FURTHER REFINEMENT OF PERFORMANCE-BASED PLASTIC DESIGN OF STRUCTURES FOR EARTHQUAKE RESISTANCE

M. Reza Bayat[1], Subhash C. Goel[2], and Shih-Ho Chao[3]

[1] *Graduate Student, University of Texas, Arlington, TX, USA*
[2] *Professor Emeritus of Civil Engineering, University of Michigan, Ann Arbor, MI, USA*
[3] *Assistant Professor of Civil Engineering, University of Texas, Arlington, TX, USA*
Email: mbayat@uta.edu, subhash@umich.edu, shchao@uta.edu

ABSTRACT :

Performance-Based Plastic Design (PBPD) method has been recently developed to achieve enhanced performance of earthquake resistant structures. The design concept uses pre-selected target drift and yield mechanism as performance limit states. The design lateral forces are derived by using an energy balance equation where the energy needed to push the structure up to the target drift is calculated as a fraction of elastic input energy which is obtained from the selected elastic design spectra. Plastic design is then performed to detail the frame members and connections in order to achieve the intended yield mechanism and behavior. The method has been successfully applied to a variety of steel framing systems and validated through extensive inelastic static and dynamic analyses. This paper presents some modifications and refinements that have been developed for implementation in the PBPD method, especially for mid to high-rise Moment Frame (MF) structures. The refinements are focused on the design of frame members that are intended to remain elastic as part of the design yield mechanism so that the structure can better achieve the desired performance. The refined procedure is applied to a 20-story steel moment frame and validated by inelastic static and dynamic analyses. The results showed that performance of the PBPD structure is much improved over that of the baseline SAC frame, which was designed by the then current code practice.

KEYWORDS: Performance-Based design, Plastic design, Moment frame, P-Delta effect, High-rise structures

1. INTRODUCTION

Current seismic design practice is based on elastic structural behavior and inelastic behavior is only considered indirectly through certain modification factors. However, it is known that the structures designed by current codes experience large inelastic deformations during major earthquakes. Therefore, the current design approaches may not provide an accurate understanding of structural behavior under a design level or a severe earthquake. Design base shear in current U.S. seismic design practice is calculated by using specified elastic spectra, and then modified by an appropriate R factor. After design of members, the inelastic drift is estimated by multiplying the elastic drift by a deflection amplification factor, C_d. This estimated drift should not exceed the given limits. But even after satisfying all of the current code criteria, the behavior of structures under design level ground motions can be somewhat unpredictable and uncontrolled.

As new codes are finding their way toward Performance-Based Design (PBD) framework, and considering the fact that the main goal of PBD is a desirable and predictable structural response, there is an increasing demand for new analysis/design procedures which are capable of predicting structural behavior in a more accurate way that can be implemented in PBD practice. Currently, PBD is carried out by performing series of design and evaluations in an iterative manner, which may not always give the desired result because an initial good design is needed to arrive at the targeted design after iterations (Krawinkler and Miranda, 2004).

Recently, a new design method has been developed at the University of Michigan, called Performance-Based Plastic Design (PBPD) method (Goel and Chao, 2008). This method directly accounts for inelastic behavior and eliminates the need for any assessment after initial design. The method uses pre-selected target drift and yield mechanism as key performance limit states, Figure 1. The design base shear for selected hazard level(s) is calculated by equating the work needed to take the structure monotonically up to the target drift to that required by an equivalent EP-SDOF system to achieve the same state. In addition, a new lateral force distribution has been developed based on relative distribution of maximum story shears obtained from inelastic dynamic analysis (Chao et al., 2007). Then plastic design is carried out to detail the frame members and connections in order to achieve the intended yield mechanism and behavior. The method has been successfully applied to steel MF, BRBF, EBF, STMF, and CBF.

The main purpose of this study was to further improve the PBPD method as applied to steel MFs regarding some practical design issues. Current methods of designing Non-Designated Yielding Members (Non-DYM), such as columns in MFs, are reviewed and modifications and refinements are presented in order for the final design to better match with the expected performance. Two main issues are addressed in this paper: (1) P-Delta effect for design of Non-DYMs, and (2) Calculation of base column plastic moment, M_{pc}, to prevent formation of soft-story mechanism and also to achieve a more realistic column design moment profile over the height of the structure. In the following, currently used method for design of Non-DYMs in PBPD is reviewed and modifications are suggested.

