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To cite this article: H Herryanto *et al* 2021 *IOP Conf. Ser.: Earth Environ. Sci.* **907** 012007

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Alternative approach in Partial Capacity Design (PCD) by using predicted post-elastic story shear distribution

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Abstract. The Capacity Design Method is an approach widely used to design earthquake resistant structures. It allows the structures to dissipate earthquake energy by forming plastic hinges through beam side sway mechanism. In the design process, the columns need to be designed stronger than the beams connected to them. Several previous studies have been conducted to propose alternative method allowing partial side sway mechanism namely the Partial Capacity Design (PCD) Method. In this method, selected columns are designed to remain elastic and the plastic hinges are allowed to occur only at the columns base. These columns are designed to resist increased forces. Despite of some successful attempts, PCD method still needs to be developed because sometimes the intended mechanism was not observed. This study proposes a new approach to improve the Partial Capacity Design (PCD) method. Symmetrical 6 and 10 story buildings with 7 bays are analyzed using seismic load for city of Surabaya. Structure behavior under non-linear static analysis is well predicted by this approach. However, under non-linear dynamic analysis, a few unexpected plastic hinges of elastic columns were observed at upper stories. But it should be noted that the earthquake used for performance analysis (maximum considered earthquake) is 50% larger than the one used for design (earthquake level corresponding to elastic design response spectrum).

1. Introduction

The Capacity Design Method is an approach widely used to design earthquake resistant structures where the desired failure mechanism is beam side sway mechanism as illustrated in Figure 1 (illustration in ATC 40 [1]). This method applies strong column-weak beam concept to prevent soft story failure during earthquake. In the design process, the columns need to be designed stronger than the beams connected to them. Several previous studies have been conducted to propose alternative methods based on Partial Capacity Design (PCD).

The idea of alternative method of PCD is started from previous study about gravity load dominant structure. According to Paulay and Priestley [2], in low-rise reinforced concrete frames, especially those with long-span beams, beams design are often governed by gravity load rather than seismic requirement. Application of capacity design will result excess column capacity and the structure possesses excess lateral capacity. In such case, development of plastic hinge in several columns are allowed to attain lower lateral capacity and partial side sway mechanism as illustrated in Figure 2.
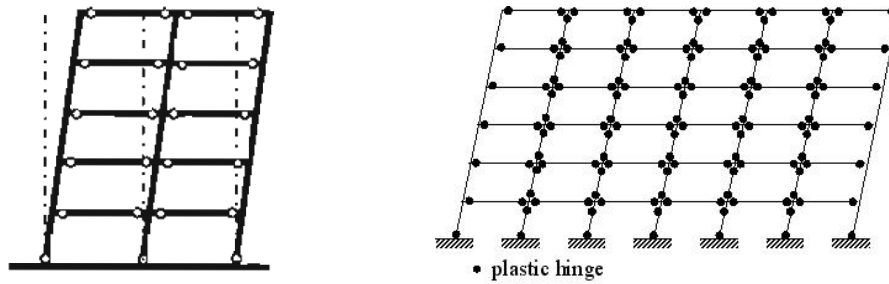
Figure 1. Beam side sway mechanism [1].   **Figure 2.** Partial side sway mechanism [3].

Muljati et al. [3], Muljati dan Lumantarna [4, 5] developed an alternative method of PCD that propose more practical design procedure than those in capacity design, where the column-beam capacity ratio is unnecessary to be checked. This method allows interior columns and beams to develop plastic hinges while maintains exterior columns to remain elastic during targeted seismic load. The targeted failure mechanism is partial side sway mechanism as shown in Figure 2. Magnification factor is used to magnify exterior column design forces to meet such condition. This method needs further research that in some case studies it failed to meet the desired failure mechanism.

This paper proposes a new approach in designing the elastic columns (exterior columns). The idea is to increase the story shear force for each story individually using the concept from previous studies. The increase of story shear force results in increase of elastic column shear force that in turn be used to determine bending moment of the column with assumption of cantilever behavior. By using this method, structure failure mechanism is expected to meet partial side sway mechanism.

