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The Use of Magnification Factor Formula in Partial Capacity Design Method for Fully Ductile Moment Resisting Frames

I. MULJATI*, B. LUMANTARNA

Department of Civil Engineering, Petra Christian University, Indonesia

Abstract

The concept of Partial Capacity Design (PCD) offers a new attractive alternative in the seismic design especially for fully ductile moment resisting frame. Different from conventional Capacity Design which attempt to maintain the condition of “strong column weak beam”, PCD allows the interior columns become plastic while the exterior columns are kept being elastic during severe earthquake. In order to maintain the elastic condition, columns are designed to sustain larger seismic load by multiplying the nominal seismic load with a Magnification Factor. The latest study on PCD recommends the use of an empirical formula to define the value of Magnification Factor. The objective of this study is to check the validity of the formula. Eight symmetrical fully ductile concrete moment resisting frames consist of 6- and 10-story, 4- and 6-span in zone 2 and 6, are designed in accordance with the Indonesian Earthquake Code (SNI 03-1726-2002) using the PCD method and formulation. The seismic performances of these buildings are evaluated using three dimensional static nonlinear pushover and dynamic nonlinear time history analysis. The results show that the empirical formula of Magnification Factor applied well. Furthermore all structure has good seismic performance and meets the failure mechanism as expected.

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Selection

Keywords: partial capacity design, seismic performance, magnification factor, failure mechanism, nonlinear analysis

1. Introduction

Partial Capacity Design (PCD) becomes more attractive due to its simplicity to be adopted in design practice. Unlike Capacity Design (CD) which is maintaining all columns to remain elastic, PCD allows some interior columns and all beams to be plastic while the other exterior columns are kept to be elastic

* Corresponding author and presenter

Email: imuljati@peter.petra.ac.id
during the targeted seismic loading. Partial side sway mechanism shown in Figure 1 is used as the targeted failure mechanism. The interior columns are designed using ultimate loading combination while the exterior columns should be designed based on the nominal seismic load multiplied by a Magnification Factor (MP). (Muljati and Lumantarna 2008) reported that the formulation of MP needs further observation because it failed to meet the failure mechanism as expected. The further research done by (Susanto 2009) found the problem. It was caused by improper application of the hinge properties assignment during modeling stage. The study recommended to apply and to check the previous MF formula in various regular structures designed as fully ductile moment resisting frame.

The aim of this study is to validate the MF formula. Eight symmetrical fully ductile concrete moment resisting frames consist of 6- and 10-story, 4- and 6-span in zone 2 and 6 of the Indonesian seismic map, are designed based on the Indonesian Seismic Code (SNI 03-1726-2002) using the PCD method and the MF formulation. The structural performance is measured based on story drift and failure mechanism criteria determined by ATC40.

2. Theoretical Background

PCD assumes that 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 according to:

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

where $n_{ex}$ and $n_{in}$ are the total number of exterior and interior columns; $S_{ex}^T$ 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 shown graphically in Figure 2.

![Figure 2: Load distribution in PCD](image)
In order to keep the exterior columns to remain elastic during the targeted seismic load, they should be
designed larger than the ordinary design seismic load as specified in the code. The magnification factor of
the external columns’ shear force is derived from:

\[
MF = \frac{\left( \frac{C^T}{C^{500}} \right) \mu - 1.60 (n_{in} R_{in}^N)}{\left( n_{ex} R_{ex}^N \right)}
\]  

(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; \( \mu \) 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.

However, during the application of the targeted seismic load the structure already in the non-linear
stage, the spectral acceleration due to the five hundred years return period earthquake, \( C^{500} \) should be
obtained from the non-linear response spectrum. Unfortunately, the non-linear response spectrum is not
provided in the code. Therefore, it is proposed to obtain the spectral acceleration in the plastic stage, \( C^T \),
using the natural period of the structure in plastic condition predicted by the correlation between the
elastic and the plastic natural period (\( T_{elastic} \) and \( T_{plastic} \)) of several structures previously observed
according to:

\[
T_{plastic \ (predicted)} = 2.969 T_{elastic} + 0.313
\]  

(3)

The procedure to obtain \( C^T \) using elastic spectral acceleration is explained graphically in Figure 3.

![Figure 3: Spectral acceleration](image)

3. Design and Analysis

Eight symmetrical fully ductile concrete moment resisting frames consist of 6- and 10-story, 4- and 6-
span with equal story height of 3.50 m, are used in this study. These buildings are assumed to be built on
soft soil in zone 2 and 6 of the Indonesian seismic map (SNI 03-1726-2002). These buildings are
designed using the proposed method with 500-year return period ground acceleration as the target seismic
load. Structural properties and dimension are shown in Table 1.
Table 1: Structural properties and dimension

<table>
<thead>
<tr>
<th>Storey</th>
<th>Zone 2</th>
<th>Zone 6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6-story</td>
<td>10-story</td>
</tr>
<tr>
<td>4-bays</td>
<td>PCD2-6-4</td>
<td>PCD2-6-6</td>
</tr>
<tr>
<td>6-bays</td>
<td>mm²</td>
<td>mm²</td>
</tr>
<tr>
<td>PC6-6-4</td>
<td>500x500</td>
<td>450x450</td>
</tr>
<tr>
<td>PC6-6-6</td>
<td>500x500</td>
<td>550x550</td>
</tr>
<tr>
<td>PC6-10-4</td>
<td>700x700</td>
<td>650x650</td>
</tr>
<tr>
<td>PC6-10-6</td>
<td>700x700</td>
<td>650x650</td>
</tr>
</tbody>
</table>

Secondary beam: 250x550
Main beam: 350x700
Slab thickness: 120 mm

The performance of the observed structures are determined by nonlinear static pushover analysis (ATC40 1996) using ETABS (Habibullah 1998) and nonlinear time history analysis using RUAUMOKO 3D (Carr 2002). The hinge properties of the beams and columns are obtained using ESDAP (Lidyawati 2003) a program for developing moment-curvature relation of sections. This program is developed at Petra Christian University, Surabaya based on the algorithm proposed by (King 1986). The ground acceleration used for the time history analysis is spectrum consistent ground acceleration modified from N-S component of El-Centro 1940. The modification is achieved using RESMAT (Lumantarna 1997), a program developed at Petra Christian University, Surabaya.

