MINIMUM DESIGN LIMIT OF PARTIAL CAPACITY DESIGN FOR REGULAR MOMENT RESISTING FRAME

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ABSTRACT

As an alternative seismic design to the most popular method capacity design (CD), partial capacity design (PCD) has been applied for various structural systems including regular and irregular moment resisting frame (MRF). PCD promotes a simpler and faster design procedures by avoiding sequential procedures in CD where column design can only be initiated after the nominal capacity of beams are known. Using the partial side sway mechanism as its safe collapse mechanism, PCD allows some columns to be plastic while the others are kept to remain elastic assured by using of a magnification factor (MF). Some MRFs' designed using this method give good performance at the targeted seismic level. However the design parameter such as member dimension, number of reinforcement and the number and choices of elastic columns configuration varied from one design to the others. This study explores the performance of "minimum design" defined as: (a) uniform column dimension avoiding minimum reinforcement, and (b) using perimeter columns as the elastic columns. Eight MRF structures with regular plan, consist of 6- and 10-storey, in zone 2 and 6 of Indonesian seismic map, will be used as case study. The design performance is verified using non linear time history analysis based on structural plastic mechanism. Results show that the "minimum design" criteria could not be used, unexpected shear distributions leading to the condition where structures fail to meet the expected collapse mechanism.

Keywords: partial capacity design, moment resisting frame, performance based design, nonlinear time history analysis.

1. INTRODUCTION

After its intensive exploration in the past decade, partial capacity design (PCD) is prospective to be applied in the design practice, especially for moment resisting frame structures. Compare to capacity design (CD), PCD offers shorter design procedure because column design can proceed together with beam design based on ultimate load combination. By maintaining safe plastic mechanism, e.g. partial beam side sway mechanism shown in Fig.1, PCD allows some columns to be plastic while the other columns (usually perimeter columns) are kept in elactic condition.

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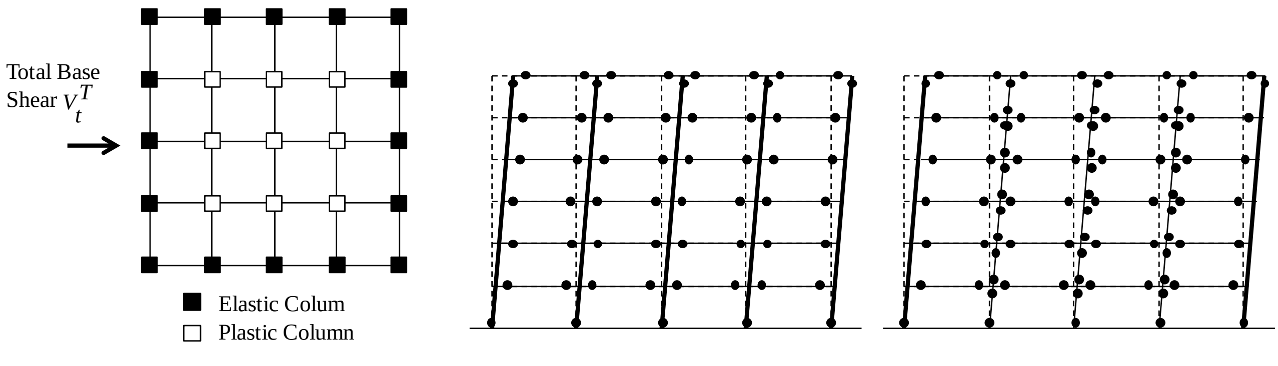


Figure 1: Partial Beam Side Sway Mechanism.

PCD assumes during the targeted seismic load, the interior columns sustain the shear force up to the nominal seismic load multiplied by the overstrength factor, f_1. Then the excess of shear force is sustained entirely by the exterior columns (Muljati and Lumantarna 2011):

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

where n_{ex} and n_{in} are the total number of exterior and interior columns; S^T_{ex} is the shear force in the exterior column due to the target seismic load; S^N_{in} is the shear force in the interior column due to the nominal seismic load; f_1 = 1.60 is the overstrength factor; and V^T_t is the total base shear due to the targeted seismic load.

