \frac{aE_h}{4r\left(\frac{u_u}{u_y} - 1\right)F_y u_y} = \frac{aE_h}{r\left[4\left(u_u - u_y\right)F_y\right]} \tag{19}$$

The term $4\left(u_u - u_y\right)F_y$ is the energy of one ultimate complete cycle in the case of elastic-perfectly-plastic. In general, Eq. (19) is basically the ratio of the hysteretic energy demand to the energy of one ultimate complete cycle (can be understood as energy capacity). However, the incorporation of the two factors: a for the energy demand (E_h) and r for the energy capacity make the damage index as 1 when the structure collapses. The damage model suffers two issues pointed out by the authors: harmonizing the definitions used and the results obtained by researchers, and understanding clearly the damage model applying for the design of a particular structure (Teran-Gilmore and Jirsa 2005).

3. A new concept for DI

3.1 Essential characteristics of a DI

The extent of damage occurring in a structure when subjected to an earthquake excitation primarily depends on two factors - the structure itself and the applied external loading. The important parameters for a structure in damage mitigation are its stiffness, strength and damping characteristics, whilst in the case of loading, its intensity, energy and frequency contents play a vital role in causing damage. It is now widely accepted that the magnitude of DI should, ideally, vary between 0 and 1. A structure should not suffer any damage when it operates within its elastic limit and hence DI should be equal to 0 at this stage. On the other hand, the maximum possible magnitude for DI should be set equal to 1 referring to the event of total collapse. It is worth noting that most of the currently available concepts produce positive DI within the elastic range while their DI exceeds 1 in the event of failure.

Load-deformation curve in FEMA 356 (ASCE 2000) neglects the damage before yield, i.e., tension cracks in the concrete. This is a commnly adopted technique to simplify the relevant formulations for performance evaluation. Tension cracks are, however, inevitable and hence should be considered in damage models although the DI at this state would be much smaller than DI after yielding. Fig. 1 is the modified version of load – deformation curve in FEMA 356 (ASCE

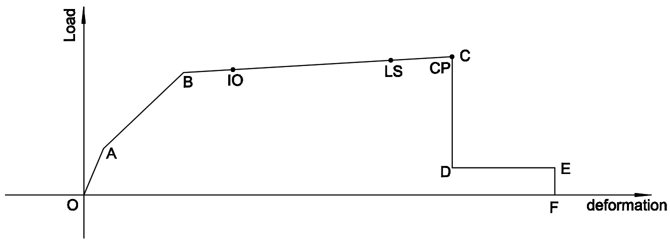


Fig. 1 Load - deformation relations considering tension cracking

2000) that includes the damage due to tension cracks, which is represented by point A.

Fig. 1 shows the relationship between load and the corresponding deformation i.e., force-displacement or moment-rotation. Three performance levels (IO, LS and CP) are also shown in Fig. 1. Points A, B and C represent the cracking, the yield and the ultimate state, respectively. The structure moves to near collapse situation when the load drops from C (ultimate load) to D (residual load), which is assumed to be 20% of the load at C (ASCE 2000), and then arrives at complete collapse at point F. The following conditions should be satisfied to achieve appropriate definitions for different states of damage.

From O to A: DI should be 0 as the RC structure (both concrete and steel) operates within the elastic range.

From A to B: DI should have a small positive magnitude; this is assumed to be a state when tension cracks start to appear i.e. concrete shows some hairline cracks but the reinforcement is yet to reach the yield limit. It is worth noting that DI for a structure operating within the region AB will be small.

From B to C: DI is assumed to increase sharply in this region producing larger positive magnitudes but will be smaller than 1. Yielding of reinforcement will initiate at this stage, resulting in larger cracks in the tension region. At the same time, the strain in the compressive concrete will rapidly increase to the strain at maximum stress. These conditions occur simultaneously and will result in an increase in damage; DI should increase sharply at this stage and will have a large value.

From C to F: DI will reach its maximum value of 1. The concrete in compression zone is assumed to have failed and the structure is unable to sustain any additional load representing a state of collapse.

In addition to the aforementioned conditions regarding the appropriate magnitude, DI should be cumulative to include the effects of cyclic loading as the extent of damage increases with an increase in number of cycles. Hence, if a structure undergoes the same amplitude in every cycle, the cumulative damage until the $(n-1)^{th}$ cycle should be less than the cumulative damage until the n^{th} cycle.

3.2 Basic concept of the proposed DI

The proposed concept for a DI is primarily based on residual deformation. Fig. 2 shows two simple structures – in Fig. 2b, a stub column is subjected to a vertical load, whilst in Fig. 2e, a column is subjected to a lateral force. The considered structures are assumed to experience a total deformation u_m at which point the applied forces are released. A portion

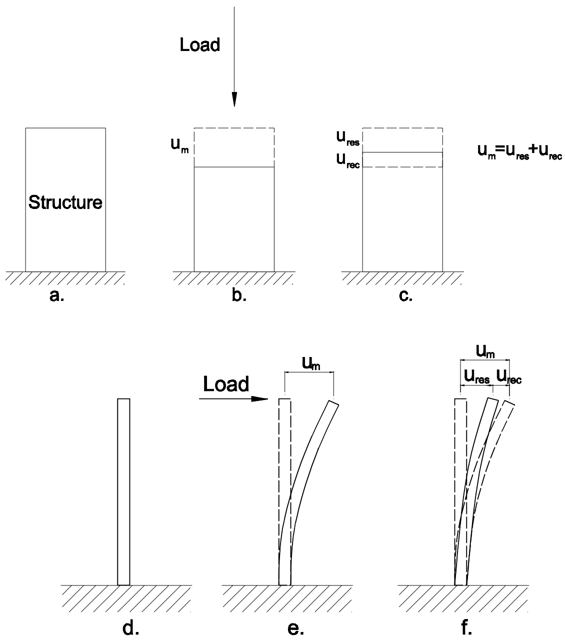


Fig. 2 Concept of the proposed DI based on residual deformation

of the total deformation u_m may be recovered (recoverable deformation - u_{rec}), whilst the rest may remain within the structure (residual deformation - u_{res}). The overall behaviour of the structure may be divided into two categories: (1) Elastic range - there is no residual deformation when the load is released and hence DI = 0 and (2) Plastic range - there will be some residual deformation left within the structure when the applied load is withdrawn and in this case, DI should produce a positive magnitude between 0 and 1.

In simple terms, initially, DI may be defined using Eq. (20) as the ratio of the residual deformation u_{res} to the total deformation u_m.

$$DI = u_{res} / u_m \tag{20}$$

Considering elastic perfectly-plastic behaviour, Eq. (20) may be modified to include the yield force F_y and expressed as shown in Eq. (21), where E_{rec} is the recoverable energy and $E_{non\text{-}rec}$ is the non-recoverable energy.

$$DI = \frac{F_y.u_{res}}{F_y.(u_{res}+u_{rec})} = \frac{F_y.u_{res}}{F_y.u_{res}+F_y.u_{rec}} = \frac{E_{non-rec}}{E_{non-rec}+2E_{rec}} \tag{21}$$

3.3 The proposed DI

Eq. (21) will produce DI = 0 when a structure operates within the elastic range. In other words, if the rotation of a structure is smaller than the cracking value, the rotation will be fully recovered and hence the residual rotation is 0. As a result, $E_{non\text{-}rec} = 0$ and thus DI becomes 0. RC structures generally experience negligible damage when working within the range between tension cracking of concrete and yield point; this leads to ignoring the effects of tension cracking whilst analysing RC structures to keep the commonly adopted procedures relatively simple. As discussed in Section 3.1, tension cracking in RC structure is a potential type of damage and thus should be allowed for in an appropriate damage model although the obtained DI will be a small positive number. The first two following damage states, as shown in Fig. 3, are proposed herein to appropriately identify the significance of tension cracks: (1) No damage: DI = 0 (before cracking point A); (2) Minor damage (operational): $0 < DI \leq DI_y$, where DI_y is the DI at yield (between points A and B).