2. Partial Capacity Design (PCD)

Muljati and Lumantarna [5] proposed an alternative to PCD method. In this method, interior columns are designed using nominal seismic load. Thus, exceeding seismic load during targeted seismic level will be sustained by exterior columns derived as :

$$n_{ex}S_{ex}^T = V_t^T - f_1 n_{in}S_{in}^N \tag{1}$$

where n_{ex} and n_{in} are the total number of exterior and interior columns, respectively; S_{ex}^T is the shear force in the exterior column due to the target seismic load; S_{in}^N is the shear force in the interior column due to the nominal seismic load; f_1 is the overstrength factor; and V_t^T is the total base shear due to the targeted seismic load. The load distribution in PCD is illustrated in Figure 3.
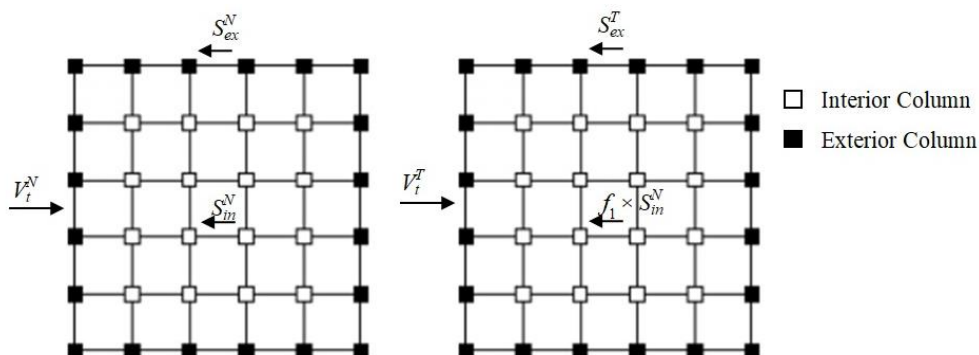

Figure 3. Load distribution in partial capacity design due to (a) nominal earthquake (b) targeted earthquake.

Magnification Factor (MF) is a ratio of exterior column shear forces due to targeted and nominal seismic loads, derived as:

$$FP = \frac{\frac{C^T}{C^{500}}\mu - f_1 n_{in} R_{in}^N}{n_{ex} R_{ex}^N} \qquad (2)$$

where C^T is the spectral acceleration of the target seismic load; C^{500} is the spectral acceleration of a five hundred years return period earthquake (Elastic Design Response Spectrum); μ is the structural ductility; n_{in} and n_{ex} are the total number of interior and exterior column; R_{in}^N and R_{ex}^N are the ratio of interior and exterior columns base shear to the total base shear due to the nominal seismic load.

During targeted seismic load, structure is designed to be in nonlinear condition so that C^T need to be determined in nonlinear response spectrum using structural plastic period T_p. Unfortunately, the nonlinear response spectrum is not provided in the code. Therefore, it is proposed to determine C^T from elastic design response spectrum by using pseudo plastic period, $T_{plastic}$ (non-linear period which corresponds to equivalent total base shear of real building with the elastic response spectrum) as illustrated in Figure 4 according to:
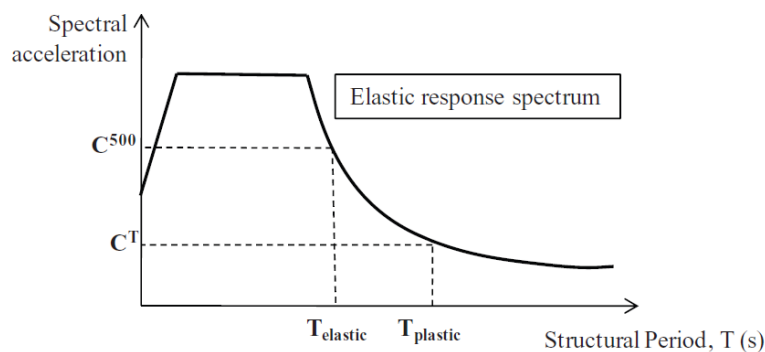
$$T_{plastic} = 2.969 T_{elastic} + 0.313 \qquad (3)$$

Figure 4. Determination of spectral acceleration in PCD [5].

