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Seismic Damage Assessment of Reinforced Concrete Structure using Non-linear Static Analyses

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Abstract

In this paper, stiffness based damage index (DI_c) is introduced and expressed as a simple formula based on nonlinear response got from nonlinear static procedures. It is useful because only once the pushover has to be performed for the inertia loads obtained from equivalent static method given in BIS 1893, to show the degree of damage of structure in question. It is employed to the damage assessment of example RC frames representing different structures. To extend DI_c for different performance levels defined in FEMA 356, the damage values are related to drift based damage index. Results show that DI_c agrees with drift damage values and is valuable tool for practical applications.

Keywords: *performance based seismic engineering, damage indices, damage function, FEMA performance levels, pushover analyses*

1. Introduction

Design and construction in India is generally regulated at the state or local level using codes provided by the bureau of Indian standards. When adopted and enforced by local authorities, building codes are intended to establish minimum requirements for providing safety to life and property from fire and other hazards. These seismic design codes allow designing a reinforced concrete structure which can experience the repairable damages during minor and moderate earthquakes. During strong earthquakes, these structures have experienced irreparable damages or they were collapsing. With an aim to communicate the safety-related decisions, design practice has been moved towards the predictive method of assessing potential seismic performance, known as Performance Based Seismic Design (PBSD). PBSD refers to the methodology in which structural design criteria's are expressed in terms of achieving a set of performance objectives. Performance objectives are statements of the acceptable risk of incurring different levels of damage and the consequential losses that occur as a result of the damage (Ghobarah, 2000; FEMA 445, 2006).

The prediction of the amount of damage that a structure is likely to sustain in its design life is a probabilistic problem, However many researchers had put forth various deterministic approaches of damage assessment involving different engineering demand parameters (EDPs) and they proved to be a valuable tool. These EDPs includes, stress, strain, displacement, curvature, deformation, base shear, strength, stiffness and dissipated energy. A damage index represents a phenomenon of damage involving different combinations of these EDPs. In the literature, two types of damage assessment procedures appear, the first procedure is based on the balance between some demand on the structure and the corresponding capacity of the structure, and the second procedure is based on the degradation of some structural property (Powell and Allahabadi, 1988).

These EDPs can be computed by linear structural analyses, but the more rational approach is the use of nonlinear structural analyses. The performance evaluation procedures appearing in PBSD documents are able to perform nonlinear static and dynamic analyses. All such procedures have been used in research, but have not found wide use in the formal damage assessment. Evaluating a nonlinear dynamic response is tedious job and involves more calculations. The nonlinear static procedure involves ease and less time-consuming process, has become common in practice. PBSD had strong favor to the nonlinear static analyses and provide the various performance evaluation procedure based on it (Ghobarah *et al*., 1999).

In this paper a damage index is introduced using nonlinear responses obtained from the output of the nonlinear static analyses (displacement-controlled) performed on RC frames. Damage value is obtained in respect to the degradation in stiffness. This paper attempts to correlate these damage value with various performance levels defined in PBSD.

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2. Damage Index

The concept of damage and damage-ability in a structural design is of high importance. The prediction of the amount of seismic damage that a RC structure is likely to sustain during its design life is a probabilistic problem. Many researchers had provided deterministic approach, which had proved to be a valuable tool. These deterministic approaches involve the calibration of damage indicators using the analytical and experimental data for the computation of a damage value. (Powell and Allahabadi, 1988). The practical situations where the damage indicators can be employed are; (a) post-earthquake damage assessment, (b) reliability studies of existing structures, and (c) seismic performance of novel types of structures (Kappos, 1997).

Damages to a reinforced concrete structures are generally related to the failure of its component members, which occurs because of the crushing of concrete. This phenomenon initiates with the spalling of concrete cover and later of the crushing of confined concrete core, which are not easy to define even under predominant flexural conditions. The criteria's used to define the state of failure are illustrated in Fig. 1 (Kappos, 1997).

The criteria's used to define the state of failure of RC member is related to some EDPs. A damage is a combination of EDPs representing different phenomenon of failure resulting a non-dimensional value known as Damage Index (DI) which ranges between 0 (undamaged state) to 1 (damaged state) (Kappos, 1997).

There are two basic procedures which are employed to compute a damage index, the first is based on demand versus capacity, which involves the estimation of some demand on a structure, substructure or member and estimation of the corresponding capacity (supply) and the second is based on estimation of property for a structure, substructure, or member in its undamaged state and a corresponding estimation in its damage state. The possible choice of EDPs are, displacement, deformation, stiffness, strength, and energy dissipation capacity (Powell and Allahabadi, 1988).

The damage quantification appearing in literature can be broadly categorized into (i) empirical DIs, and (ii) analytical DIs. (Williams and Sexsmith, 1995). Empirical and analytical approaches have been used to yield various structural damage. The empirical damage models are based on statistics of observed structural damage following a seismic event. Empirical systems cannot predict the reserve strength and response characteristics of a structure for a specified degree of damage because, i) the systems do not comply with the mechanics of materials with respect to inelastic cyclic deformation; ii) future earthquakes may have different intensities, duration, and frequency content; iii) a present code modification according to the post-earthquake experiences may change damage statistics; and iv) the shifted dynamic characteristics of structures due to repairs and damages resulting from past earthquakes (Ghobarah *et al.*, 1999).

The analytical damage models may involve various degrees of complexities caused by the characteristics of the structure and the seismic response. Analytical damage models are broadly divided into structural parameter-based and vibration response-based DIs. The structural parameter-based DIs depend on the geometry of structural elements, such as column and wall area and their general material properties. In the absence of the field observation of damaged structures due to seismic loads, calibration is performed using nonlinear dynamic analyses. Vibration response-based indices use the structural response measurements for a single excitation event and calculate damage-related physical factors, such as peak acceleration, peak velocity, etc. (Wang *et al.*, 2007). An updated review of all available DIs in related literature is presented in Table 1.

The development of damage models began with the development of the first damage model, which was based on the ductility concept. Ductility-based damage models were expressed as a function of member rotation, curvature, and characteristic displacement, as defined in Eqs. (1)-(3). Because of being easy to quantify, these indices are used to assess the performance but fail to account for the effect of strength and stiffness degradation under the cyclic loads. The need for structural safety under cyclic loading against plastic incursions leads to the development of DIs expressed in terms of kinematic or cyclic ductility as measures of collapse (Eqs. (4)-(6)). The kinematic or cyclic ductility DI faced difficulties related to the difference between the characteristics of expected earthquake and earthquake used in their calibration (such as intensity, duration, and frequency content), but are strongly supported because of their prediction that is typically similar to ductility DIs (Williams *et al.*, 1999).