A gradual increase in loading initiates yielding in the tensile reinforcement, increases the crack width and then results in failure in the concrete compression zone. Significant damage in RC structures occurs after yielding and hence DI should be large and increase sharply til the structure reaches its ultimate point. After the ultimate point, the DI should sharply increase and have values close to 1 to capture the damage state when the capacity of RC structures approaches to zero. The rate of increase of DI at this stage corresponding the dropping rate of the load will be the largest when compared to any other states. RC structures typically experience three performance levels (ASCE, 2000) – IO, LS and CP while operating in the plastic range. Additional three intermediate states of damage - light damage, moderate damage and severe damage are proposed in the current research based on those performance levels. The aforementioned analysis leads to the proposed damage classification as shown in Fig. 3 and the damage levels shown in Table 1. It is worth noting that the legends in the first column of Table 1 are used to express the damage levels in the studied cases in Section 4.

The proposed definition of DI, Eq. (21), is modified in the following sections to include the recognised essential characteristics of a DI.

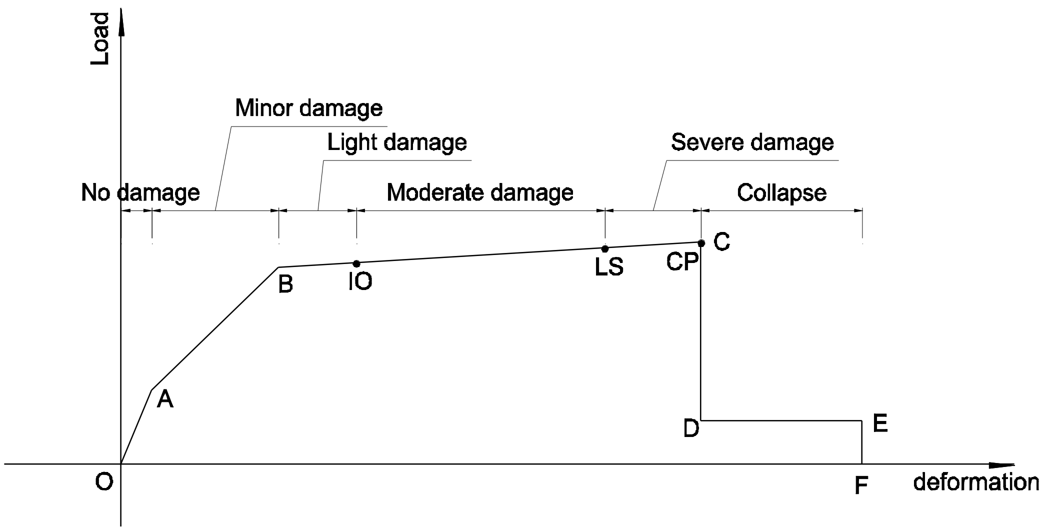


Fig. 3 Damage classification proposed in the current study

Table 1 Damage levels

Legends	Damage index	Description
.	0 - 0.05	No or minor
+	0.05 - 0.25	Light
x	0.25 - 0.50	Moderate
▲	0.50 - 0.75	Severe
●	0.75 - 1.00	Collapse

3.3.1 DI for monotonic loading

Eq. (22) gives DI for an individual structural element i.e. beam, column etc., whilst the DIs for individual elements may be combined using Eqs. (23) and (24) to obtain the overall DI for a structure. It is worth mentioning that Eqs. (23) and (24) exploit the concept proposed by Park and Ang (1985). Eqs. (25) and (26) define the parameters of the model, in which, $E_{non\text{-}rec,y}$ and $E_{non\text{-}rec,collapse}$ are non-recoverable energy at yield and at collapse, respectively, under monotonic loading. $DI_{k,element}$ is the damage index of the k^{th} element and $\lambda_{k,\ element}$ is the weighting factor based on hysteretic energy. It is worth noting that $E_{non\text{-}rec}$ is always less than or equal to $E_{non\text{-}rec,collapse}$, thus N is always larger than or equal to i. When i approaches to the value of N, the exponent in Eq. (22) approaches the value of zero and thus, DI tends to be one.

$$DI = \left[\frac{E_{non-rec}}{E_{non-rec} + E_{rec}} \right]^{(N-i)} \tag{22}$$

$$DI_{structure} = \sum_{k=1}^{n} \left(\lambda_{k,element} . DI_{k,element} \right) \tag{23}$$

$$\lambda_{k,element} = \left[\frac{E_{non-rec,k}}{\sum E_{non-rec,k}} \right]_{element} \tag{24}$$

$$N = \frac{E_{non-rec,collapse}}{E_{non-rec,y}} \tag{25}$$

$$i = \frac{E_{non-rec}}{E_{non-rec,y}} \tag{26}$$

Energy parameters used in the proposed concept are determined as shown in Fig. 4 by assuming that the unloading branch will be parallel to the initial branch before cracking as represented by OA.

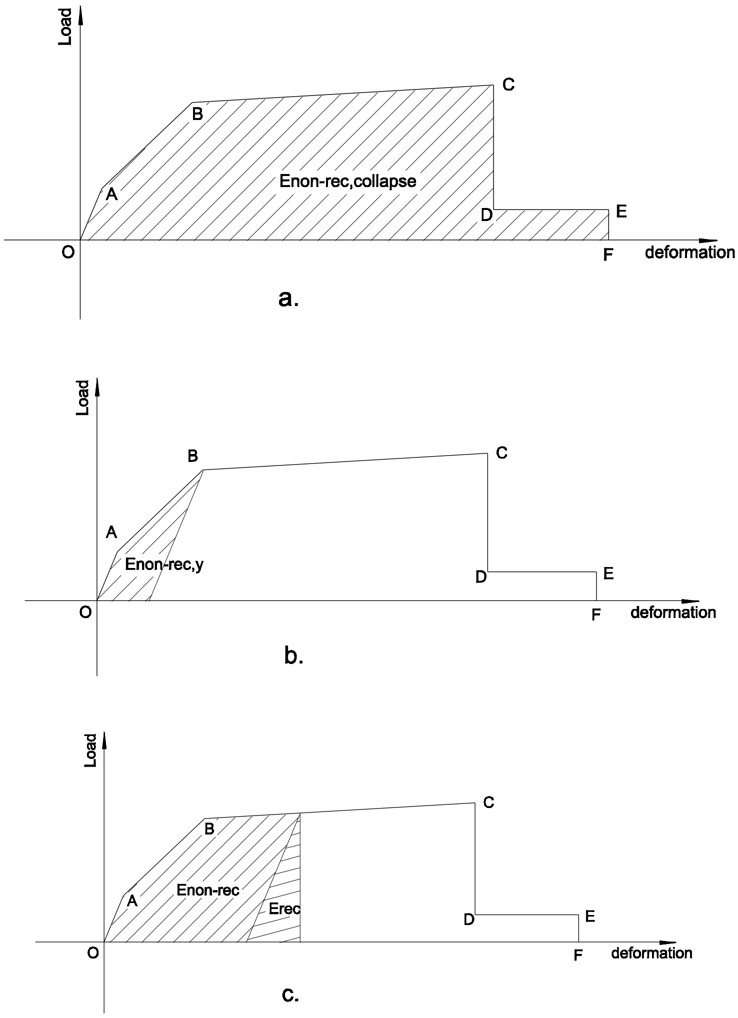


Fig. 4 Definition of energy parameters a. $E_{non\text{-}rec,collapse}$, b. $E_{non\text{-}rec,y}$ and c. $E_{non\text{-}rec}$ and E_{rec}

