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PERFORMANCE-BASED DESIGN AND SEISMIC RESPONSE OF MASS IRREGULAR REINFORCED CONCRETE SPECIAL MOMENT FRAME (RC-SMF)

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ABSTRACT

This research work concentrates on the investigation of the seismic response of mass irregularities in Reinforced Concrete Special Moment Frame (RC-SMF) which were designed by performance-based plastic design method. This design method uses the modified energy-work balance concept for calculating the design base shear and beam-column design moment, as well as the inelastic state-based formula to distribute the lateral forces. Three 8-story RC-SMF structures with mass irregularity at 1st, 4th and 8th floor, respectively, and one 8-story RC-SMF structure without mass irregularity were considered in this study. These 4 RC-SMF buildings were subjected to 20 ground motions with 2% target drift and a seismic hazard level of 1% probability of exceedance in 50 years. The seismic response of these buildings was observed by Incremental Dynamic Analysis (IDA) with an intensity measure ranging from 0.1 to 3.5g using OpenSees software. It is found that the response curves ideally show the behavior of four RC-SMF structure under strong ground motions.

Keywords: *Mass irregular structure, Performance-based plastic design, IDA.*

1. INTRODUCTION

The perfect regular structure is an idealized concept to analyze the structure. However, real-life buildings are almost always irregular. Due to the usage of buildings, the common irregularities found in the structures are mass, geometry and stiffness irregularities. Hence, it is important to evaluate the seismic performance of irregular type structures that will provide the behavior of the structural member under strong ground shaking. A number of researches have been performed to investigate the irregularities effects in buildings during big earthquakes (Chopra and Goel 2004) on the limit capacity of buildings. Moreover, the damage occurs in code-compliant irregular type RC structure due to inelastic behavior shows the need for more research on designing procedure and seismic response evaluation of structures that have irregularities.

The current design code generally considers the elastic deformation although the structure undergoes large inelastic deformation under strong earthquakes. Additionally, the elastic analysis accounts the equivalent static design force to calculate the required strengths of elements. However, strength and deformation demand require proper inelastic behavior of elements to prevent undesired failure or collapse. Hence, the Performance-Based Plastic Design (PBPD) has been developed with a series of research works by several researchers. The important concept of this method is the target drift and yield mechanism used to achieve the target building performance and demonstrate the inelastic response of elements during a strong earthquake (Leelataviwat et al. 1999; Lee and Goel 2001). The modified work-energy concept is then used to calculate the design base shear and beam-column moment (Lee and Goel 2001). A lateral force distribution method that considers the inelastic response of the structure is used in this PBPD method for getting the more accurate seismic response (Chao et al. 2007). A number of comprehensive researches using PBPD method have been performed on the different type of structure like eccentrically braced frame, special moment truss, high-rise buckling-restrained braced frames, tall hybrid coupled walls, reinforced concrete special moment frame etc. (Chao and Goel 2005; Chao and Goel 2006; Chao and Goel 2008; Liao and Goel 2014; Chan-anan et al. 2016). From those studies, the seismic response of the structures which were designed by PBPD method have evenly distributed story drift over height while the structures designed with seismic code-based method have undesired story drift or soft story failure over height. However, research works have only been performed on Reinforced Concrete Special Moment Frame (RC-SMF). Therefore, further research is needed using this methodology for another type of RC structure like RC-SMF with irregularity (mass, geometry and stiffness), RC moment frame with shear wall, Reinforced Concrete Ordinary Moment Frame (RC-OMF), Reinforced Concrete Intermediate Moment Frame (RC-IMF) etc. In this study, the RC-SMF with mass irregularity is taken for designing and analyzing using the proposed seismic design method.

The main objective of this research was to apply the PBPD method to the mass irregular RC-SMF structures to analyze the seismic response of the structures through the IDA curve. Four 8 story RC-SMF structures were selected to design it's all elements using PBPD design method which has mass irregularity at 1st, 4th and 8th floor and one without mass irregularity, respectively. All four RC-SMF structures were assumed as multipurpose buildings, located in Padang Indonesia on soil type E (soft soil). The advantage of this method is to set hazard level and performance limit depends on the necessity of the structures and later evaluate the response of structures by performing the nonlinear time history analysis. Similarly, these structures were designed for 2% target drift with a seismic hazard level of 1% probability of exceedance in 50 years. The nonlinear model was implemented on OpenSees 2.5.0, a PEER's open-source structural analysis and simulation software. Moreover, the analysis results used to present as seismic responses of structures through incremental dynamic analysis (Vamvatsikos and Cornell 2002).

