Comparison of Seismic Responses for Reinforced Concrete Buildings with Mass and Stiffness Irregularities Using Pushover and Nonlinear Time History Analysis

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Abstract. Pushover analysis or also known as nonlinear static procedures (NSP) have been recognized in recent years for practical evaluation of seismic demands and for structural design by estimating a structural building capacities and deformation demands. By comparing these demands and capacities at the performance level interest, the seismic performance of a building can be evaluated. However, the accuracy of NSP for assessment irregular building is not yet a fully satisfactory solution, since irregularities of a building influence the dynamic responses of the building. The objective of the study presented herein is to understand the nonlinear behaviour of six story RC building with mass irregularities at different floors and stiffness irregularity at first story (soft story) using NSP. For the purpose of comparison on the performance level obtained with NSP, nonlinear time history analysis (THA) were also performed under ground motion excitation with compatible to response spectra design. Finally, formation plastic hinges and their progressive development from elastic level to collapse prevention are presented and discussed.

1. Introduction

Performance based seismic design has become popular among structural engineer society over the past twenty years due to its potential benefit in assessment, design and better understanding of inelastic structural behaviour during major earthquakes [1,2]. The key parameters of performance based seismic design lies the accurate estimation of seismic demand and structures capacity. This goal can be achieved only using either nonlinear static procedures (NSP) or by performing nonlinear dynamic time history analysis ([3-13]). The best way to investigate the seismic behavior of a structure under ground motion excitation is the nonlinear dynamic time history analysis (THA) that represents the most rigorous and accurate approach. Nonetheless, it should be kept in mind that the response is sensitive to the structural properties and input ground motion, therefore several analyses are required with increased complexity, computational costs and time consumption. This is the reason why NSP is widely used instead of THA, regardless its limitations for assessment seismic performance of a building.

Iwan [9] demonstrated the inability of pushover methods to predict demands for pulse-like near fault ground motions. Krawinkler and Seneviratna [10] declare that NSP procedures will provide insight into structural aspects that control performance under earthquake when implemented with caution and good judgment. For structure with first mode dominated, NSP will very likely provide good estimates of global and local inelastic deformation demand. Furthermore, Goel and Chadwell [8] investigate of current NSP for concrete building using recorded strong motion data. It was confirm that the NSP either underestimate or overestimate the peak roof displacement for several buildings under consideration, poor estimate of drifts in upper stories due to higher mode effect, and unreliable estimate of story shear and overturning moments. Lignos and Gantes [12] performed improvement pushover called modal pushover analysis (MPA) and THA for structure with irregular mass distribution. The results revealed that story drift is significant difference if the structure is elastic, demands are under estimated at the upper stories. For inelastic state, MPA shows adequately results in the upper stories. Base shear is overestimate and tendency increasing if mass discontinuity is located at lower stories. Moreover, Lu et al.[14] studied applicability of NSP to evaluate seismic performance of steel arch bridge, and it was noted that NSP acceptable accurate in prediction displacement capacities compare to THA. Currently, the applicability of NSP as a tool for evaluate seismic behavior of structural building is still pros and cons among of structural engineering community.

The main objective of this paper to study comparison of seismic performance of six story reinforced concrete structure with mass irregularity and stiffness or strength irregularity using NSP and THA. In order to verify the applicability of pushover procedures for estimating the overall seismic demands, the pushover results from the frames are compared with the dynamic analysis. In addition, to provide a realistic basis for this comparison, the selection and scaling of the ground motions used are carefully assessed with regard to the design spectrum. Mass irregularity is the presence of heavy mass on certain floor of a building. In this case, additional mass equal to 50% of uniform mass each story are imposed to second, fourth , and sixth story, whereas for stiffness irregularity is designed at the first story by increasing the height of first story columns. The comparison between both nonlinear analysis in term of response quantities, i.e., story displacement, inter-story drift, and base shear are presented and compared. Additionally, formation plastic hinge at the first yield to collapse prevention are also described.

