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FRAGILITY CURVES OF RC SMF DESIGNED USING PBPD SUBJECTED TO SEISMIC GROUND MOTIONS

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ملخص البحث

طريقة التصميم اللدن القائم على الأداء (PBPD) شاع إستخدامها مؤخراً فى التصميم الزلزالى للمنشآت. تستهدف هذه الطريقة الإزاحة الأفقية 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). فى هذا البحث تم عمل التحليل اللاخطى الإستاتيكى, بالإضافة الي التحليل الديناميكى بإستخدام مجموعة من سجلات الحركة الأرضية لدراسة خمسة مستويات للتصميم الزلزالى لتقييم أداء المنشأت. تم تحليل النتائج للإطارات ذات الركيزة الثابتة للوصول الى منحنيات الهشاشة عند مختلف مستويات الأداء.

ABSTRACT

Performance-Based Plastic Design (PBPD) methodology has been widely used for seismic design of building structures. This method uses a pre-selected target drift and yield mechanisms as key performance objectives. Reinforced concrete special moment frames (RC SMF) as part of seismic force-resisting systems are used in this research, for concrete structures designed according to ACI-318/ASCE-07 and also according to PBPD. Incremental dynamic analysis (IDA) in addition to pushover analysis using SAP2000 software were conducted under a set of ground motion records. The peak accelerations of the records were scaled to provide a set of records with varying ground accelerations. Five levels of performance based seismic designs, operational phase (OP), immediate occupancy (IO), damage control (DC), life safety (LS), and collapse prevention (CP), were considered to assess structural performance. Numerical results obtained for fixed-base support conditions, and fragility curves for several performance limits were generated for both types of models.

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

1. INTRODUCTION

Performance-Based Plastic Design (PBPD) methodology is a derivative of the Performance based Seismic design PBSD method. PBPD is recognized as an ideal method for use in the future practice of seismic design. Performance-based Plastic design method is a direct design method starting from the pre-quantified performance objectives, in which plastic design is performed to detail the frame members and connections in order to achieve the intended yield mechanism and behavior. Control of drift and yielding is also built into the design process from the very start, eliminating or minimizing the need for lengthy iterations to reach the final design [1-7].

Seismic fragility analysis is a tool that aims to evaluate the performance of structures under earthquake events and is an important part of risk analysis of buildings.

It shows the probability to express level of damage at specified ground motion records. Some parameters, such as peak ground acceleration (PGA), peak ground velocity (PGV) and damage index (DI), can be used to develop fragility curves. In this research, peak ground acceleration was selected because it is used to conduct nonlinear history analysis and damage index was also used following Equation 1 to describe the damage state of the structure exposed to increasing ground motion intensity [8-13]. Inelastic Displacement Ductility Ratio (IDDR) was defined as:

$$IDDR = \frac{\delta_m - \delta_y}{\delta_u - \delta_y} = \frac{\mu_m - 1}{\mu_u - 1} \tag{1}$$

where δ_m is the maximum displacement from the dynamic analysis of the structure, δ_y is the yield displacement from the Pushover analysis, δ_u is the final displacement of the failure state from the Pushover analysis. In this research, collapse prevention limit will be used as a final displacement, $\mu_m = \delta_m / \delta_y$ is the displacement ductility demand by the earthquake and $\mu_u = \delta_u / \delta_y$ is the maximum displacement ductility demand by Pushover analysis [14 - 15].

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 [16].

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 [17], was selected for this study. These structures were redesigned by the PBPD approach as introduced in reference [1]. The frames are used to support both vertical and horizontal loads - Figure 1. These structures were redesigned by the PBPD approach with the configuration presented in Table 1 [1]. The baseline structures and the PBPD structures were evaluated for a set of pre-defined earthquake ground motions, using incremental dynamic analysis (IDA) as will be described later. Fragility curves were developed for each structure for several performance levels and the damage index IDDR.