During cyclic loading, energy dissipates in the structure accounting

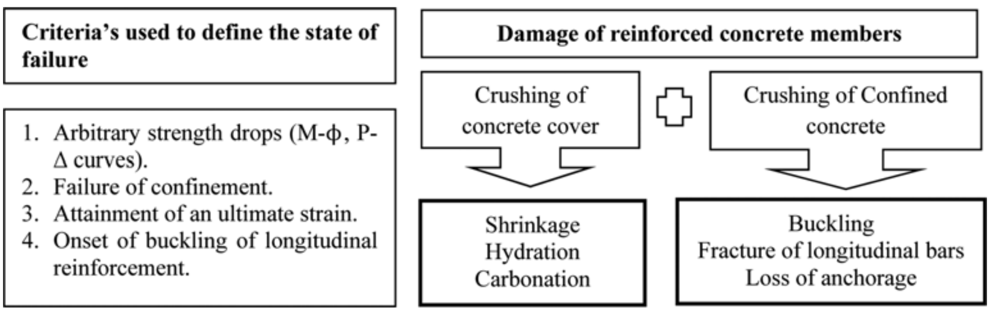

Fig. 1. Identification of the Degree of Damage in RC Members (Kappos, 1997)

Table 1. Summary of Available DIs with Their Parameter Values

Damage Index	Description		Formulation
A. Vibration response-based DIs			
1. Local DIs			
Newmark and Rosenblueth (1971)	DI is defined in terms of ductility factor expressed as a function of rotation (θ)	$\mu_r(\theta) = \frac{\theta_m}{\theta_y} = 1 + \frac{\theta_m - \theta_y}{\theta_y}$	(1)
Banon *et al.* (1981)	DI is defined in terms of ductility factor expressed as a function of curvature (ϕ)	$\mu_r(\phi) = \frac{\phi_m}{\phi_y} = 1 + \frac{\phi_m - \phi_y}{\phi_y}$	(2)
Park (1986)	DI is defined in terms of ductility factor expressed as a function of characteristic member displacements (δ)	$\mu_r(\delta) = \frac{\delta_m}{\delta_y} = 1 + \frac{\delta_m - \delta_y}{\delta_y}$	(3)
Lybas and sozen (1977)	DI is defined as the ratio of initial stiffness to maximum elastic stiffness.	$DI = \frac{K_o}{K_m}$	(4)
Banon *et al.* (1981)	Flexural Damage Ratio (FDR) is defined in terms of stiffness degradation.	$DI = \frac{M_u \phi_m}{M_m \phi_u}$	(5)
Roufaiel and Meyer (1987)	Modified FDR (MFDR) is defined in terms of increment in flexibility before and after a failure	$DI = \dfrac{\frac{\theta_m}{M_m} - \frac{\theta_y}{M_y}}{\frac{\theta_u}{M_u} - \frac{\theta_y}{M_y}}$	(6)
2. Cumulative DIs			
Banon and veneziano (1982)	Based on the normalized cumulative rotation	$DI = \dfrac{\sum_{i=1}^{n} \phi_{im} - \phi_y}{\phi_u}$	(7)
Stephens and Yao (1987)	Based on the cumulative displacement ductility	$DI = \sum_{i=1}^{n} \left[\frac{\Delta d^{+}}{\Delta d_f}\right]^{1-br}$ b = 0.77 (recommended)	(8)
Jeang and Iwan (1988)	Force-based DI accounting the effects of combining cycles with various amplitudes	$DI = \sum_{i=1}^{n} \left[\frac{n_i \mu_i^s}{C}\right]$	(9)
3. Combined DIs			
Banon and veneziano (1982)	DI is expressed as a linear combination of maximum displacement, failure displacements, and hysteretic energy dissipation	$DI = \sqrt{\left(\left[\frac{d_m}{d_y - 1}\right]^2 + \left[\frac{2E_h}{F_y d_y}\right]^{0.38}\right)^2}$	(10)
Park and Ang (1985)	DI is expressed as linear combination of maximum plastic displacement and plastic dissipated energy	$DI = \frac{d_m}{d_u} + \beta_e \frac{\int dE}{F_y d_u}$	(11)
Niu and Ren (1996)	Similar to Park?Ang DI but formulated with different constants	$DI = \frac{\theta_m}{\theta_u} + \alpha\left(\frac{E}{E_u}\right)^{\beta}$	(12)
4. Global DI			
Roufaiel and Mayer (1987)	Strength-based global DIs	$DI = GDP\frac{d_m - d_y}{d_u - d_y}$	(13)
Park, Ang, Wen (1985)	Hysteretic energy weighted average	$D_{storey} = \dfrac{\sum_{i=1}^{N} D_i E_i}{\sum_{i=1}^{N} E_i}$ $D_{global} = \dfrac{\sum_{storey,i=1}^{N} D_{storey,i} E_{storey,i}}{\sum_{storey,i=1}^{N} E_{storey,i}}$	(14)

Table 1. (continued)