2. PERFORMANCE-BASED PLASTIC DESIGN METHOD

The Performance-Based Plastic Design (PBPD) method uses elastic-plastic single-degree-of-freedom-system to represent the work-energy balance concept which is more rational design concept than the code-based method which is developed by Leelataviwat et al. (1999) and modified by Lee and Goel (2001). The work-energy equation is made by external work equal to the internal work of an elastic-plastic single-degree-of-freedom-system. The external work is calculated by pushing the structure up to target drift monotonically which is equal to the energy need of an equivalent elastic-plastic single-degree-of-freedom (EP-SDOF) system as shown in Fig. 1. The work-energy balance equation is given by (Liao and Goel, 2014):

$$\left(E_e + E_p\right) = \gamma.\left(\frac{1}{2}M.S_v^2\right) = \frac{1}{2}\gamma.M\left(\frac{T}{2\pi}S_a.g\right)^2 \tag{1}$$

where E_e and Ep are elastic and plastic components of total energy needed, respectively, to push the structure up to the target drift. The γ is energy modification factor, T is natural period and M is total mass of the structure. The S_v and Sa are the design pseudo-spectral velocity and pseudo-spectral acceleration, respectively. The energy modification factor γ is obtained from the inelastic spectra (Lee and Goel 2001) as shown by Eq. 2.

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \tag{2}$$

where Rμ is ductility reduction factor and μs is structural ductility factor.

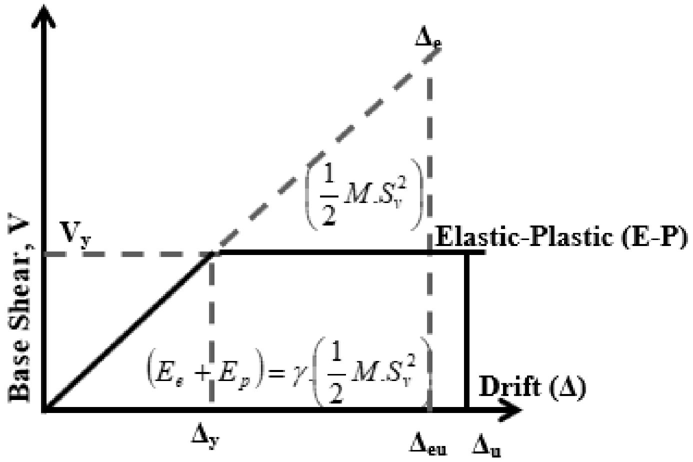

Fig. 1 Required energy by elastic-plastic-single degree-of-freedom system (work-energy balance concept)

Following Liao and Goel (2014), the design base shear formula of PBPD method is obtained from the work-energy balance equation (Eq. 1) and the simplified form of the equation is shown in Eq. 3.

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4.\gamma.S_a^2}}{2} \tag{3}$$

where V_y/W is design base shear coefficient, V_y is the design base shear, W is the total seismic weight of the structure, and α is a dimensionless parameter given by:

$$\alpha = \left(\sum_{j=1}^{n}(\beta_i - \beta_{i+1}).h_i\right).\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}} \left(\frac{\theta_p 8\pi^2}{T^2 g}\right) \tag{4}$$

where β is the shear distribution factor, θ_p is plastic component of target drift, w_n and w_j is top floor seismic weight and j^{th} floor seismic weight respectively, and h_n and h_j is the structure height at top floor and j^{th} floor respectively. T is calculated according to ASCE/SEI 7-16 (2017).