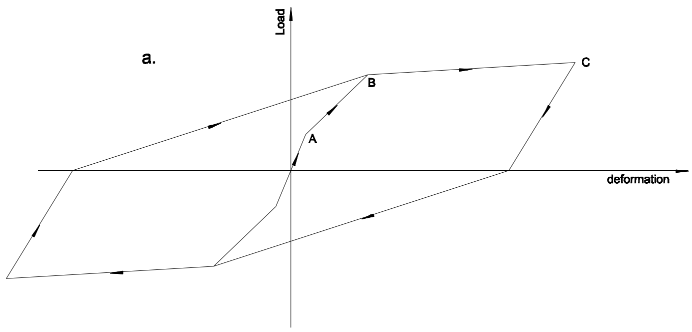


Fig. 5 A structure subjected to one complete cycle based on Takeda model (Takeda *et al*. 1970): a. one ultimate cycle; b. one yielding cycle

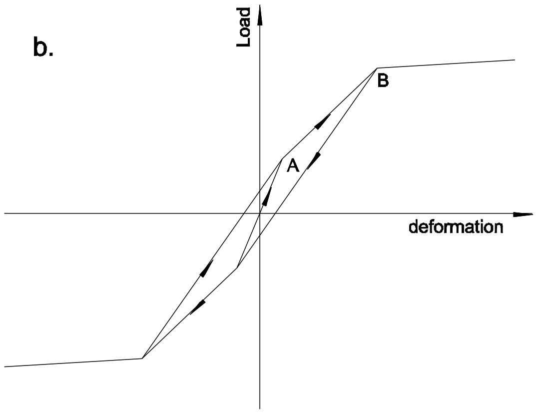


Fig. 5 Continued

3.3.2 DI for cyclic loading

The number of cycles has a significant influence on the extent of damage and this effect should be considered whilst devising an appropriate formulation for DI. The proposed Eqs. (22), (25) and (26) are modified as shown in Eqs. (27), (28) and (29):

$$DI = \left[\frac{E_h}{E_h + E_{rec}} \right]^{\alpha(N-i)} \tag{27}$$

$$N = \frac{E_{h,collapse}}{E_{h,1y}} = \frac{\gamma E_{h,1collapse}}{E_{h,1y}} \tag{28}$$

$$i = \frac{E_h}{E_{h,1y}} \tag{29}$$

where $E_{h,1collapse}$ and $E_{h,1y}$ are the hysteretic energy of one complete ultimate (Fig. 5a) and yielding cycle (Fig. 5b), respectively. Eq. (28) and (29) define the proposed parameters N and i. N is the equivalent number of yielding cycles to collapse whilst i is the equivalent number of yielding cycles at the current time of loading ($i \leq N$). γ is a factor that takes into account the difference between the theoretically defined $E_{h,1collapse}$ and the real $E_{h,\ collapse}$. This is due to different approaches to determine the energy absorbed by structures and the conditions that the structures undergo such as a large number of small amplitude cycles or a small number of large amplitude cycles (Teran-Gilmore and Jirsa 2005). For simplification, $\gamma = 1$ is used in this study. The use of $\gamma = 1$ may result in an underestimation or overestimation of $E_{h,\ collapse}$ for the above atypical loading conditions. More research is needed to quantify the factor γ. α is a modification factor which takes into account the effect of number of cycles. The sensitivity analysis of this factor is performed in Section 4.2.

4. Validation of the proposed DI

4.1 For static load cases

4.1.1 Beams and columns

Cantilever beams/columns with a cross-section of 400 mm × 400 mm have been analysed. The axial force for the beam is neglected while that of 0.2 $f_c^{'}A_g$ (Ghobarah and Said 2001), in which $f_c^{'}$ is concrete strength and A_g is the cross-sectional area, is adopted for columns in this study case. The total longitudinal reinforcement ratios are assumed to be 1% and 1.5%. Distance from the concrete surface to the centre of the longitudinal reinforcement is 50mm. The stirrup spacing is 100 mm. Behaviour of concrete is assumed to follow the modified the Kent and Park (1971) model made by Park *et al*. (1982) with f_c' = 30MPa, f_t' = $0.75\sqrt{f_c^{'}}$ MPa, ε_{cu} = 0.004 and the modulus of elasticity of concrete is taken as $E_c = 5000\sqrt{f_c^{'}}$ MPa. Simple model for steel (Park and Paulay

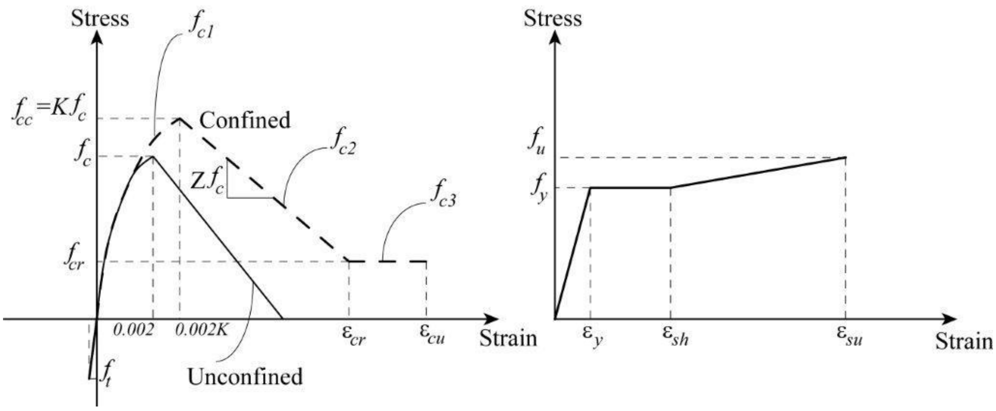
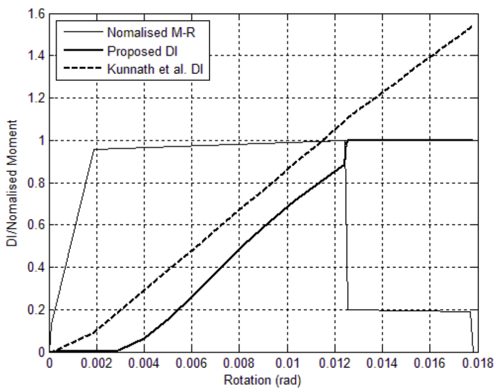
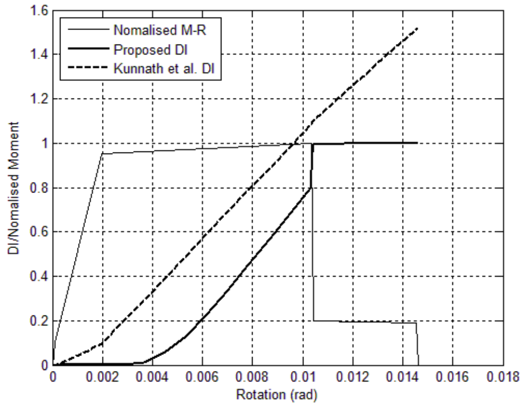