Damage Index	Description		Formulation
Bracci (1989)	Gravity load weighted average	$D_{storey} = \dfrac{\sum_{i=1}^{N} W_i D_i^{b+1}}{\sum_{i=1}^{N} W_i D_i^{b}}$ $D_{global} = \dfrac{\sum_{storey,i=1}^{N} W_{storey,i} D_{storey,i}^{b+1}}{\sum_{storey,i=1}^{N} W_{storey,i} D_{storey,i}^{b}}$	(15)
B. Strength parameter-based DIs			
Allemag and Brown (1982)	Modal Assurance Criterion (MAC) Is applied to correlate the two sets of mode shape, that is, damaged and undamaged. The value ranges from 0 to 1. When two sets fit each other, the value is closer to 1, and value 0 implies no correlation.	$MAC(\phi_{ut}, \phi_{dt}) = \dfrac{\lvert \phi_{ut}^T \phi_{dt} \rvert^2}{(\phi_{ut}^T \phi_{ut})(\phi_{dt}^T \phi_{dt})}$	(16)
Lieven and Ewins (1988)	Coordinate MAC (COMAC) Examining the changes in mode shape caused by damage is a better approach for combining data of various modes to obtain a single parameter	$COMAC_j = \dfrac{[\sum_n (\phi_{nj})_d (\phi_{nj})_u]^2}{[\sum_n (\phi_{nj})_d^2 \sum_n (\phi_{nj})_u^2]}$	(17)
Pandey *et al.* (1991)	Modal Flexibility DI (MFDI) This method involves comparison of the flexibility matrices obtained from the two sets of mode shapes	$MFDI = 1 - \dfrac{\sum_{i=1}^{N} \dfrac{\phi_{ij}^2}{\omega_j^{*2}}}{\sum_{i=1}^{N} \dfrac{\phi_{ij}^{*2}}{\omega_j^{*2}}}$	(18)
Pandey and Biswas (1994)	Storey DI (SDI) This index represents a percentage reduction in storey stiffness before and after damage (expressed in terms of floor mass, modal frequency, and mode shape). The value 0 implies no damage, and 1 indicates collapse.	$SDI = 1 - \dfrac{\omega_j^{*2} \sum_{i=1}^{N} \dfrac{m_i \phi_{ij}^*}{\Delta \phi_{ij}^*}}{\omega_j^2 \sum_{i=1}^{N} \dfrac{m_i \phi_{ij}}{\Delta \phi_{ij}}}$	(19)
Wang *et al.* (2007)	Approximate SDI (ASDI) For most buildings, the floor mass distribution is generally uniform; thus, the approximate value of the SDI can be represented as ASDI	$ASDI = 1 - \dfrac{\omega_j^{*2} \sum_{i=1}^{N} \dfrac{\phi_{ij}^*}{\Delta \phi_{ij}^*}}{\omega_j^2 \sum_{i=1}^{N} \dfrac{\phi_{ij}}{\Delta \phi_{ij}}}$	(20)
Ghobarah *et al.* (1999)	Stiffness Damage Index (DI_k) This index represents the change in the stiffness of a structure by performing pushover analyses on the structure twice: one before subjecting the structure to the earthquake, and one after subjecting the structure to ground motion	$(DI)_K = 1 - \left(\dfrac{K_{final}}{K_{initial}}\right)$	(21)

only for ductility, which is not an accurate measure of damage. This concept results in the development of cumulative DIs, which are expressed as a function of plastic deformation and absorbed hysteretic energy (Eqs. (7)-(9)). Cumulative DIs have proven to be a simple measure for structural degradation during a seismic event but were found dependent on the duration and intensity of an earthquake, and they failed to represent the complex behavior of concrete (Sinha and Shiradhonkar, 2012; Mihaiţă, 2013).

The concept of assessing damage according to the combined effects of strength, ductility, and energy dissipation leads to the development of combined DIs, as defined in Eqs. (10)-(12). Amongst all the cumulative DIs, the Park–Ang DI is widely supported by scientists as it was found to be consistent with the observed damage statistics for both concrete and steel structures. DIs defined in Eq. (13)-(15) are used to measure the damage to the entire structure and its characteristics. They inform about the global damage expressed as a function of the distribution and severity of local damage (Sinha and Shiradhonkar, 2012; Mihaiţă, 2013).

DIs defined in Eqs. (16)-(20) are based on the modal frequency, mode shape, or both. The damage is always accompanied by reduction of stiffness and modal frequency; however, determining the damage location only by observing the changes of modal frequencies is extremely difficult. DIs, which account for changes in mode shape, were used but were found to have low sensitivity to damage. Later DIs, which considered both modal frequencies and mode shapes, to detect the occurrence and location of damage were proposed. These DIs involved a tedious process of evaluating flexibility or stiffness matrices for every incremental time step. These damage models which uses the physical, measurable parameter needs knowledge of non-linear dynamic response of the structure during an earthquake which is a tedious job to be followed.

Next, a simple method of comparing the changes in stiffness before and after an earthquake was suggested by Ghobarah *et al.* (1999) (refer Eq. (21)). This DI was found to be an easy method of quantification, as it does not need any dynamic analyses to be performed. But it had some limitations as; (1) it does not address the cumulative effect, and (2) for a moderate damage and collapse stage the damage value exceeds the 1. In this paper a modification is proposed to Ghorbarah *et al.*, DI by adding some cumulative parameters and extended to evaluate damage value at various performance levels.

3. Damage Functions

The increase in natural periods of a building during an earthquake indicates that there has been a damage to the structure as a loss of stiffness. One measure of stiffness degradation is period lengthening and other can be obtained from direct computation. To calculate stiffness parameter and stiffness ratio it needs to perform an inelastic dynamic analyses or a static cyclic analyses of a structure involving more time and computational efforts (Powell and Allahabadi, 1988). To overcome these analytical difficulties, Ghobarah *et al.* (1999) had given a new approach for determining the change in stiffness of the structure. The approach is to perform pushover analyses of the structure twice; once before subjecting the structure to the earthquake and once after subjecting to the ground motion. The stiffness damage index $(DI)_k$ of the whole frame is calculated as;

$$DI_k = 1 - \frac{K_{final}}{K_{initial}}$$

Where K_{inital} represents the initial slope of the base shear-top deflection relationship resulting from pushover analyses of the frame before subjecting it to the earthquake ground motion and K_{final} is the initial slope of the same relationship but after subjecting the frame to the earthquake (time history). The stiffness damage index was advantageous for its concise procedure and no need to perform any dynamic analyses in its evaluation, but it had some limitations. The damage value was found to be inconsistent for ductile structures and to account the cumulative effect.

In the present work, to eliminate the limitations of the stiffness damage index, a cumulative effect has been introduced by using the same relation as defined in Eq. (22), rewritten as;

$$DI_c = 1 - \frac{K_c}{K_0} \tag{22}$$

Where DI_c is damage at collapse, K_c is stiffness at collapse and K_o is stiffness at operational level, all these parameters are measured for a single pushover conducted on the structure.

