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ESTIMATION OF DAMAGE INDEX FOR REINFORCED CONCRETE SMRF DESIGNED USING PERFORMANCE-BASED PLASTIC DESIGN METHOD

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ملخص البحث :

طريقة التصميم اللدن القائم على الأداء (PBPD) شاع إستخدامها مؤخراً فى التصميم الزلزالى للمنشآت. تستهدف هذه الطريقة إزاحة أفقية و آلية الحركة عند الوصول الى حمل الخضوع للمنشأ، و هما قيم مختارة سلفاً قبل البدأ فى التصميم. فى هذا البحث تم تحليل إطارات من الخرسانة المسلحة المقاومة للعزوم و ذات ممطولية كافية، و ذلك طبقاً للكود الأمريكى ACI-318/ASCE-07 و أيضاً بطريقة (PBPD) بإستخدام التحليل اللاخطى الإستاتيكى، و ذلك لتقييم معامل التهشم عند خمسة مستويات للتصميم الزلزالى لدراسة أداء المنشأت.

ABSTRACT

Performance-Based Plastic Design (PBPD) method is widely extended for seismic design of building structures. A pre-selected target drift and yield mechanisms is used as key performance objectives. In this research, reinforced concrete special moment frames (RC SMF) were analyzed. Two designs were considered in the analysis, one design according to ACI-318/ASCE-07, and the other according to PBPD.

Using static nonlinear analysis (Pushover analysis), stiffness-based damage indices were obtained for both types of frames to assess their structural performance at five levels of performance based seismic designs, operational phase (OP), immediate occupancy (IO), damage control (DC), life safety (LS), and collapse prevention (CP).

KEYWORDS: Performance-Based Plastic Design (PBPD); Reinforced Concrete Special Moment Frames (RC SMF); Damage Index (DI); Pushover Analysis.

1. INTRODUCTION

Performance-Based Plastic Design (PBPD) method was derived from the Performance based Seismic design PBSD method. Performance-based Plastic design method starting from the pre-defined performance objectives, in which the intended yield mechanism is achieved through performing plastic design. Plastic design controls drift and yielding of frame members from the beginning to minimize the lengthy iterations to reach the final design [1-7].

Stiffness-based damage index used is following Equation 1 to describe the damage state of the structure. This index accounts for the cumulative effects by studying the degradation of structural stiffness for every incremental displacement at every interval step of pushover. [8-9].

$$DI_C = 1 - \frac{\sum_{i=0}^{n=c} K_i (d_i - d_{i-1})}{K_0 \; d_i} \tag{1}$$

where:

ID_c = Damage index at step c,

K_o = Initial stiffness,

K_i = Building stiffness at step i,

d_i = Roof displacement at step i,

d_{i-1} = Roof displacement at step i-1,

Drift limits were linked to performance levels as follow, 0.5% for operational phase (OP), 1.0% for immediate occupancy (IO), 1.5% for damage control (DC), 2.0% for life safety (LS), and 2.5% for collapse prevention (CP), to assess structural performance [10].

Structural performance levels “Life Safety (LS)”, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. While structural performance level “Collapse Prevention (CP)”, means the post-earthquake damage state in which the building is on the verge of partial or total collapse. However, all significant components of the gravity-load-resisting system must continue to carry their gravity load demands. Structural performance levels for allowable drift shall not exceed 2% and 4% for LS and CP, respectively. In this study the allowable drift for CP will be limited to 2.5% only.

2. STATEMENT OF THE PROBLEM (PROBLEM FORMULATION)

Four baseline RC structures (4, 8, 12 and 20-story internal RC special moment frame structure) as used in the FEMA P695 [11], was selected for this study. Building configuration is illustrated in Figure 1. The frames are used to support both vertical and lateral loads. These (code-based design) structures were redesigned by the PBPD approach as shown in Table 1 [1].

2.1. Input Data

The building is designed to sustain the following loading data:

- Design floor dead load = 8.38 kN/m^2 (175 psf).
- Design floor live load = 2.40 kN/m^2 (50 psf).

2.2. Material Properties

- Concrete cylinder compressive strength fc' = 34.5 - 41.4 MPa (5.0 - 6.0 ksi)
- Reinforcement rebar yield strength fy = 413.7 MPa (60.0 ksi)

Table 1: Building Configuration and Design Parameters.