Fig. 6 Material models adopted for concrete and steel



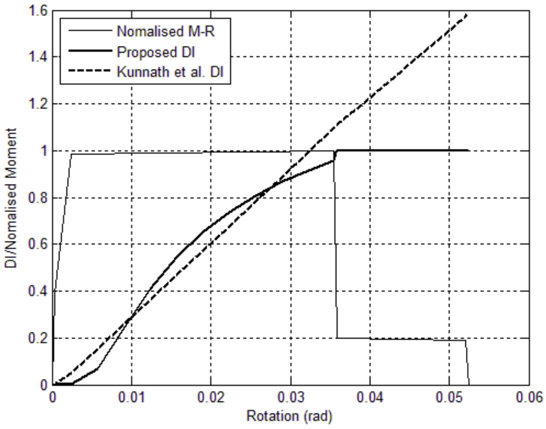
(a)

Fig. 7 Comparison of the proposed and the Kunnath *et al*. (1992) damage models for the considered cantilever beams: a) $\rho = \rho$'= 0.50%; b) $\rho = \rho$'= 0.75%

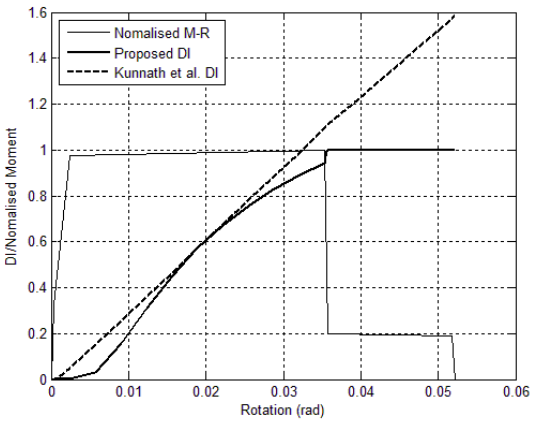


(b)

Fig. 7 Continued



(a)



(b)

Fig. 8 Comparison of the proposed and the Kunnath *et al*. (1992) damage models for the considered cantilever columns: a) $\rho = \rho' = 0.50\%$; b) $\rho = \rho' = 0.75\%$

1975) is used with E_s = 2×10^5 MPa, f_y = 500 MPa, f_u= 540 MPa, ε_{sh} = 0.015 and ε_{su} = 0.05. The 10mm stirrup reinforcement with f_y = 300 MPa is used. Fig. 6 shows the material models used in the considered study case.

The moment-curvature curves up to the ultimate compression stress of concrete are obtained using MATLAB based on fibre model, in which the cross section is discretised into many fibres. The strain distribution is assumed linear and the stress on each fibre is based on the material model, with the strain defined at the centroid of that fibre. The iterated loops of strain distribution will stop when the equilibrium conditions are achieved. Then, simple plastic hinge model with plastic hinge length L_p = 0.5d, as proposed by Paulay and Priestley (1992), is used to obtain rotations. FEMA 356 (ASCE 2000) guidelines are followed to obtain the post-ultimate behaviour leading to collapse. It is worth noting that the confinement effect is taken into account for the columns while it is ignored for the beams in this study case. Also, possible buckling of the longitudinal reinforcement is neglected.

Figs. 7 and 8 compare the proposed DIs with those calculated following the model proposed by Kunnath *et al*. (1992), which is a modified version of the model proposed by Park and Ang (1985), for cantilever beams and columns respectively. Normalised moment – rotation curves are also plotted. Both models give DI = 0 when the structure works in the elastic range (before crack). However, Kunnath *et al*. (1992) model produces values larger than 1 when approaching the collapse state, whilst the upper limit for the proposed model is 1. In addition, Kunnath *et al*. (1992) model produces much larger values for DI when the structures work around their yielding point. This consequently leads to the difference between the two models in the plastic range, especially for the beams.

4.1.2 Frames

The experiment performed by Mehrabi (1994) is revisited. The dimensions of columns and beam were 177.8 × 177.8 mm and 152.4 × 228.6 mm, respectively. The reinforcement arrangement is shown in Fig. 9. The compressive strength of concrete was f_c' = 30.89 MPa. The concrete strain at peak stress was ε_{cu} = 0.0018 and the modulus of elasticity was $E_c = 21926$ MPa. The properties of reinforcement are shown in Table 2. The axial force on each column was 293.582 kN. Further details of the frame can be found in the Ref. (Mehrabi 1994).

Each beam and column of the frame is discretised to a number of elements, on which the damage is assumed to be uniform in the element. The number of elements is selected so as to properly distinguish the regions of damage along the beam or column members. In this case, the number of element for each column and beam are chosen as 10 which is considered to be large enough for the above mentioned purpose. The lengths of each column and beam element are 153.67 and 231.14 mm respectively. Each element is modelled by non-linear Link element which is available in SAP 2000 (Computers and Structures Inc 2009). The moment-rotation is required for the non-linear Link property; hence, the moment-curvature obtained using MATLAB based on fibre model, is conversed to moment-rotation by multiplying the length of the element. Fig. 10 shows the model of the frame with non-linear Link elements.

Analysis of the frame subjected to monotonic lateral load is performed until the displacement reaches around 75 mm. Fig. 11 shows the analytical force-displacement relationship in comparison with the experimental result obtained by Mehrabi (1994). In spite of some differences, they quite match overall.

The proposed damage model is then applied to obtain the damage indices for the elements.

These damage indices distributed around the frame are plotted as shown in Fig. 12b in comparison to the damage pattern observed from the experiment performed by Mehrabi (1994) shown in Fig. 12a. It should be noted that the damage levels in Fig. 12b are referred to Table 1. The proposed damage model well quantifies the severe damage state of the frame although it cannot explain the cracks in the middle regions of the beam and columns. The more severe damage occuring in columns

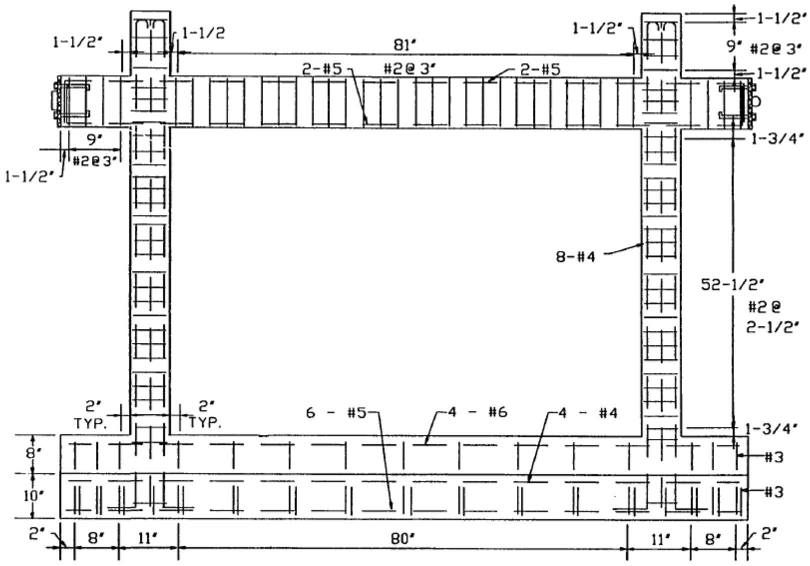


Fig. 9 Reinforcement arrangement (Mehrabi 1994)

Table 2 Properties of reinforcement

Bar size	Bar type	Diameter (mm)	Yield stress f_y (MPa)	Ultimate stress f_u (MPa)
#2	Plain	6.35	367.6	449.6
#4	Deformed	12.7	420.7	662.1
#5	Deformed	15.9	413.8	662.1