2. Nonlinear static procedures (NSP)

Although THA is the most accurate analysis to evaluate seismic demand, the application NSP is generally considered to be more appropriate for seismic design due to its simplicity and ease to use. This method is based on assumption that the response of the MDOF structure can be related to the response of an equivalent SDOF. This is the reason why the NSP is known as the most use tool in the engineering practice for assessment of seismic behaviour of structures, and currently has resulted in guidelines such as ATC-40 [2], FEMA-356 [4], and FEMA-440 [5] and standards such as ASCE 41-13 [1].

NSP is conducted by applying the gravity loads followed by lateral load is gradually increased along a direction under consideration. The investigated building is pushed according to predefined lateral load pattern to obtain the target displacement. At each load step, the base shear and the roof displacement can be plotted to created capacity curve or pushover curve. It results is assumption of maximum base shear that the structure is capable to withstand during earthquake event. The distribution of lateral load pattern along the height of the structure and nonlinear modelling assumption led to different pushover curves.

2.1. Lateral load pattern

There are three lateral load pattern proposed in FEMA-356, namely (a) inverted triangular distribution, (b) uniform distribution, (c) distribution of forces proportional to fundamental mode (mode 1). Ghaffarzadeh et al.[7] studied response seismic demand of RC frames using NSP. The results show that push (a) pattern and push (c) pattern yielded similar results and reasonably accurate estimates of the maximum displacement. Although, slightly overestimate in the upper stories, while push (b) pattern overestimate demands at the lower stories. Moreover, the applicability lateral load pattern on evaluation of seismic deformation demands using NSP were investigated by Kunnath and Kalkan [11]. It was found that in all cases, push (a) pattern provided closest results to the mean time history analysis, and other two load patterns tend to overestimate demands at the lower stories.

IOP Conf. Series: Materials Science and Engineering **180** (2017) 012145 doi:10.1088/1757-899X/180/1/012145

2.2. Structural performance level

The seismic performance of a building structure is measured by the stage of damage under a certain seismic hazard in which is quantified by roof displacement and deformation of the structural members. Figure 1 shows force-deformation relation for plastic hinge in pushover analysis. This guidelines and standards previously mentioned define force-deformation criteria for potential locations of plastic hinge. There are five points labelled A, B, C, D, and E are used to define the force-deformation behaviour of the plastic hinge, and three points labelled IO (immediate occupancy), LS (life safety), and CP (collapse prevention) are used to define acceptance criteria for the hinge. There are six levels of structural performance in ASCE 41-13, i.e., Immediate occupancy (S-1), Damage control range (S-2), Life safety (S-3), Limited safety range (S-4), Collapse prevention (S-5), and Not considered (S-6) . Two levels of seismic hazard are commonly defined for buildings, namely (a) design basic earthquake (DBE): an earthquake with a 10% probability in 50 years of being exceeded. This is an earthquake with a 500 years reoccurrence period, and (b) maximum considered earthquake (MCE): an earthquake with a 2% probability in 50 years of being exceeded. This is an earthquake with a 2500 years reoccurrence period. The case study building were designed based on design spectrum that constructed for DBE seismic hazard level according to Indonesian seismic design. Target displacement in this study is determined based on the displacement coefficient method defined in ASCE 41-13.

2.3. Procedures to determine target displacement

The displacement coefficient method currently documented in FEMA-440 and adopted in the ASCE-41-13 standard is the improvement of the basic displacement procedures in FEMA-356. This method is accomplished by modifying the elastic response of equivalent SDOF system with coefficient $C_0, C_1,$ and C_2 is expressed as:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^{\ 2}}{4\pi^2} g \tag{1}$$

where, S_a is response spectrum acceleration at the effective fundamental period and damping ratio of the building, g and is gravity acceleration, T_e is effective fundamental period computed from

$$T_e = T_i \left(K_i / K_e\right)^{0.5} \tag{2}$$

in which K_i, K_e is the elastic stiffness and the effective stiffness of the building in the direction under consideration, respectively, obtained by idealizing the pushover curve as a bilinear relationship, C_0 is modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system obtained from table 7-5 in ASCE 41-13, C_1 is modification factor to relate expected maximum inelastic displacement to displacement calculated for linear elastic response computed from