In order to keep the exterior columns to remain elastic during the targeted seismic load, they should be magnified by a magnification factor (MF) which is given as (Muljati et al. 2012):

$$MF = \frac{\left(\dfrac{C^T}{C^{500}}\right)\mu - 1.60\left(n_{in}R_{in}^{N}\right)}{\left(n_{ex}R_{ex}^{N}\right)} \tag{2}$$

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; μ is the structural ductility; n_{in} and n_{ex} are the total number of interior and exterior column; R^N_{in} and R^N_{ex} are the ratio of interior and exterior columns' base shear to the total base shear due to the nominal seismic load.

During the application of the targeted seismic load structure is already in the non-linear stage, the spectral acceleration C^T is obtained from the targeted seismic load, which in this case the five hundred years return period earthquake, C^T should be obtained using the so called "plastic period" $T_{plastic}$ from the targeted response spectrum (Muljati and Lumantarna 2011):

$$T_{plastic\ (predicted)} = 2.969T_{elastic} + 0.313 \tag{3}$$

Previous researches show that the empirical formula of MF is valid to be applied in PCD method especially for regular fully ductile moment resisting frame up to 10-story (Muljati and Lumantarna 2011). The value of MF is able to keep exterior columns to be in elastic condition as expected in the determined collapse mechanism. Structures have a good seismic performance in terms of story drift and damage indices. PCD is also reported applicable for MRF structures with vertical set-back (Lumantarna and Pudjisuryadi 2012). However, since the columns are not designed using the capacity of the beams connected to them, one should be very careful, and not use excessive reinforcement in the beams, only minimum round ups should be applied.

All survival MRF structures designed using PCD involing wide choice of design parameters such as column dimension, configuration and number of elastic/pastic column, and number of reinforcement. It is thought that to be use in design, some guides are needed to determine the minimum condition of these design parameters. This study observes the possibility of using: (a) uniform column dimension with no minimum reinforcement, and (b) perimeter columns as the elastic columns.

2. OBSERVED STRUCTURES

This study uses eight regular concrete MRF structures consist of two different regular layout plan (A and B), 6- and 10-story, equal-span and -story height of 8m and 3.50m (Fig.2). These buildings are assumed to be built on soft soil in zone 2 and 6 of Indonesian seismic map (SNI 03-1726-2002) and designed using the proposed method with 500-year return period ground acceleration as the target seismic load. For further discussion, the variants will be named based on the plan type, followed by number of story and the seismic zone. For example A10-2 indicates structure with floor plan type A, 10-story, in zone 2. Unlike the previous studies, this study intentionally choose huge structures in order to check the applicability range of PCD. In the preliminary design, beam and column dimension is predicted to be 1/12 times its span and $P/(0.35f_c')$ respectively, where P is axial force due to gravity load, and f_c' is the concrete strength.

The limitation of natural fundamental period as stated in SNI 03-1726-2002 clause 5.6 is not considered to avoid the use of minimum reinforcement in the design. Columns use uniform dimension, and perimeter columns are assigned as the elastic column. The performance of the structures are checked by nonlinear time history analysis using SAP 2000 Nonlinear (CSI 2007) with the consistent ground acceleration spectrum, modified from N-S component of El-Centro 1940. The modification is achieved using Seismomatch (Seismosoft 2011). Moment-rotation relationship is modeled as biliner using Cumbia (Montejo 2007). The acceptance criteria for evaluating structural performance are based on failure mechanism of the structure. The maximum drift specified by the Indonesian standard is 0.02.