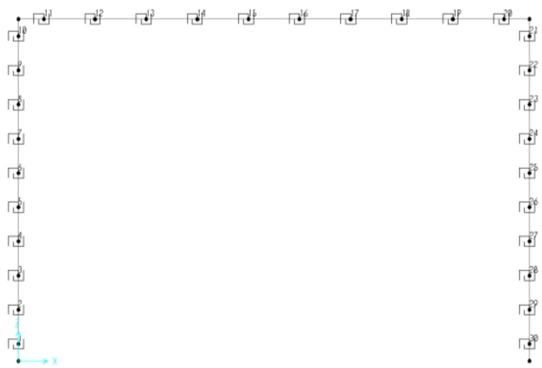


Fig. 10 Model with non-linear link elements

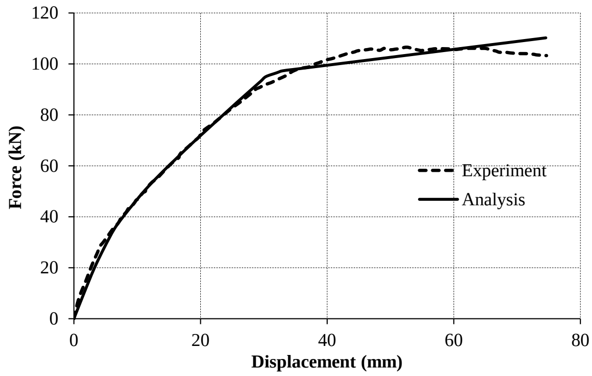
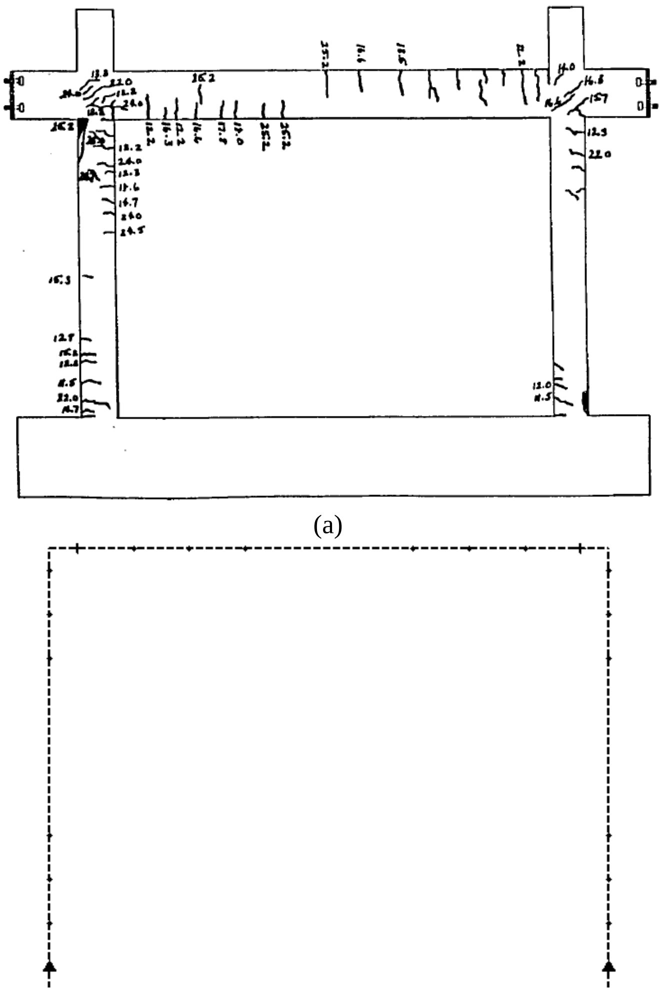


Fig. 11 Experimental (Mehrabi 1994) and analytical force-displacement relationship



(a)

(b)

Fig. 12 Damage of the specimen: a) Experiment (Mehrabi 1994); b) Damage analysis using the proposed damage index comparing to beams can be explained by the higher load and deformation of the columns. The damage of the columns in the bottom regions is more severe than that in the top. This can be explained by the rotation at the top of columns can be reduced due to the rotation from the beam; in addition, the bottoms of the columns are fixed.

4.2 For cyclic load cases

4.2.1 Columns

Results from experiments performed by Tanaka (1990) are employed in this study case. His specimens 1 to 4 with cross-sectional dimensions of 400 × 400 mm, and length L = 1600 mm are re-visited. The concrete strength was 25.6 MPa. Longitudinal reinforcement with total ratio (ρ + ρ') was 1.57% comprising six 20-mm-diameter bars with the yield strength f_y = 474 MPa. Transverse reinforcement was ϕ12 mm with the spacing of 80 mm and the yield strength of f_{yh} = 333 MPa. Axial load was 819 kN. The analytical static force-displacement relationship is computed based on fibre model. The simple plastic hinge length L_p = 0.5d proposed by Paulay and Priestley (1992) and FEMA 356 (ASCE 2000) guidelines are employed. The average value of 0.1 (Prakash and

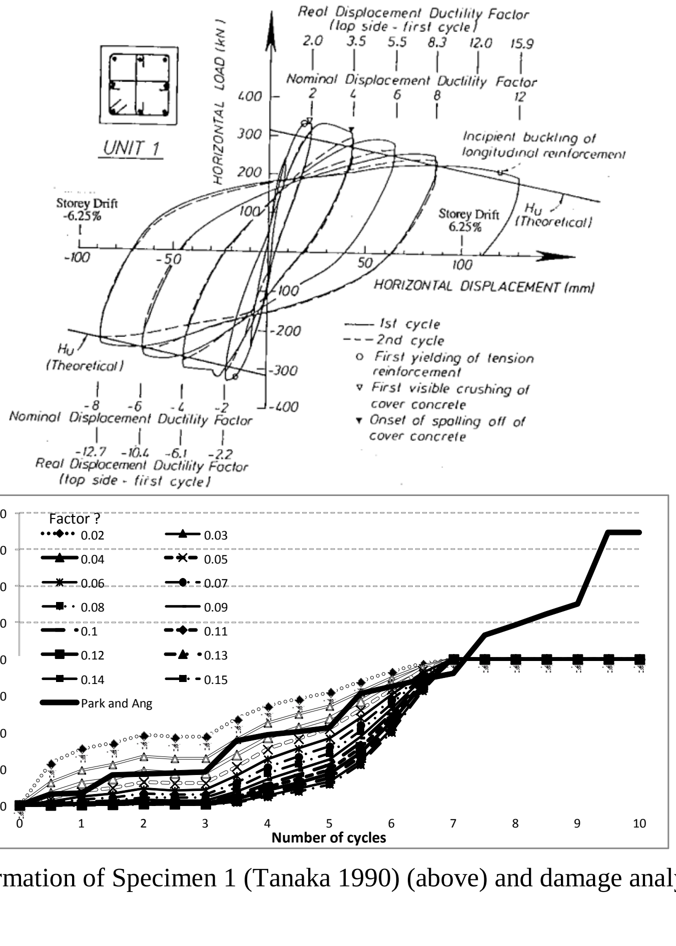


Fig. 13 Force-deformation of Specimen 1 (Tanaka 1990) (above) and damage analysis (bottom)