$$C_1 = \begin{cases} 1.0; & T_e > 1.0s, \\ 1 + \dfrac{R-1}{aT_e^{\ 2}}; & 0.2s < T_e \le 1.0s, \\ 1 + \dfrac{1}{800}\left(\dfrac{R-1}{T_e}\right)^2; & T_e \le 0.2s \end{cases} \tag{3}$$

where a is equal to 130 for soil site class A and B, 90 for soil site class C, and 60 for soil site classes. D, E, and F, and R is the ratio of elastic and yield strengths is given as follows:

$$R = \frac{S_a}{V_y / W} C_m \tag{4}$$

in which V_y is the yield strength estimated from pushover curve, W is the effective seismic weight, and Cm is the effective modal mass factor at the fundamental mode of the building, C_2 is modification factor to represent the effect of pinched hysteretic shape, stiffness degradation, and strength deterioration on maximum displacement response computed from

$$C_2 = \begin{cases} 1.0; & T_e > 0.7s \\ 1 + \dfrac{1}{800}\left(\dfrac{R-1}{T_e}\right)^2; & T_e \le 0.7s \end{cases} \tag{5}$$

To avoid dynamic instability, ASCE 41-13 limit the R as

$$R \le R_{\max} = \frac{\Delta_d}{\Delta_y} + \frac{|\alpha_e|^{-h}}{4};\ h = 1.0 + 0.15\ln(T_e) \tag{6}$$

in which Δ_d is the deformation corresponding to peak strength, Δ_y is the yield deformation, and α_e is the effective negative post yield slope given by

$$\alpha_e = \alpha_{P-\Delta} + \lambda(\alpha_2 - \alpha_{P-\Delta}) \tag{7}$$

where α_2 is the negative post yield slope ratio and $\alpha_{p-\Delta}$ is the negative slope ratio caused by *P*-Δ effects defined in Figure 2, and λ is the near-field effect factor given as 0.8 for $S_1 \ge 0.6$ and 0.2 for $S_1 < 0.6$ (S_1 is defined as the 1-second spectral acceleration for the Maximum Considered Earthquake).

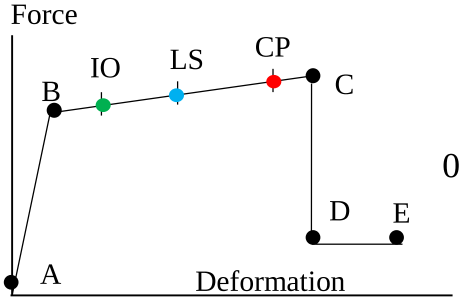


Figure 1. Force-deformation for pushover hinge

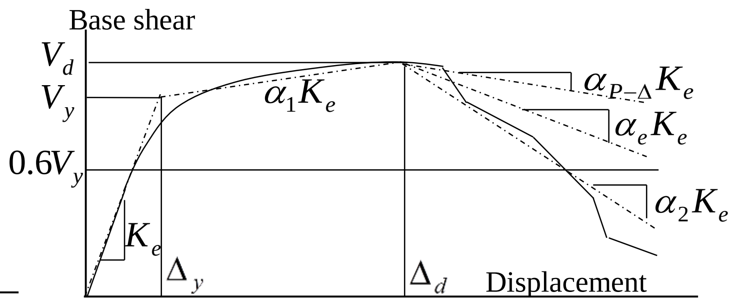


Figure 2. Idealize force-deformation curves

3. Description of buildings used in evaluation

The building selected for this study is six story of three bay reinforced concrete (RC) frames as shown in Figure 3. Four cases of building with different mass, i.e., building having uniform mass is designated as M_0, building having irregular mass is introduced at second, fourth , and sixth story designated as M_2, M_4, and M_6, respectively. Mass irregularity at an individual floor is considered by a 45 kN/m design load, with respect to 30 kN/m of the other floors, while live load for all building are equal to 15 kN/m, and the corresponding first mode vibration periods are 1.57s, 1.59s, 1.68s, and 1.76s, respectively. For building with stiffness irregularity at first story is similar to M_0 but the first story height is defined 5m, and denoted as S_T, with fundamental period of 1.67s. The reinforcement of the beams and columns are listed in table 1 and table 2. All concrete columns are specified to 40 MPa, all concrete beams are designed 350x700mm of 30MPa, while yield stress for all reinforcement is 400 MPa. The pulse type ground motion records of Northridge earthquake compatible to response spectrum design as shown in Figure 4 were applied in THA.

