

Evaluation of Various Proposed Lateral Load Patterns for Seismic Design of Steel Moment Resisting Frames



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SUMMARY:

The seismic design of buildings is normally based on the equivalent lateral forces provided in seismic design guidelines. The height-wise distribution of these lateral design loads predominantly correspond to the first vibration mode. However, as structures exceed their elastic limits in severe earthquakes, these load patterns may not represent the nonlinear response, and therefore they would not necessarily lead to efficient distribution of strength within the structure. A brief review of alternative lateral load patterns resulting from investigations on nonlinear seismic response of structures is presented. Due to the limits caused by idealizations and simplifications inherent in such investigations, the practicality and accuracy of the proposed load patterns should be evaluated and verified before proceeding into the practice. This paper examines the efficiency of various patterns for seismic design of 5 and 10-storey SMRFs. The nonlinear response and seismic performance of these models are studied under five different seismic records.

Keywords: Seismic design, Lateral load pattern, Nonlinear response history analysis, Steel moment resisting frames, Ductility demand

1. INTRODUCTION

The seismic performance of different code-based designed lateral load resisting structural systems have been broadly studied over the last three decades. In the light of these investigations, it was found that the lateral load distribution used by current seismic design guidelines does not always lead to the uniform distribution of ductility demand and damage. Hence the employment of such lateral load patterns does not guarantee the optimal distribution of structural materials throughout the structures in the nonlinear range of behavior. (Karami Mohammadi et al., 2004; Moghaddam et al., 2006, 2008; Hajrasouliha et al., 2009, 2012).

Chopra evaluated the ductility demands of several shear building with elastoplastic behaviour subjected to El-Centro earthquake of 1940 (Chopra, 2001). The storey shear strength of these models conformed to the height-wise distribution pattern of the earthquake forces specified in the Uniform Building Code. He found that this distribution pattern does not lead to equal ductility demand in all stories, and that in most cases the ductility demands in the first storey is the largest among all stories.

Moghaddam proportioned the relative storey yield strength of a number of shear buildings in accordance with some arbitrarily chosen distribution patterns as well as the distribution pattern suggested by the UBC. The ductility and displacement demands of these models were calculated. It was concluded that: (a) the pattern suggested by the code does not lead to a uniform distribution of ductility, and (b) a uniform distribution of ductility with a relatively smaller maximum ductility demand can be obtained from other patterns (Moghaddam, 1996; Moghaddam et al., 1999). These findings have been confirmed by further investigations (Moghaddam et al., 2006, 2008; Karami Mohammadi et al., 2004).

Lee et al. and Chao et al. analyzed a series of steel moment and braced frames subjected to a wide variety of earthquake records. They showed that in general there is a discrepancy between the earthquake induced shear forces and the forces determined by assuming code-based design load distribution patterns (Lee et al., 2001; Chao et al., 2007). Based on the results of their studies, they suggested a new lateral force distribution for seismic loads to address the influence of increasing higher mode effects in the inelastic range of behaviour. However, the effects of ground motion characteristics and the degree of nonlinearity were not considered in their suggested load distribution.

Moghaddam and Hajirasouliha developed an effective optimisation method to find optimum lateral load distribution for seismic design of shear-building structures to obtain uniform storey ductility. They showed that, for the same target storey-ductility demand, structures designed with the average of optimum load patterns for a set of earthquakes with similar characteristics, have relatively lower structural weight compared to those designed conventionally (Moghaddam and Hajirasouliha, 2006; Hajirasouliha et al., 2009, 2012). Their proposed load pattern is a function of structural performance level (i.e. storey ductility), and therefore, is suitable for performance-based seismic design of structures.

The extensive researches conducted by Goel et al. leads to development of a new seismic design lateral force distribution based on inelastic state of a structure and also a new methodology (i.e. Performance-Based Plastic Design (PBPD)) for seismic design of a wide diversity of steel framing systems (such as Moment-Resisting Frames (MRF), Eccentrically-Braced Frames (EBF), Special Truss Moment Frames (STMF), etc.). In these studies, performance limit states are expressed by predictable global yield mechanism and pre-designated target drift limit. The design base shear for each performance level is derived by an energy-based method where the required energy 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 (Lee, et al., 2001, 2004; Leelataviwat, 2002; Chao and Goel, 2005, 2006, 2007; Goel et al. 2008, 2010). By using a similar approach (storey shear distribution), Park and Medina (2006) proposed a design lateral force distribution for moment frame structures (Park, 2007; Park and Medina, 2007).

Some other notable studies also conducted in this field of investigations (Kato et al., 1982; Deguchi et al., 2008; Motamedi and Nateghi-A., 2008; etc.).