2. CURRENT METHOD OF DESIGNING NON-DESIGNATED YIELDING MEMBERS

It is commonly agreed that in order to achieve a desirable performance of structures during earthquakes, columns should remain elastic (except at the base). The current practice involves iterative design-evaluation procedures with the possibility of not converging to a proper design. To overcome this difficulty, "column-tree analysis" was introduced as part of PBPD framework (Leelataviwat et al., 1999) so that more realistic column design moments can be obtained. In this method, column design moments are obtained by considering the equilibrium of the entire column tree as part of the yield mechanism in the design limit state. Figure 2 shows the free-body diagram of exterior column-tree of a moment frame at target drift.

An important factor that amplifies the moments in design of columns, as Non-DYMs, is P-Delta effect. To obtain a proper design in PBPD, this effect needs to be accounted for in the design procedure. Several methods are currently used for this purpose. The most common methods include: Amplification of first-order analysis moments by B1 and B2 factors; Direct second-order analysis by using appropriate structural analysis programs (AISC, 2005).

Two other methods were studied and are presented in this paper. The results are compared with those obtained by using the B2-factor method as applied to the 20-story SAC frame.

3. PROPOSED METHODS TO ACCOUNT FOR P-DELTA EFFECT IN COLUMN DESIGN

3.1. Direct P-Delta Consideration in Column Tree Analysis

In this method, which can be considered a more direct way of considering P-Delta effect, the column tree is considered in an assumed deflected shape at target drift (linear deflected shape is assumed herein), and the gravity loads are applied (Figure 3) on a lumped gravity column. At this point, the equilibrium equation of the column tree is formulated to obtain lateral forces, F_L, and the resulting column moments are calculated.

3.2. Pushover Analysis of the Frame

In this method, a non-linear static pushover analysis of the entire frame is carried out up to the target drift by applying design lateral force distribution. The Designated Yielding Members (DYM) are modeled to behave inelastically, while the non-DYM are modeled (or "forced") to behave elastically. P-Delta effect is captured by applying the floor gravity loads on “gravity columns” which can be lumped into one, if desired.

Inelastic static pushover and dynamic time-history analyses of the frame were carried out by using the above two methods and also the B2-factor method. It can be seen in Figure 5b and 5c that, while the column moments obtained from pushover analysis are significantly smaller than the ones obtained from column tree analysis without P-Delta, the amplified moments by the B2-factor and the Direct P-Delta method are somewhat close to each other, except that B2-factor would lead to relatively heavier columns at first lower stories and Direct P-Delta method would lead to somewhat heavier columns in mid and top stories.

Dynamic analysis results were used to determine which of the proposed methods is more suitable for use in PBPD method in order to account for the P-Delta effect. As can be seen in Figure 6, the column tree analysis results amplified by B2-factor give the most appropriate design moments for columns in all stories, while also being on the safe side. This can also be affirmed by comparing the performances of the frames designed by these three methods under dynamic analysis. The column design moments obtained from pushover analysis are also shown in the Figure 6. As can be seen, except for the first story, pushover method underestimates the required column moments, which would lead to a weak and flexible structure. Also, having insufficient column stiffness in the upper stories by using results of pushover analysis can lead to undesirable performance under dynamic analysis due to higher mode effects, especially for mid to high-rise structures.

4. MODIFIED PLASTIC MOMENT AT THE BASE

The plastic moment of columns at base, M_{pc}, can be obtained by using the following (Goel and Chao, 2008):

$$M_{pc} = \frac{1.1Vh_1}{4} \tag{4.1}$$

where V is the base shear for equivalent one bay model. The 1.1 factor was considered as margin to account for possible overloading due to strain hardening and uncertainty in material strength in order to prevent formation of soft-story mechanism. But it is known that some other factors could potentially cause the base shear of the designed frame to be greater than the design base shear. Therefore, those factors also should be accounted for in the calculation of M_{pc} in order to prevent formation of a soft-story mechanism at the first level.

