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Residual Capacity of Seismically Damaged Reinforced Concrete Moment Frames

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Abstract

Reinforced Concrete (RC) structures are generally designed for safety conditions, where earthquake energy is dissipated through yielding of the reinforcement and its inelastic deformation. Their seismic behaviour has been the subject of extensive studies in the past two decades. The most common tool to assess their seismic damage is the Maximum Inter-storey Drift (MID). Due to the complexity involved in conducting nonlinear dynamic analysis, researchers recently emphasized that Residual Inter-storey Drift (RID) might be an easier alternative. This study aims at identifying differences arising from evaluating the seismic damage using MID and Maximum RID (MRID). A six-storey building designed and detailed according to current seismic codes is used in the study. Pushover analysis is conducted to define the collapse limit using both MID and MRID. The observed damage of the building at these limits when subjected to six earthquake records is obtained using nonlinear dynamic analysis. The two methods are found to be significantly different and it was concluded that the MRID is a better tool to judge on the seismic damage state of a building.

Keywords: seismic, reinforced concrete, moment frame, damage, residual drifts.

Introduction

Evaluating the seismic performance of reinforced concrete structures was the focus of the research community in the last two decades. Among many criteria, the Maximum Inter-storey Drift (MID) is the most commonly used criterion to assess the global damage. Different values for MID at collapse were proposed by researchers including 2%[1], 2.5%[2], 3%[3,4], 4%[5], and 5.6%[6]. A statistical experimental study[7] showed that the MID at collapse for small-scale bare frames varies between 3% and 15% with an average of 4.0%. The reported large variation in the values of MID results from differences in design approaches and variations of columns ductility along the frame height. Although using a single value of MID may be a tool to make a general judgment on the global damage level, it is not an accurate method to identify locations of local damage. Another major drawback for using MID is that a nonlinear dynamic analysis is needed to evaluate its value. FEMA 273[5] introduced the use of RID to eliminate the need for dynamic analysis as it can be evaluated by field measurements. A value of 3 % was proposed for RID at collapse[5].

The main objective of this study is to provide a comparison between the MID and MRID as methods to evaluate the seismic damage. A six-storey building located in California was designed according to ACI requirements[8]. The designed building was subjected to a static pushover analysis to define the collapse limit in terms of MID and MRID. Incremental dynamic analyses using six ground motion records were then conducted.

Yielding and crushing limit states

Local yielding of elements is defined when the tensile strain in the longitudinal reinforcement reaches its yield strain (ε_y= 0.002). A number of criteria were suggested by different researchers to identify concrete crushing of individual members. These criteria include using a value for ultimate curvature[9, 10] or assuming that the crushing strain is 0.003[11]. The crushing strain is expected to depend on the type of concrete and its confinement. Its value varies from 0.0025 to 0.006 for unconfined concrete[12] and from 0.015 to 0.05 for confined concrete[13].

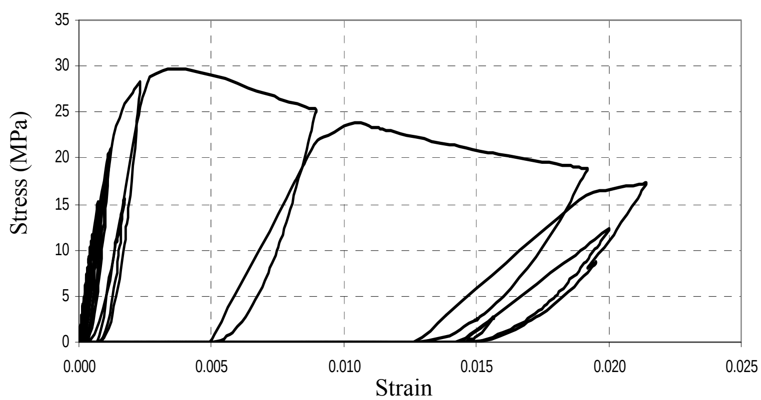


Figure 1 - Stress-strain curve of Confined concrete[14]

A typical compressive stress-strain curve for confined concrete including unloading and reloading branches is shown in Fig. 1. The envelope for this curve is very close to the stress-strain curve for the monotonic compressive test[12]. For static pushover analysis, crushing can be assumed to occur when the confined concrete strain reaches the lower bound value of 0.015. For dynamic analysis and due the loading and reloading paths, instantaneously reaching this lower bound value of 0.015 will not represent the crushing state of the core concrete. The residual concrete strain can give better representation about state of the damage in the building.