3. Development of Partial Capacity Design (PCD)

This study proposes design procedure based on previous study by Muljati and Lumantarna [5]. Beams and plastic columns (interior columns) are designed using nominal seismic load from Elastic Design Response Spectrum (EDRS) following provisions in SNI 1726:2012 [6], except the minimum column-beam capacity ratio. Elastic columns (exterior columns) are designed by increasing column forces using additional story shear force. Load distribution in this method is illustrated in Figure 5.
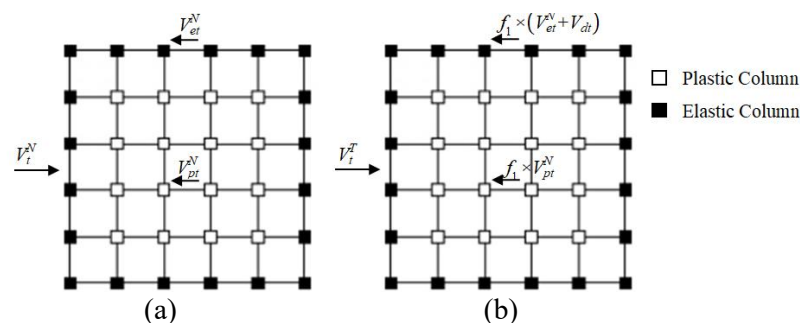

Figure 5. Load distribution in developed partial capacity design partial capacity design due to (a) nominal earthquake (b) targeted earthquake.

In design process, plastic columns are designed using nominal seismic load. During targeted seismic load, plastic columns are predicted to sustain shear force equal to design shear from nominal seismic load including their overstrength factor. Thus, shear force exceeding nominal seismic shear will be sustained by elastic columns as in Figure 5 (b). The exceeding shear force is treated as additional story shear to nominal seismic story shear. The additional story shear for design is derived as:

$$\frac{V_t^T}{f_1} - V_t^N = \left(V_{et}^N + V_{dt} + V_{pt}^N\right) - \left(V_{et}^N + V_{pt}^N\right) \tag{4}$$

$$V_{dt} = \left(\frac{C^T}{C^D}\mu - 1\right)V_t^N \tag{5}$$

where V_{dt} is total additional story shear; V_{et}^N and V_{pt}^N are total shear force of elastic and plastic columns due to nominal seismic R=8; V_t^T and V_t^N are total story shear due to targeted seismic load and nominal seismic load R=8; C^T and C^D are spectral acceleration of the target seismic load and the design seismic load; and R is response modification coefficient ($f_1\mu$ as in SNI-03-1726-2002 [7]). Similar to the previous study, C^T is determined in elastic design response spectrum using $T_{plastic}$ in Equation 3.

The total additional story shear is used to calculate additional shear force of elastic columns regarding the stiffness of each column. The additional shear force then be used to calculate additional bending moment of elastic columns. During design seismic load or targeted seismic load as well, a portion of bending moment of elastic columns from connected beams resistance is predicted to be limited to the design bending moment due to nominal seismic R=8 including overstrength factor. Thus, the additional shear force of elastic columns will be addition to a portion of bending moment where the columns behave as vertical cantilevers. Elastic columns shear force and bending moment are illustrated in Figure 6.
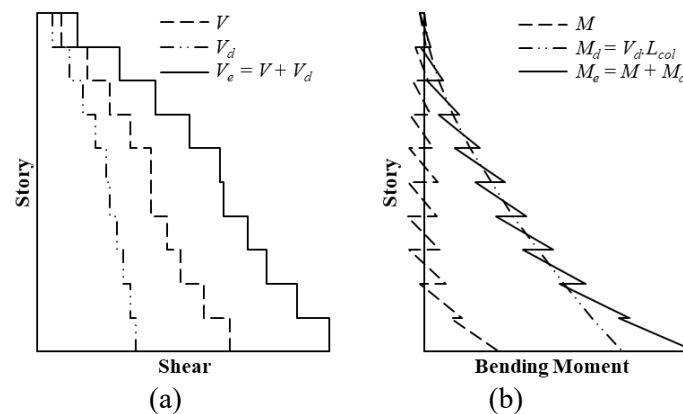

Figure 6. Additional forces to elastic column in (a) shear (b) bending moment.