The lateral design force distribution is based on inelastic dynamic response and higher mode effects (Chao et. al. 2007) as follow:

$$F_i = (\beta_i - \beta_{i+1}).V_n \tag{5}$$

where Fi is lateral force at level i, and V_n is static story shear at top-level. The static story shear and lateral force at top-level are same, hence the formula is given in Eq. 6.

$$V_n = F_n = \left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j} \right)^{0.75T^{--0.2}} .V_y \tag{6}$$

The story shear distribution factor is the ratio between static story shear and static story shear at top-level as shown in Eq. 7.

$$\frac{V_i}{V_n} = \beta_i = \left(\frac{\sum_{j=1}^{n} w_j h_j}{w_n h_n} \right)^{0.75T^{--0.2}} \tag{7}$$

The main target of PBPD method is to provide sufficient strength to beam to get desired performance level. The plastic hinge is formed at the both end of beam and only at column base. The structural strength follows the story shear distribution along the building height to prevent the yielding from concentrating at upper story level (Chao et. al 2007). The moment capacity of beam at each level is determined by plastic design which is external work equal to internal as follow:

$$\sum_{i=1}^{n} F_i h_i \theta_p = 2.M_{pc}\theta_p + \sum_{i=1}^{n} 2\beta_i \left(M_{pb-positive} + M_{pb-negetive}\right).\gamma_i \tag{8}$$

where θ_p is kinematic rotation angle and M_{pc} is the plastic moment at column base for the yield mechanism, M_{pb} is beam positive and negative moment strength.

According to PBPD, the column strength is determined by taking factored gravity load and maximum beam strength. The joint condition is based on strong-column weak-beam concepts, which is the sum of nominal flexural strength of column to the sum of nominal flexural strength of beam is more than 1.2 according to ACI 318-14, and 1.3 according to FEMA P695. The shear force in interior and exterior columns can be obtained by using (Moehle and Mahin, 1991):

$$V_i = \frac{\left|M_{pr(+)}\right|_i + \left|M_{pr(-)}\right|_i}{L^{'}} + \frac{w_{i-tributary}.L^{'}}{2} \tag{9}$$

$$V_i^{'} = \frac{\left|M_{pr(+)}\right|_i + \left|M_{pr(-)}\right|_i}{L^{'}} - \frac{w_{i-tributary}.L^{'}}{2} \tag{10}$$

where M_{pr} is the maximum beam strength which is multiplied by an over strength factor (Moehle and Mahin 1991), $L^{'}$ is the beam length between to hinge, and w_i is the unit seismic weight. The lateral forces in exterior and interior columns can be obtained by using:

$$\alpha_i F_{L-ext} = \frac{\sum_{i=1}^{n} (M_{pr-})_i + \sum_{i=1}^{n} V_i \left(\frac{L - L^{'}}{2} \right)_i + M_{pc}}{\sum_{i=1}^{n} \alpha_i h_i} \tag{11}$$

$$\alpha_i F_{L-\mathrm{int}} = \frac{\sum_{i=1}^{n}\left(\left|M_{pr-}\right|_i + \left|M_{pr+}\right|_i\right) + \sum_{i=1}^{n}[V_i + V_i^{'}].\left(\frac{L-L^{'}}{2}\right)_i + 2M_{pc}}{\sum_{i=1}^{n}\alpha_i h_i} \tag{12}$$

3. MODELING AND ANALYSIS APPROACH

In this study, eight story SMF irregular concrete structures were considered. Three structures were chosen to represent mass irregularity at 8th, 4th, and 1st story, and denoted as MI-8, MI-4, and MI-1, respectively. One structure named as MI-S to represent the structure without mass irregularity. The number of bays is the same for all structures as shown in Fig. 2. The frame of grid line no. 3 was taken from all four structures to design by PBPD and perform nonlinear time history analysis to get the seismic responses of these frames (Fig. 3). The seismic weight in each floor was 1300 KN, except for 1st, 4th and 8th floor of MI-1, MI-4, and MI-8, respectively. The seismic weight of these floor was 2100 KN, which means the mass irregularity exists on that floor according to ASCE/SEI 7-16 (2017).