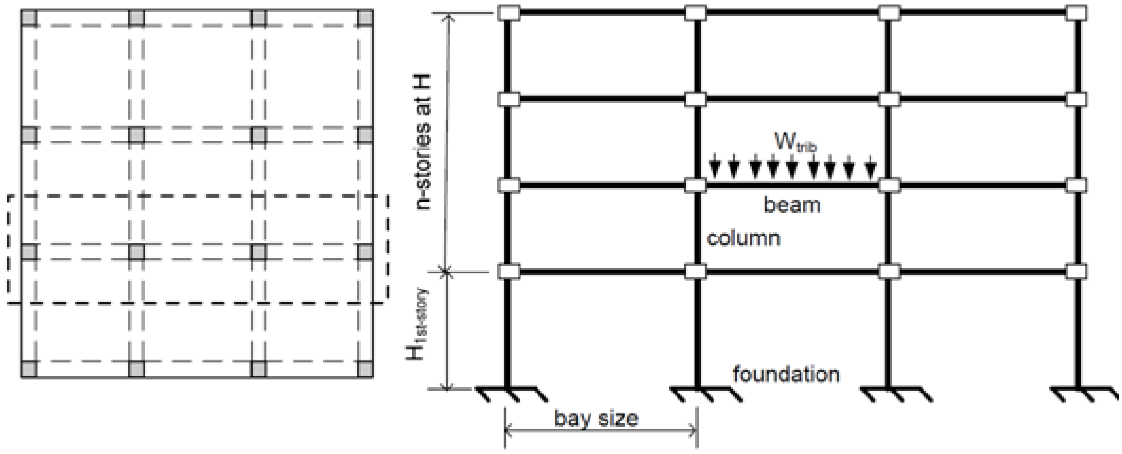

Figure 1: Typical floor plan and typical elevation of the RC SMF. [1]

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)

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)

2.3. Selected Ground Motion Records

In order to carry out incremental dynamic analyses, an appropriate set of acceleration time histories is required. Randomness in ground motion is taken into account by using 44 earthquake records. In this study, Far-Field record set includes twenty-two records (considering both X and Y components of the record that makes a total of 44 individual components) that cover FEMA P695 [17], from the Pacific Earthquake Engineering Research Center (PEER) ground motion database. For each record. Table 2 and 3 summarize the magnitude, year, and name of the event, as well as the name of the recording station. The twenty-two records occurred between 1971 and 1999. Event magnitudes range from M6.5 to M7.6 with an average magnitude of M7.0 and site-source average distance is 16.4 km for the Far-Field record set. Notice that a minimum of 7 time-history records must be applied to the structure, to be allowed to use average results instead of the most unfavorable ones, as suggested by several modern seismic codes [UBC,1997; EC8-1, 2005; ECP 201, 2012], however all the 22 pairs of records were used in this study to cover a wider range of results. Finally, the 44 records have been scaled in order to match their PGA with the target PGA that ranges from 0.1g to 1.0g.

Table 2: Parameters of recorded ground motions for the far-field record set.

ID Number	Name	Record Sequence Number	Year	Magnitude	PGA_{max} (g)	PGV_{max} (m/s)
1	San Fernando	68	1971	6.6	0.21	0.19
2	Friuli, Italy	125	1976	6.5	0.35	0.31
3	Imperial Valley	169	1979	6.5	0.35	0.33
4	Imperial Valley	174	1979	6.5	0.38	0.42
5	Superstition Hills	721	1987	6.5	0.36	0.46
6	Superstition Hills	725	1987	6.5	0.45	0.36
7	Loma Prieta	752	1989	6.9	0.53	0.35
8	Loma Prieta	767	1989	6.9	0.56	0.45
9	Cape Mendocino	829	1992	7.0	0.55	0.44
10	Landers	848	1992	7.3	0.42	0.42
11	Landers	900	1992	7.3	0.24	0.52
12	Northridge	953	1994	6.7	0.52	0.63
13	Northridge	960	1994	6.7	0.48	0.45
14	Kobe, Japan	1111	1995	6.9	0.51	0.37
15	Kobe, Japan	1116	1995	6.9	0.24	0.38
16	Kocaeli, Turkey	1148	1999	7.5	0.22	0.4
17	Kocaeli, Turkey	1158	1999	7.5	0.36	0.59
18	Chi-Chi, Taiwan	1244	1999	7.6	0.44	1.15
19	Chi-Chi, Taiwan	1485	1999	7.6	0.51	0.39
20	Duzce, Turkey	1602	1999	7.1	0.82	0.62
21	Manjil, Iran	1633	1990	7.4	0.51	0.54
22	Hector Mine	1787	1999	7.1	0.34	0.42

Table 3: Recording station and component data for the far-field record set.