In this paper, various proposed lateral load patterns for seismic design of steel moment resisting frames are evaluated using nonlinear response history analyses of two building examples subjected to a wide range of natural earthquake records.

2. CASE STUDIES

2.1. Description of Buildings Used in Evaluation

To evaluate the proposed seismic lateral load patterns, two 5-bay moment-resisting steel frames with 5 and 10 storeys (as shown in Fig. 2.1) were examined. Uniformly distributed dead load of 35.316 kN/m were assumed to be applied on all beams and uniform service live load have been considered as 11.772 and 8.829 kN/m for interior storeys and roof, respectively.

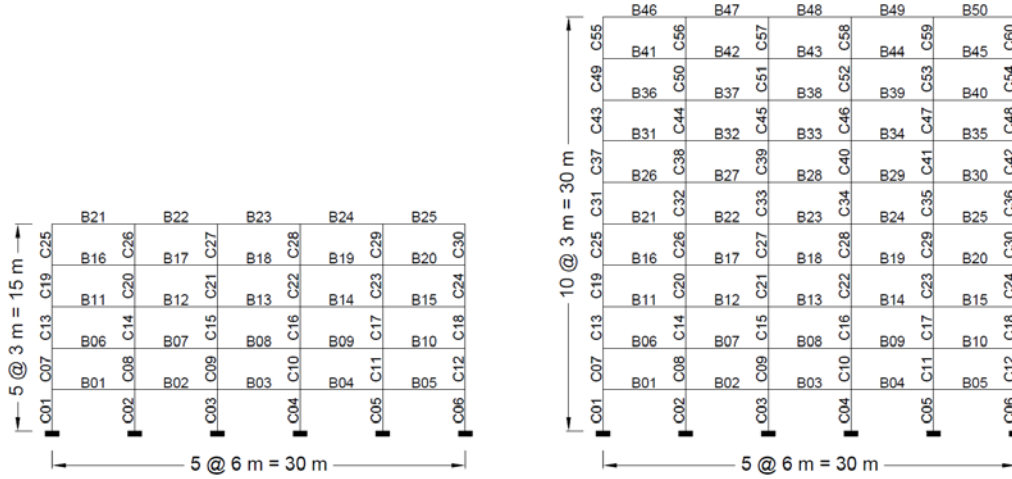


Figure 2.1. Mathematical models for 5-bay moment-resisting steel frames with 5 and 10 storeys

2.2. Ground Motions

To investigate the efficiency of various proposed lateral load patterns, five medium-to-strong ground motion records obtained from PEER ground motion database (Pacific Earthquake Engineering Research Center (PEER), 2000) were used as listed in Table 2.1. All of these selected records correspond to sites of soil profile as USGS type C which is similar to soil type D of ASCE/SEI 7-10 (American Society of Civil Engineers (ASCE), 2010) and were recorded in a low-to-moderate distance from the fault rupture (between 5 and 15 km) with rather high magnitudes (i.e. $M_s > 6.7$). These records are used directly without being normalized.

Table 2.1. Characteristics of ground motions used in this study

EQ. #	Earthquake	Record/Component	Station	Magnitude (Ms)	PGA (g)	PGV (cm/s)	PGD (cm)
a	Duzce, Turkey 1999	DUZCE/DZC270	Duzce	7.3	0.535	83.5	51.59
b	Imperial Valley 1979	IMPVALL/HE04140	955 El Centro Array #4	6.9	0.485	37.4	20.23
c	Loma Prieta 1989	LOMAP/G03000	47381 Gilroy Array #3	7.1	0.555	35.7	8.21
d	Cape Mendocino 1992	CAPEMEND/PET090	89156 Petrolia	7.1	0.662	89.7	29.55
e	Northridge 1994	NORTHR/NWH360	24279 Newhall - Fire Sta	6.7	0.59	97.2	38.05

2.3. Considered Seismic Lateral Load Patterns

Several seismic lateral load patterns were reviewed and evaluated in this study as described below.

2.3.1. Code-compliant (ASCE/SEI 7-10) lateral load pattern (P.1)

In ASCE/SEI 7-10 (American Society of Civil Engineers (ASCE), 2010), the lateral seismic force (F_i) induced at any level shall be determined from the following equations:

$$F_i = \frac{w_i h_i^k}{\sum_{j=1}^n w_j h_j^k} V; \quad i = 1, 2, \dots, n; \quad k = \begin{cases} 1 & ; \text{if } T \leq 0.5 \\ 0.5T + 0.75 \text{ or } 2 & ; \text{if } 0.5 < T < 2.5 \\ 2 & ; \text{if } T \geq 2.5 \end{cases} \quad (2.1)$$

In which w_i is the portion of the total effective seismic weight of the structure (W) located or assigned to level i , h_i is the height from the base to level i , n is the number of storeys, V is the total design

lateral force or shear at the base of the structure, k is an exponent related to the structure period and T is the fundamental period of the structure in the direction under consideration.