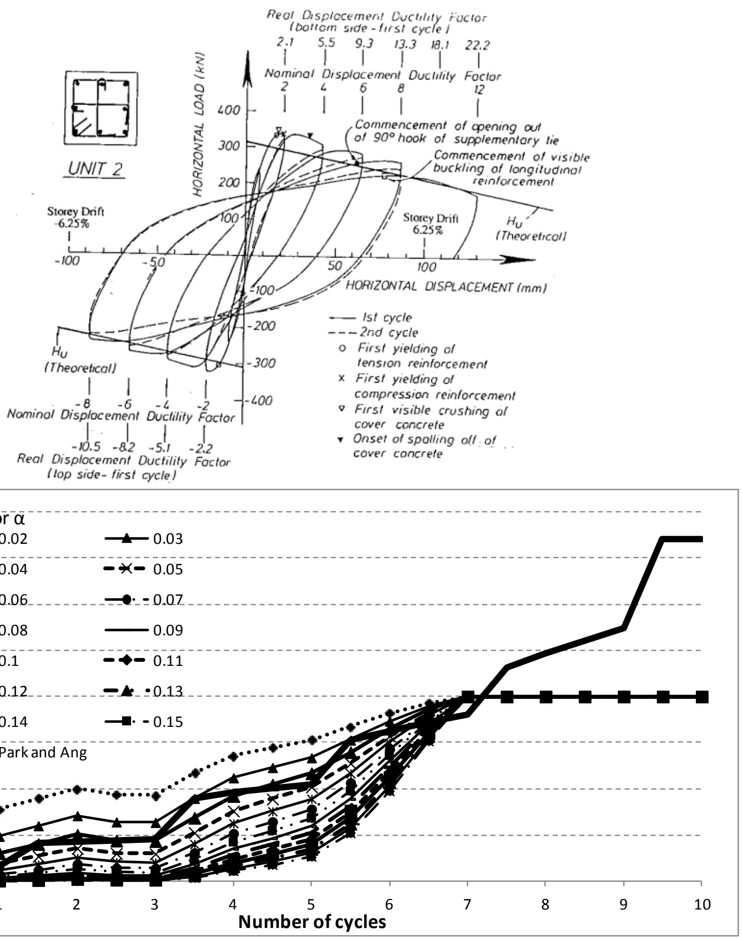


Fig. 14 Force-deformation of Specimen 2 (Tanaka 1990) (above) and damage analysis (bottom)

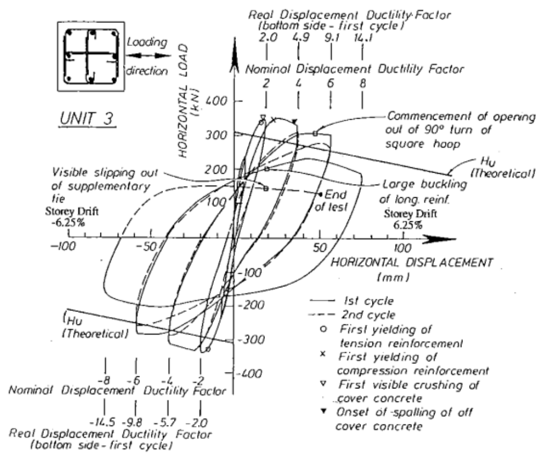


Fig. 15 Force-deformation of Specimen 3 (Tanaka 1990) (above) and damage analysis (bottom)

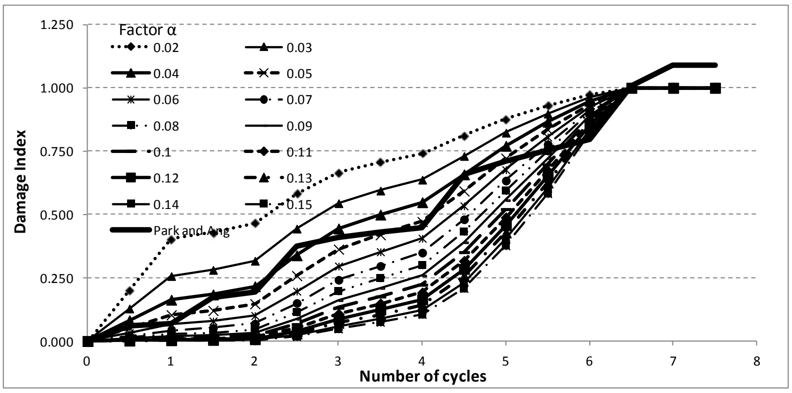


Fig. 15 Continued

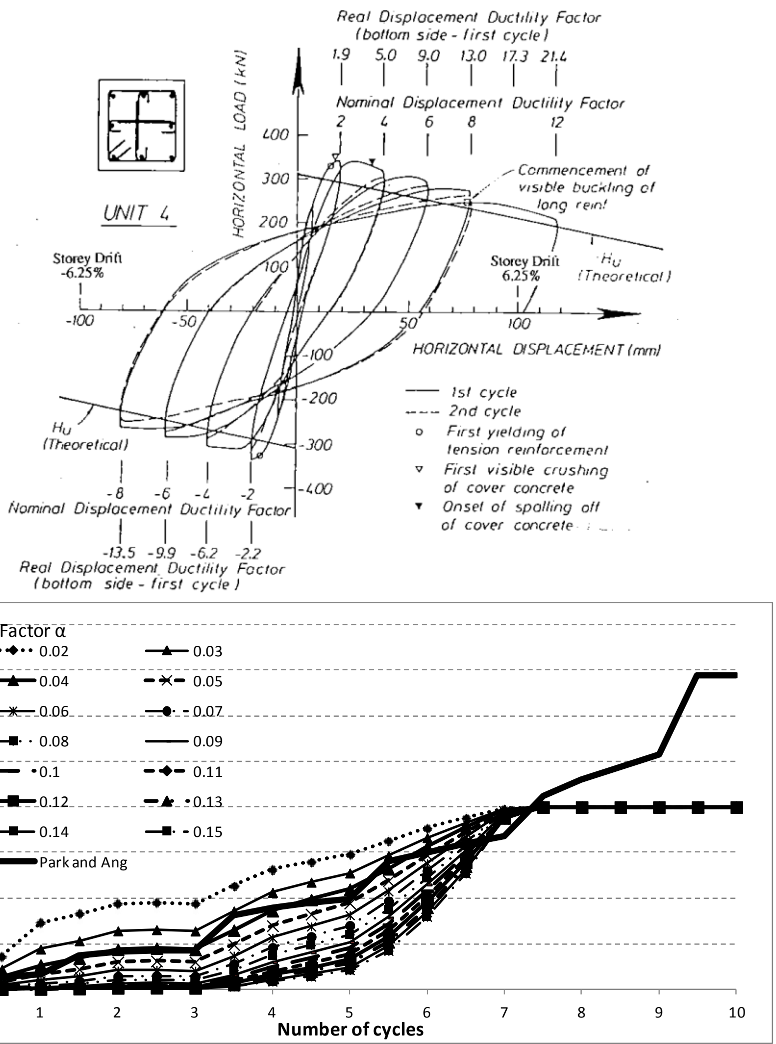


Fig. 16 Force-deformation of specimen 4 (Tanaka 1990) (above) and damage analysis (bottom)

Table 3 Damage indices at first yield of tension reinforcement

Specimen	Damage index			
	Proposed			Park and Ang
	α = 0.04	α = 0.05	α = 0.06	
1	0.181	0.118	0.077	0.205
2	0.201	0.134	0.090	0.203
3	0.184	0.120	0.079	0.170
4	0.171	0.110	0.071	0.188
Average	0.184	0.121	0.079	0.192

Table 4 Damage indices at the onset of spalling of concrete cover

Specimen	Damage index			
	Proposed			Park and Ang
	α = 0.04	α = 0.05	α = 0.06	
1	0.345	0.264	0.202	0.443
2	0.343	0.262	0.201	0.445
3	0.338	0.258	0.197	0.373
4	0.324	0.245	0.185	0.408
Average	0.337	0.257	0.196	0.417

Table 5 Properties of reinforcement

Reinforcement	Diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Young modulus (MPa)	Ultimatestrain
D4	5.715	468.86	503.34	214089.8	0.15
D5	6.401	262.01	372.33	214089.8	0.15
12 ga.	2.770	399.91	441.28	206160.5	0.13
11 ga.	3.048	386.12	482.65	205471	0.13

Belarbi 2010) is used for factor β in Park and Ang model while the factor α of the proposed model is changed from 0.02 to 0.15 to obtain damage indices which cover Park and Ang indices. This later suggests a reasonable value for the factor α. Figs. 13 to 16 show the sensitivity of the proposed damage index with the modification factor α in comparison with Park and Ang model based on the experimental results obtained by Tanaka (1990).