ID Number	Recording Station	Horizontal Records	
		X - Component	Y - Component
1	LA - Hollywood Stor	SFERN/PEL090	SFERN/PEL180
2	Tolmezzo	FRIULI/A-TMZ000	FRIULI/A-TMZ270
3	Delta	IMPVALL/H-DLT262	IMPVALL/H-DLT352
4	El Centro Array #11	IMPVALL/H-E11140	IMPVALL/H-E11230
5	El Centro Imp. Co.	SUPERST/B-ICC000	SUPERST/B-ICC090
6	Poe Road (temp)	SUPERST/B-POE270	SUPERST/B-POE360
7	Capitola	LOMAP/CAP000	LOMAP/CAP090
8	Gilroy Array #3	LOMAP/G03000	LOMAP/G03090
9	Rio Dell Overpass	CAPEMEND/RIO270	CAPEMEND/RIO360
10	Coolwater SCE	LANDERS/CLW-LN	LANDERS/CLW-TR
11	Yermo Fire Station	LANDERS/YER270	LANDERS/YER360
12	Beverly Hills - Mulhol	NORTHR/MUL009	NORTHR/MUL279
13	Canyon Country-WLC	NORTHR/LOS000	NORTHR/LOS270
14	Nishi-Akashi	KOBE/NIS000	KOBE/NIS090
15	Shin-Osaka	KOBE/SHI000	KOBE/SHI090
16	Arcelik	KOCAELI/ARC000	KOCAELI/ARC090
17	Duzce	KOCAELI/DZC180	KOCAELI/DZC270
18	CHY101	CHICHI/CHY101-E	CHICHI/CHY101-N
19	TCU045	CHICHI/TCU045-E	CHICHI/TCU045-N
20	Bolu	DUZCE/BOL000	DUZCE/BOL090
21	Abbar	MANJIL/ABBAR--L	MANJIL/ABBAR--T
22	Hector	HECTOR/HEC000	HECTOR/HEC090

3. MODEL DESCRIPTION

2D finite-element models of the structures were generated using SAP2000 v20. The column bases are fixed and the effects of gravity loads and second-order effects are considered through the geometric nonlinearities. Nonlinear dynamic time history analysis was performed to evaluate the structural response of the building subject to the previously mentioned ground motions in addition to pushover analysis. Stiffness modifiers utilized for beams and columns for dynamic analysis are 0.35 and 0.70 respectively.

4. RESULTS AND DISCUSSION

4.1. Pushover Analysis Results

To evaluate the performance behavior of structures designed based on the two methods (Code design and PBPD), a static nonlinear analysis (Pushover analysis) is conducted. For static nonlinear analysis the equivalent static load pattern is selected and the structures are pushed over a specified drift of roof. The roof displacement of each structure and the performance of structures designed based on PBPD method is compared with structures designed based on the conventional method, in addition to design base shear for each case - Figure 2.

It is also noted that in the structures that are designed by conventional method many columns yielded, however no column yielded in the structures that are designed by PBPD method. It can be concluded that more energy is dissipated in structures that are designed by PBPD method and expected yield mechanism is reached.

