QUANTIFICATION OF DAMAGE USING HINGE PATTERN IN RC MOMENT RESISTING FRAME BUILDINGS

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QUANTIFICATION OF DAMAGE USING HINGE PATTERN IN RC MOMENT RESISTING FRAME BUILDINGS

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Abstract

Damage index models are used for quantification of damage in any numerical model of a building. While using any particular damage model there is high possibility that it might give similar result for two identical structures, each having different attributes. So, though currently available damage index models precisely quantify damage in the structure but fails to identify the reason or attributes/irregularities present in it. Therefore, for quantitative assessment, a unique aspect should be used which will depend on structural as well as architectural features. This paper presents new method for quantification of damage for reinforced concrete (RC) moment resisting framed (MRF) structures using the pattern of hinge formation in structure when it is subjected to a monotonic loading. A two-dimensional RC MRF structures with different building irregularities were modeled. As a primary step to estimate the damage from the pattern of hinge formation, it is necessary to understand and study the relation between damage and hinge pattern. For this purpose, energy-based damage index model is used. The relation between damage index and pattern of hinge formation is studied with the help of regression analysis. A multiple regression analysis is performed taking damage index as dependent variable and the number of hinges formed in each damage state as independent variables. The paper also discusses two different approaches for regression analysis determining the corresponding coefficients. The regression analysis resulted in an equation which can approximately quantify the damage in the structure using total number of hinges formed in each structural member.

Keywords: Damage index, hinge pattern, hinge status, regression analysis

1. Introduction

Knowing the probable future loss or the damage state of any structure due to earthquake is very important for developing a resilient structure. Seismic assessment methods are very helpful in predicting probable future damage state or estimating present state of any structure. But even for developing these assessment methods, it is important to perform the damage analysis of different structures. In past, many attempts were made using empirical as well as theoretical approaches to yield various estimates of structural damage [1]. Empirical approach is purely a statistical study based on observed damage in the buildings after earthquake. Although, these damage observations are subjective, they provide useful information on the overall seismic performance of structural systems [2]. However, this approach has many drawbacks as it underestimates the reserved strength and response characteristics of structure. Apart from these there are many seismic damage index models, which are used for predicting the possible damage. These damage indices have been formulated using response parameters of the structure that are obtained through analytical evaluation of structural response [3]. The analytical damage models are broadly divided into two classes (a) strength-based damage indices and (b) response-based damage indices [1]. A strength-based damage index depends on geometry of structural elements such as column, wall area and their general material properties [4]. Whereas response-based damage index depends on structure's time period, deformation, interstorey deformation etc.

Apart from seismic assessment methods, damage indices are used in the field of post-earthquake damage assessment and play important role in decision regarding retrofitting of structure. Therefore, it is important to make sure the precision of predicted damage state of structure when some damage index is used. Important decisions concerning the residual strength and safety of a damaged structure are usually based on a single overall or global damage in a structure [5]. This global damage is nothing but the combination of local damages in each structural member. However, the development of analytical models is very complex since the index should apply to various structural systems at advanced stages of inelastic deformation and up to collapse [5]. Therefore, it is important to understand the contribution of each structural member in the overall global damage. This can be understood using plastic hinge pattern at different stages of nonlinear static analysis.

The objective of this study is to develop the new damage assessment equation for numerical models which is purely based on response of each individual frame. The proposed damage estimation equation uses hinge pattern, developed after each monotonic loading step. This response-based assessment approach will be helpful in developing preliminary seismic assessment methods such as rapid visual survey.

2. Non-Linear Static Analysis

Though elastic analysis gives appreciable results for determination of elastic capacity of structure, it over-estimates the same result without considering the yielding of member. So, to predict actual response of structure beyond yield point or elastic limit, inelastic analysis plays an important role. Out of various methods of nonlinear analysis, nonlinear static (static pushover) analysis have comparatively more advantages. One major advantage of this method is a significant reduction of computational effort while maintaining the credibility of the results at an acceptable level [6]. During nonlinear static analysis a series of incremental static load is applied on a structure and the response of structure at each step is recorded. This nonlinear static analysis produces the capacity curve which is nothing but the base shear verses deformation of structure. Based on the capacity curve, a target displacement which is an estimate of the displacement that the design earthquake will produce on the building is determined [7]. The extent of damage experienced by the structure at this target displacement is considered as representative of the damage experienced by the building when subjected to design level ground shaking [7]. Under incremental static loading various structural members in a structure yields at different stages. *Therefore, at global level the overall change in the performance of structure is dominated by plastic yielding effects due to which structure experiences loss of strength and stiffness at each step.* A typical capacity curve along with the associated damage states is shown in Fig. 1.