The notation V and M are shear force and bending moment due to nominal seismic R=8; V_d and M_d are additional shear force and bending moment; V_e and M_e are predicted shear force and bending moment due to nominal seismic R=1.6 (targeted seismic at design level); and L_{col} is column length.

4. Design and results
Symmetrical 6 and 10 story buildings with 7 bays at 8 m span and 3.5 m inter-story height as illustrated in Figure 7 are observed. Loads consisting self-weight, superimpose dead load 1.5 kN/m², and perimeter dead load 4.2 kN/m, and live load 1 kN/m² for rooftop, and live load 2.4 kN/m² for other floors are applied to the structures. Structural dimension and properties are tabulated in Table 1.
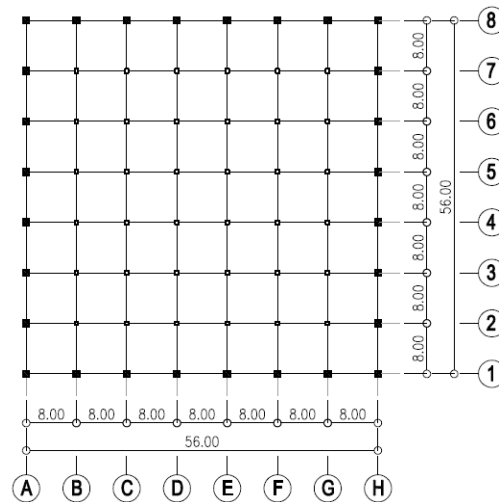
Figure 7. Typical structural plan of 6 and 10 story buildings.

Table 1. Structure dimension and properties.

Story	6 Story Building		10 Story Building	
	Elastic Column	Plastic Column	Elastic Column	Plastic Column
9-10	-	-	600x600	350x350
7-8	-	-	750x750	425x425
5-6	600x600	350x350	900x900	500x500
3-4	750x750	425x425	1000x1000	575x575
1-2	900x900	500x500	1100x1100	650x650

Primary Beam Dimension: 300x700 mm

Slab Thickness: 120 mm

Concrete Compression Strength, $F_c' = 30$ MPa

Longitudinal Reinforcement Yield Stress, $F_y = 420$ MPa

Transversal Reinforcement Yield Stress, $F_{yt} = 420$ MPa

Both structures are designed and analyzed using seismic load for city of Surabaya at site class C as in SNI 1726:2012. Structure performance are evaluated based on story drift and failure mechanism through Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP) analysis as in FEMA 273 [8].

4.1. Story shear distribution to columns

Story shear distribution to the columns in 6 and 10 story buildings as in Table 2 and Table 3 are obtained from NSP analysis. Actual story shear values due to targeted seismic V_t^{Ta} of 6 story building are close to the predicted values V_t^T, meanwhile in 10 story building, V_t^{Ta} are lower V_t^T for all stories. For both structures, actual total shear of plastic columns V_{pt}^{Ta} are greater than predicted, that presumed due to overstrength in structural element. The results show that structure behavior under non-linear static analysis is well predicted for both 6 and 10 story buildings.

Table 2. Story shear distribution to the columns in 6 story building.