4. Analytical model

The computer program SAP 2000 Ver.10.1 was used to perform finite element analysis of the RC frame using either NSP or THA. The structure is modeled as a 2D assemblage of elements connected at nodes. Lumped mass is applied at nodes and node has 3 DOF. One dimensional element is used in the finite element meshing, and foundation were modeled as fixed support at ground level. To account the inelastic behavior of beams and columns component, both end of each element are modeled as concentrated plastic hinge as described in FEMA-356. Non-linier geometry option was also turn on to consider effect P-delta. For THA, Rayleigh damping was constructed through the first and the third mode of vibration were specified 5% of critical damping. Direct integration method using Newmark-Beta was selected for numerical solution. In addition, for NSP procedures, lateral load pattern is selected as inverted triangular.

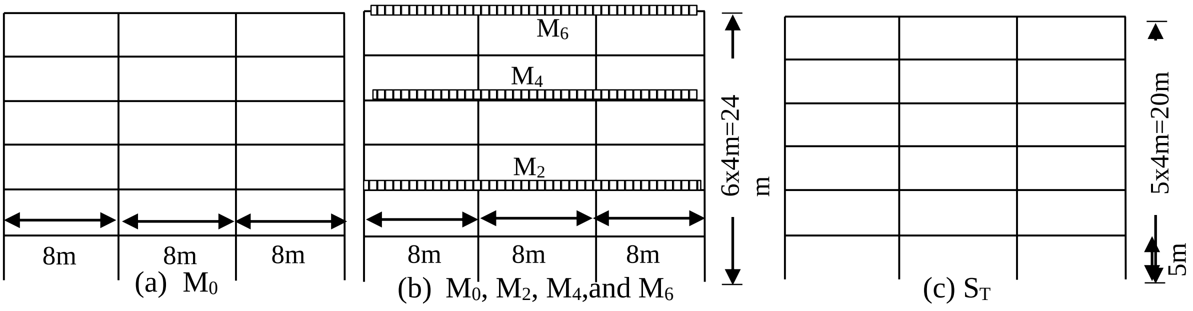


Figure 3. Schematic of elevation for the 6 story RC frames

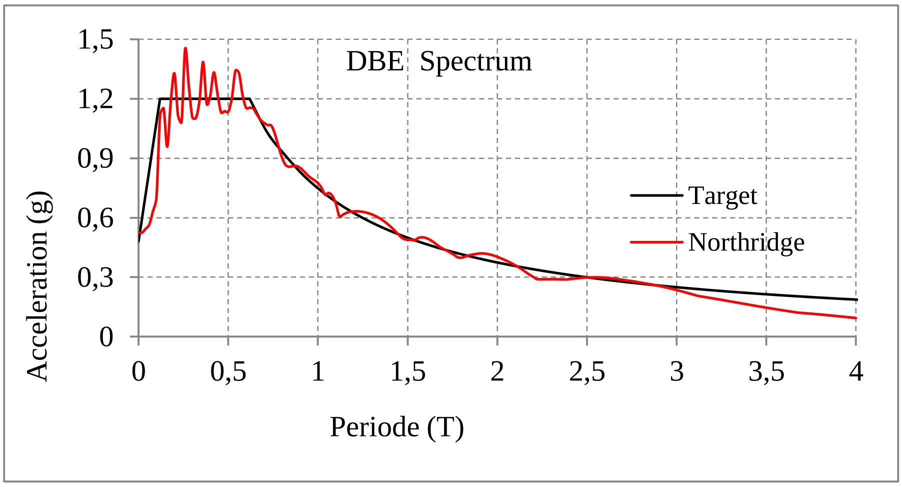


Figure 4. Scale ground motion to spectrum design

Table 1. Detail of beam reinforcement

Frames type	Story 1	Story 2	Story 3	Story 4	Story 5	Story 6
M_0, S_T	5D22/ 3D22	4D22 +2D19/ 3D22	4D22 +2D19/ 3D22	4D22 +1D19/ 3D22	2D22 +3D19/ 3D22	2D22 +3D19/ 3D22
M_2	4D22 +2D19/ 3D22	6D25/ 3D25	4D22 +2D19/ 3D22	5D22/ 3D22	6D19/ 3D19	6D19/ 3D19
M_4	4D22 +2D19/ 3D22	6D22/ 3D22	6D22/ 3D22	4D25 +2D2/ 3D35	6D19/ 3D19	6D19/ 3D19
M_6	4D22 +2D19/ 3D22	6D22/ 3D22	6D22/ 3D22	4D22 +2D19/ 3D22	2D22 +4D19/ 3D22	6D22/ 3D22

5. Discussion of results

5.1 Pushover curves and global performance level

Figure 5 present pushover curves obtained from NSP. These curves provide global behavior of the frame. The slope of curves is gradually reduced with increase the lateral displacement. It demonstrates that all the pushover curves show reasonable similar trend, in which yielding and performance point occur at the almost the same displacement except for irregular mass