The influencing factors include: design resistance factor, $\emptyset$ for beams (0.9), yield overstrength for beams ($R_y = 1.1$), strain, hardening of beams (1.1), and also an oversize factor when selecting design sections (1.1). Taking these factors into account, and also considering a 10% margin, a resultant factor of $(1/0.9)\times(1.1)\times(1.1)\times(1.1)=1.48$ would be needed. Therefore, a value of 1.5 is used in the modified equation, Figure 4:

$$M_{pc} = \frac{1.5Vh_1}{4} \tag{4.2}$$

Another issue with using Eqn. (4.1) is that the moment at the top of first story columns generally turns out to be greater than the moment at the base (M_{pc}), and therefore the design moment for this column would be a larger moment. This issue can also be resolved by using the modified equation for M_{pc}, because by considering all possible sources of overstrength in the calculation of M_{pc} and corresponding lateral forces the moment at the top of first story columns would always be less or equal to M_{pc}. This was verified in 4, 10, and 20-story frames and shown in Figure 5a for the 20-story frame.

5. RE-DESIGN OF 20-STORY SAC LA FRAME BY PBPD METHOD

The above mentioned modifications are more beneficial for design of tall moment frames. The study was done for 4, 10 and 20-story frames, but only the results of the latter are shown in this paper due to limited space.
The LA 20-story structure, originally designed as part of the SAC Steel Research Program (Gupta and Krawinkler 1999; Lee and Goel, 2001), was re-designed by PBPD method with the above modifications for column design. To be consistent with the original design, the same building code (UBC 1994) was used for PBPD as well. The target yield mechanism is similar to the one shown in Figure 1 with the beam plastic hinges assumed at 6 inches from the column face. PBPD lateral force distribution (Chao et al, 2007) was used to distribute the base shear over the height of structure. Significant design parameters are listed in Tables 5.1. The beams were designed as full sections assuming ductile unreinforced flange connections with the columns.

The design moments for interior columns are shown in Figure 6. The results obtained by using B2-factor method are shown here. In the following, the responses of SAC and PBPD frames under inelastic pushover and time-history analyses are briefly compared. It should be mentioned that weight of the PBPD frame was only 13% greater than that of the SAC frame, while the response results showed much superior performance. More details can be found in the reference by Goel and Chao (2008).

6. COMPARISON OF PERFORMANCES OF THE SAC AND PBPD FRAMES

Nonlinear static (pushover) and dynamic (time-history) analyses were carried out for the SAC and PBPD frames by using Perform-3D program (CSI, 2007). A lumped gravity column was added which enables the model to capture the P-Delta effect as well as the contribution of all gravity columns in total response of the structure. Strength degradation of moment-rotation behavior of plastic hinges and panel zone deformations were also modeled.

Some results from pushover analysis are shown in Figure 7. It can be seen that the plastic hinges tend to form in lower stories in the SAC frame and experience relatively larger plastic rotations while the plastic hinges are much better distributed in the PBPD frame with less plastic rotations at the same roof drift. It can also be seen from the deflected shapes of the frames at 3.5% roof drift and the distribution of plastic hinges that SAC frame has greater tendency to develop a soft-story type of mechanism in the lower part of the frame. On the contrary, none of the columns in the PBPD frame experience yielding as intended (except at the base) even at 3.5% roof drift.

Inelastic dynamic analyses of both frames were carried out by using 10%/50yrs and 2%/50yrs SAC ground motions for LA site (Somerville et al., 1997). The results showed very good behavior of the PBPD frame under 10%/50yrs as well as 2%/50yrs hazard levels. No plastic hinging was observed in the columns of the PBPD frame (except at the base), whereas significant plastic hinging was observed in the SAC frame columns under both sets of ground motions (Goel and Chao, 2008). Moreover, collapse of the SAC frame was seen for three ground motions, namely LA30, LA35, and LA36. Even though those ground motions caused large story drifts in the PBPD frame, but the structure did not lose stability, Figure 8.

7. CONCLUSIONS

Refinements have been proposed for “column-tree analysis” within the PBPD method to achieve better performance for taller moment frames. Considering different methods of accounting for P-Delta effect, the B2-factor method seems to give more realistic column design moments at this stage of the study. Also, column plastic moment at the base was modified to account for most of the possible sources of over-strength. The 20-story SAC LA frame was redesigned by the refined PBPD method, and significant improvements in its performance were noticed.

8. ACKNOWLEDGEMENT

The results presented in this paper are part of a broader research effort on further advancement of the PBPD method sponsored by Schuff Steel Company, for which the authors are most grateful.