2.3.2. Lateral load pattern proposed by Goel et al. (P.2)

The format of Goel et al. design lateral force distribution is as follows (Chao and Goel, 2005, 2006, 2007; Goel et al. 2008, 2010):

$$F_i = C_{iv}V$$

$$C_{iv} = (\beta_i - \beta_{i+1}) \left(\frac{w_n h_n}{\sum_{j=1}^n w_j h_j} \right)^{\alpha T^{-0.2}} ; \quad \beta_{n+1} = 0 \quad (2.2)$$

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{j=i}^n w_j h_j}{w_n h_n} \right)^{\alpha T^{-0.2}} ; \quad i = 1, 2, \dots, n$$

Where F_i is the lateral force at level i , V is the total design base shear, β_i is the shear distribution factor at level i , w_j is the seismic weight at level j , h_j is the seismic height of level j from the base, n is the number of storeys, T is the fundamental period, V_i is the storey shear force at level i . The value of parameter α was originally proposed as 0.5 by Lee and Goel, which was later modified to 0.75 based on more extensive nonlinear response history analyses on a wide variety of steel framing systems (Lee and Goel, 2001).

2.3.3. Lateral load pattern proposed by Hajirasouliha and Moghaddam (P.3)

The Hajirasouliha and Moghaddam lateral load pattern can be expressed as follows (Moghaddam et al., 2006, 2008; Hajirasouliha et al., 2009, 2012):

$$F_i = \frac{w_i \phi_i}{\sum_{j=1}^n w_j \phi_j} V ; \quad \phi_i = (a_i T + b_i) \mu_T^{\frac{c_i T + d_i}{100}} ; \quad i = 1, 2, \dots, n \quad (2.3)$$

Where F_i is the optimum seismic design lateral force at i^{th} storey with the seismic weight of w_i for a structure with fundamental period of T and target ductility demand of μ_T . V is the total design base shear and a_i , b_i , c_i , d_i are constant coefficients at i^{th} storey that should be calculated for each set of design earthquakes. The proposed values of these constant coefficients for site class C (as categorized in ASCE/SEI 7-10) is given in Table 2.2 as a function of relative height (Hajirasouliha and Pilakoutas, 2012). These coefficients could be obtained at each level of the structure by interpolating the corresponding values given in Table 2.2.

Table 2.2. Modified coefficients for Eqn. 2.3 as a function of relative height (site class C)

Relative Height	a	b	c	d
0	6.14	20.15	6.89	62.35
0.1	3.17	32.81	6.40	45.75
0.2	0.24	45.50	5.91	29.19
0.3	-1.92	58.78	5.03	16.09
0.4	-2.86	71.75	2.63	7.89
0.5	-4.33	87.18	0.85	0.90
0.6	-5.71	104.33	-0.33	-5.23
0.7	-5.79	122.37	-1.76	-8.52
0.8	-2.95	141.16	-3.20	-10.23
0.9	4.79	160.50	-4.70	-10.46
1	21.96	184.07	-6.84	-8.61

2.3.4. Lateral load pattern proposed by Park and Medina (P.4)

The proposed lateral load pattern by Park and Medina for regular steel moment-resisting frames is given by the following expression (Park, 2007; Park and Medina 2007):

$$\begin{aligned}
F_i &= \left(\frac{\left(1 - \frac{F_t}{V_y}\right) w_i h_i^k}{\sum_{j=1}^n w_j h_j^k} + \delta_{in} \frac{F_t}{V_y} \right) V_y; \quad i = 1, 2, \dots, n; \quad \delta_{in} = \begin{cases} 0; & \text{if } i \neq n \\ 1; & \text{if } i = n \end{cases} \\
k &= 0.56 - 0.17\mu_T; \quad 1 \leq \mu_T \leq 5 \\
\frac{F_t}{V_y} &= 0.32 - 0.0016H - 0.13k; \quad 22m \leq H \leq 66m
\end{aligned} \tag{2.4}$$

Where F_i is the lateral force at level i , F_t is the portion of the base shear that is applied as a concentrated force at the top of the structure, w_i is the seismic weight at level i , h_i is the seismic height of level i from the base, n is the number of storeys, V_y is the base shear strength to achieve a specified target storey ductility ratio of μ_T and H is the total height of the structure from the base.