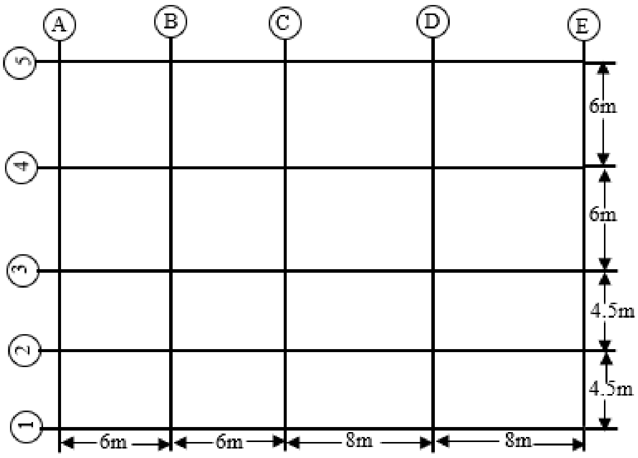

Fig. 2 Floor plan of MI-8, MI-4, MI-1, and MI-S

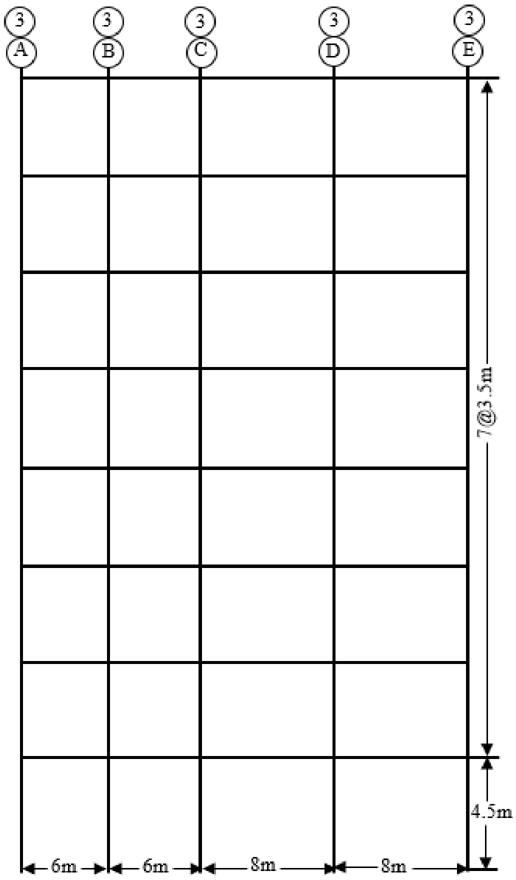

Fig. 3 Frame of grid line 3

The structures were considered as a multi-purpose building which located in Padang, Indonesia on soil type E. The design response spectrum (Sa) was taken from the Indonesia national earthquake database for soil type E as shown in fig. 4 (PUSKIM 2018).

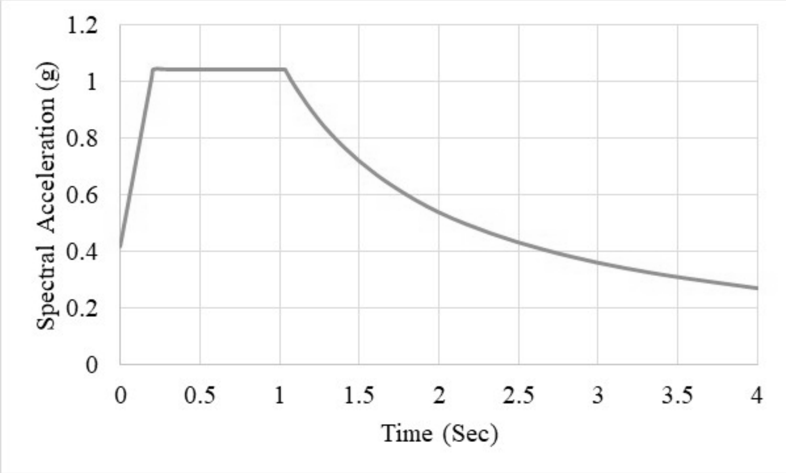

Fig. 4 Design response spectrum for soil type E in Padang, Indonesia

All structure's natural period was assumed the same as 1.35 sec following ASCE approximation formula. The total seismic weight and other parameters are given in Table 1. The design base shear varies from 2010 kN to 2170 kN, which was distributed as lateral force over structure height using Eqs. 5 -7.