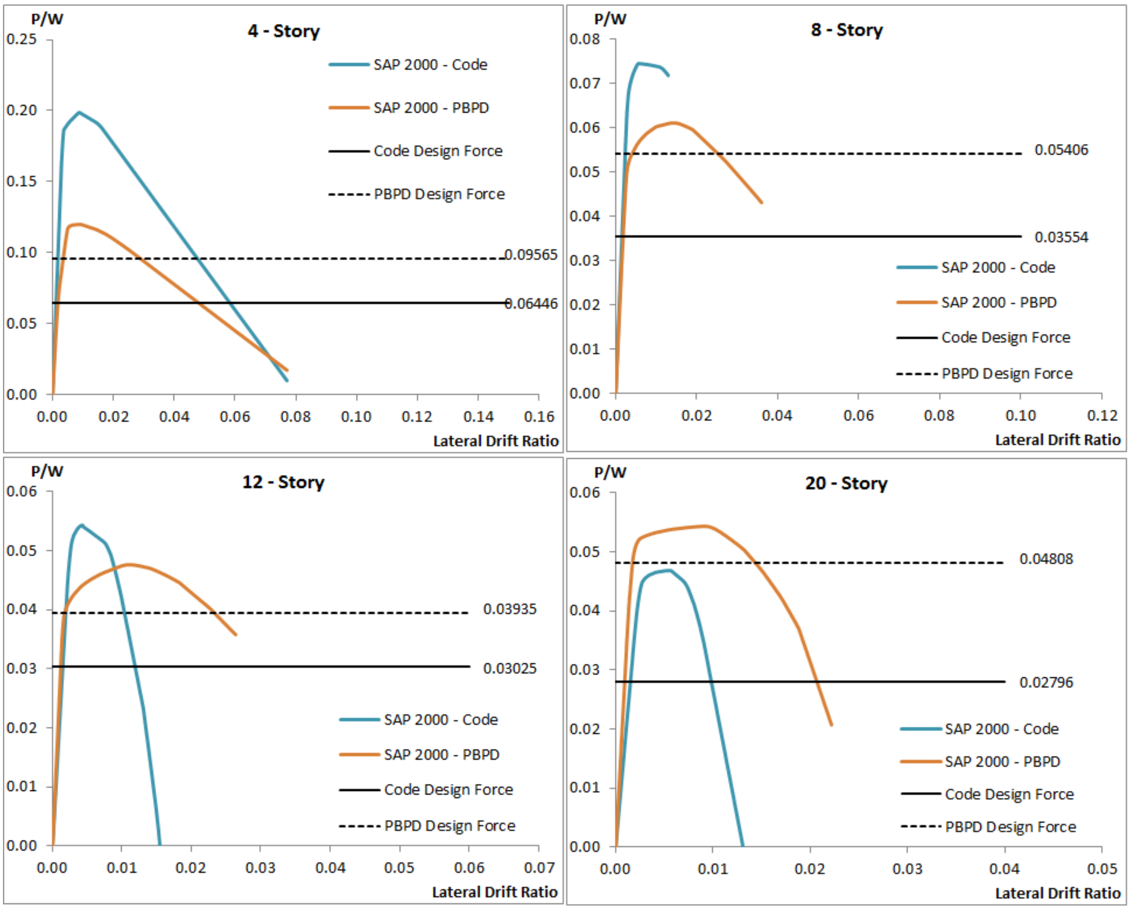

Figure 2: Pushover results - 4, 8, 12 and 20 story.

4.2. Fragility Curves

Incremental Dynamic Analysis (IDA) was performed using SAP2000 software under the previously indicated set of ground motion records. Ten intensity levels of acceleration were used, starting from 0.1g to 1.0g. The maximum displacement results from IDA for all records were recorded, and used to develop fragility curves for all structures based on the reference performance levels Figure 3 to 6. The outputs of pushover analysis (P-Delta Curve) were used to determine the ultimate base shear capacity of the structure and its corresponding roof displacement. Maximum roof displacement results from IDA and that corresponding to structural capacity were used to develop another fragility group of curves describing the probability of exceeding the roof displacement at which structure reaches its capacity (Figure 7).

At the design ground acceleration, the desired performance levels were met for both design methods. The probability of exceeding the roof displacement at which structure reaches its capacity decreased (Displacement corresponding to structural maximum capacity from pushover analysis) when using PBPD method. except for the 4-story structure as shown in Figure 7.

The damage state of the structure exposed to increasing ground motion intensity was calculated based on Equation 1 and presented in Figure 8. The calculation is based on the yield displacement extracted from pushover idealized bilinear curve, maximum displacement from the IDA results and final displacement corresponding to 2.5% roof drift. Damage index calculated for structures designed using PBPD is almost the same as that of the code method for the 12 and 20-story structures. It increased for the 4 and 8-story structures designed using PBPD.

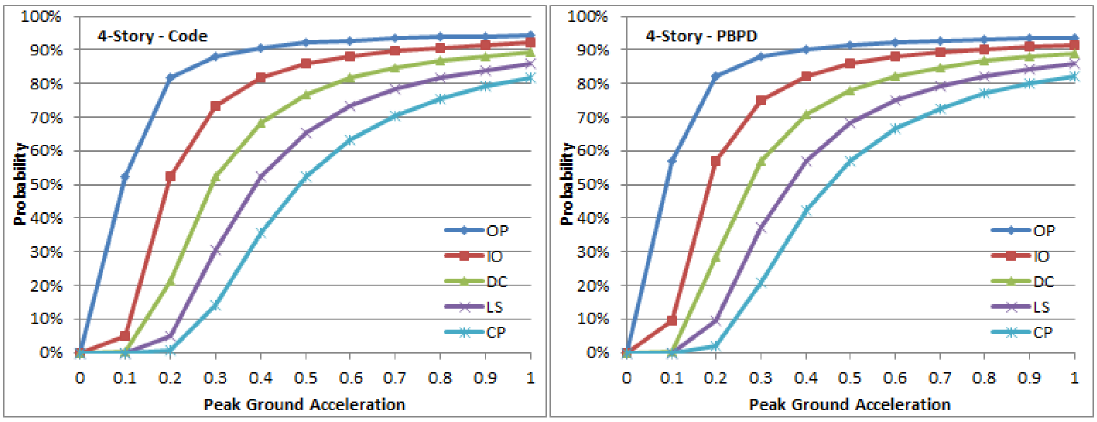

Figure 3: Fragility curves - Probability of exceeding performance levels - 4 story.