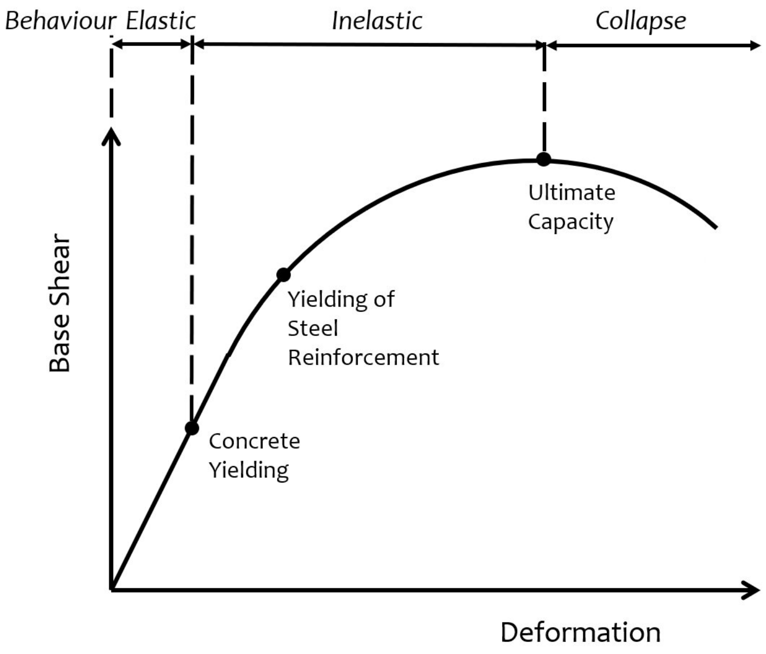


Fig 1 – Typical structural performance and accociated damage state

For studying the performance of structure using nonlinear static analysis, two different plasticity models can be used which are (a) distributed plasticity model and (b) concentrated plasticity model. In this study concentrated plasticity models i.e., plastic hinges are used for the analysis. To make it easy for practile use, many documents such as FEMA-356 [8] and ATC-40 [9] have provided default properties for hinges. However, though these documents provide the hinge properties for several ranges of detailing, some structural analysis programs such as SAP2000 implements the averaged values [10]. SAP2000 [11] program is used for the nonlinear static analysis in this study.

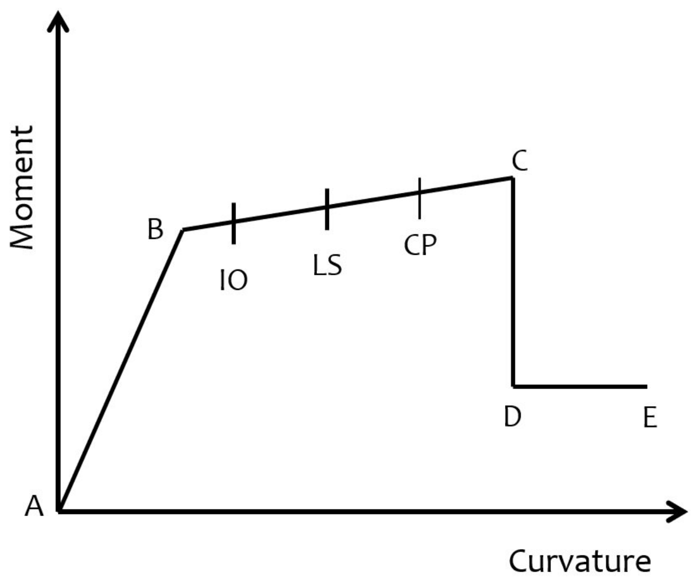


Fig. 2(a) – Typpical moment curvature relationship of a hinge

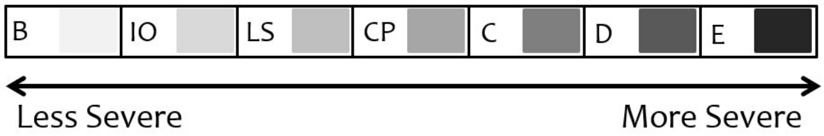


Fig. 2(b) – Hinge severity level [12]