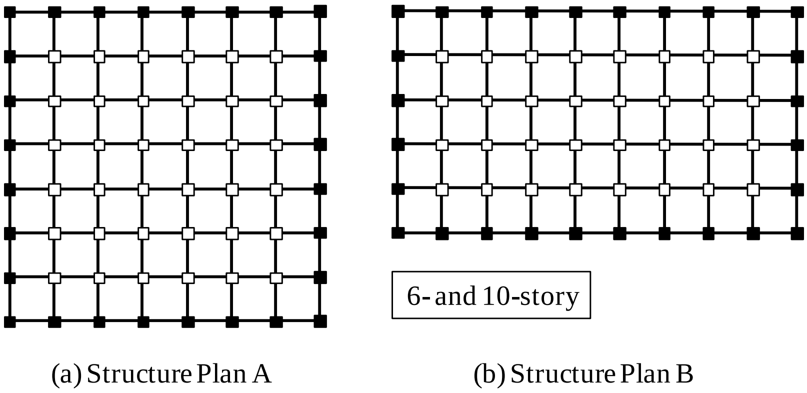


Figure 2: Stuctural Plan.

3. RESULTS AND ANALYSIS

3.1. Structural Member Dimension

The limitation to use uniform dimension for both elastic and plastic columns is very difficult to be achieved due to drift limitation and minimum reinforcement. Unavoidable use of minimum reinforcement occurs in some members of structures in zone 2. Design procedure involving a lot of itterations before the final dimension are found (see Table 1).

Table 1: Structural member dimension (mm)

Structure	Beam	Column Range
A6-2	300x700	450 – 550
A10-2	300x700	450 – 650
A6-6	500x700	550 – 900
A10-6	500x650	500 – 1000
B6-2	300x700	400 – 550
B10-2	300x700	400 – 600
B6-6	300x700 and 300x650	550 – 950
B6-10	300x700	500 – 800

3.2. Base Shear Distribution

The choice of structural plan seem plays important factor. The huge number of bays and the use of uniform dimension of colums, do affect the structural response. Perimeter columns which are expected as the main component, framed the system likely as a tube, do not work well. The interior columns resist larger portion of base shear than the exterior columns do.

3.3. Structural Mechanism

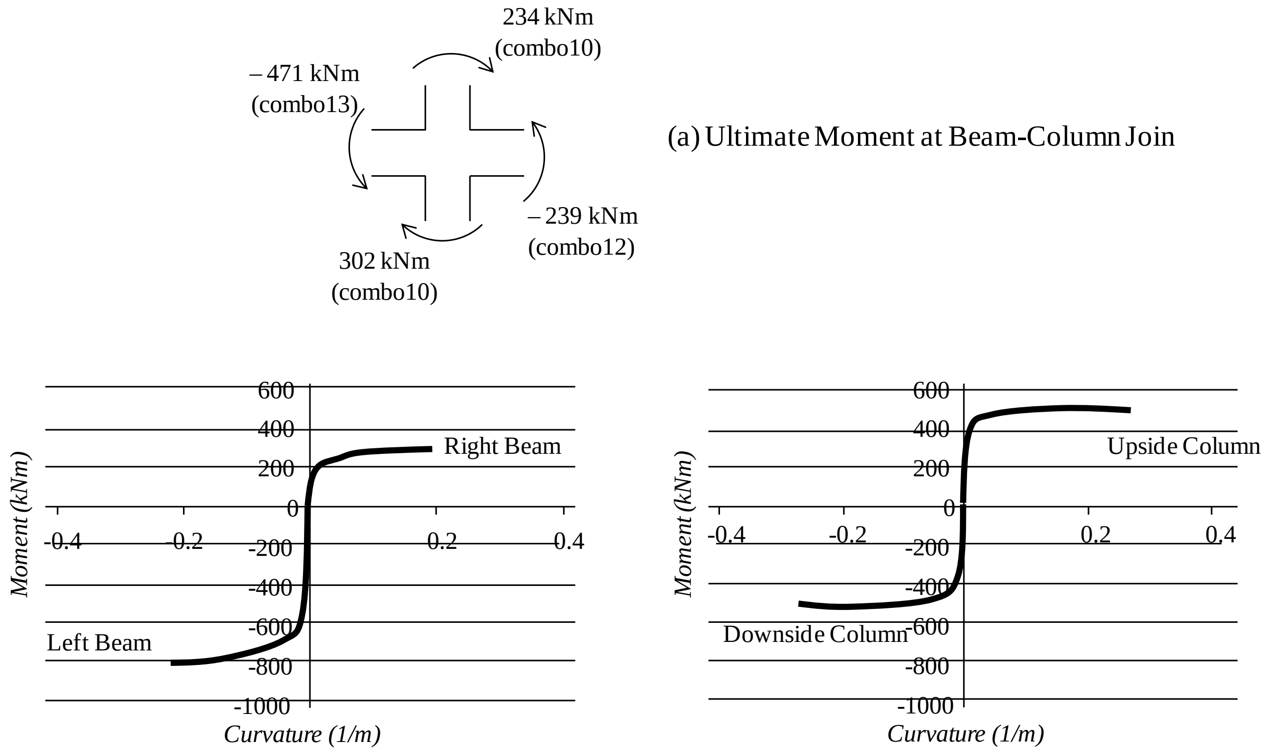
The summary of failure mechanism of all structures are shown in Table 2. The yielding of plastic columns resulting unexpected condition, leading to failure condition. Some structures even experience soft storey mechanism and yielding at the elastic columns prior to their collapse.