Figs. 13 to 16 show that the two models demonstrate a good agreement at a particular value of α around 0.04. This indicates that the proposed model with this modification factor can produce the damage indices similar to the Park and Ang model. However, the reliability associated with the Park and Ang model itself is an issue. For example, the collapse probability with DI = 1, computed according to Park and Ang's model, is around 50% with a standard deviation of 0.54 (Johnson *et al*. 2009). Hence, values of 0.04-0.06 are proposed instead for the modification factor α. Tables 3 and 4 show the damage indices obtained using the proposed and Park and Ang models for the states of first yield of steel and the onset of spalling of the concrete cover respectively. These damage states are shown on the experimental force-deformation curves in Figs. 13 to 16. The average damage indices at first yield of steel are 0.184 (α = 0.04), 0.121 (α = 0.05) and 0.079 (α = 0.06). Either of them shows the light damage state based on the damage levels shown in Table 1. This agrees with the average Park and Ang damage index of 0.192 representing damage with light cracking throughout.

Similarly, the average damage indices at the onset of spalling of cover concrete are 0.337 (α = 0.04), 0.257 (α = 0.05) and 0.196 (α = 0.06). The first two values indicate the moderate damage state, while the third value shows light damage. The average Park and Ang damage index of 0.417 indicated the severe damage which was described as concrete crushing and reinforcement exposed (Park and Ang, 1985) seems to be overestimated because this is the onset of the spalling of the concrete corresponding to the moderate damage level defined by Park and Ang (1985). Inspite of lower values of DI resulting from using α = 0.06 in comparison to using others, the proposed damage indices increase with a faster rate after the onset of the spalling as observed in the Figs. 13 to 16. This value of 0.06 is proposed for α and it demonstrates well for the frame in Section 4.2.2.

4.2.2 Frames

The one-third scale three-storey RC frame designed only for gravity load shown in Fig. 17 was tested by Bracci (1992). Its dimensions (in inches) and reinforcement details are shown Fig. 18. The average elastic modulus E_c of concrete was 24200 MPa and the average strength f_c' of 27.2 MPa (varying from 20.2 to 34.2 MPa). Table 5 shows four types of reinforcement and their properties used for the frame.

The total weight of each floor was found to be approximately 120 kN, which included the self-weight of beams, columns, slabs and additional weights attached to the model as can be seen in Fig. 17. Further details of the model can be found in Bracci (1992) and Bracci *et al*. (1995). The N21E ground acceleration component of the Taft earthquake, which occurred on 21 July 1952 at the Lincoln School Tunnel site in California, was used for the experiment. Minor, moderate and severe shaking were represented by different peak ground accelerations (PGA) of 0.05g, 0.20g and 0.30g, respectively.

Table 6 shows the axial loads in columns which are assumed to be constant during earthquakes. The moment-curvature curves up to the ultimate compression stress of concrete were obtained using a fibre model. FEMA 356 (ASCE 2000) guidelines were followed to obtain the post-ultimate behaviour leading to collapse. Moment-rotation curves used for the nonlinear analyses were then obtained using the plastic hinge technique. The plastic hinge length $L_p = d$ proposed by Sheikh and Khoury(1993) and based on the observation from the experimental damage of the frame was adopted in this study case. The nonlinear Link element following hysteretic Takeda model (Takeda *et al*. 1970) in SAP2000 was employed to model the structure as shown in Fig. 19. Table 7 shows the first structural frequencies by comparison with those from the experiment. The first and second modes demonstrate a good match but there is a little difference in the third mode.

Table 8 presents the maximum inter-storey drifts and maximum storey displacements obtained from the model in comparison with those from the experiment (Bracci *et al*. 1995). Though not an exact match, an overall good approximation is demonstrated by the model.

The proposed damage model is employed to identify, locate and quantify the damage imparted to the structure during the earthquakes. Figs. 20a, 21a and 22a present the experimental damage

Fig. 17 Model of three storey frame (Bracci *et al*. 1995)

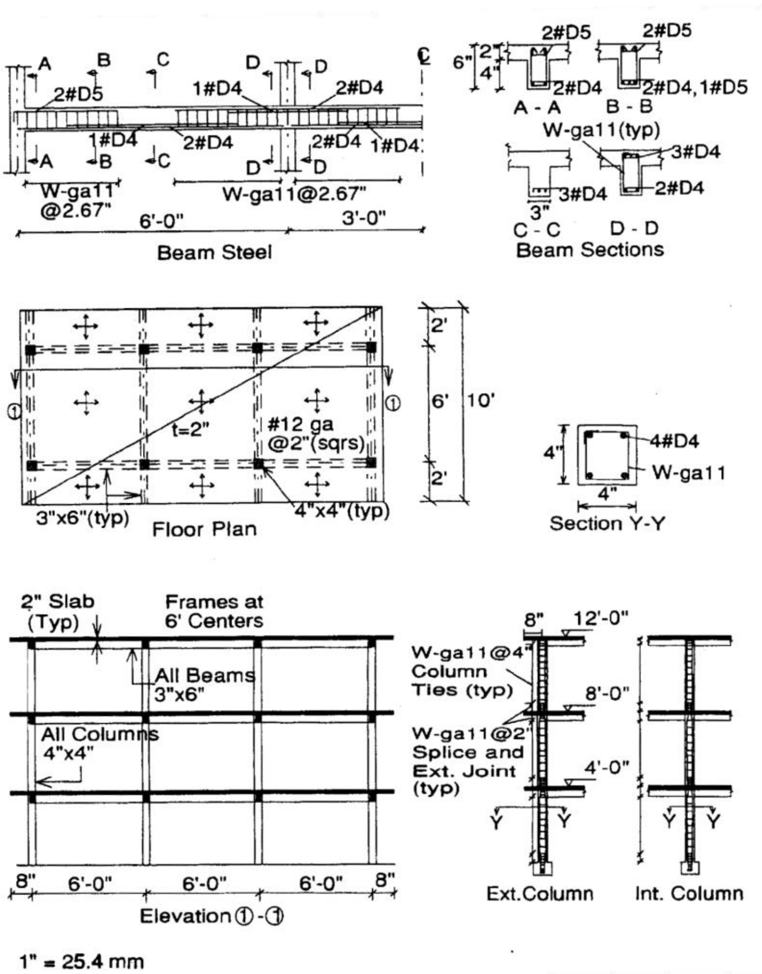


Fig. 18 Dimensions and reinforcement arrangement of three storey frame model (Bracci *et al*. 1995)