Design Parameters	4 - Story	8 - Story	12 - Story	20 - Story
ID Number	1010	1012	1014	1021
Number of Floors	4	8	12	20
First Story Height - H_1 m (ft)	4.572 (15)			
Upper Story Height - H_n m (ft)	3.962 (13)			
Bay Size m (ft)	9.144 (30)	6.096 (20)		
Total Height m (ft)	16.459 (54)	32.309 (106)	48.158 (158)	79.858 (262)
Code Compliant Base Shear kN (kip)	858.5 (193)	418.1 (94)	547.1 (123)	907.4 (204)
PBPD Compliant Base Shear kN (kip)	1243.7 (279.6)	632.5 (142.2)	746 (167.7)	1567.1 (352.3)

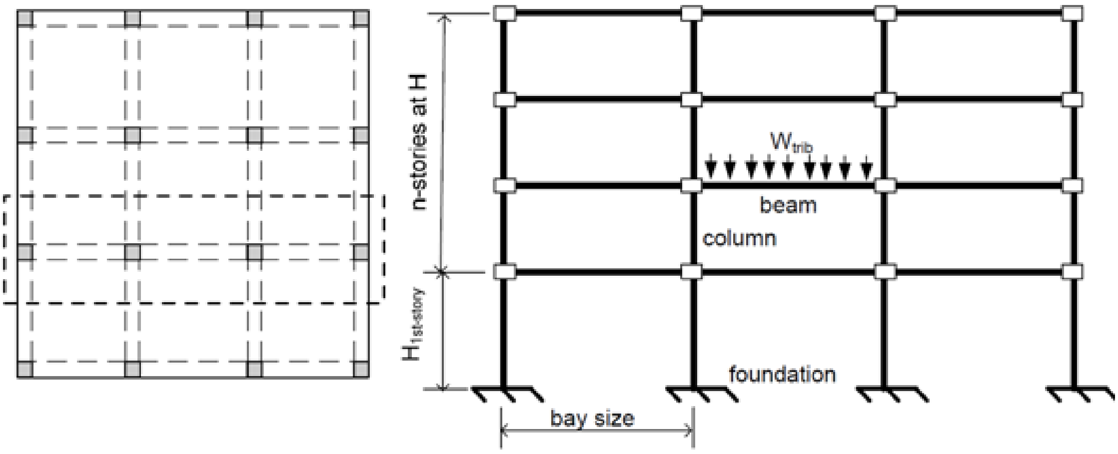


Figure 1: Typical floor plan and typical elevation of the RC special moment frame. [1]

3. MODEL DESCRIPTION

SeismoStruct 2016 software analysis package was used in this study to perform pushover analysis. 2D-models were created for all baseline frames and the frames redesigned using PBPD as well. P-Delta curve is generated for each frame considering a triangular lateral loading pattern.

4. RESULTS AND DISCUSSION

4.1. Nonlinear Static Pushover Analyses

The output of a pushover analysis is presented in the form of P-Delta curve, which is typically base shear versus roof top displacement plot. This relation can also be modified by normalizing the values through dividing roof displacement by the building height, and dividing base shear by the building weight, as presented in Figure 2. It was noticed that frames designed using PBPD provide more ductility for all frames, although their maximum base shear capacities are less than their corresponding values for frames designed following code. The 20-story frame was prematurely lost its stiffness due to the formation of plastic hinges in columns that caused an excessive deformation and a decrease in the frame lateral load resistance. This loss in stiffness occurred at a loading value exceeds the design base shear by about 65%.