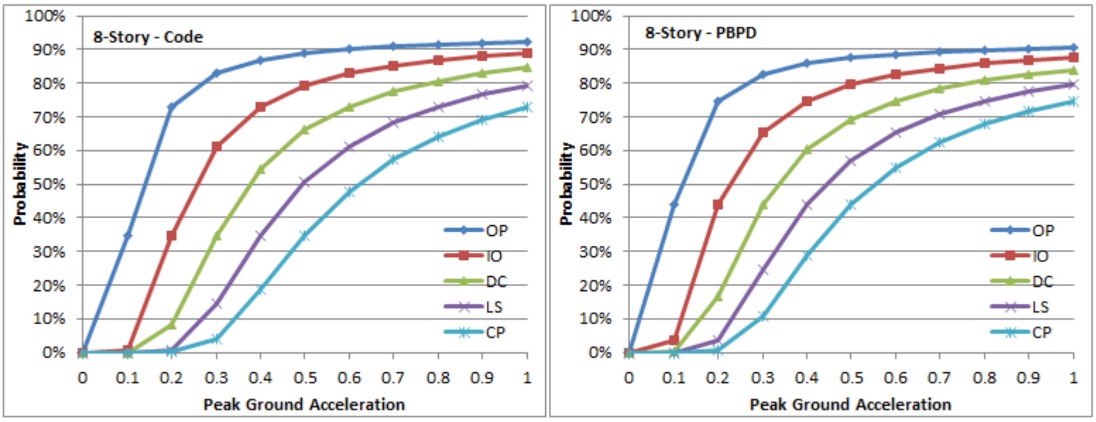

Figure 4: Fragility curves - Probability of exceeding performance levels - 8 story.

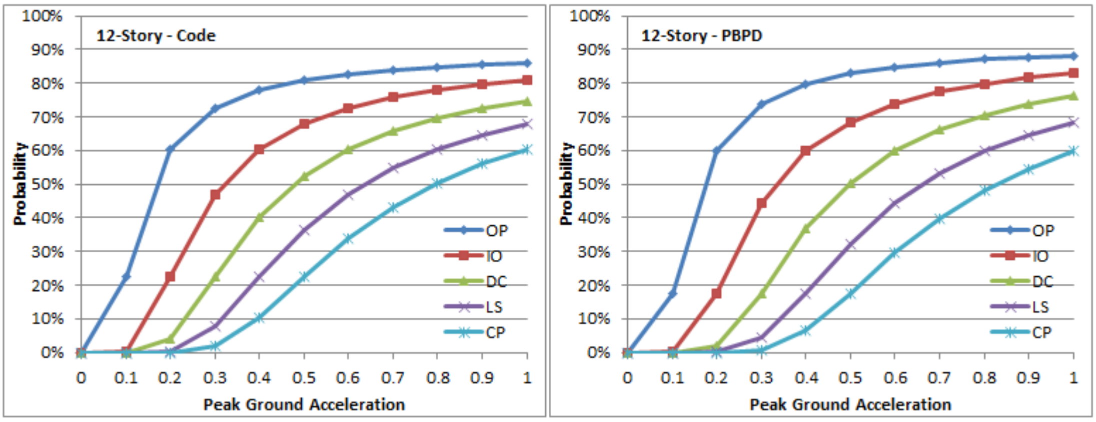

Figure 5: Fragility curves - Probability of exceeding performance levels - 12 story.