frame in sixth story (M_6). For M_6 frame yielding and performance point occur at the displacement larger than other frames. Furthermore, the smallest and the largest displacement at the performance point occurred for uniform mass frame (M_0) and for irregular mass frame in sixth floor, respectively. A comparison of NSP and THA in term of roof displacement, base shear and global performance level are listed in table 3. It was noted that NSP poor estimate in roof displacement and base shear (at the performance point) compare to THA. Additionally, the roof displacement at near collapse obtained from NSP close to the results of THA. Also, the results indicates the displacement ratio is more inaccurate than base shear ratio. Moreover, NSP provides global performance of all frames fall into damage control (S-2), whereas THA exhibit global performance level of all frames is within the Limited safety range (S-4). Furthermore, the capacity curve (base shear) of frame M_6 and frame S_T obtained from both methods are smaller than others frames. This demonstrated that heavy mass in placed on the top story and frame that having soft story provide poor seismic performance

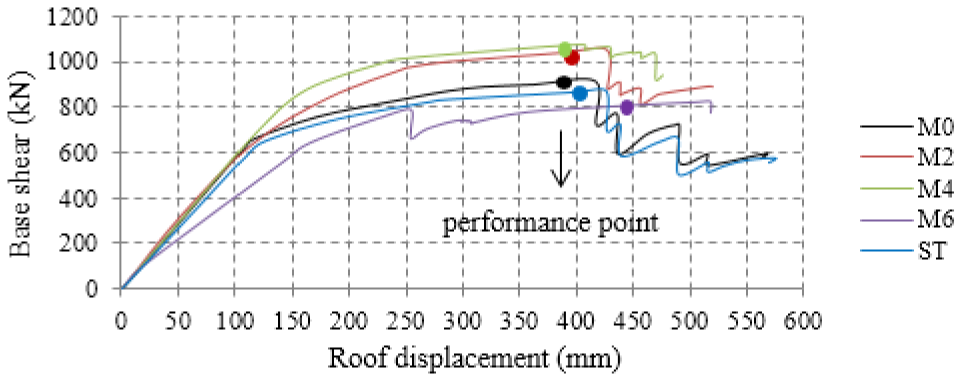


Figure 5. Pushover curves (capacity curves)

Table 2. Detail of column reinforcement

Frames type	Columns size & reinforcement		
	Stories 1&2 600x600mm	Stories 3&4 500x600mm	Stories 5&6 450x600mm
M_0, M_2, M_4, M_6, and S_T	20D-22mm	16D-22mm	12D-22mm

Table 3. Comparison of NSP to THA

Frames type	NSP (at the performance point)		NSP (at near collapse)	THA		NSP (performance point) / THA		Global Performance level	
	BS (kN)	Disp. mm (%)	Disp. mm (%)	BS (kN)	Disp. mm (%)	BS ratio	Disp. ratio	NSP	THA
M0	892	328 (1.37)	568 (2.4)	1254	499 (2.08)	0.71	0.66	S-2	S-4
M2	1019	335 (1.40)	520 (2.2)	1333	513 (2.14)	0.76	0.65	S-2	S-4
M4	1049	331(1.38)	476 (2.0)	1315	610 (2.54)	0.80	0.54	S-2	S-4
M6	788	386 (1.61)	518 (2.2)	1177	574 (2.39)	0.67	0.67	S-2	S-4
ST	852	343 (1.43)	575 (2.4)	1124	557 (2.32)	0.76	0.63	S-2	S-4