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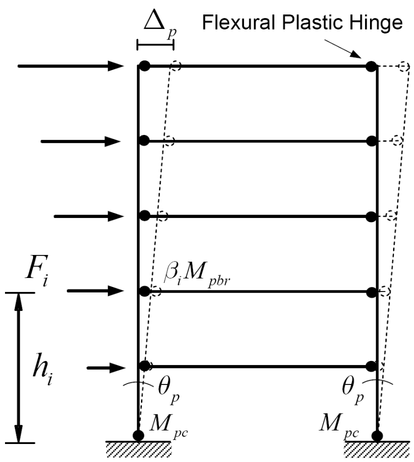


Figure 1 Desirable yield mechanism for Moment Frame

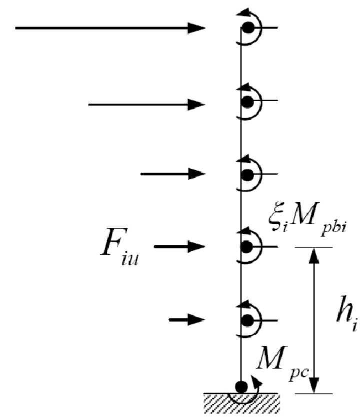


Figure 2 Free-body diagram of an exterior column

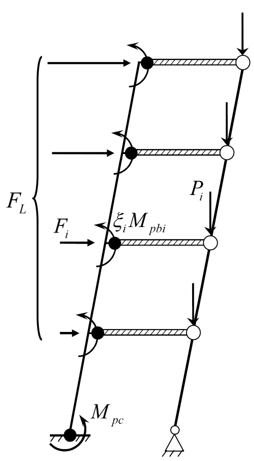


Figure 3 Column tree and gravity column in "Direct P-Delta" method

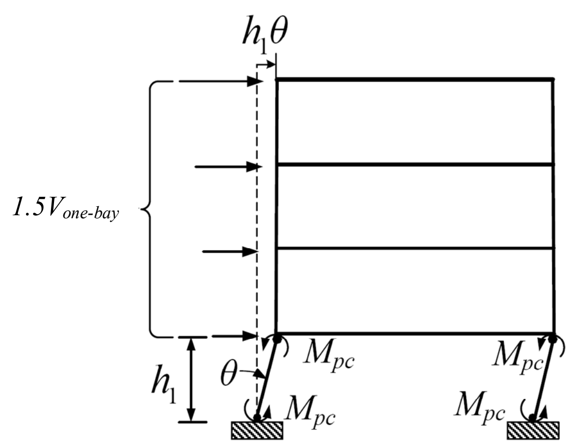
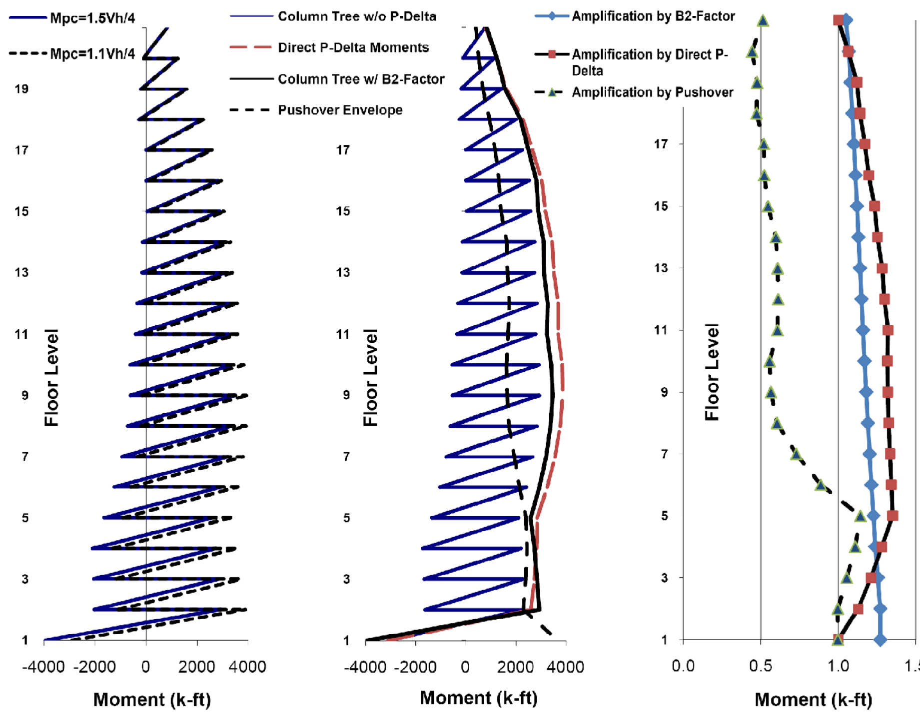