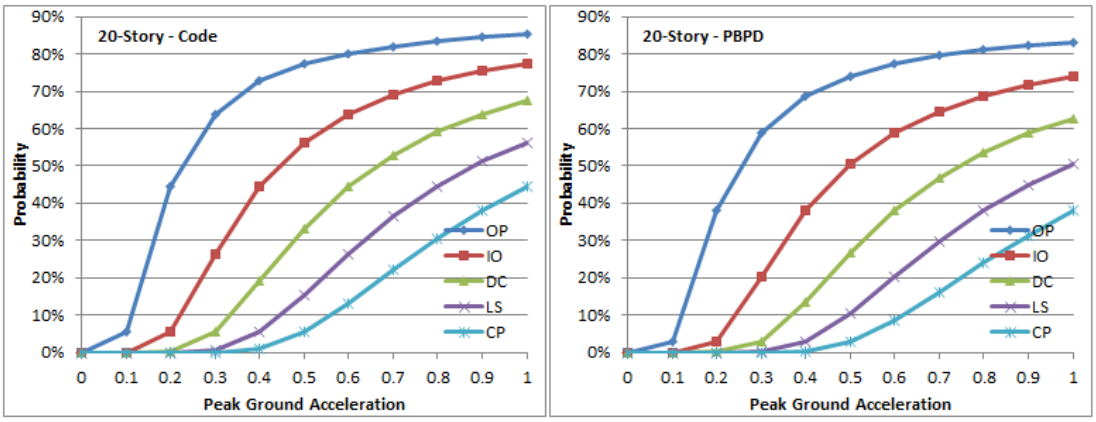

Figure 6: Fragility curves - Probability of exceeding performance levels - 20 story.

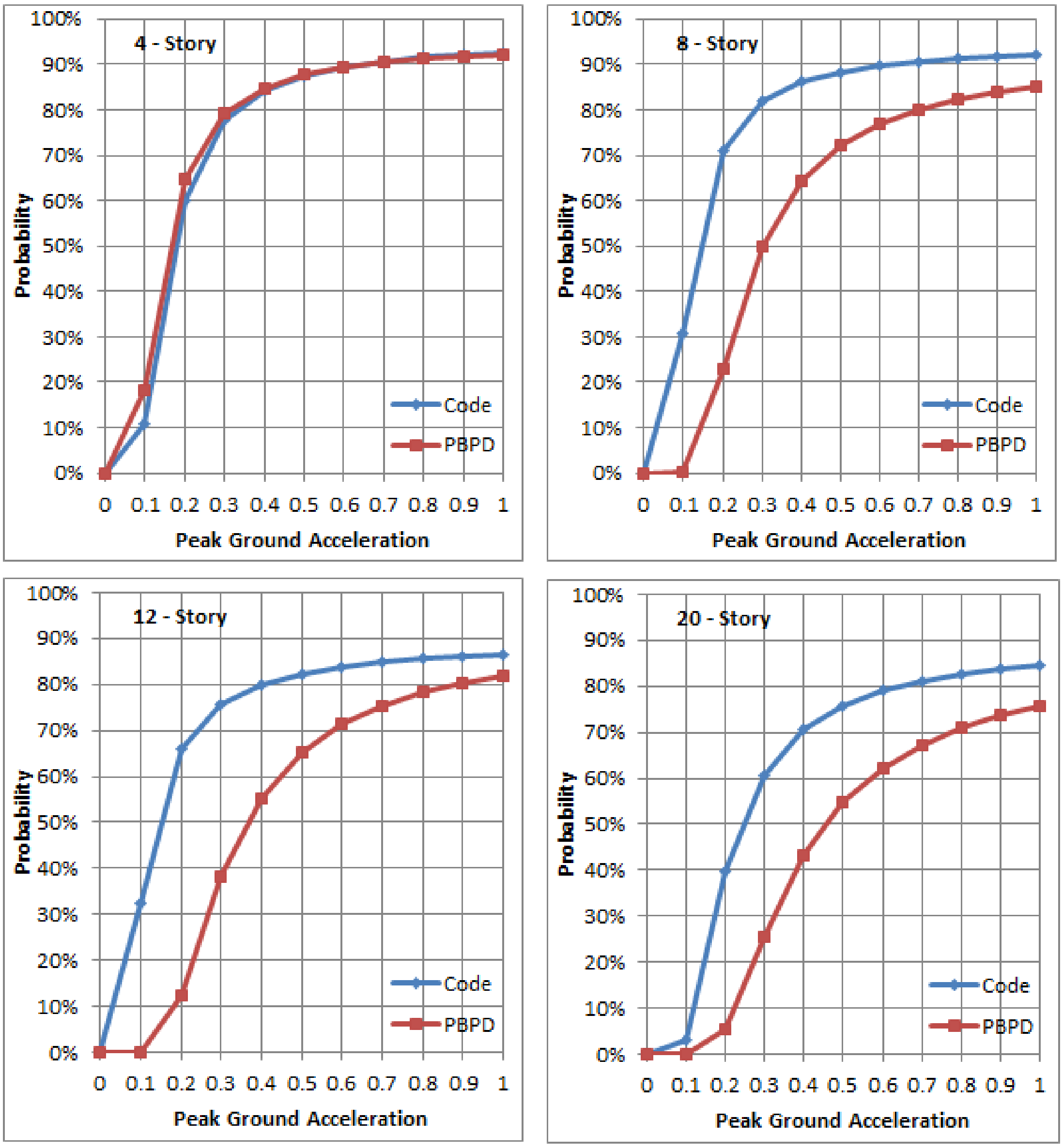

Figure 7: Fragility curves - Probability of exceeding the roof displacement at which structure reaches its capacity - 4, 8, 12 and 20 story.