Case Study

A symmetric six-storey RC office building was assumed. The selected dimensions and layout of the building are shown in Fig. 2. The building was selected to be in a high seismic region, California. The design was conducted according to the regulations of the International Building Code[15] and the ACI requirements[8]. The concrete unconfined compressive strength was assumed to be 28 MPa and the reinforcing steel yield strength was 400 MPa. The frame was designed for the critical combination of gravity and seismic loading. When calculating the dead load, the weight of the structural elements and the masonry walls were included. The live load was assumed to be 4.8 kN/m^2. The building was designed as a Special Moment Frame (SMF). Section dimensions and reinforcement details for a typical frame are given in Fig. 2.

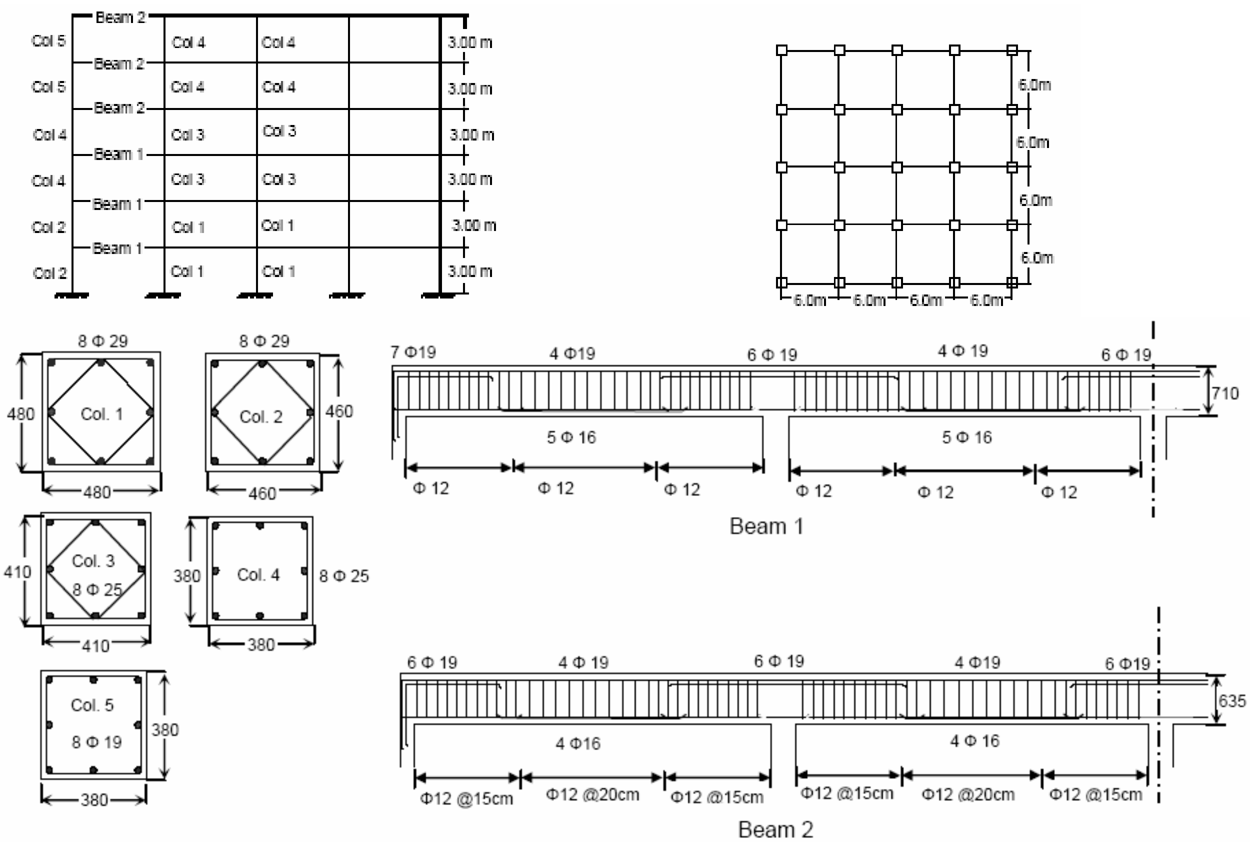
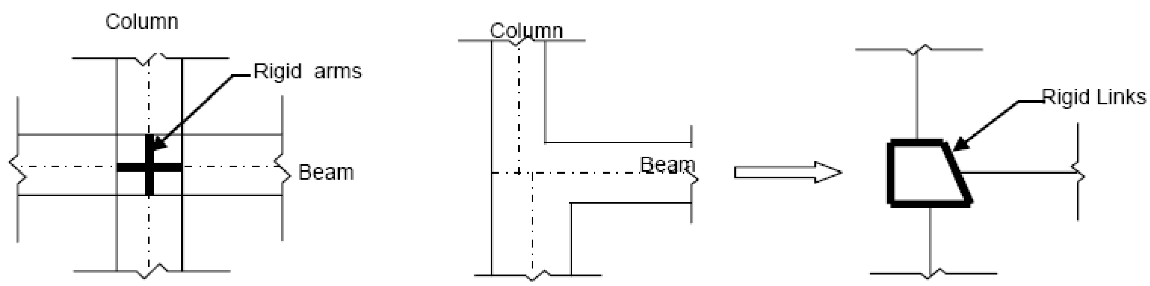