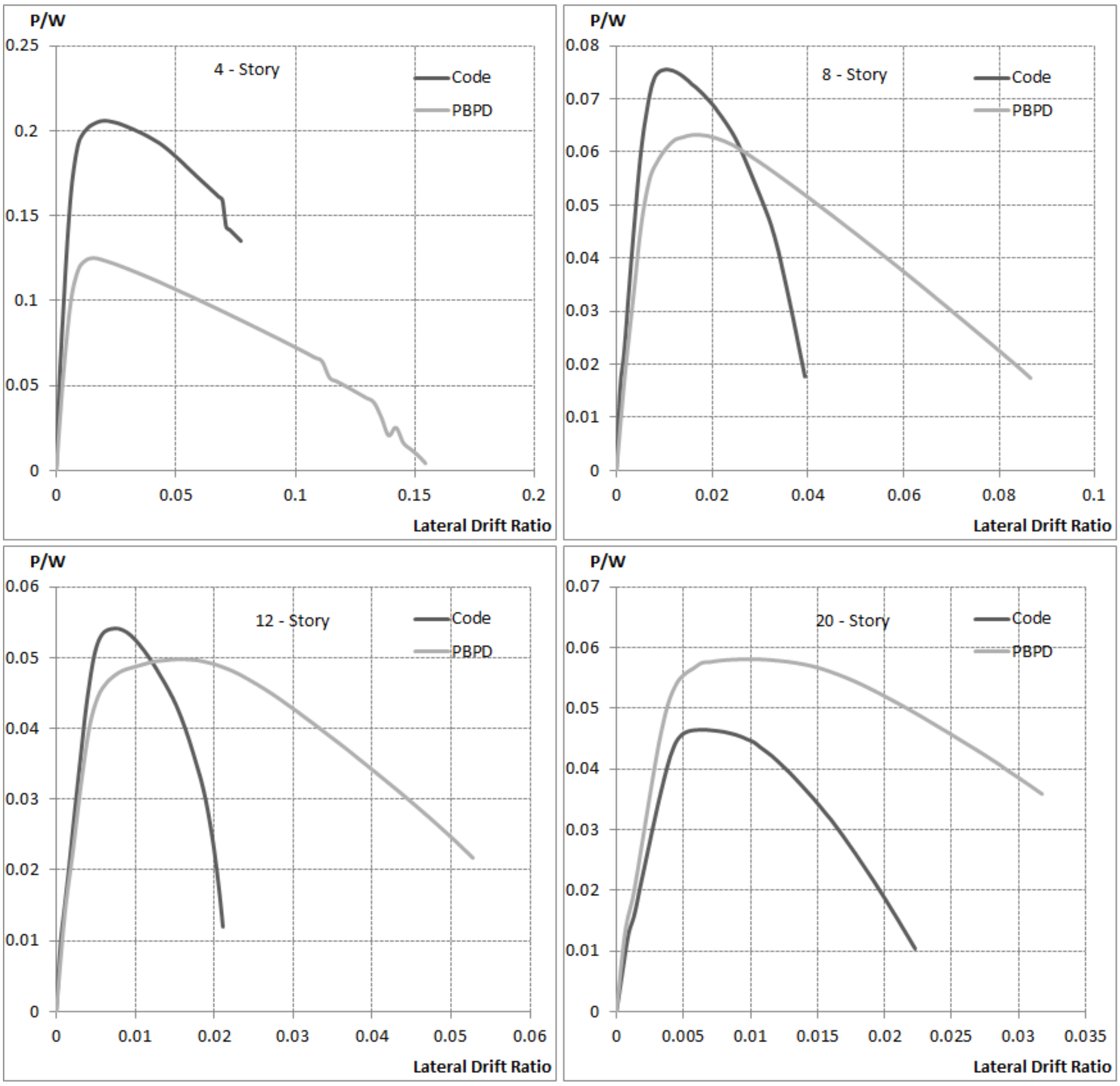


Figure 2: Base shear ratio versus Lateral drift ratio for 4, 8, 12 and 20-Story.

4.2. Degradation of Structural Stiffness

As shown in Figure 3, in the inelastic phase of loading, the stiffness of the structure has a downfall curve. The structural stiffness at different performance levels is presented in Figure 4 - 5.