Table 1 Design parameters of four structures

Structure, RC-SMF	MI-8	MI-4	MI-1	MI-S
Floor height, h (m)	29	29	29	29
Period, T (sec)	1.35	1.35	1.35	1.35
Total seismic weight, W (kN)	11342.4	11342.4	11342.4	10675.2
Dimensionless parameter, α	1.43	1.36	1.37	1.38
Spectral acceleration, S_a	0.797	0.797	0.797	0.797
Design Base Shear, V_y (kN)	2100	2170	2150	2010

In this research, the distributed plasticity approached was used to model the beam-column element in OpenSees. The main advantage of distributed plasticity is it has several integration points over element length as shown in Fig. 5. As a result, the integration points show yielding conditions over the length of the element. The Gauss-Lobatto integration approach places integration point at the both end of element; hence it is the most common way to evaluate the force-based element responses for distributed plasticity (Neuenhofer and Filippou, 1997; Scott and Fenves, 2006). In this study, fiber section with suitable number of patches and layers was used to define the beam-column sections and force-beam-column element. Five Gauss-Lobatto integration points along the element length was used to define a beam-column element. The uniaxial hysteretic constitutive material model namely ConcreteCM (Mander et. al 1988; Chang and Mander 1994) and Giuffre-Menegotto-Pinto steel material (Filippou et. al. 1983) was used to model the core and cover concrete, and steel rebars.

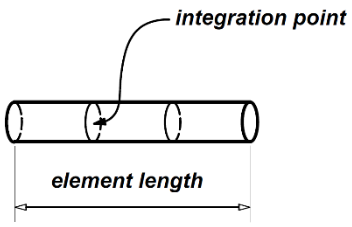

Fig. 5 Element with Gauss-Lobatto integration point

The nonlinear modelling needs earthquakes data to get seismic responses of structure. PEER ground motion database contains wide ranges of ground motions (PEER 2018). In this study, 20 ground motions with magnitude ranges M 6.0 to M 7.7 were selected to show the responses of the structures through IDA curves. To obtain IDA curve, the nonlinear analysis using OpenSees was performed, where the intensity measures started from 0.1g until structure collapse. The ground motions were scaled in different IM (Intensity Measure) level with 5% damping ratio. This step was done by using SeismoSingnal2018.

4. RESULTS AND DISCUSSION

The PBPD method considers the nonlinear behavior and target performance level into the design process from the start. The design output of the column and beam are shown in Figs. 6 to 9.

Firstly, the design base shear and lateral force distribution of MI-1, MI-4, and MI-8 are almost the same although the mass irregularity exists on different floors in these structures. As a result, the design outcome shows that the beam-column properties of these structures nearly similar. On the other hand, the MI-S contains 800 kN less seismic weight than the other three structures, therefore the design base shear and lateral force distribution is less. However, there are few dissimilarities in the beam-column size among these four structures. Finally, the nonlinear analysis performed in OpenSees shows the satisfactory performance of these structures under severe ground motions.

Secondly, the positive and negative beam reinforcement ratio of MI-1, MI-4, and MI-8 at 8^{th} floor is the same. Although, the MI-8 has an extra 800 kN more load at 8^{th} floor. The rebar ratio of the beam at 4^{th} floor of MI-1 and MI-4 is same, but mass irregularity exists at 4^{th} floor in MI-4. Moreover, the rebar ratio of beam at 1^{st} floor of MI-1 and MI-4 is same, however, the 1^{st} floor of MI-1 has extra load. The positive and negative moment of beam is calculated by eq. (8) which is not only based on seismic weight of structure but also based on the shear distribution factor and plastic component of target drift.

Thirdly, the column rebar properties are determined by eq. (9 to 12) which is based on maximum beam strength factor, shear force in column and distance between beam hinge. The column size of these structures is almost the same. The new design approach considers the strong column weak beam mechanism which is justified in nonlinear analysis by achieving the performance of structure. The uniform distribution of interstory drift showed that all columns dissipate earthquake energy properly and minimize the excessive inelastic deformation. The IDA curve (figs. 10 to 13)

The time history analyses of were performed to create incremental dynamic analysis (IDA) curve for each structure. The response of the structure plotted in IDA curve was the maximum interstory drift, and intensity measure (IM). The IM was spectral acceleration, S_a at T_1 with 5% damping ratio of the scaled ground motion. The S_a value range was 0.1g to 3.3g with an increment of 0.1g, but it depended on the structure collapse. The analysis results as IDA curve is shown in Figs.10 to 13. The IDA curve shows that maximum story drift lies within the target drift of all four structures. The drift of four structures increased linearly up to yield drift, and the structure collapsed after the target drift. The uniform distribution of story drift implies that the PBPD considers no soft story failure and it is justified in this study.