Figure 2 – Layout and cross-sections of beams and columns for the six-storey building

Analytical model

The finite element program ZEUS-NL[16] was utilized. The program is capable of representing spread of inelasticity within the member length using the fiber analysis approach taking into account both geometric and material nonlinearity.

As the structure is symmetric, a 2D model was used. Beams and columns were modeled using cubic elasto-plastic elements. In order to achieve accurate results and to monitor local damage at the ends of each element, beam elements are divided into six elements according to the distribution of longitudinal and transverse reinforcements. Columns are divided into three elements. The frame beams are modeled as a T-section, the effective flange width of such beams is assumed to be the beam width plus 7% of the clear span of the beam on either side of the web[17]. Rigid elements are used, as shown in Fig. 3a, to model the intermediate beam-column connections. Due to the different alignment for the edge columns at the second floor, the arrangement of rigid arms illustrated in Fig. 3b was used at these edge connections. The shown arrangement of rigid elements allows accurate modelling of the forces transferred between members meeting at these connections.

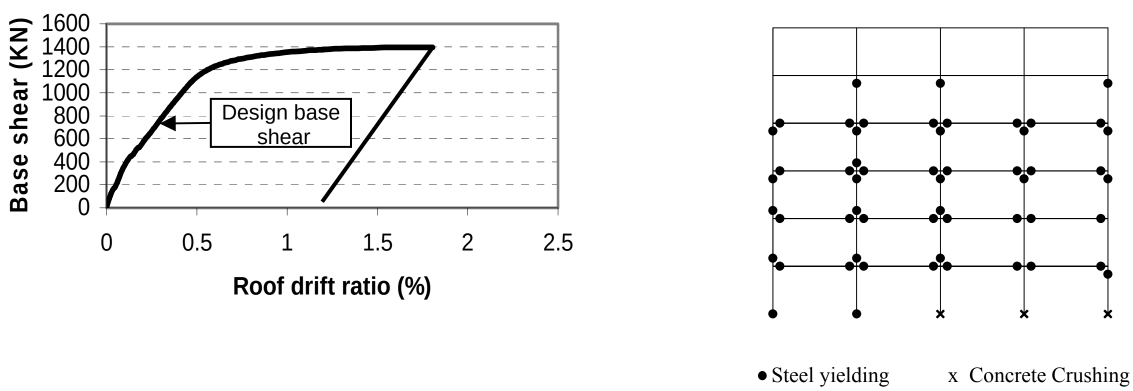


a) Intermediate connection b) Edge connection in the second floor

Figure 3 - Rigid arms for modeling the intermediate and edge beam column connections

Static Pushover Analysis

Inelastic pushover analysis was performed using ZEUS-NL. It allowed investigating the failure mechanism and determining the limit states of the moment frame. The vertical distribution of the lateral load was taken similar to the distribution used for the design. A force controlled pushover analysis was employed up to the maximum force resistance, followed by unloading path to identify the residual drifts.