A typical moment-curvature curve of a plastic hinge is shown in Fig. 2(a) where as Fig. 2(b) shows the increasing damage severity of each hinge state [12]. The points B and C shown in Fig. 2(a) represents yield and ultimate value. However, points between B and C which are IO, LS and CP, represents ultimate capacity at different acceptance criteria i.e., immediate occupance, life safety and collapse prevention respectively. Point D represents loss of strength and finally point E represents the collapse stage of the member. FEMA-356 [8] and ATC-40 [9] documents recommends limit states up to collapse prevention only but SAP2000 program adopts limit state up to complete collapse state i.e., point E. As the rotation of assigned plastic hinge reaches the rotation limit of each of the above mentioned state, damage in the member progresses from lower to higher state. For example a structural member having plastic hinge of damage state life safety will have less damage compared to a similar member having plastic hinge of damage state D.

3. Description of Structure and Modeling

Three two-dimensional reinforced concrete bare frames of different geometry are considered for this study as shown in Fig (3). For all the three frames the height of each storey is 3 m and length of each bay is 3 m. All the three frames are named as *ideal frames* because these frames do not have any irregular building attributes (vulnerable parameters) which affects its performance. The frames are designed for both gravity and seismic loads as per Indian standards. For gravity loading, self-weight of member, weight of wall, total dead weight from slab and participating live load from slab (i.e., 25% of live load) is considered.