The pushover curve represents the degradation of structures capacity for each increment in displacement which in turn shows that the stiffness degradation also follows the same path. From this it can be concluded that damage estimated using Eq. (22), is for the first yield of the structure. Whereas the structure still possess the reserve strength which was not utilized, such a quantification may lead towards misjudgment of the actual behavior of structures. To overcome the above limitations, a new concept has been put forth which includes;

$$(1 - DI_c)K_0 = K_c$$
$$(1 - DI_c)K_0 d_c = V_c$$

For and incremental steps of pushover the equation may be written as (refer Fig. 2);

$$K_0 d_1 = V_1$$
$$K_1(d_2 - d_1) = V_2$$
$$K_2(d_3 - d_2) = V_3$$
$$K_n(d_c - d_n) = V_c$$

$$\sum_{i=0}^{n=1} K_0 d_1 + K_1(d_2 - d_1) + K_2(d_3 - d_2) + K_n(d_c - d_n) = V_c$$

$$DI_c = 1 - \frac{V_c}{K_0 d_c} = 1 - \frac{\sum_{i=0}^{n=1} K_0 d_1 + K_1(d_2 - d_1) + K_2(d_3 - d_2) + \ldots + K_n(d_c - d_n)}{K_0 d_c} \tag{23}$$

Where K_0 represents the stiffness at operational level, d_c is the current displacement, d_n is the displacement at any nth point, V_c is the current force.

When extended to various performance level the DI values are;

For immediate occupancy level,

$$DI_{IO} = 1 - \frac{\sum_{i=0}^{n=1} K_0 d_1 + K_1(d_2 - d_1) + K_2(d_3 - d_2) + \ldots + (d_{IO} - d_n)}{K_0 d_c} \tag{24}$$

For life safety range,

$$DI_{LS} = 1 - \frac{\sum_{i=0}^{n=1} K_0 d_1 + K_1(d_2 - d_1) + K_2(d_3 - d_2) + \ldots + K_{LS}(d_{LS} - d_n)}{K_0 d_c} \tag{25}$$

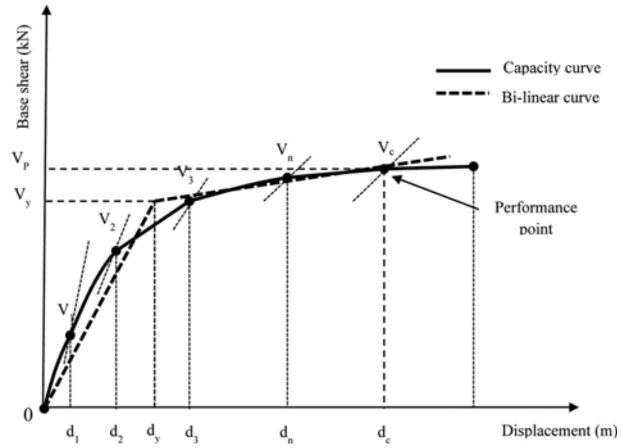

Fig. 2. Various Nonlinear Parameter of Pushover Curve

Table 2. Building Performance Levels as per FEMA 356

Non-structural Performance Levels	Structural Performance Levels					
	SP-1 (I.O)	SP-2 (DCR)	SP-3 (L.S)	SP-4 (L.S.R)	Sp-5 (S.S)	Sp-6 (N.C)
NP-A (Operational)	1-A	2-A	NR	NR	NR	NR
NP-B (Immediate Occupancy)	1-B	2-B	3-B	NR	NR	NR
NP-C (Life Safety)	1-C	2-C	3-C	4-C	5-C	6-C
NP-D (Hazard Reduced)	NR	2-D	3-D	4-D	5-D	6-D
NP-E (Not Considered)	NR	NR	3-E	4-E	5-E	NR

NR = Not recommended levels

For collapse prevention range,

$$DI_{CP} = 1 - \frac{\sum_{i=0}^{n=1} K_0 d_1 + K_1(d_2 - d_1) + K_2(d_3 - d_2) + \ldots + K_{CP}(d_{CP} - d_n)}{K_0 d_c} \quad (26)$$

4. FEMA Performance Levels

FEMA 273, later modified by FEMA 356, defines building performance levels of a structure obtained from various combinations of structural and nonstructural performance levels, as illustrated in Table 2. The performance levels of structure at several stages are;

1. **Operational:** The structural response is restricted to linear limit;
2. **Immediate occupancy:** The structure will be safe and in service after the earthquake;
3. **Damage control range:** It's a damage state between life safety and immediate occupancy performance level;
4. **Life safety:** The structure is damaged but still remains at a marginal level of collapse;
5. **Limited safety range:** A damage state between collapse prevention and life safety performance level;
6. **Collapse prevention level:** The structure is able to resist the gravity loads, but retains no margin against collapse.
7. **Collapse:** The structure is not able to provide any life safety and is not meant for any further service.

These performance levels were assessed by using two damage variable viz. drift and plastic deformation. Drift is the rooftop displacement of the structure over the height of the structure, and plastic deformation depends on plastic hinges yielding from collapse mechanism due to transient or permanent drift. To relate the damage index proposed in this work with FEMA 356 discrete performance levels, collapse mechanism of structure was followed. Damage value was obtained for plastic hinge falling in particular performance level.

5. Example RC frames

For the purpose of this study, 2-D symmetric-in-plan intermediate RC frames of different types of buildings, which can be considered as similar to typical office building frame, were designed, based on BIS 456 guidelines. These buildings were assumed to be located in the seismic zone V (severest zone as referred in BIS 1893 on soil type II. The height of the model is presumed as 3 m, and the beam span is 4m. The distance between the frames is assumed to be 4m. All study frame have the same plan arrangement with three numbers of bays in each direction. These buildings generally represent the low, middle height, and high-rise building respectively.

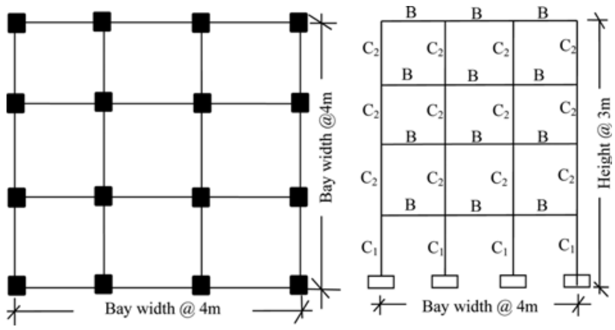

Fig. 3. Structural Arrangement of Four Storey RC Frame

Figure 3, represents the typical layout and member designations of a four storey RC frame. These frames sustain the mean dead load of 4.6 kN/m^2 for all frames and the mean intensity of live load between typical floors and roof is assumed to be 4 kN/m^2 and 1.5 kN/m^2, respectively. Selection properties of selected members of frames are presented in Table 3. The seismic demands on the building are calculated following the BIS 1893. The design base shear on the building is derived as;

$$V_b = \frac{z}{2}\frac{I}{R}\frac{S_a}{g}W \quad (27)$$

Where, z denotes zone factor (= 0.36 for zone V), I is structures importance factor (= 1 for these buildings), R is response reduction factor (= 5 for ductile frame), S_a is spectral acceleration (base of natural frequency, $T_a = 0.075(h)^{0.75}$) and W is the seismic weight of the structure.