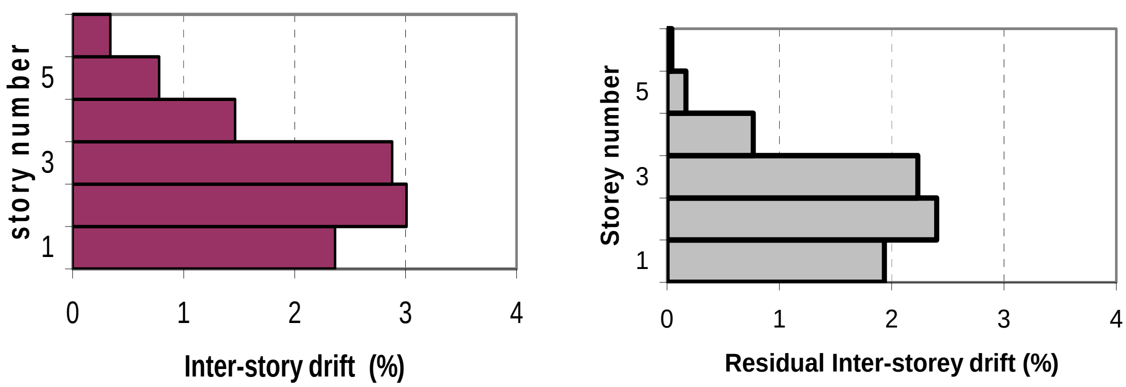


a) Relationship between roof drift and base shear b) Observed Damage at failure

Figure 4 - Pushover analysis results

Fig. 4a shows the pushover curve for a moment frame of the six-storey building. The building lateral capacity is 1.8 times the design base shear. Fig. 5a shows the distribution of inter-storey drift over the building height at collapse. The MID is observed in the second storey and was used by Elfeki and Youssef[18] to define the global damage levels. In this paper only the collapse limit is considered to define the building capacity. It was found that the second floor reaches its maximum capacity at 3% inter-storey drift[18]. This value is matching the collapse limit values recommended by Broderick and Elnashai[3] and Kappos[4]. At this level of drift, three columns were considered crushed, Fig. 4b. The corresponding residual inter-storey drift distribution through the height is shown in Fig. 5b. The figure shows that the Maximum Residual Inter-storey Drift (MRID) is equal to 2.4% at the second floor. This value is considered as the residual collapse limit.

In the following sections a comprehensive study on the behaviour of the building under the effect of six earthquake records will be presented. A comparison between the building damage states when using the both the MID and MRID will be illustrated.



a) Inter-storey drift distribution b) Residual Inter-storey drift

Fig 5 - Comparison between MID and RID at collapse limit

Seismic response analysis

Eigen value analysis was performed to determine the horizontal periods of the structure. The fundamental period of vibration were found equal to 0.5006 second. The first four mode shapes for horizontal direction are shown in Fig. 6.

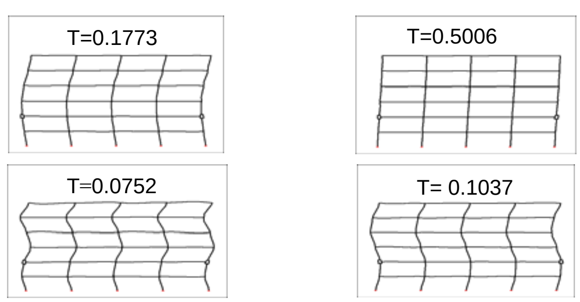


Fig 6 - First four mode shapes of the six-storey RC building

Selection of ground motion records

Six earthquakes records are selected to conduct dynamic analysis on the designed RC building. The criteria used in this selection are to cover a wide range of ground motion frequencies represented by the (A/v) ratio (the ratio between the peak ground acceleration and the peak ground velocity). The characteristics of the chosen records are presented in table 1.

Using a reliable method to scale the selected ground motion records is very important to conduct incremental dynamic analysis. Many methods have been proposed for scaling the ground motion records such as using the Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), and the 5% damped spectral acceleration at the structure's first-mode period [Sa(T1, 5%)]. Using Sa(T1,5%) to scale the records was found to be a reliable method[19, 20], T1 = 0.50056 second. Fig. 7 shows the scaled spectral acceleration for the earthquakes chosen for the analysis with the design spectra.