5.2 Story displacement and inter-story drift demand

Figure 6 and Figure 7 show comparison of story displacement and inter-story drift obtained from NSP and THA. In general, the Figure indicates that story displacement and inter-story drift exhibit identical shapes regardless of analysis method utilized , and the maximum inter-story drift location close to middle story. Furthermore, in term of inter-story drift and corresponding to local performance level in third story, NSP procedures reveal that frame M_6 produce limited safety range (S-4) performance level , while frame M_2 and M_0 exhibit life safety (S-3) performance level. The remaining frames exhibit damage control (S-2) performance level. Moreover, the story displacement and inter-story obtained from NSP are always smaller than the results of THA.

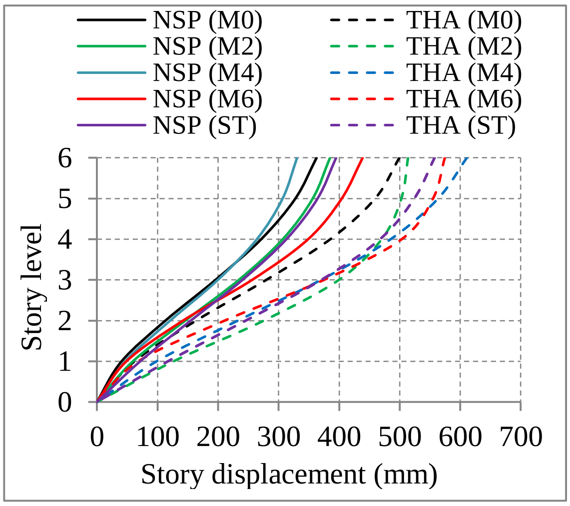


Figure 6. NSP vs. THA for story displacement

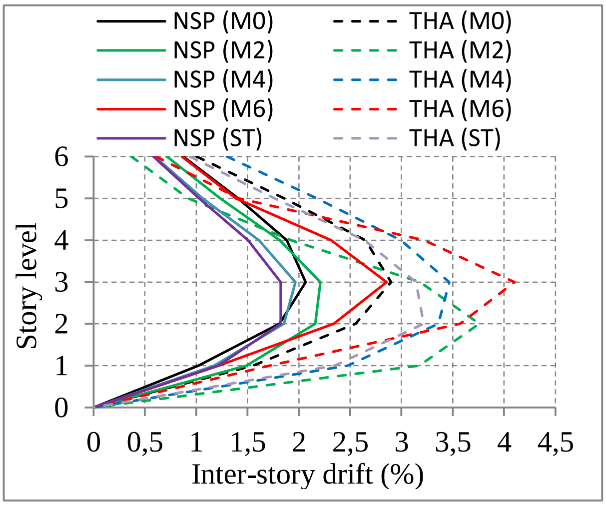
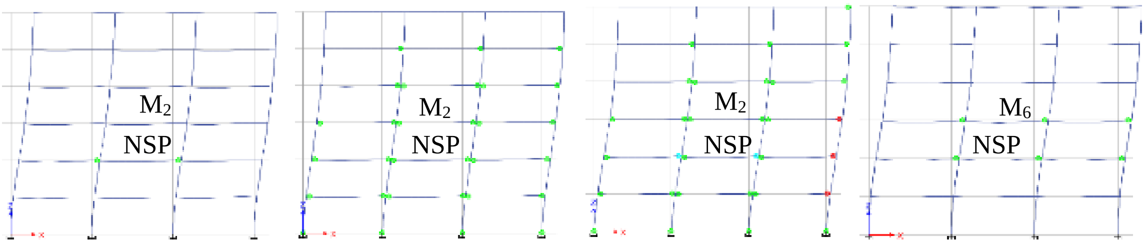