The mass irregularity at 4^{th} floor has less probability of collapse than other structure. In comparison between mass irregularity effect at 1^{st} and 4^{th} floor, the section size and rebar properties of these two structures are very similar. However, the collapse probability of the 1^{st} floor mass irregularity is higher than the 4^{th} floor mass irregularity. In addition, the interstory drift and probability of collapse due to mass irregularity at 8th floor was also higher than regular structure. Moreover, the seismic responses of these structures also showed that the maximum drift values are under yield drift in S_a value 1g.

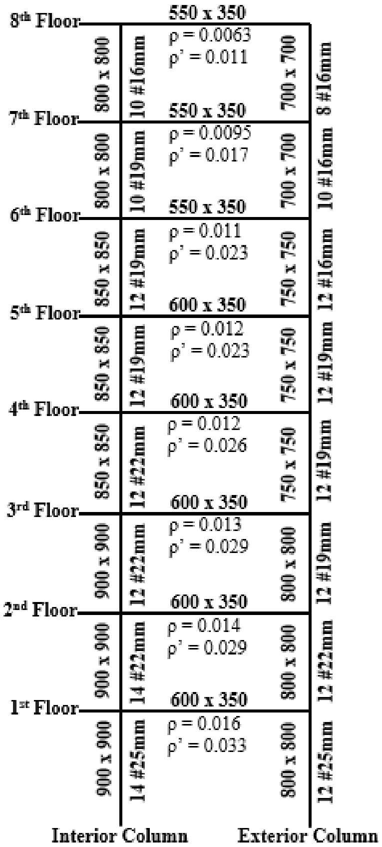

Fig. 6 Colum-beam properties of MI-1

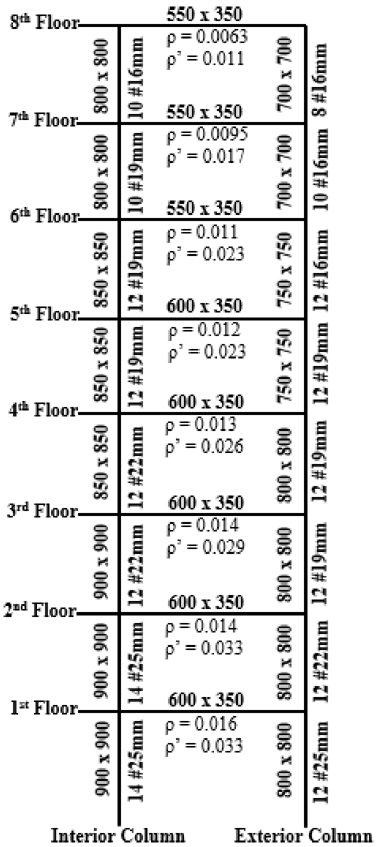

Fig. 7 Colum-beam properties of MI-4

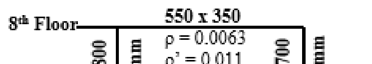

Fig. 8 Colum-beam properties of MI-8

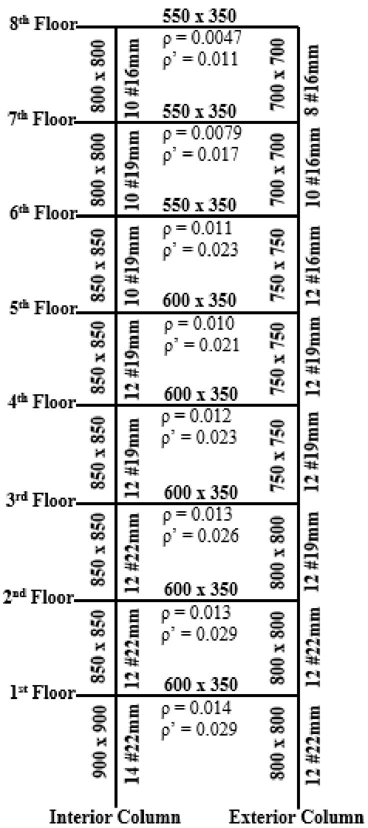

Fig. 9 Colum-beam properties of MI-S

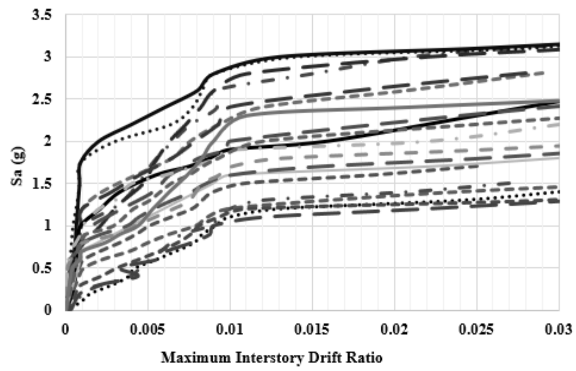

Fig. 10 IDA curve of MI-1(T_1=1.35sec)