Table 2: Summary of Structural Mechanism

Structure	Direction	Structural Mechanism	Partial Side Sway Mechanism Achievement
A6-2	x, y	Structure survives under 20 second exication. However all elastic columns experience yielding at story-3 leading to soft story mechanism.	NO
A10-2	x, y	Structure exhibit partial side sway mechanism. All exterior columns still in elastic condition. However almost interior columns at story-8 experience yielding, and the fracture one resulting structural collapse at t = 5.45 sec.	NO
A6-6	x, y	Although exterior columns at story-3 experience yielding, but the failure of structure is caused by the fracture of interior column at story-3, at t = 5.92 sec.	NO
A10-6	x, y	Structure survives under 20 second exication although exterior columns at story-2, -7, and -8 experience yielding. Some interior columns also experience yielding.	NO
B6-2	x	Structure experiences soft story mechanism at story-1, -2, and -3. Structure collapse due to fracture at interior column at story-4, at t = 13.98 sec.	NO
	y	Although exterior columns at story-3, -4, and -5 experience yielding, but the failure of structure is caused by the fracture of interior column at story-2, -3, -4, and -5 at t = 4.54 sec.	NO
B10-2	x	Structure survives under 20 second exitation, although structure exhibits soft story mechanism at story-9.	NO
	y	Structure exhibit partial side sway mechanism. All exterior columns still in elastic condition. However fracture occurence at interior column in story-9 leadings to structural collapse at t = 4.58 sec .	NO
B6-6	x	Structure survives under 20 second excitation and fulfill the Partial Side Sway Mechanism	YES
	y	Structure survives under 20 second excitation and fulfill the Partial Side Sway Mechanism	YES
B10-6	x	Structure survives under 20 second excitation and fulfill the Partial Side Sway Mechanism	YES
	y	Structure survives under 20 sec exitation, although structure exhibits soft story mechanism at story-2.	NO

3.4. Joint Equilibrium

Further observation on the interior columns which failed show that bending moments used in the design in one beam-column join are not in equilibrium as shown in Fig.3a. This is due to the fact that maximum ultimate load for each beam and column are taken from different load combination case, resulting the unbalance condition at the beam-column join. This results in the interior column could be designed using lower load combination than the beam. The moment capacity of the left beam of ±800kNm (Fig.3b) is larger than the column capacity of ±500kNm (Fig.3c). In other cases also found that beams have larger strain hardening stage than the interior columns. As the result, column fails prior to the beam failure.



(a) Ultimate Moment at Beam-Column Join

(b) Moment-curvature of Beam

(b) Moment-curvature of Interior Column

Figure 3: Unbalance Condition in Beam-Column Join.

4. CONCLUDING REMARKS

The minimum design limit of partial capacity design determined by the use of: (a) uniform column dimension without minimum reinforcement; and (b) perimeter columns as the elastic column is not suitable to be applied the design practice due to its difficulty to find the best suited member dimension and its poor failure mechanism.

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