Figure 4, shows the fundamental period of these frames on the 5% damping pseudo acceleration design spectrum specified in BIS 1893 for soil type II in zone V. The structural design of these buildings is not unique solution available for the demand calculated. Based on the same demand, different designer may select different solutions. The RC design solution adopted in the present work are based on common practices adopted by

Table 3. RC Section Details for the Study Frames (with the SCWB design criterion)

RCMRF designation	Members	Floors	Width (mm)	Depth (mm)
S4B3	Beams	1-4	300	300
	Column	1-1	450	450
		2-4	300	300
S6B3	Beams	1-6	300	300
	Columns	1-3	450	450
		4-6	300	300
S8B3	Beams	1-8	300	300
	Columns	1-2	600	600
		3-5	450	450
		6-8	300	300
S10B3	Beams	1-10	300	300
	Columns	1-1	750	750
		2-4	600	600
		5-7	450	450
		8-10	300	300
S12B3	Beams	1-12	300	300
	Columns	1-3	750	750
		4-6	600	600
		7-9	450	450
		10-12	300	300

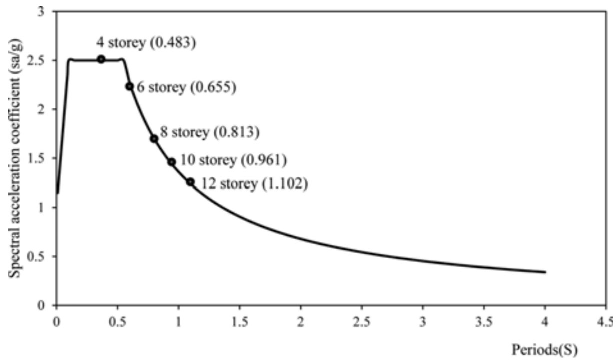

Fig. 4. 5% Damping Response Spectrum for Soil Type II in Zone V, as per BIS 1893

engineers. For a planar frame, all columns in a storey have same section or chosen to remain same and similarly the beams.

The column remains same up to two or three storeys depending on the building height. The RC section design ensures the strong-column-weak-beam behavior. The RC section details are provided in Table 3.

A strong-column-weak-behaviors requirement is considered in the design. The RC sections are designed with M25 grade concrete (having 28 days characteristics cube strength of 25 MPa) and Fe 415 grade reinforcements (having a characteristic yield strength of 415 MPa).

6. Nonlinear Static Pushover Analyses

The pushover procedure involves the application of gravity loads and representative lateral load pattern on a structure for a monitored displacement of 4% of the height of the structure. The equivalent lateral load distribution adopted for this pushover analysis is as suggested in BIS 1893;

Table 4. Lateral Loads Acting on Example Building as Per IS 1893

Storey level	Storey height (m)	Storey weight (kN)	$W_i h_i^2$	$Q_i = \frac{W_i h_i^2}{\sum W_i h_i^2}$ kN
Roof	12	237	34128	65.34
3rd floor	9	267	21627	41.40
2nd floor	6	267	9612	18.40
1st floor	3	267	2403	4.60

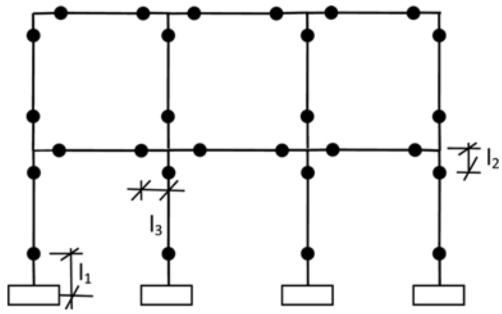

Fig. 5. Hinge Locations at Columns and Beams of RC Frames

$$Q_i = V_b \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} \tag{28}$$

Where, Q_i is the equivalent lateral force on the ith floor; W_i is the seismic weight on the ith floor; h_i is the height up to the i^{th} floor and n is the total number of storeys.

The values of lateral load obtained, for example four storey RC frame are tabulated in Table 4. Pushover analyses have been performed using SAP 2000 V 14.0, which is a general purpose structural analysis program for static and dynamic analyses of structures.

Beam and column elements were modeled as a non-linear element with lumped plasticity by defining plastic hinges at both ends of beams and columns. The plastic hinges were located at user defined locations obtained from recommendation of Inel *et al.*, 2006 as shown in Fig. 5. The plastic hinge lengths are obtained by following simple expression, 29-33 given by park and paulay, 1975.

$$l_p = 0.5H \tag{29}$$

$$l_p = 0.5 \times (300) = 150\ mm \tag{30}$$

$$l_1 = \frac{l_p}{2} = 75\ mm \tag{31}$$

$$l_2 = H_{beam} - \frac{l_p}{2} = 225\ mm \tag{32}$$

$$l_3 = \frac{H_{column}}{2} - \frac{l_p}{2} = 150 mm\ for\ storey\ 1\ and\ 75 mm\ for\ other\ storey \tag{33}$$

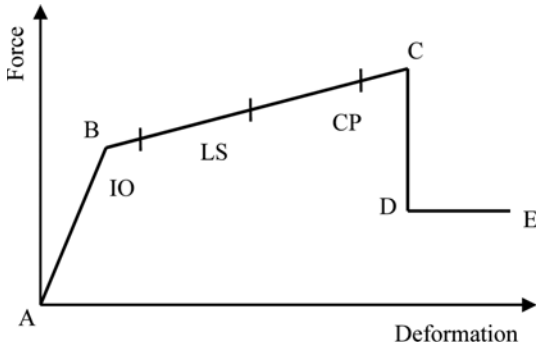

Fig. 6. Force-deformation Relationship of a Typical Plastic Hinge

SAP 2000 implements the plastic hinge properties described in FEMA 356 (or ATC 40) as shown in Fig. 6. Five points labeled A, B, C. D and E define the force-deformation behavior of plastic hinge. The values assigned to each of these points vary depending on the type of element, material properties, longitudinal and transverse steel content, and the axial load level of the element. Table 5-6, represents plastic rotation limits for RC beams and columns. SAP 2000 provides default-hinge properties and recommends beams to be assigned to the concentrate M3 plastic hinges and columns with the P-M3 plastic hinge, which was adopted in the present work.