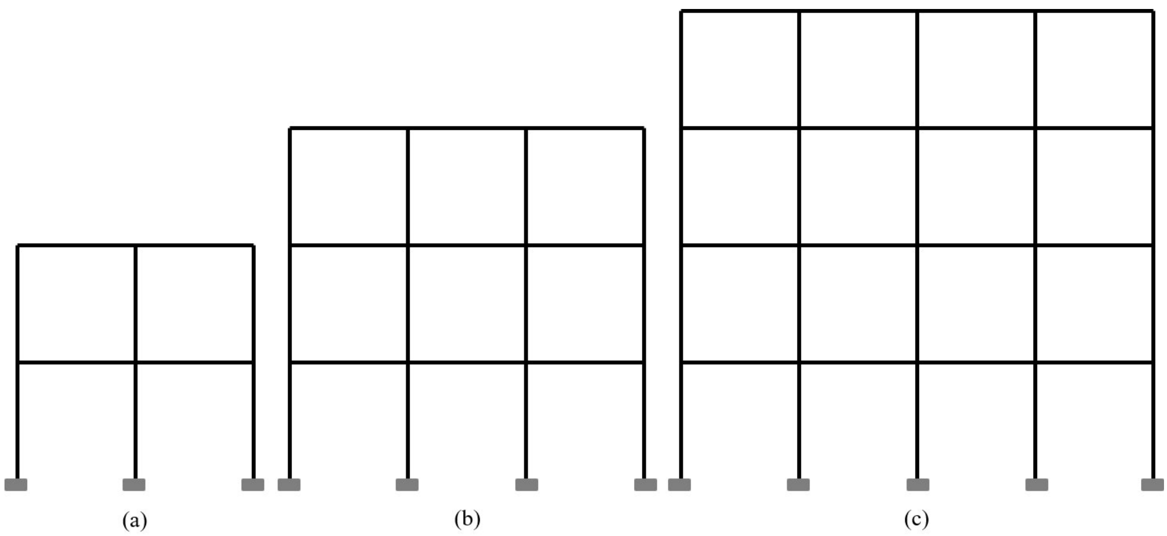


Fig. 3 – Ideal frame models (a) Model A; (b) Model B; (c) Model C

For seismic loading, a design ground acceleration of 0.36g (i.e., seismic zone V as per IS-1893:2016 Part I [14]), soil type II (medium as per IS-1893:2016 Part I [14]) are assumed. As the frames are to be used as ideal or reference frames for further comparative study, all the frames are designed as ductile moment resisting frames therefore, response reduction factor 5 is considered in design. Material properties are assumed to be 20 MPa as compressive strength of concrete and 415 MPa as yield strength of both longitudinal and transverse reinforcements. Initial stiffness values for beams and columns are considered as per IS-1893:2016 which are 0.35I for beams and 0.7I for columns. The spacing of 150 mm is considered for transverse reinforcement throughout the length of structural members.

Model A (i.e., 2-storey & 2-bay) have total base dimension as 6 m, Model B (i.e., 3-storey & 3-bay) have total base dimension as 9 m and Model C (i.e., 4-storey & 4-bay) have total base dimension as 12 m. Typical floor to floor height for all the storey in all frames is 3 m. All the frames are designed as per Indian Standards guidelines i.e., IS-456:2000 [13] and IS-1893:2016 [14] using default design option in the SAP2000 program. The basic necessary requirements of ductile design provisions as per IS-13920:2016 [15] were implemented in each frame during modeling. For example, the minimum dimension criteria for beams and columns, criteria for minimum transverse reinforcement spacing and moment capacity ratio (i.e., $M_C/M_B \geq 1.4$).

After design is complete, a nonlinear static analysis is performed on each frame. The nonlinearity of beams and columns is modeled with concentrated plasticity by defining plastic hinges at both the ends of beams and columns as shown in Fig. (4). Hinges are assigned at five percent relative distance from both the ends of beams and columns.

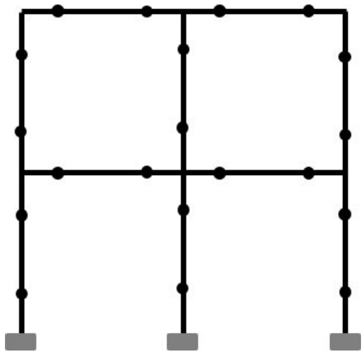


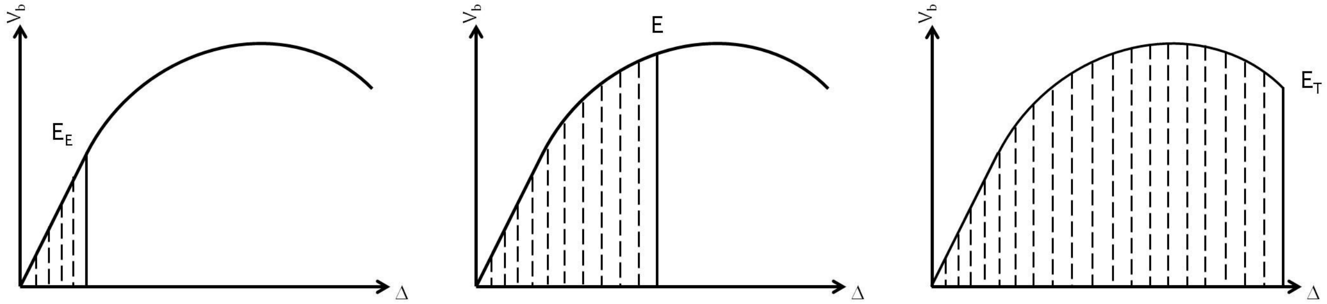
Fig. 4 – Assigned hinge locations for beams and columns

SAP2000 program implements all seven hinge properties as shown in Fig (2). The properties of hinges IO, LS and CP are as per FEMA-356 document. The limiting values assigned by the program to each of these hinge state vary depending on type of element, material property and amount of steel provided. In the present study SAP2000 default hinges are assigned to each member. SAP2000 recommends default hinge property of PMM hinge for columns and M3 hinge for beams. In a reinforced concrete frames shear failure takes place when the strength of concrete is very low and amount of transverse reinforcement is insufficient. But as all the *ideal* frames are modeled as ductile moment resisting frames, only flexural hinges are assigned.