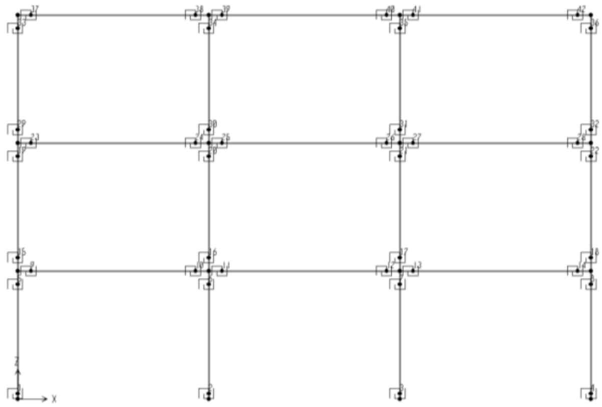
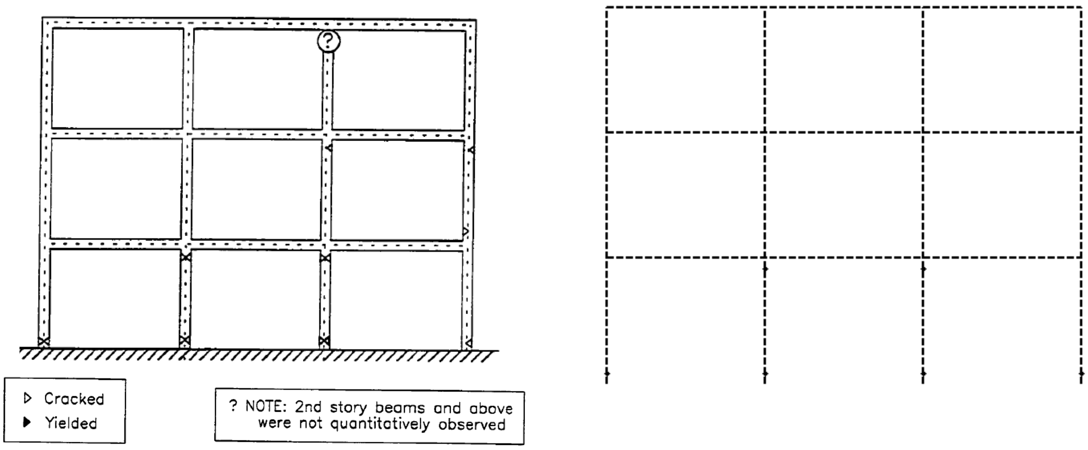
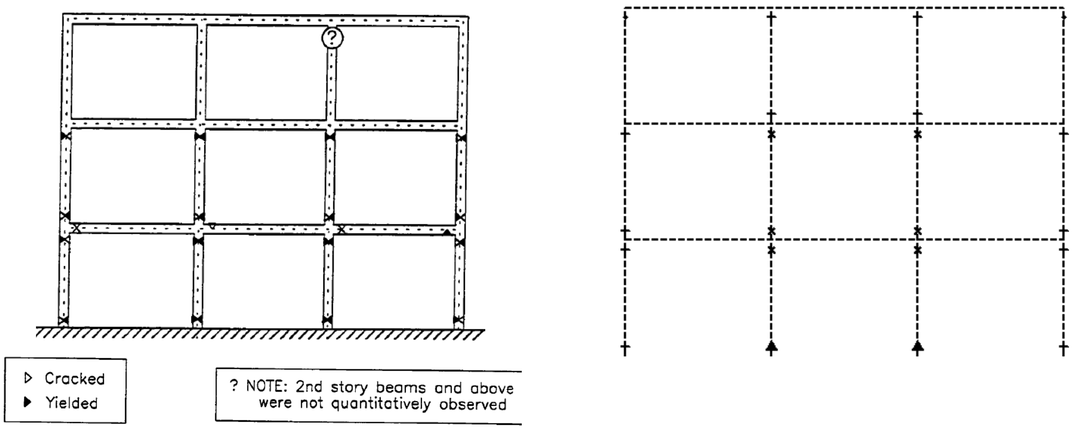


Fig. 19 Modelling of the three-storey frame with nonlinear LINK elements



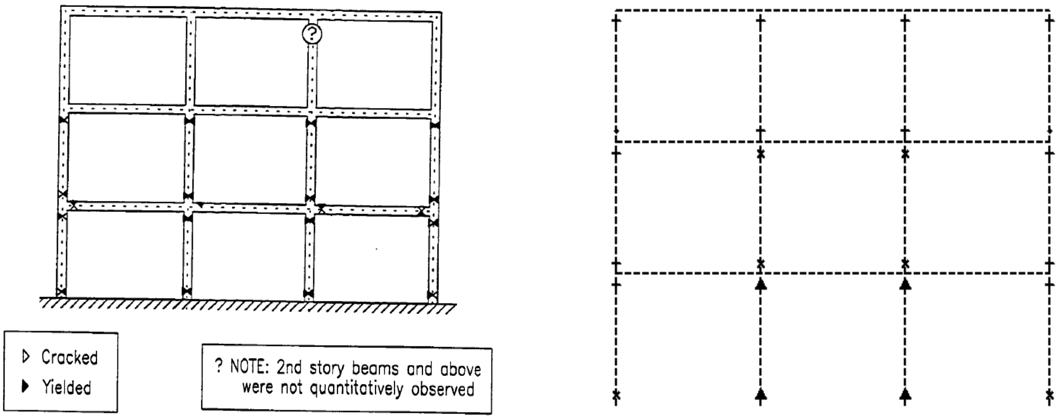
(a) (b)

Fig. 20 Modelling of the three-storey frame with nonlinear LINK elements



(a) (b)

Fig. 21 Damage analysis – Taft 0.20g: a) Experiment (Bracci 1992); b) Analytical damage state



(a) (b)

Fig. 22 Damage analysis – Taft 0.30g: a) Experiment (Bracci 1992); b) Analytical damage state

Table 6 Axial load in columns

Storey	Axial load (kN)	
	External column	Internal column
1	30	60
2	20	40
3	10	20

Table 7 Modal frequencies (Hz)

Mode	Experiment (Bracci *et al.* 1995)	Model
1	1.78	1.70
2	5.32	5.30
3	7.89	9.03

Table 8 Comparison between experimental (Bracci *et al.* 1995) and analytical results

PGA	Storey	Maximum inter-storey drift (%)		Maximum storey displacement (mm)	
		Experiment	Model	Experiment	Model
0.05g	3	0.23	0.21	7.6	7.9
	2	0.24	0.25	5.6	5.6
	1	0.28	0.23	3.6	2.8
0.20g	3	0.54	0.83	33.5	38.9
	2	1.07	1.17	29.0	30.7
	1	1.33	1.31	16.3	16.0
0.3g	3	0.89	1.18	59.7	58.4
	2	2.24	1.91	52.1	46.1
	1	2.03	1.96	24.6	23.9

states obtained from (Bracci 1992) while Figs. 20b, 21b and 22b show the analytical damage states for Taft PGAs of 0.05g, 0.20g and 0.30g, respectively. It should be noted that the analytical damage states are plotted for different damage index levels as shown in Table 1. The damage indices less than 0.005, which can be ignored, are not shown in these Figs. The damage states obtained from analyses are overall close to those obtained from experiment.

5. Conclusions

A detailed review of available concepts for DI has been presented in this paper with brief details on their relative merits and drawbacks. Literature review identified two important characteristics for a DI - the effects of cyclic loading on structural damage should be incorporated in an appropriate way and the proposed model should vary between 0 and 1. The most widely used DI model was proposed by Park and Ang (1985) considering changes in both deformation and energy during an earthquake, which was later modified by Kunnath *et al*. (1992). However, this model does not specify any upper limit for DI, and, therefore, produces erratic results when a structure approaches collapse.

The formulation of a new concept for damage index based on residual deformation has been presented in the current paper. The proposed model has been developed both for static and cyclic loading and it takes into account the whole response of structures. In addition to including the effects of cyclic loading, the proposed DI satisfies the essential characteristics for an appropriate damage model and produces rational values of damage indices. It also shows a good agreement with the Park and Ang (1985) model at a specific value of modification factor α. A damage classification was proposed following the performance levels defined in FEMA 356 (ASCE, 2000). A number of case studies have demonstrated rational performances of the proposed model as a useful tool for design and assessment of RC structures. Although the model demonstrated to work properly for the studied cases, more research is encouraged to investigate the hysteretic energy at collapse or the factor γ and to more accurately define the damage levels for wider range of axial loads and reinforcement ratios.

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