Story	Story Shear			Total Shear of Plastic Column			Total Shear of Elastic Column		
	V_t^T (kN)	V_t^{Ta} (kN)	Ratio	V_{pt}^T (kN)	V_{pt}^{Ta} (kN)	Ratio	V_{et}^T (kN)	V_{et}^{Ta} (kN)	Ratio
6	5708	5337	0.94	1746	3589	2.06	3962	1748	0.44
5	10619	10443	0.98	2613	4397	1.68	8006	6045	0.76
4	14395	14508	1.01	4855	7667	1.58	9539	6842	0.72
3	17072	17414	1.02	4705	6407	1.36	12367	11007	0.89
2	18716	19054	1.02	5813	7858	1.35	12904	11196	0.87
1	19422	19639	1.01	3606	5810	1.61	15816	13829	0.87

Table 3. Story shear distribution to the columns in 10 story building.

Story	Story Shear			Total Shear of Plastic Column			Total Shear of Elastic Column		
	V_t^T (kN)	V_t^{Ta} (kN)	Ratio	V_{pt}^T (kN)	V_{pt}^{Ta} (kN)	Ratio	V_{et}^T (kN)	V_{et}^{Ta} (kN)	Ratio
10	4618	3222	0.70	1407	2813	2.00	3212	408	0.13
9	8870	6521	0.74	2155	3528	1.64	6715	2993	0.45
8	12480	9558	0.77	4207	5978	1.42	8273	3580	0.43
7	15474	12273	0.79	4410	5400	1.22	11064	6873	0.62
6	17886	14592	0.82	6518	7133	1.09	11369	7459	0.66
5	19749	16488	0.83	5980	7009	1.17	13769	9479	0.69
4	21099	17916	0.85	7984	8165	1.02	13115	9751	0.74
3	21987	18883	0.86	6730	7419	1.10	15257	11464	0.75
2	22476	19409	0.86	6993	8324	1.19	15484	11084	0.72
1	22651	19585	0.86	4734	5756	1.22	17917	13829	0.77

4.2. Drift
Story drift for 6 and 10 story buildings from NSP and NDP analysis due to seismic at level of design seismic (EDRS) and Maximum Considered Earthquake (MCE_R) are shown in Figure 8 and Figure 9. Drift limit are taken as 2 % for design seismic and 4% for MCE_R as in in FEMA 273 [8]. Both structures meet the drift requirement.
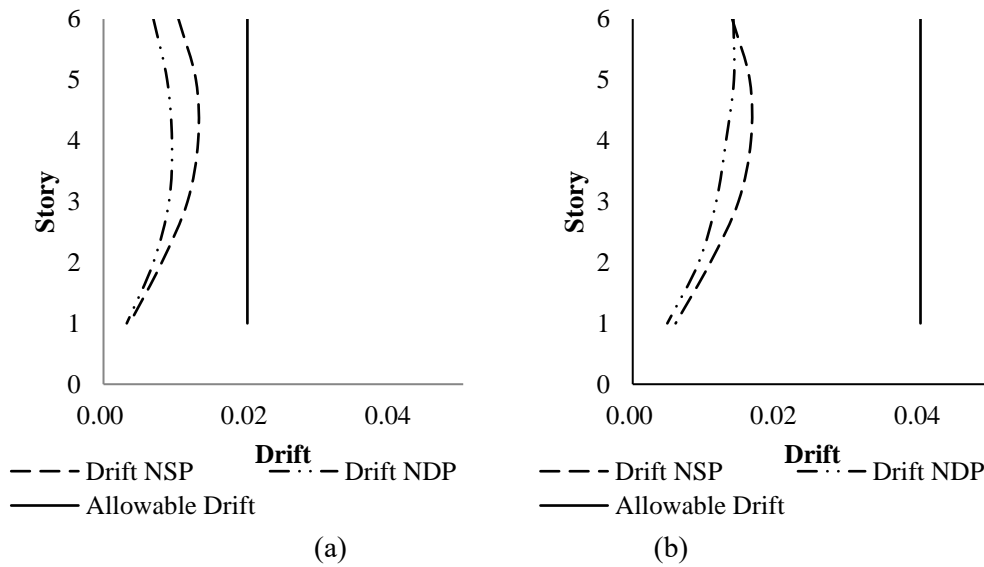
Figure 8. Story drift of 6 story building due to seismic at level (a) EDRS (b) MCE$_R$.
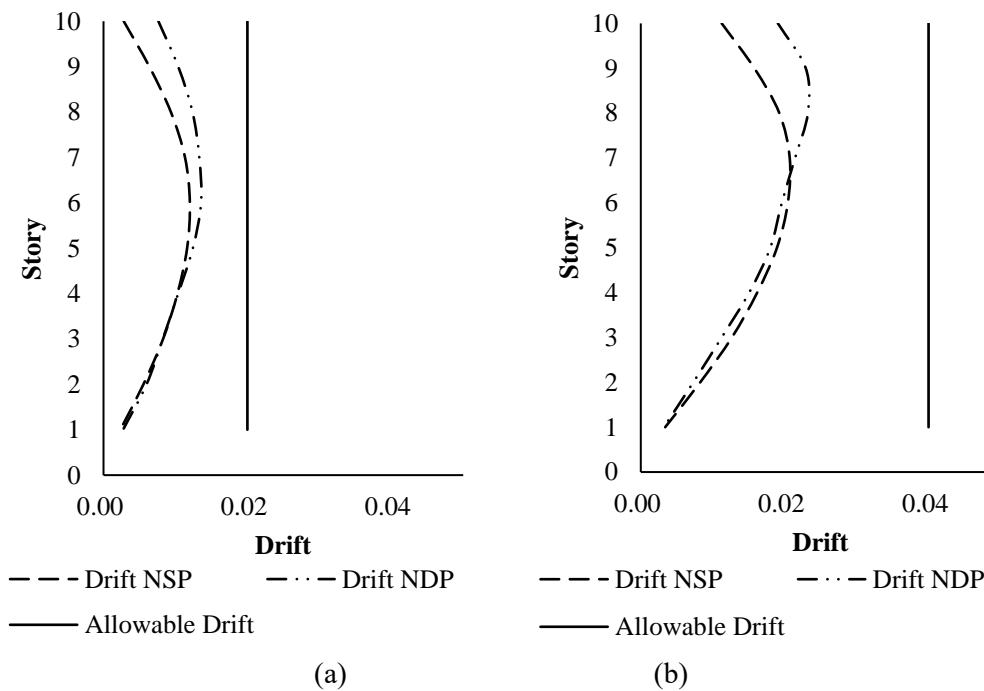