The pushover procedure involves the application of design gravity loads before applying incremental lateral forces. The gravity loads are applied as distributed elemental loads based on yield line theory and concentrated loads from secondary beams. First the static analysis is performed in a single step. The state of structure at this stage is saved and subsequently the static lateral incremental load pushover analysis is conducted starting from this state of structure. The analysis is load control of gravity loads and deformation control for lateral loads. The P-Δ effects were considered in the analyses. The output on of a nonlinear analysis is presented in a form of pushover curve, which is typically roof top displacement versus base shear plot.

7. Computation of DI for the Study Frames

Figure 7, shows pushover curve for the study four storey RC frame. The nonlinear responses obtained from this pushover curve is used to calculate the proposed damage index. This damage index has a direct relationship with structural stiffness variations. The damage index represents the degree of damage that may occur at each performance level. Therefore, the study of degradation in stiffness of structure was done, which will be useful in PBSD.

Table 7, represents the collapse mechanism of the study four storey RC frame. The rooftop displacement and base shear values at different performance levels were recorded. Table 8, shows the stiffness values corresponding to the various performance levels and sequence of plastic hinge formation. Table 9, represents the calculated values of stiffness and damage index corresponding to different performance levels.

It is clear from Fig. 8, that the stiffness of the structure has a

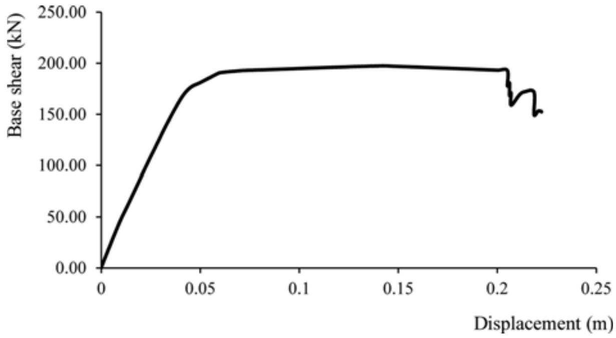

Fig. 7. Pushover Curve of Studied RC Frame

Table 5. Plastic Rotation Limits for RC Beams Controlled by Flexure (*FEMA* 356)

Conditions			Modelling Parameters			Acceptance Criteria				
			Plastic rotation angle (radians)		Residual strength ratio	Plastic rotation angle (radians)				
						Performance level				
$\frac{P}{A_g f_c'}$	Trans. Reinf.	$\frac{V}{b_w d\sqrt{f_c'}}$				IO	Component type			
							Primary		Secondary	
			a	b	c		LS	CP	LS	CP
≤ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.010	0.02	0.02	0.03

C indicates the transverse reinforcement meets the criteria for ductile detailing

Table 6. Plastic Rotation Limits for RC Columns Controlled by Flexure (*FEMA* 356)

Conditions			Modelling Parameters			Acceptance Criteria				
			Plastic rotation angle (radians)		Residual strength ratio	Plastic rotation angle (radians)				
						Performance level				
$\frac{\rho-\rho'}{\rho'_{bal}}$	Trans. Reinf.	$\frac{V}{b_w d\sqrt{f_c'}}$				IO	Component type			
							Primary		Secondary	
			a	b	c		LS	CP	LS	CP
≥ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03

C indicates the transverse reinforcement meets the criteria for ductile detailing

Table 7. Pushover Analysis Result for Study four Storey RC Frame

Step No.	Displ. (m)	Base force (kN)	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	Beyond E
0	0	0	48	8	0	0	0	0	0	0
1	0.00029	2.021	47	9	0	0	0	0	0	0
15	0.072	192.31	30	25	1	0	0	0	0	0
35	0.166	195.58	26	11	18	1	0	0	0	0
43	0.204	192.74	25	8	13	9	0	1	0	0
44	0.204	178.06	25	8	13	9	0	0	1	0
63	0.227	37.86	25	8	7	6	0	0	9	1

Table 8. Calculation of Stiffness Degradation and Damage Index for a Study Frame

Step No.	Stiffness kN/m	Incremental displacement (m)	Cumulative Force (V_c) (kN)	K_o*d_c (kN)	Summation K_n*d_n (kN)	DI_c	Remarks
0	0	0	0.00	0	0	0	
1	6968.97	0.00029	2.02	2.021	2.021	0.00	First hinge formation
15	2649.40	0.0072	19.27	505.87	283.86	0.44	First hinge in IO to LS
35	1176.23	0.0048	5.65	1158.78	441.85	0.62	First hinge in LS to CP
43	941.70	0.0048	4.52	1426.38	481.64	0.66	First hinge in C to D
44	869.95	4E-06	0.00	1426.41	481.64	0.66	First hinge in D to E
63	166.29	0.00088	0.15	1586.76	497.20	0.69	First hinge in beyond E

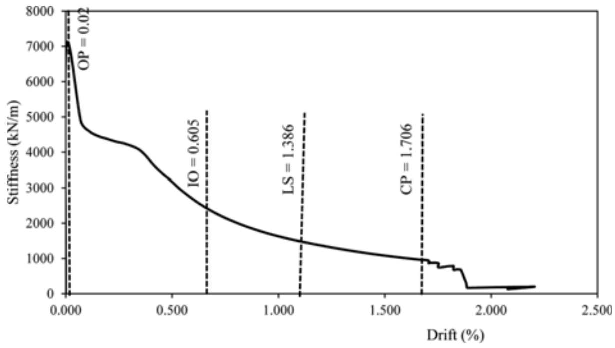

Fig. 8. Variation of Structural Stiffness at Different Performance Levels for Study Four Storey RC Frame

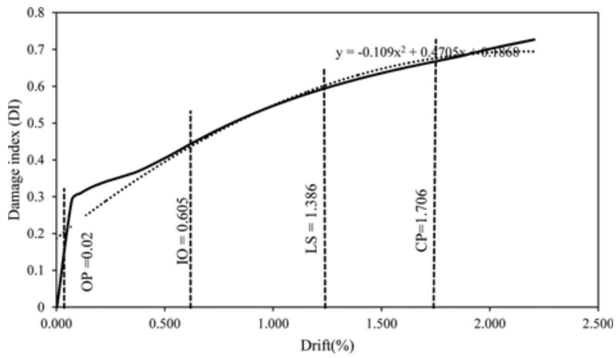

Fig. 9. Variations of Damage Index at Different Performance Levels for Study Four Storey RC Frames

downfall curve at different performance levels while entering in inelastic phase. From this graph it will be easier to classify various performance levels based on the structural strength. When the structural strength has been measured in terms of loss of stiffness, it was found that at immediate occupancy level structure has about 56.11% of the intact stiffness, of life safety level structure has about 38.13% of the intact stiffness, and at collapse prevention level structure has about 33.97%.