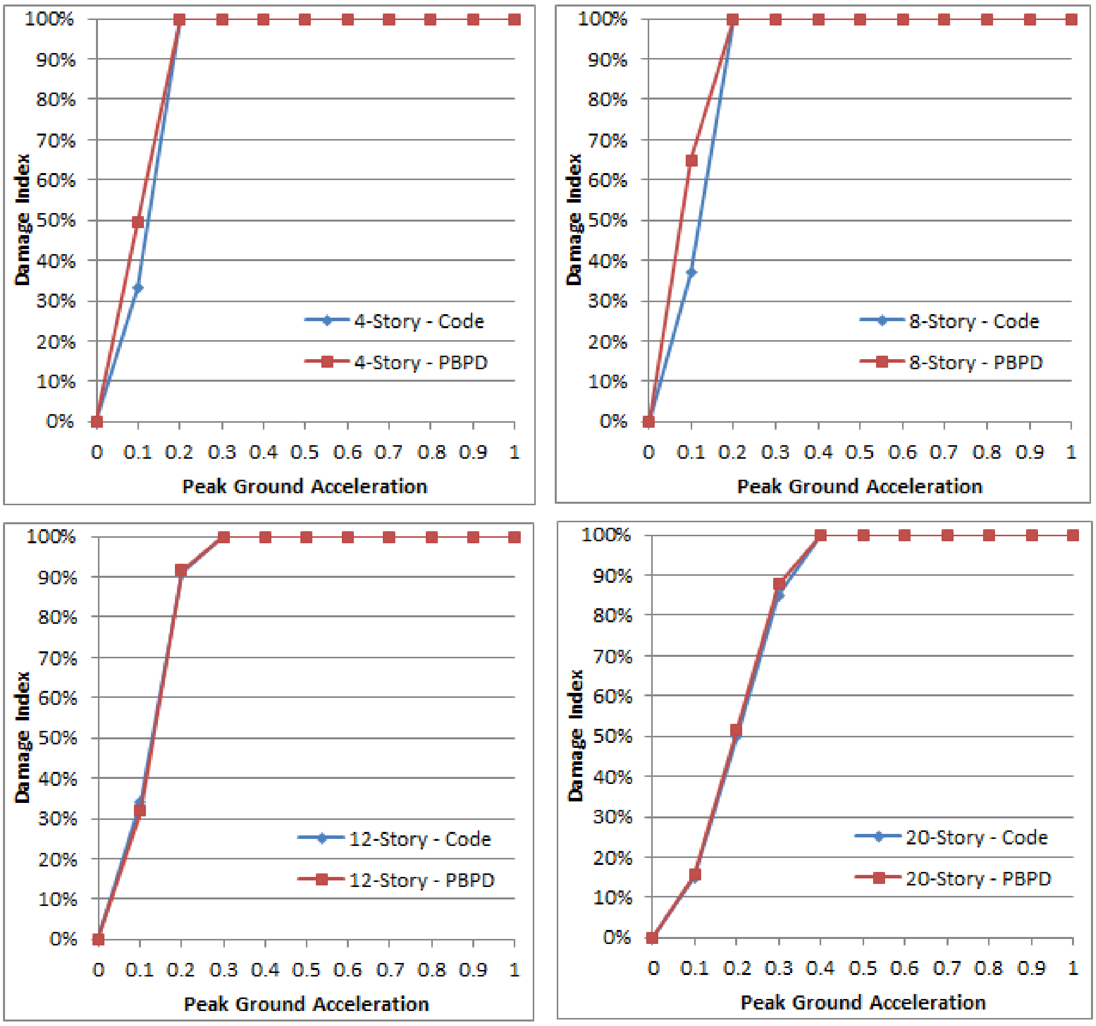

Figure 8: Damage Index Fragility curves - 4, 8, 12 and 20 story.