Table 1. The earthquake records used in the dynamic analyses

Earthquake	Date	Ms Magnitude	Station	PGA (g)	A/v
Northridge USA	17/1/94	6.7	Arleta-Nordhoff	0.34	Inter.
Imperial Valley USA	15/10/79	6.9	El Centro Array #6 (E06)	0.439	Low
Loma Prieta USA	18/10/89	7.1	Capitola (CAP)	0.53	High
Whittier USA	1/10/87	5.7	Whittier Dam	0.316	High
San Fernando	9/2/71	6.6	Pacoima Dam	1.23	Inter.
Morgan Hill (USA)	24/4/84	6.1	Gillory Array #2 (G02)	0.212	High

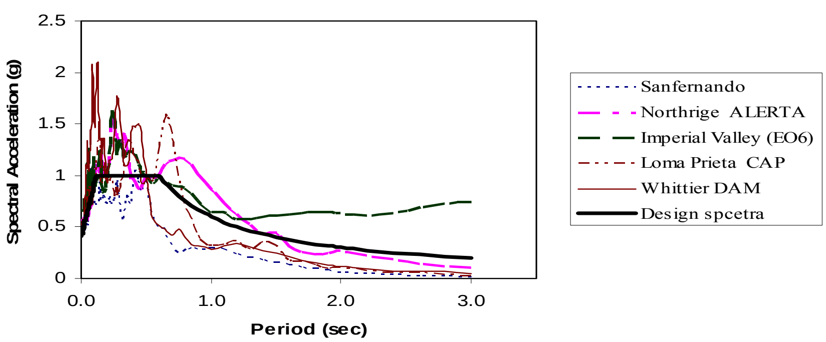


Figure 7 - Spectral accelerations

Each earthquake record is scaled to different Sa(0.50056 second, 5%) levels. The non-linear dynamic analysis was conducted using each scaled record. At each level of Sa the MID and the MRID values are tracked up to reaching the value of the collapse and residual collapse limit, respectively.

The local damage of the individual elements was tracked in term of yielding and crushing at the collapse limit using both the 3% MID and 2.4% MRID, respectively. A comparison between the damage experienced by the building at collapse defined using the two limits are illustrated in Fig. 8 for the six earthquake records. It can be noticed from the figure that the damage state of the building at 2.4% MRID is representing the actual collapse where there is a minimum of three columns that reached the crushing state. It can also be observed that the 3% MID is not representing the building failure. The figure shows that in case of Whittier and Morgan Hill records, the damage states at both 3% MID and 2.4% MRID are identical. This similarity is because at the same level of Sa in these two cases, the building reached its collapse limit corresponding to MID or MRID.