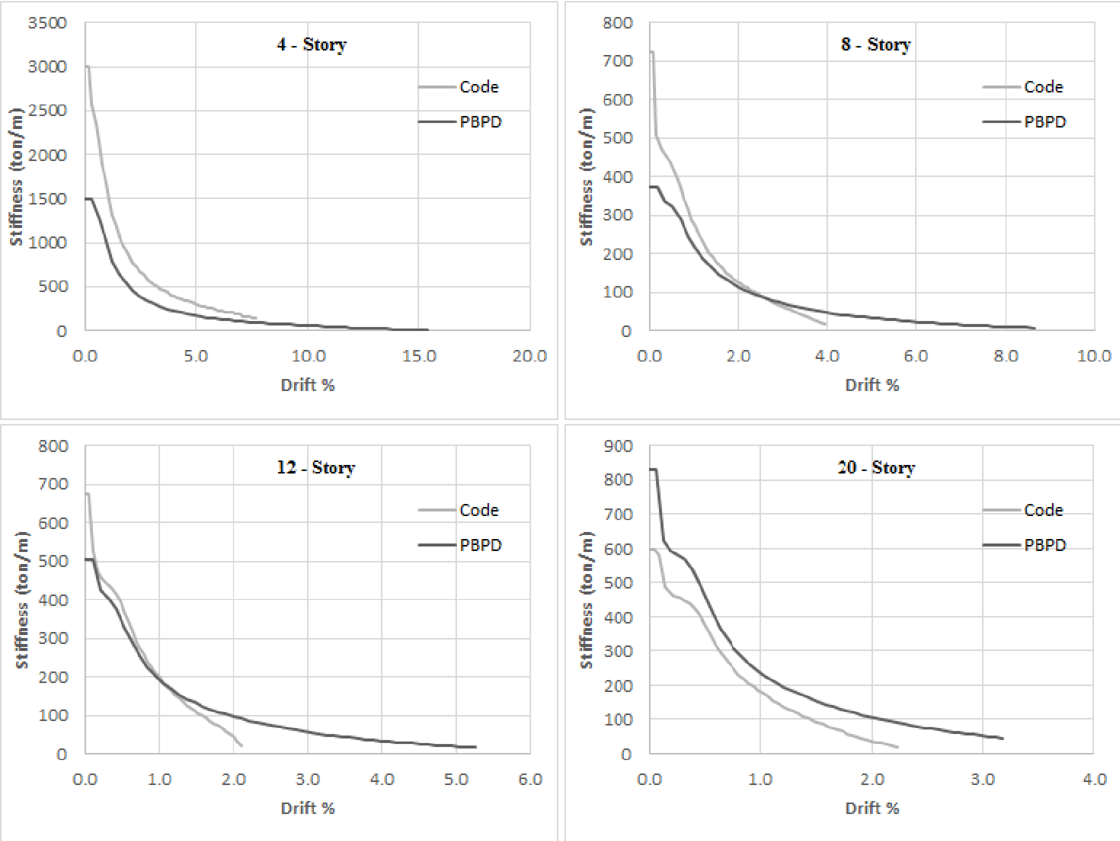
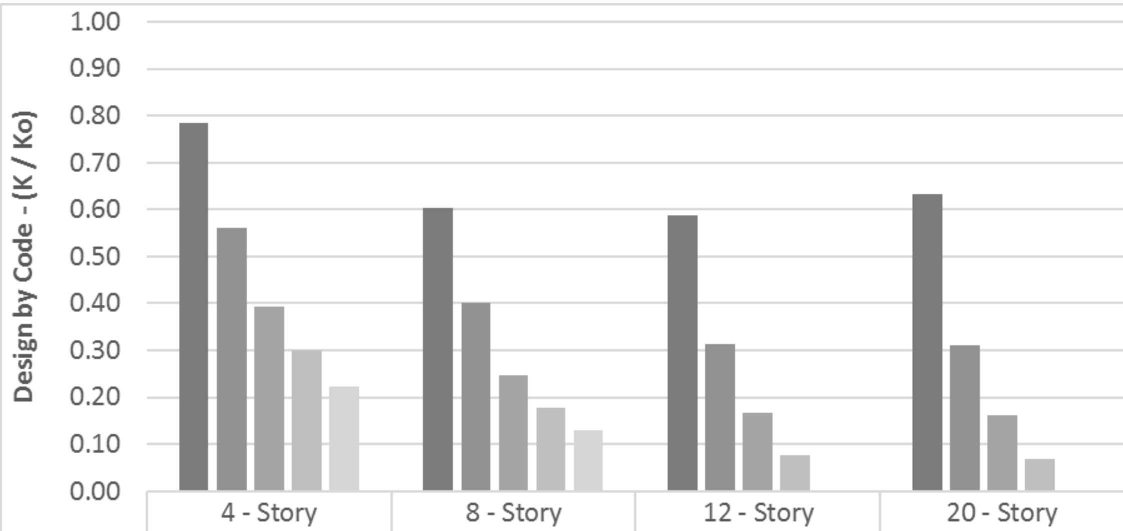
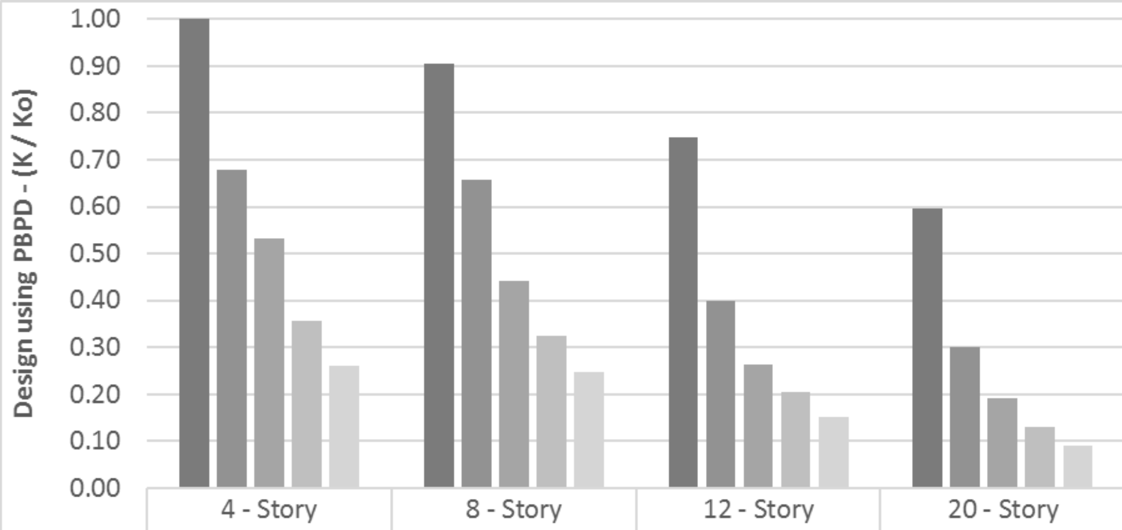