Variation of damage index with respect to percentage drift of the study four storey RC frame is shown in Fig. 9. From these variations, it is clear that DI equals to zero at operational level (Formation of the first plastic hinge), DI equals to 0.44 at immediate occupancy, DI equals to 0.66 at life safety range, DI equals to 0.66 at collapse prevention level, and DI equals to 0.69 at collapse.

A collapse zone cannot be decided using a pushover, while the damage index, which is calculated using drift reveal collapse zone. The drift damage criterion is simple and popular indices employed to determine the global damage index. PBSD documents FEMA 356 and ATC 40 had put forth the procedures for evaluation of the performance level of the structure using drift based damage index. Drift based damage index can be calculated from pushover analysis using the following relation (Habibi *et al*., 2013);

$$DI_{drift} = \frac{\Delta_m}{H} \tag{34}$$

Where, Δ_m represents the target displacement at the performance level under consideration and H is the height of the structure.

FEMA 356, had provided various drift limits to define a performance level for a structure. The prescribed and calculated value of drift and damage index values are presented in Table 9. From Fig. 9, it can be concluded that for the study RC frame the calculated damage values follow the drift path described in FEMA 356. Hence, a collapse zone can be introduced.

The drift obtained at LS performance level is 1.386 %, which is lower than the code prescribed value, not consistent with each other. Similarly drift obtain at collapse is 2.078 which, is lower

Table 9. Drift Base Damage Index for the RC Frame Under Study

Performance levels	Drift based damage index		Stiffness based damage index	
	Calculated value (%)	Prescribed value (%)	Calculated value	Prescribed value
Operational level	0.002	< 0.7	0	0
Immediate occupancy level	0.605	1	0.44	0.462
Life safety level	1.386	2	0.62	0.548
Collapse prevention level	1.706	4	0.66	0.692
Collapse	2.078	> 4	0.69	0.325

than code prescribed value. The reason behind this inconsistencies, maybe restrictions of plastic rotation in members. The performance levels are determined based on plastic rotation of plastic hinges, the fulfillment of restrictions of plastic hinges has caused inconsistencies between the drift in the model and code prescribed values.

To overcome this a relationship is proposed between the stiffness and drift index, by running an exponential trend line approximating the behavior of the structure. The exponential relation is expressed in Eq. (35);

$$DI = -0.109x^2 + 0.4705x + 0.1868 \tag{35}$$

Where, DI represent stiffness based damage value, and x represent corresponding to drift index at desired deformation level.

A limiting value for a particular performance level can be obtained by substituting the drift limits described in FEMA 356, as shown in Table 9.

With an intention to study variation of proposed damage index on different structures, the RC frames discussed in Table 3, were designed and analyzed. The pushover curve obtained for these structures are shown in Fig. 10 and corresponding nonlinear responses are tabulated in Table 10. Fig. 11-13, shows the degradation in stiffness, and damage value obtain for the study RC frames.

From pushover results it can be concluded that, with the increase in the number of storeys there is associated increase in inelastic behavior of a structure which can be represented by an increase in rooftop displacement. The mean percentage increase in displacement corresponding to performance point and ultimate point is observed to be 43.81% and 29.96% respectively.

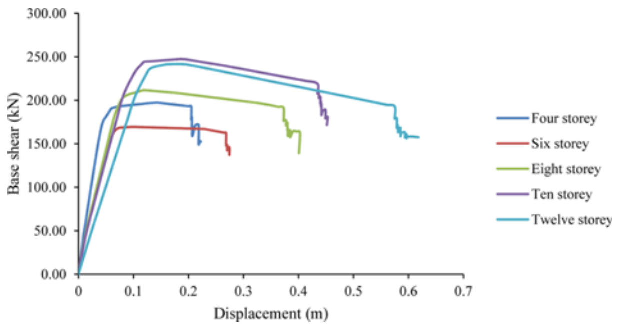

Fig. 10. Pushover Curve for Different Study RC Frames

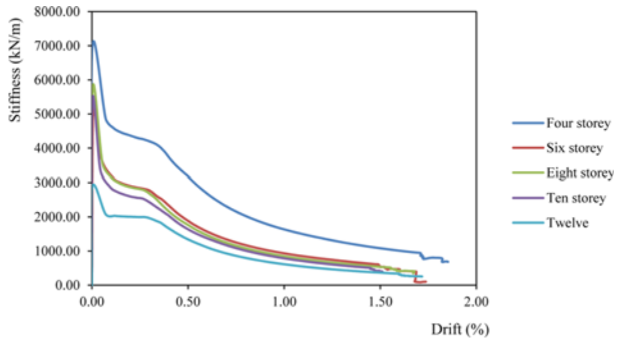

Fig. 11. Degradation of Structural Stiffness for Different Study RC Frames

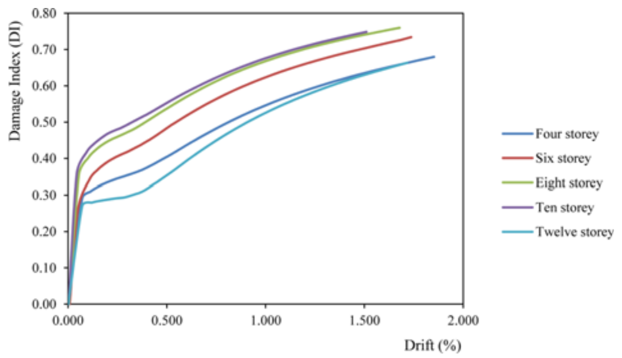

Fig. 12. Variation of Damage Value for Different Study RC Frames

Figure 11, shows that degradation in stiffness of structure has a downfall trend irrespective of variation in height of the structure. Ghobarah *et al.* in Eq. (21), had provided damage value related to first crack to the structure, which is approximate method of damage quantification, from pushover results its understood that, structure possess an ability to support load up to formation of collapse mechanism or at collapse. Introduced damage index provides an extension to Ghorbarah *et al.*, damage index using which damage value at different performance levels defined in FEMA 356 can be evaluated.