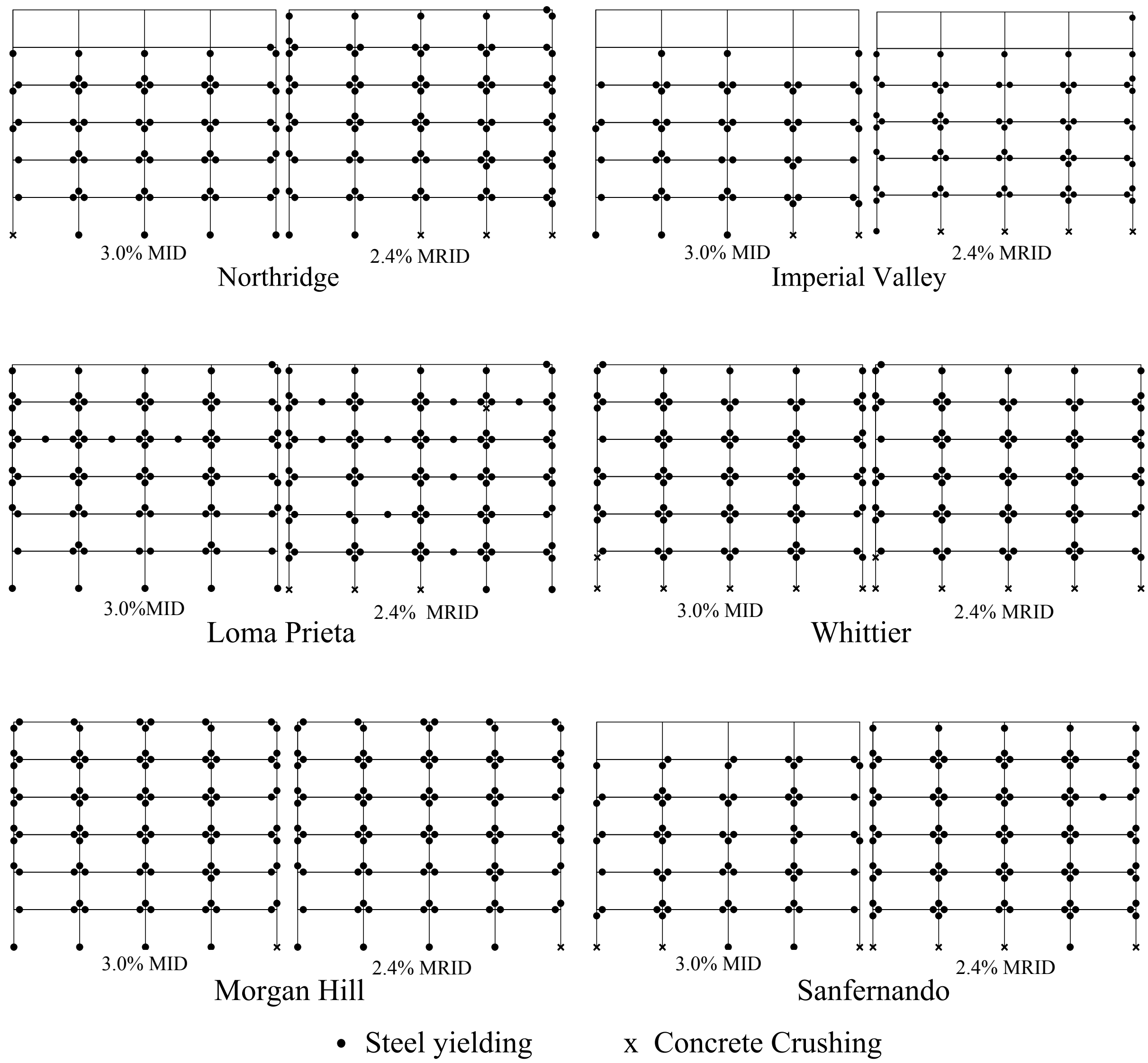


Figure 8 – Comparison between the damage at MID of 3% and at MRID of 2.4%

Table 2 presents some data obtained during the dynamic analysis, which may help in comparing the two methodologies. For each earthquake record, the table gives the number of the storey experiencing the MID and the one experiencing the MRID. The corresponding RID and inter-storey drift for the same storey are also given. The table shows that at 2.4% MRID, the corresponding inter-storey drift ratio is varying between 3.1% and 5.0%. These values for ID are much higher than the collapse limit, 3%, estimated using the pushover analysis. This can be explained with reference to figure 8 by the fact that during dynamic analysis and before reaching the residual collapse limit, 2.4% MRID, the ID may reach high values for an instance. In case of Morgan Hill earthquake the third storey sustained 6.24% ID while the RID at the end of the analysis was only 1.27 % at the same storey level. The MID seems to be very conservative and is not representing the actual damage of the structure following an earthquake.

Table 2 – Positions of MID and MRID and their corresponding RID and ID

Earthquake record	Storey experienced MID			Storey experienced MRID		
	Storey No.	MID (%)	Corresponding RID (%)	Storey No.	MRID	Corresponding ID (%)
Northridge	2nd	4.8	2.44	2nd	2.44	4.8
Imperial Valley	2nd	4.35	2.68	2nd	2.68	4.35
Loma Prieta	5th	4.9	2.25	5th	2.25	4.9
Whittier	1st	3.1	2.47	1st	2.47	3.1
Morgan Hill	3rd	6.24	1.27	1st	2.57	5
Sanfernando	2nd	5.25	2.56	1st	2.6	5

It can be noticed from the table that the positions of the MID and the MRID are affected by the frequency content of each earthquake. Figure 8 explains that collapse is occurring due to the crushing of the first storey columns in all the records, while table 2 shows that the MRID is not always happening in the first floor. It can be concluded that although both the MID and the MRID can be used for evaluating the global damage state of the building, they cannot predict the position of the maximum damage.

Conclusions

This paper focuses on evaluating the use of MID and MRID to assess the severity of seismic damage in RC framed buildings. A six-storey reinforced concrete structure is used for that purpose. Pushover analysis is used to estimate the building collapse limits in terms of MID and MRID. The building is then subjected to six earthquake records scaled to different intensities. The observe damage at these limits is compared.

The results from the dynamic analyses show that the MRID is representing accurately the building failure, while the MID is very conservative. The MID is reached during the dynamic analysis is instantaneous and is not expected to lead to collapse of the building. Although the MRID was found to be an accurate criterion to define the damage state of the building, it was concluded that it could not predict the position of the maximum damage. Further work is needed to develop a methodology capable of predicting the global damage and its location.

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