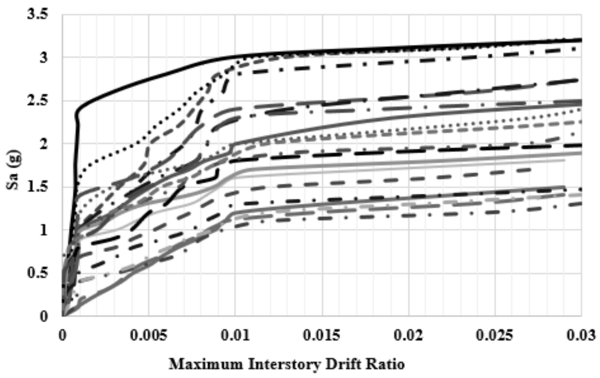

Fig. 11 IDA curve of MI-4 (T_1=1.35sec)

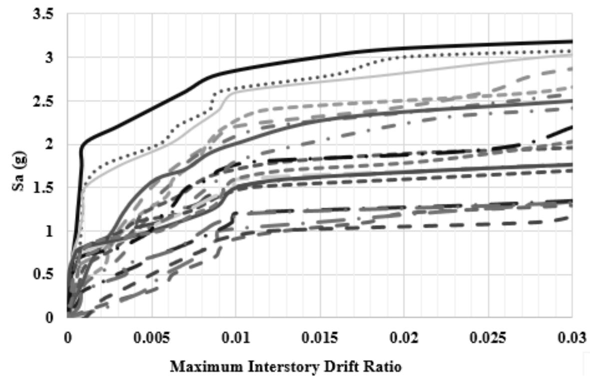

Fig. 12 IDA curve of MI-8 (T_1=1.35sec)

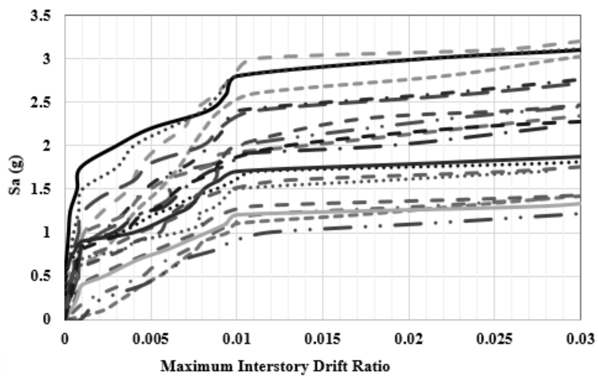

Fig. 13 IDA curve of MI-S (T_1=1.35sec)

5. CONCLUSIONS

The main objective of this study is to predict seismic performance of mass irregular and regular RC-SMF designed by PBPD method. The seismic responses from time history analysis are shown through IDA curves. The seismic performance of all four structure achieved the target performance hence it validated the PBPD method. The study needs to be extended for more cases and needs comparison with code based designed structure in order to get most suitable method for practice. The mass irregularity effects up to mid-floor has less chance of collapse than top floor. The interstory drift and collapse probability at spectral acceleration 1.5g is very low, but as spectral acceleration increased the collapse probability elevates highly.

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