Figure 12, represents variation of damage value at every incremental step of pushover, referring to collapse mechanism of structure, damage value of different performance levels can be evaluated. Damage value for different performance levels are illustrated in Fig. 13. According to the obtained results, increase in the storey number shows reduction in damage index. Amongst different performance level IO represents more damage value

Table 10. Pushover aNalysis Results of All Studied Frames

Studied RCMRF	Height (m)	At performance point			% drift at different performance levels			
		Base shear (kN)	Displ. (m)	% Drift	OP	IO	LS	CP
S4B3	12	144.80	0.034	0.283	0.02	0.60	1.39	1.71
S6B3	18	163.26	0.065	0.361	0.01	0.53	1.20	1.49
S8B3	24	201.72	0.080	0.333	0.004	0.54	1.24	1.55
S10B3	30	229.31	0.100	0.333	0.003	0.52	1.17	1.44
S12B3	36	237.78	0.136	0.412	0.002	0.51	1.24	1.56

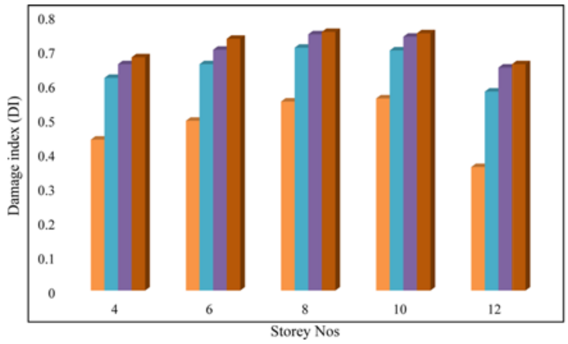

Fig. 13. Damage Values at Different Performance Levels of Study RC Frames

and is much influenced with storey height compared to other performance levels.

8. Conclusions

Assessment of the structural damages of RC structures by inspection will be proper for a class of building to show buildings and structural components representing life-safety hazards. Nonlinear dynamic analysis has been extensively used in seismic damage assessment, but was found to be inconsistent for determination of the behavior of existing RC structures which are dependent on inelastic displacement and deformation up to collapse. The pushover analysis is promising simple and efficient approach of evaluation of inelastic lateral loads resistance of large class of existing and new structures, provided that its limitations are fully addressed. Stiffness damage index given by Ghobarah *et al.*, in Eq. (22), provides a simple and ease approach of damage evaluation of RC structures using nonlinear static procedure. For stiffness damage index evaluation pushover analysis has been performed twice, once before subjecting RC structure to earthquake, and once after subjecting to earthquake time history. Stiffness damage index has limitation that it does not addresses the cumulative effects, and evaluates the structures damage for first crack resulting to higher damage value. In present paper an attempt had been made to extend stiffness damage index to account for cumulative effects by studying degradation of structure stiffness for every incremental displacement at every interval step of pushover. Introduced damage index was formulated for values got from only one pushover analysis, and is able to evaluate damage value of any displacement or any force corresponding to capacity curve. The lateral load pattern adopted represents the dynamic inertia load obtain from equivalent static load described in BIS 1893. The study RC frames represent low, middle height and high-rise structures. Results obtained from pushover analysis carried on study RC frames show that the inelastic deformation increases with increase in storey height of structures. The stiffness degradation of study RC frame were found to have downfall curve irrespective of increase in storey height. The stiffness degradation follows the collapse mechanism of the structure and found to consistent with drift based damage index presented in PBSD document, hence a collapse zone can be defined. Damage value corresponding to IO performance level was found to have higher contribution for damage, hence stiffness at IO level may be checked and alter to have optimized design of RC section securing life safety design. Introduced damage index offer quick and approximate method of global damage assessment.

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Notations

ATC = Applied Technological Council
DI = Damage Index
EDPs = Engineering Demand Parameters
FEMA = Federal Emergency Management Agency
PBSD = Performance Based Seismic Design
b = Counts severity of the damage elements
C = Constant value
D_i = Local damage index at location i
d_m, d_y = Maximum displacement and yield displacement, respectively
dE = Incremental dissipated hysteric energy
E_i, E_h = Dissipated energy and dissipated hysteretic energy, respectively
F_m, F_y = Maximum force during previous cycle, and failure force during loading cycle, respectively
g = Acceleration due to gravity

K_o, K_m, K_u, K_e = Initial, maximum, ultimate, and elastic bending stiffness, respectively
K_i = Elastic stiffness of the building
M_u = Ultimate bending moment resulting from pushover analysis
n, n_i = Number of hysteretic cycles and number of cycles with inelastic deformation, respectively
R = Ratio of elastic and yield strengths
S_a = Response spectrum acceleration at effective fundamental period and damping ratio of the building under consideration
T_a, T_m, T_d = Natural period at initial stage, maximum softening, and final softening, respectively
T_0 = Initial period of vibration of a nonlinear system
T_c = Characteristic period of ground motion
V_T = Total shear force
W = Total building weight
α = Post-yield stiffness ratio
β_e = Parameter representing the cyclic loading
μ = Maximum displacement ductility ratio
κ = Adjustment factor for approximate account of changes in hysteretic behavior of reinforced concrete structure.
ζ = Equivalent ductility ratios
ϕ_m, ϕ_u = Maximum and ultimate curvatures, respectively
μ_u, μ_m =Ductility under monotonic loading, and ductility attained during seismic response, respectively.
δ_{max} = Roof displacement
Δd^+ = Incremental increase of positive displacements
Δd^- = Incremental decrease of negative displacements
Δd_f = Recommended 10% of floor height
Δd_{df} = Value of Δd^+ for a cyclic load that leads to failure
K_i = Elastic stiffness of the building
K_e = Effective stiffness of the building obtained by idealizing the pushover curve as a bilinear relationship.

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