Figure 3: Degradation of structural stiffness with lateral drift for 4, 8, 12 and 20-Story.



	4 - Story	8 - Story	12 - Story	20 - Story
OP	0.784	0.603	0.589	0.634
IO	0.560	0.401	0.313	0.311
DC	0.393	0.247	0.167	0.162
LS	0.299	0.177	0.075	0.070
CP	0.223	0.131	0.000	0.000

Figure 4: Structural stiffness for frames designed following code at different performance levels.



	4 - Story	8 - Story	12 - Story	20 - Story
OP	1.000	0.905	0.747	0.597
IO	0.679	0.658	0.400	0.300
DC	0.531	0.443	0.264	0.192
LS	0.356	0.323	0.204	0.131
CP	0.261	0.247	0.152	0.091

Figure 5: Structural stiffness for frames designed using PBPD at different performance levels.

4.3. Calculation of Damage Index

Using Equation 1, variation of damage indices with lateral drift ratio is presented in Figure 6, for 4, 8, 12 and 20-story frames. The damage indices at different performance levels are presented in Figure 7 - 8. All values of damage indices calculated for frames designed using PBPD, exceeds the corresponding values for frames designed following code. The 12 and 20-story structures designed by code did not reached the 2.5% drift ratio, therefore, the damage indices for that cases were set to be equal to 1.0.