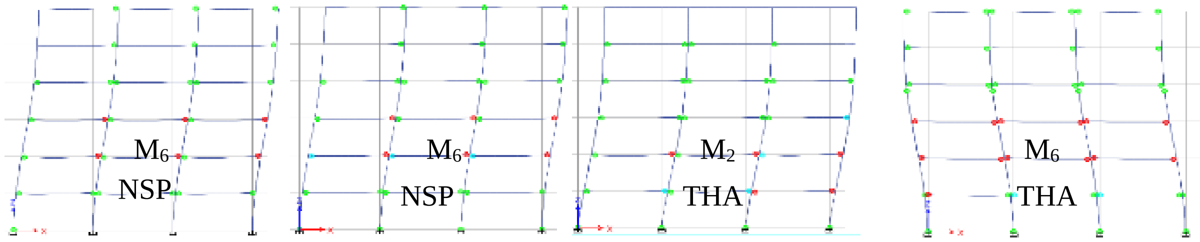
Figure 7. NSP vs. THA for inter-story drift

5.3 Formation and status of plastic hinges

The formation and status plastic hinge and deformed shape for M_2 and M_6 frames obtained from NSP and THA displayed in Figure 8. The frame M_2 was selected as representation of M_0, M_4, and S_T frames, since those frames demonstrate acceptable similar behavior, and due to space limitation. The hinge formation shows the strong column-weak beam failure pattern as expected. The plastic hinges at the first yield formed on second floor for M_2 frame, while for M_6 frame occurs on the second and third floors simultaneously. Additionally, local performance in element or story level can be observed through plastic hinge status. NSP shows all beams in frame M_2 are within the IO (S-1) level, while in frame M_6 vary, i.e., IO (S-1), LS (S-3), and CP (S-5) levels. Furthermore, THA present the local performance level for both frames fall into IO to CP levels. Moreover, M_6 frame experience more damages compare to M_2 frame using both methods. Furthermore, the status and formation of the hinges formed on the frames indicate that severe damages concentrated at beams on the lower to the middle floors.



(a) At first yield (b) At target disp. (c) At maximum disp. (d) At first yield



(e) At target disp. (f) At maximum disp. (g) At expected disp. (h) At expected disp.

Figure 8. Formation and status of plastic hinges at difference stages of the displacement.

6. Conclusion

The principal objective this paper is to evaluate the accuracy NSP procedures in comparison to THA, in order to estimate seismic demand of six story reinforced concrete buildings (RC frames) with uniform mass, irregular mass and first soft story. There are three generic frames where mass irregularities is introduced at the second, fourth, and sixth stories, one frames having uniform mass, and one frame designed with the soft first story were investigated. The general results confirm that response quantities were influenced by mass irregularity in placed at different story and stiffness irregularity at first story. Although, Pushover procedures or NSP demonstrate insufficient to estimate target displacement at the roof and base shear demand, however, NSP provide better understanding inelastic frames behaviour that can be used to identify weak elements or story through plastic hinges status and formation. In this case study, the differences between NSP and THA is more significant in roof displacement rather than base shear and the results obtained from NSP are always smaller than results of the THA. Moreover, in term of global performance level, NSP produce damage control (S-2) performance level, while THA exhibit limited safety range (S-4) performance level for the whole frame cases. Evaluation of the frames in local performance level (plastic hinge status) indicate that frame M_6 varies from IO to the CP performance level, and the remaining frames are identical performance level (damage control). Furthermore, both procedures indicate that frames having mass irregularity in top story (frame M_6) or having soft first story (frame S_T) show the poor performance level (lower base shear capacity and higher displacement demand). Neglecting the higher modes effect, ignore the potential redistribution of inertia forces and simplicity in assumption may lead to NSP procedures significant underestimation, as

IOP Conf. Series: Materials Science and Engineering **180** (2017) 012145 doi:10.1088/1757-899X/180/1/012145

yielding and cracking govern the inelastic structural behaviour. Although, pros and cons among of structural engineers in accuracy of NSP for evaluate seismic demand of structural building, the need of improvement NSP procedures is to discuss the limitations and shortcomings involved in the use of current ones.

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