4. Methodology

Stiffness, strength and ductility are three aspects which are derived from the capacity curve. But using these aspects it is very difficult to estimate overall performance or condition of building. Therefore, a quantitative assessment of building is also necessary. This quantitative assessment of structure can be derived with the help of damage indices. For the present study, an energy-based damage index proposed by A. Vimala in 2013 is used [16]. This damage index is useful to decide the damage state of structure based on deformation, during and after seismic event. This index helps to reveal the amount of damage to the structure and the margin left to reach the failure stage.

$$DI = \frac{(E - E_E)}{(E_T - E_E)} \times 100 \tag{1}$$



(a) (b) (c)

Fig. 5 – Estimation of energy-based damage from capacity curve [16]

From Eq. (1) and Fig. (5), E is energy dissipated by the structure at desired displacement where damage is to be estimated, E_E is initial yield energy of the structure and E_T is total energy dissipation capacity of the structure. The main advantage of this index is that it gives the clear information regarding the distribution of damage among the structure from which intermediate damage states can also be estimated.

The damage index used here to calculate the percentage of damage in frame depends on load-deformation curve only as area under the curve gives energy. As the damage calculation is based on capacity curve, there are high chances that the frames with two different building attributes (irregularities) may have same damage index value. In similar way stiffness, strength and ductility values of one frame may also match with the other frames. Therefore, for quantitative assessment of frames a unique response parameter should be used which will also depend on the presence of irregularities in the structure. The close observation of response of the frames indicates that the formation of hinge pattern at any stage during nonlinear static analysis can be used for damage assessment. Therefore, hinge pattern and hinge states of all structural members during the nonlinear static analysis are taken into consideration. Further, for the prediction of damage from hinge pattern and hinge states, it is also necessary to understand and study the relation between damage and combination of various hinge pattern. This relation is attempted in this study with the help of regression analysis.

The multiple regression analysis is performed where damage index values which are calculated using Eq. (1) are considered as dependent variable and the number of hinges formed of each state at every stage of nonlinear static analysis are considered as independent variable.

$$y = \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \ldots + \beta_7 x_7 \quad (2)$$

The regression analysis generates an equation Eq. (2) to describe statistical relation between any two data sets. So, in Eq. (2) β_1 represents weight of the first damage state (i.e., B to IO) and x_1 represents the total number of hinges formed in first damage state at various stages of analysis. In similar way β_2 and β_3 represents the weights of second damage state (i.e., IO to LS) and third damage state (i.e., LS to CP) and so on. Whereas, x_2 and x_3 represents total number of hinges formed in second damage state and third damage state and son on. The best fit model is decided based on various checks. For example, the model with relatively high R^2 value and relatively small ε value be called as best fit model. R^2 is known as coefficient of determination and represents that up to what extent variance in independent variables explains the variance in dependent variable whereas, ε is standard error of estimation. R^2 value close to zero explains very poor correlation between dependent and independent variables and vice-versa.

5. Results and Discussion

The capacity curves of all three frames are shown in Fig. (6). The strength of frame increases as the total number of vertical as well as horizontal members increases. The ultimate strength of Model A (i.e., 2 storey 2 bay) is nearly 95 kN, it is 175 kN for Model B (i.e., 3 storey 3 bay) and nearly 300 kN for Model C (i.e., 4 storey 4 bay). The interesting point to note here is the initial stiffness. The initial stiffness is referred as the capacity of structure to resist deformation till structure is in elastic state. Though the cross section of structural members is different in all three frames, but the stiffness values are almost same.

Initial stiffness and ultimate strength are two aspects that play an important role till the structure is in repairable damage stage. Once the structure undergoes irreparable damage stage (i.e., plastic stage or nonlinear stage), both goes on reducing and at the same time damage in the structure increases. In such case, life safety in the structure is ensured when structure have enough ductility. Close observation of capacity curve shows that the ductility of frame decreases as the geometry of structure increases. Maximum displacement of Model A is nearly 0.15 m, for Model B it is 0.23 ma and finally for Model C it is 0.3 m. Moreover, the ductility (which is ratio of maximum displacement to yield displacement) of Model A, Model B and Model C is 6.75, 5.20 and 4.07 respectively.