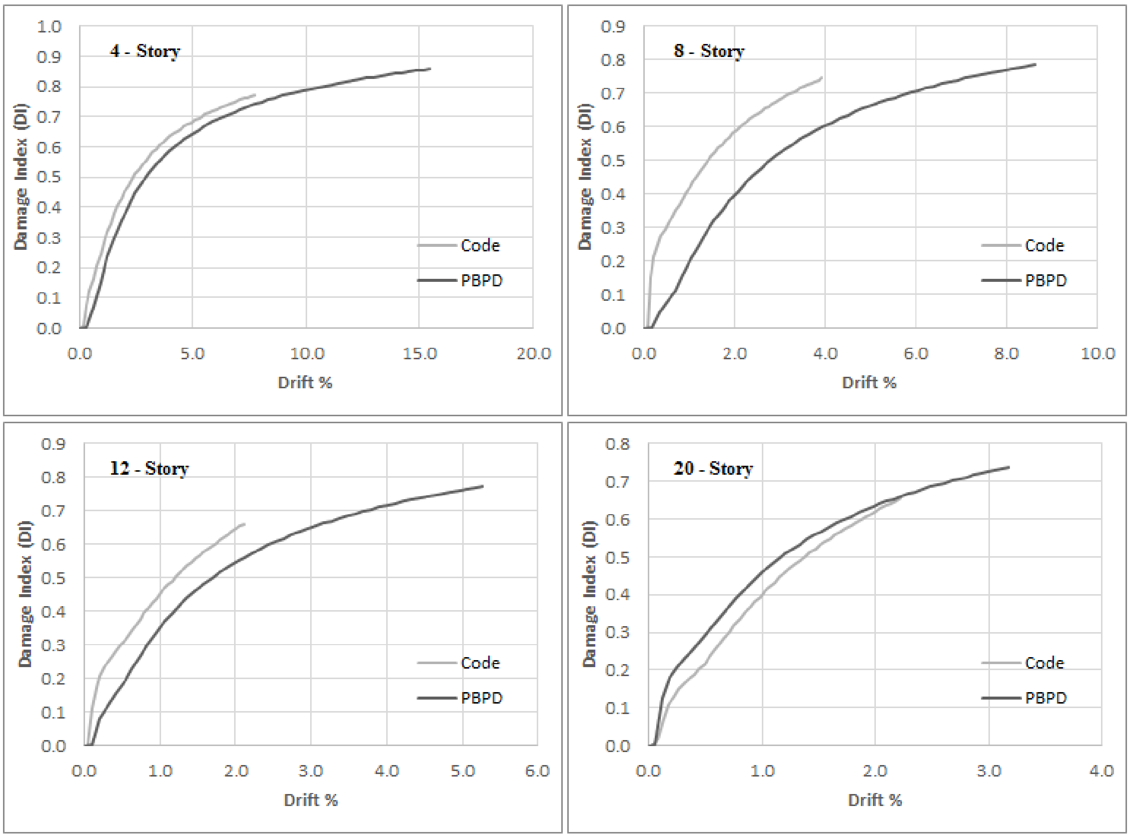
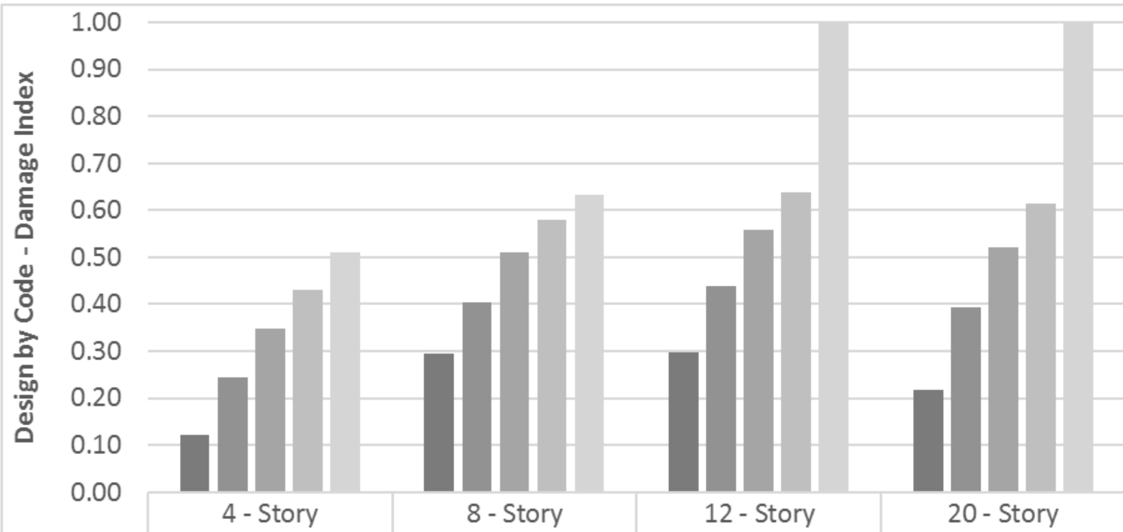
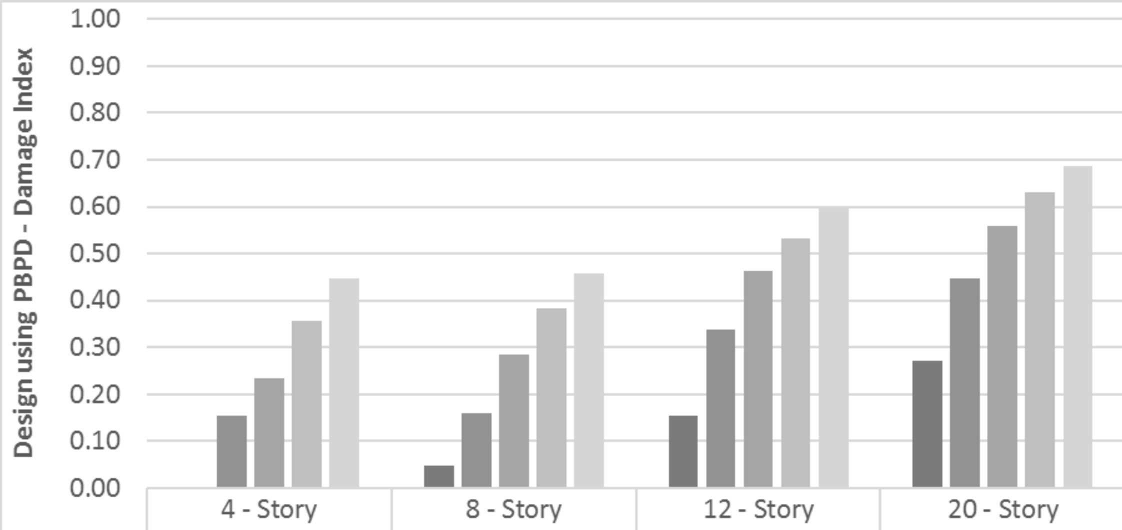