The acceptance criteria for evaluating structural performance are based on story drift and failure mechanism of the structure. The maximum drift specified by the Indonesian standard is 0.02. And the maximum damage index at the plastic hinge is determined based on ATC40 which are 0.25, 0.40, and 1.00 for immediate occupancy, life safety, and collapse prevention respectively.

4. Structural Performance

The elastic and plastic period of structures determined using Equation (3) is shown in Table 2. It can be seen that the actual period during plastic stage, \( T_{\text{plastic (actual)}} \) are closely to the predicted value, \( T_{\text{plastic (predicted)}} \).

Table 2: Structural period and magnification factor

<table>
<thead>
<tr>
<th>Buildings</th>
<th>PCD2-6-4</th>
<th>PCD2-6-6</th>
<th>PCD2-10-4</th>
<th>PCD2-10-6</th>
<th>PCD6-6-4</th>
<th>PCD6-6-6</th>
<th>PCD6-10-4</th>
<th>PCD6-10-6</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_{\text{elastic}} )</td>
<td>1.05</td>
<td>1.15</td>
<td>1.68</td>
<td>1.76</td>
<td>1.05</td>
<td>1.08</td>
<td>1.68</td>
<td>1.69</td>
</tr>
<tr>
<td>( T_{\text{plastic (predicted)}} )</td>
<td>3.44</td>
<td>3.74</td>
<td>5.30</td>
<td>5.52</td>
<td>3.44</td>
<td>3.51</td>
<td>5.30</td>
<td>5.32</td>
</tr>
<tr>
<td>MF</td>
<td>1.64</td>
<td>1.68</td>
<td>1.73</td>
<td>1.79</td>
<td>1.64</td>
<td>1.66</td>
<td>1.73</td>
<td>1.78</td>
</tr>
<tr>
<td>( T_{\text{plastic (actual)}} )</td>
<td>3.55</td>
<td>3.94</td>
<td>5.55</td>
<td>6.02</td>
<td>2.95</td>
<td>3.19</td>
<td>4.59</td>
<td>4.68</td>
</tr>
</tbody>
</table>
Figure 4 shows the story drift of all structures resulting from pushover (PO) and time history analysis (TH). Structures in zone 2 have acceptable drift, but structures in zone 6 especially predicted from pushover analysis, having slightly higher value than the maximum drift specified by the standard (0.02).

<table>
<thead>
<tr>
<th>Zone 2</th>
<th>Zone 6</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="6-story" /></td>
<td><img src="image2" alt="6-story" /></td>
</tr>
<tr>
<td><img src="image3" alt="10-story" /></td>
<td><img src="image4" alt="10-story" /></td>
</tr>
</tbody>
</table>

Figure 4: Story drift of structures

To observe the failure mechanism, Figure 5 shows plastic hinges location in the 10-story with 6-bay building in zone 6 (PCD6-10-6), which resulting unacceptable drift as predicted by pushover analysis. Due to space limitation, results for the other buildings can be found in (Buntoro and Weliyanto 2009; Atmadja and Wijoyo 2009). It can be seen that some interior columns become plastic as expected in partial side sway mechanism. However some plastic hinges take place at the exterior columns. Although pushover analysis resulting unacceptable story drift but the failure mechanism is excellent, and the maximum damage index is in the range of life safety. On the other hand, time history analysis detects unacceptable failure mechanism due to the occurrence of plastic hinges at the exterior columns at the seventh-story (circle marked).

The occurrence of plastic hinges at the exterior columns is caused by improper application of the overstrength factor $f_1$ in Equation (2). According to SNI 03-1726-2002, the overstrength factor $f_1$ consists of material- and structural-overstrength-factor. In this study, the design of section applies theoretical reinforcement area. Therefore, there is no material overstrength factor as much as 1.28 as defined by SNI 03-1726-2002. And the factor of $f_1$ should be taken 1.25 representing the structural overstrength factor.
only. After revising the value of $f_1$ in the MF formula, then all structures perform very well in the term of story drift and failure mechanism as expected. The value of MF is raised from 1.78 to 2.49 for PCD6-10-6. For the other structures, the value of MF is varied from 2.04 to 2.49.

<table>
<thead>
<tr>
<th>Exterior Frame</th>
<th>Interior Frame</th>
</tr>
</thead>
</table>

![Figure 5: Plastic hinge configuration](image)

5. Conclusion

The empirical formula of Magnification Factor is valid to be applied in partial capacity design method especially for the regular fully ductile moment resisting frame up to 10-story. The value of Magnification Factor is able to keep the exterior columns to be in elastic condition as expected in the determined collapse mechanism which is partial side sway mechanism. Structures have a good seismic performance in terms of story drift and damage indices.

Acknowledgments

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References