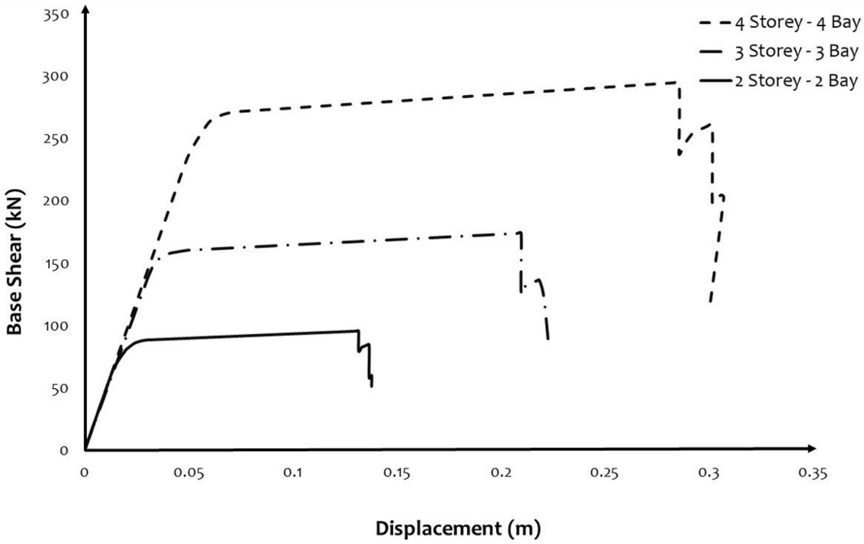


Fig. 6 – Capacity curves of Ideal Frames

In this study two different approaches are used while performing multiple regression analysis. In first attempt the total number of hinges formed at each step during analysis is considered for overall frame whereas in second attempt total number of hinges formed in columns and beams are considered separately at each step of analysis. The weights or β values obtained using these two approaches are presented in Table 1.

Table 1 – Weights of each Hinge Status determined from regression analysis

Status of Hinge	**Approach 1**	**Approach 2**	
	Weights (β values) Overall	***Weights (β values) for Beams***	***Weights (β values) for Columns***
B to IO	1.042	1.267	-0.507
IO to LS	2.781	2.786	2.406
LS to CP	4.357	4.323	4.318
CP to C	3.903	-	4.992
C to D	4.507	4.691	5.468
D to E	4.530	4.165	5.652
Beyond E	5.752	-	6.975
Constant	-7.241	-6.553	

In first approach there are total seven independent variable, each representing the one hinge status irrespective of beams and columns and damage index as dependent variable. However, in second approach there are total fourteen dependent variable where each hinge status is considered separately for beams and columns. The regression analysis does not give any coefficient value if data set has a greater number of zeros. This can be seen in the results of approach 2. In second approach, when total number of hinges in each status are further separated for beams and columns then it is observed that no hinges of category CP to C and beyond E are formed in beams. Therefore, regression analysis could not find weight for those hinge statuses. While comparing the best fit model among two approaches it is observed that the R^2 value for first approach is 0.990 and it is 0.993 for second approach. However, the standard error estimation for first approach is nearly 3.15 and the same for second approach is 2.89. This clearly shows that damage in columns has greater global effect compared to similar damage in beams. Therefore, considering hinge pattern separately for beams and columns gives more approximate estimate of damage in structure.

6. Conclusion

Studying and comparing the response parameters such as stiffness, strength and ductility may not be helpful always as there is a possibility that two different structures may have similar stiffness or ductility. Therefore, it becomes difficult to understand the behavior of structure. The only parameter which can give the damage estimation in more precise manner is the post yield damage states of each structural member. The total number of hinges formed in each damage state gives much valuable information on how the local damage progresses to global damage. Moreover, when number of hinges is considered separately for beams and columns then it gives better understanding on how local damage in beam or column affects global performance.

The correlation of number of hinges in each damage state with the global damage will be very useful in the preliminary assessment methods. Hinge pattern and hinge status study may help in identifying the specific irregularities present in the structure. Moreover, this information can be used in taking decisions regarding retrofitting of structure.

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