Figure 9. Story drift of 10 story building due to seismic at level (a) EDRS (b) MCE$_R$.

4.3. Failure mechanism

Failure mechanism of 6 and 10 story buildings are observed by the occurrence of plastic hinges. State from NSP analysis is observed at the Performance Point and state from NDP analysis are observed at the last time step of analysis. Results for both structures are shown in Table 4 and Table 5 respectively.

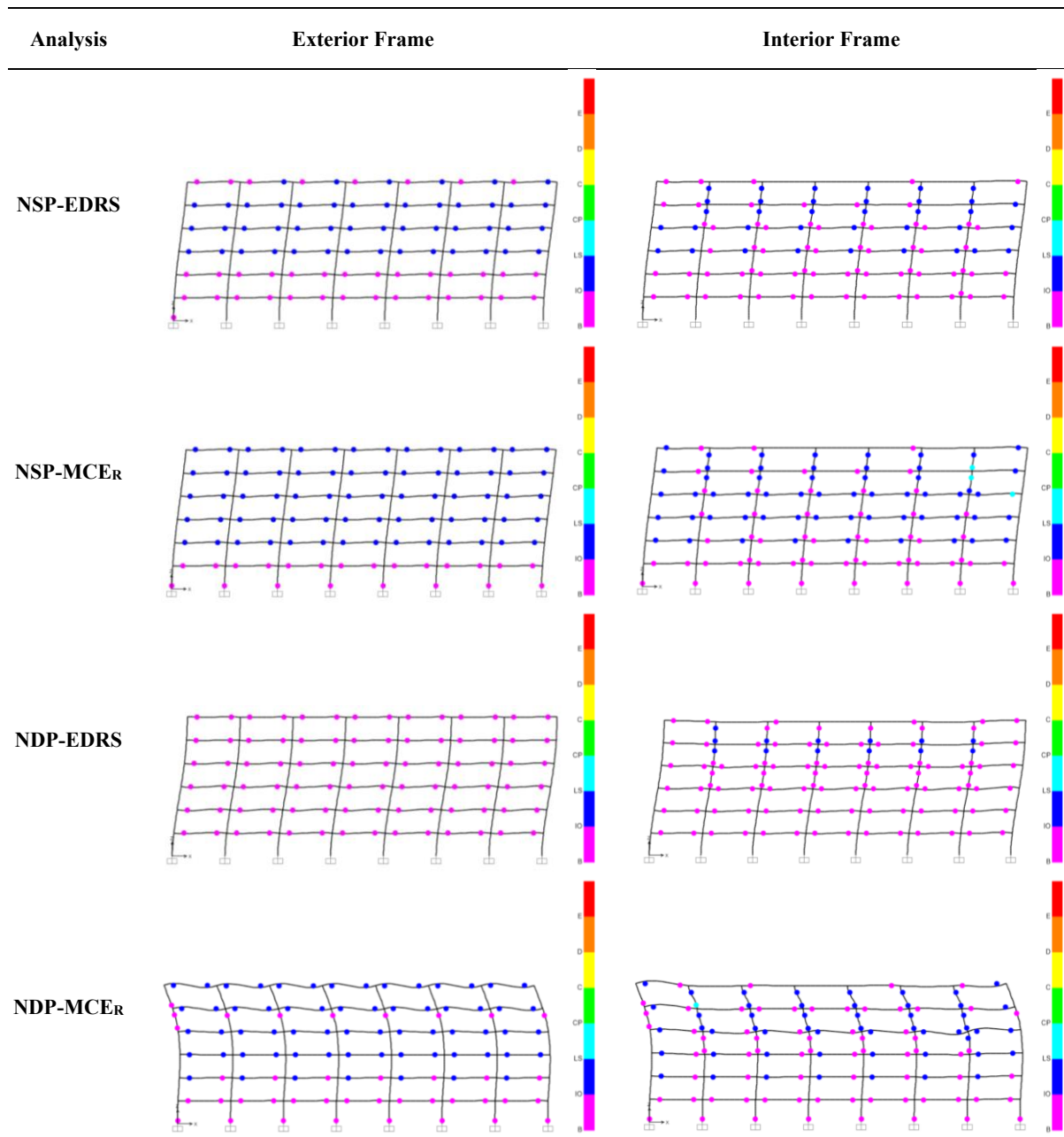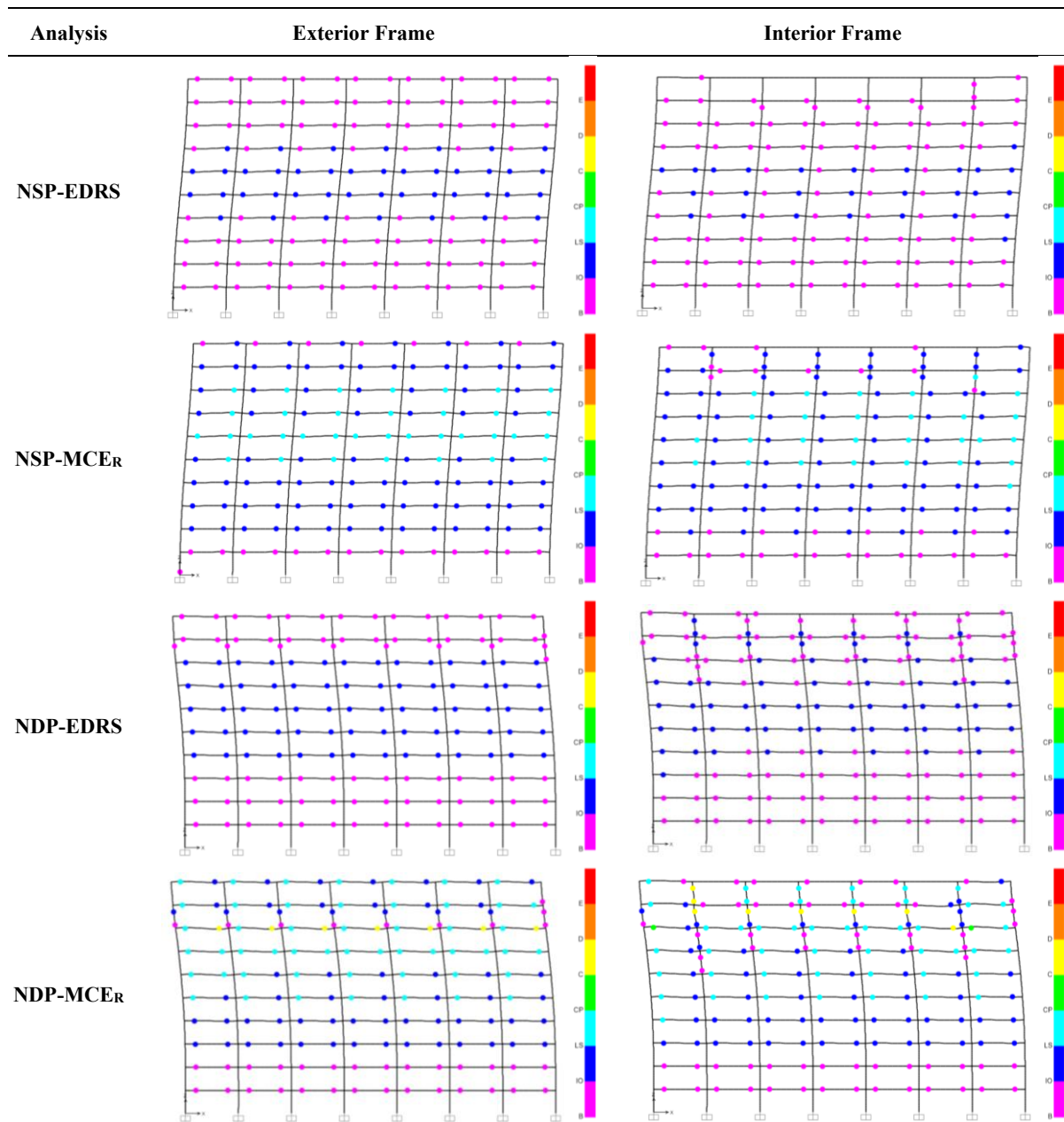**Table 4.** Plastic hinges in 6-story building.