Figure 6: Variation of damage indices with lateral drift ratio for 4, 8, 12 and 20-Story.



	4 - Story	8 - Story	12 - Story	20 - Story
OP	0.123	0.295	0.297	0.217
IO	0.244	0.403	0.439	0.394
DC	0.349	0.511	0.559	0.522
LS	0.430	0.580	0.638	0.614
CP	0.510	0.632	1.000	1.000

Figure 7: Damage indices for frames designed following code at different performance levels.



	4 - Story	8 - Story	12 - Story	20 - Story
OP	0.004	0.048	0.153	0.272
IO	0.154	0.159	0.337	0.446
DC	0.233	0.285	0.463	0.558
LS	0.358	0.382	0.533	0.632
CP	0.448	0.457	0.597	0.685

Figure 8: Damage indices for frames designed using PBPD at different performance levels.

5. CONCLUSIONS

The PBPD method as a direct design method where the drift control and the selection of yield mechanism are initially assumed in the design work, proved that it is an effective method to reach a better performance for reinforced concrete moment resisting frames with fixed base support. It does not need lengthy iterations to achieve a suitable final design. On the other hand, studying damage indices for structures designed by the PBPD method and comparing it with corresponding structures designed using traditional code method introduces a better overview of expected seismic performance of reinforced concrete special moment resisting frames designed by both methods .

This paper presents an assessment of original code design and PBPD methods to design reinforced concrete special moment frames RC SMF systems using damage indices. Main conclusions are as follows:

a. In the inelastic phase of loading, the stiffness of the structure has a downfall curve.
b. Frames designed using PBPD have more ductility than other frames designed following the code-based method, in spite of having lower base shear capacities than the corresponding values for frames designed following code.
c. At different performance levels, frames designed using PBPD showed an improvement regarding the level of damage.

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