Figure 4 One-bay frame with soft-story mechanism

Table 5.1 Design parameters for PBPD 20-story frame

Design Parameters	**10% in 50 years Hazard (2/3MCE)**	**2% in 50 years Hazard (MCE)**
S_a	0.36 g	0.54 g
T	2.299 sec.	2.299 sec.
Yield Drift Ratio θ_y	1.0%	1.0%
Target Drift Ratio θ_u	2%	3%
Inelastic Drift Ratio $\theta_p = \theta_u - \theta_y$	1%	2%
$\mu_s = \theta_u / \theta_y$	2.0	3
R_μ	2.0	3
γ	0.75	0.438
α	0.942	1.884
V / W	0.094	0.037
Design Base Shear V *	1146 kips (governs)	457 kips



a) b) c)

Figure 5 a) Comparing column moments for $M_{pc} = 1.5Vh_1/4$ and $M_{pc} = 1.1Vh_1/4$, b) Comparing column design moments for interior column tree by different methods, c) P-Delta amplification by different methods

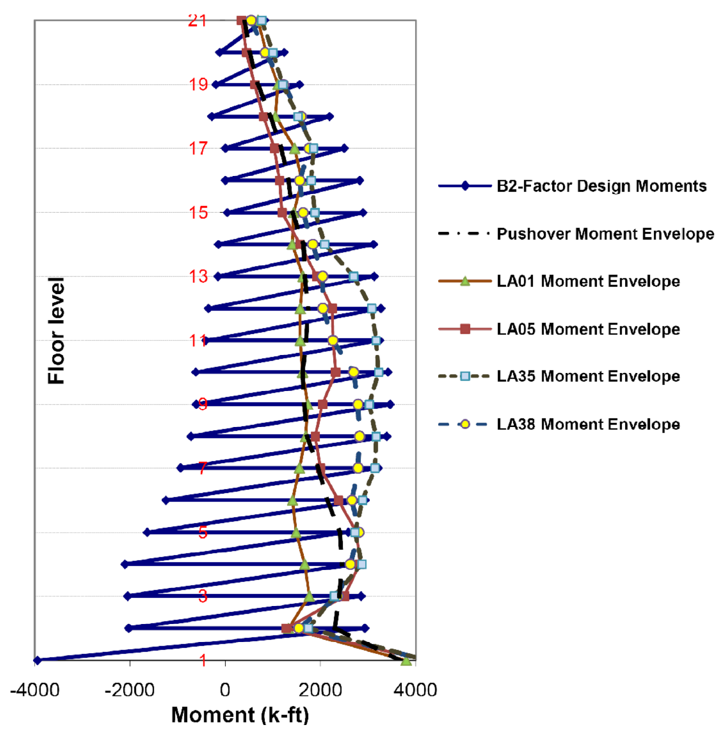
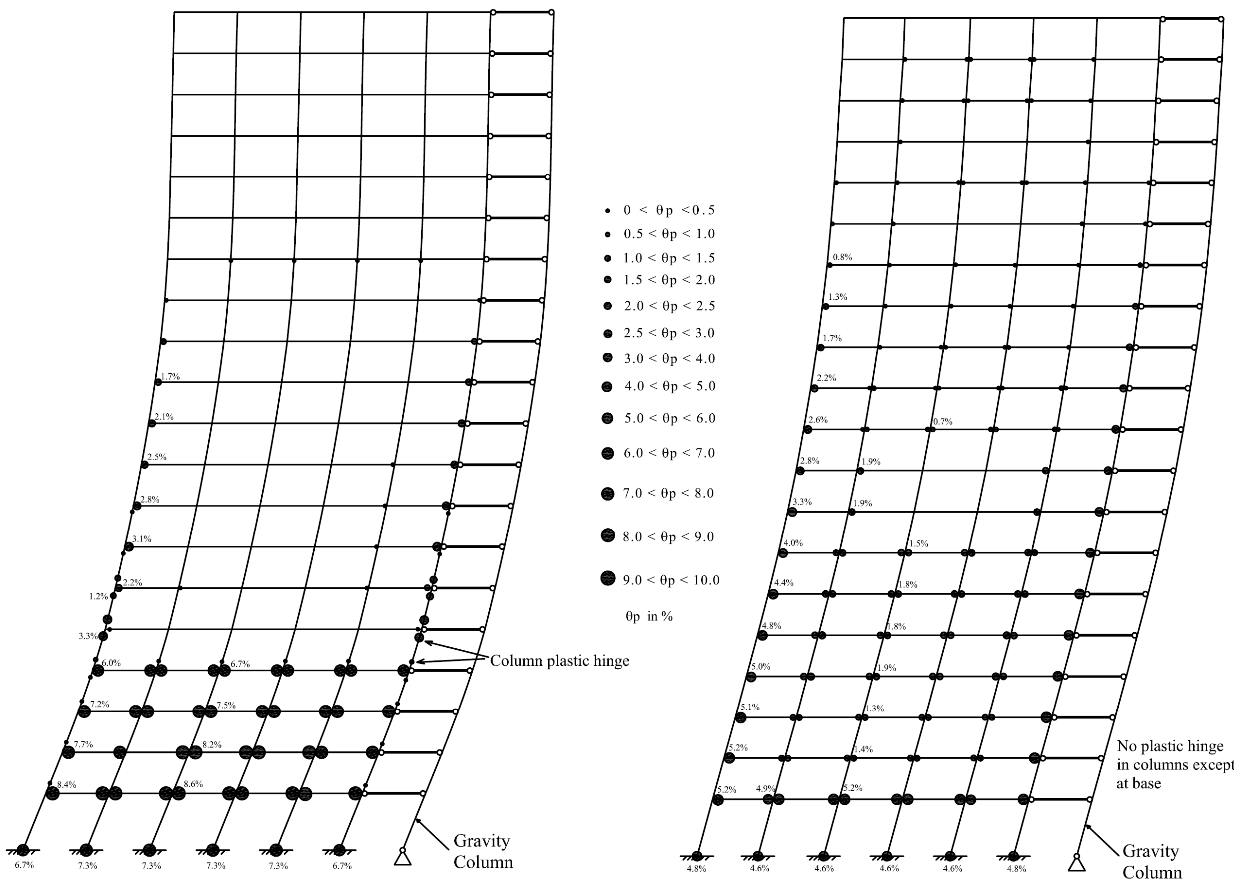
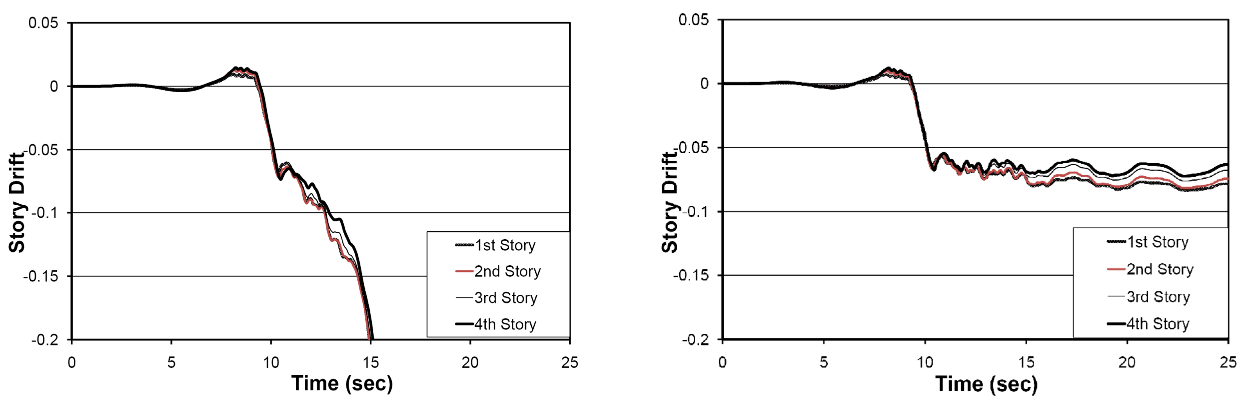


Figure 6 Comparison of interior column moments from inelastic dynamic analysis with different design methods



a) b)

. Figure 7 PH rotations at 3.5% roof drift, a) SAC frame, b) PBPD frame; Pushover Analysis



a) SAC frame b) PBPD frame

Figure 8 Story drift vs. time plots for a) SAC and b) PBPD frames under LA35 ground motion