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 fragility curves 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 RC SMF systems using fragility curves. Main conclusions are as follows:

a. Design base shear and strength

i. Design base shear needed for PBPD is greater than that of code traditional method.

ii. Strength of structures designed using PBPD method is less than that of the code traditional method.

iii. For the structures designed by the PBPD method, no hinges appear in columns before reaching the strength of the structure.

iv. The area under P-Delta curve which represents the energy dissipated by the structure is higher in case of structures designed using PBPD method.

b. Performance objectives and damage indices

i. At the design ground acceleration, the desired performance levels were met for both design methods.

ii. The probability of exceeding the roof displacement at which structure reaches its capacity decreased (Displacement corresponding to structural maximum capacity from pushover analysis) when using PBPD method except for the 4-story structure.

iii. Damage index calculated for structures designed using PBPD is almost the same as that of the code method for the 12 and 20-story structures. It increased for the 4 and 8-story structures designed using PBPD.

iv. In general, enhancement in the behavior of all structures were noticed when using PBPD method.

REFERENCES

[1] Liao, W. C., & Goel, S. C. (2014). Performance-based seismic design of RC SMF using target drift and yield mechanism as performance criteria. *Advances in Structural Engineering*, *17*(4), 529-542.

[2] Goel, S. C., Liao, W. C., Reza Bayat, M., & Chao, S. H. (2010). Performance-based plastic design (PBPD) method for earthquake-resistant structures: an overview. *The structural design of tall and special buildings*, *19*(1-2), 115-137.

[3] Banihashemi, M. R., Mirzagoltabar, A. R., & Tavakoli, H. R. (2015). Development of the performance based plastic design for steel moment resistant frame. *International Journal of Steel Structures*, *15*(1), 51-62.

[4] Banihashemi, M. R., Mirzagoltabar, A. R., & Tavakoli, H. R. (2015). Performance-based plastic design method for steel concentric braced frames. *International Journal of Advanced Structural Engineering (IJASE)*, *7*(3), 281-293.

[5] Subhash C. Goel, Shih-Ho Chao, Sutat Leelataviwat, Soon-Sik Lee (2008, October 12-17). Performance-based plastic design (PBPD) method for earthquake-resistant structures. *The 14th World Conference on Earthquake Engineering*. Beijing, China.

[6] Lee, S. S., Goel, S. C., & Chao, S. H. (2004, August). Performance-based seismic design of steel moment frames using target drift and yield mechanism. In *13th world conference on Earthquake Engineering*. Vancouver, BC: Canada.

[7] Zameeruddin, M., & Sangle, K. K. (2016, May). Review on Recent developments in the performance-based seismic design of reinforced concrete structures. In *Structures* (Vol. 6, pp. 119-133). Elsevier.

[8] Saruddin, S. N. A., & Nazri, F. M. (2015). Fragility curves for low-and mid-rise buildings in Malaysia. *Procedia Engineering*, *125*, 873-878.

[9] Federal Emergency Management Agency. (2006). Next-Generation Performance-Based Seismic Design Guidelines - Program Plan for New and Existing Buildings. FEMA 445, prepared by Applied Technology Council (ATC-58 Project).

[10] Rota, M., Penna, A., & Magenes, G. (2008). A procedure for deriving analytical fragility curves for masonry buildings. *The 14th World Conference on Earthquake Engineering*. Beijing, China.

[11] Ibrahim, Y. E., & El-Shami, M. M. (2011). Seismic fragility curves for mid-rise reinforced concrete frames in Kingdom of Saudi Arabia. *The IES Journal Part A: Civil & Structural Engineering*, *4*(4), 213-223.

[12] Majd, M., Hosseini, M., & Moein Amini, A. (2012). Developing Fragility Curves for Steel Building with X-Bracing by Nonlinear Time History Analyses. *Proceedings of the 15WCEE, Lisbon, Portugal*, 24-28.

[13] Bakhshi, A., & Asadi, P. (2013). Probabilistic evaluation of seismic design parameters of RC frames based on fragility curves. *Scientia Iranica*, *20*(2), 231-241.

[14] Nateghi, F., & Shahsavar, V. L. (2004, August). Development of fragility and reliability curves for seismic evaluation of a major prestressed concrete bridge. In *13th World Conference on Earthquake Engineering, Vancouver, BC Canada*.

[15] Saini, A. S., Vosooghi, A., & Saiidi, M. S. PROBABILISTIC PERFORMANCE-BASED SEISMIC DESIGN OF BRIDGE COLUMNS. *The 4th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Kos Island, Greece, 12–14 June 2013*

[16] Council, B. S. S. (2000). Prestandard and commentary for the seismic rehabilitation of buildings. Report FEMA-356, Washington, DC.

[17] Applied Technology Council. (2009). Quantification of building seismic performance factors. US Department of Homeland Security, FEMA P-695.