Analysis	Exterior Frame	Interior Frame
NSP-EDRS		
NSP-MCE$_R$		
NDP-EDRS		
NDP-MCE$_R$		

Table 5. Plastic hinges in 10-story building.

Analysis	Exterior Frame	Interior Frame
NSP-EDRS		
NSP-MCE$_R$		
NDP-EDRS		
NDP-MCE$_R$		

NSP analysis of 6 and 10 story buildings show that the failure mechanisms of both buildings meet the partial side sway mechanism, but need to be optimised. Only NSP-MCE$_R$ analysis of 6 story building show the plastic hinge occurrence at all of the column base. NDP analysis show the failure mechanism that similar to those in NPS analysis. Unfortunately, plastic hinges of elastic columns occur at upper story of 6 story building in NDP-MCE$_R$ analysis and at upper story of 10 story building in both NDP-EDRS and NDP-MCE$_R$ analysis. This condition is presumed due to elastic column internal forces that predicted based on one way loading.

5. Conclusions
Structure behavior under non-linear static analysis is well predicted by this new approach. However, under non-linear dynamic analysis, a few unexpected plastic hinges of elastic columns were observed

at upper stories. But it should be noted that the earthquake used for performance analysis (maximum considered earthquake) is 50% larger than the one used for design (earthquake level corresponding to elastic design response spectrum). Further researches need to be conducted to observe dynamic behavior of the structure.

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