2.3.5. Lateral load pattern proposed by Building Center of Japan (P.5)

The seismic code of Japan (BCJ, 1997) has the provision for the following pattern of storey shear strength distribution:

$$V_{yi} = C_B A_i \alpha_i W_t; \quad \alpha_i = \frac{W_i}{W_t}; \quad A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \frac{2T}{1+3T}; \quad T = 0.03H \tag{2.5}$$

Where V_{yi} is the i^{th} storey shear strength, C_B is the base shear coefficient, A_i is the shear coefficient distribution which represents the vertical distribution of the seismic load, W_i is the seismic weight above the i^{th} storey, W_t is the the total weight of the structure, T is the fundamental period of the structure and H is the total height of the structure from the base.

2.3.6. Lateral load pattern proposed by Deguchi et al. (P.6)

Deguchi et al. proposed the following pattern of storey shear strength distribution (Deguchi et al., 2008):

$$V_{yi} = C_B A_i \alpha_i W_t; \quad \alpha_i = \frac{W_i}{W_t}; \quad A_i = \frac{1}{\sqrt{\alpha_i}} \tag{2.6}$$

All the parameters are the same as defined in section 2.3.5.

2.3.7. Lateral load pattern proposed by Kato et al. (P.7)

Kato et al. (1982) found the optimum shear coefficient distribution in order to develop uniform cumulative plastic deformation at each storey through a trial-and-error dynamic response analysis as follows (Kato et al., 1982):

$$\begin{aligned}
V_{yi} &= C_B A_i \alpha_i W_t; \quad \alpha_i = \frac{W_i}{W_t} \\
A_i &= 1 + 1.5927\xi_i - 11.8519\xi_i^2 + 42.5833\xi_i^3 - 59.4827\xi_i^4 + 30.1586\xi_i^5 \\
\xi_i &= 1 - \alpha_i
\end{aligned} \tag{2.7}$$

All the parameters are the same as defined in section 2.3.5.

2.3.8. Lateral load pattern proposed by Motamedi and Nateghi-A. (P.8)

Motamedi et al. proposed a triangular-rectangular lateral load pattern based on current triangular distribution in Iranian seismic code as shown in Fig. 2.2. The value of B' in the proposed pattern is equal to $\frac{2b}{3}$ (Motamedi and Nateghi-A., 2008).

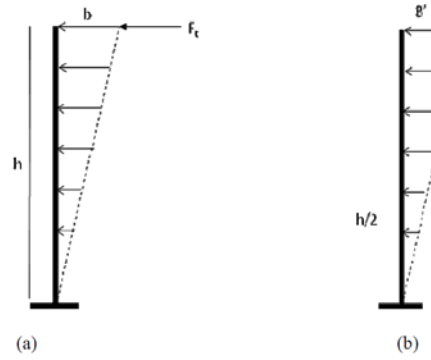


Figure 2.2. Lateral load distribution: a) Triangular common pattern used in the current Iranian Seismic Code; b) Lateral load pattern proposed by Motamedi et al. (Motamedi and Nateghi-A., 2008)

2.3.9. Lateral load pattern proposed by Moghaddam and Karami Mohammadi (P.9)

Moghaddam and Karami Mohammadi lateral load pattern can be introduced as a concentrated load at top level (F_t) accompanied by a uniform distribution of the rest of base shear (V) over the total height of the structure (Karami Mohammadi et al., 2004; Moghaddam and Karami Mohammadi, 2006). It can be expressed as follows:

$$F_i = \frac{1}{n}(V - F_t) + \delta_{in} F_t; \quad i = 1, 2, \dots, n; \quad \delta_{in} = \begin{cases} 0; & \text{if } i \neq n \\ 1; & \text{if } i = n \end{cases} \quad (2.9)$$

$$F_t = \alpha TV$$

$$\alpha = (0.9 - 0.04\mu_T)e^{-(0.6+0.03\mu_T)T}$$

All the other parameters are the same as defined in section 2.3.3.

2.2. Modelling of Moment-Resisting Steel Structures

In the present study, all nonlinear response history analyses were performed using the OpenSees platform (Pacific Earthquake Engineering Research Center, 2012). Steel material was modelled with STEEL01 which is a uniaxial bilinear steel material with hardening on the basis of Menegotto-Pinto equations. Values of initial modulus of elasticity, yield strength and strain-hardening ratio of the material are considered as 199.9 Gpa, 235.44 Mpa and 2% respectively.

Each member was modelled with a single force-based, distributed plasticity fibre beam-column element. Force-deformation relations for each section are obtained by step-by-step integration of predefined stress-strain curve of section fibres according to Gauss-Lobatto's method with seven integration points (Mazzoni et al., 2007).

IPB and IPE sections, according to DIN standard, are chosen for columns and beams, respectively. To eliminate the over-strength effect, conceptual auxiliary sections have been artificially developed by assuming a continuous variation of section properties. To achieve this goal, section dimensions (total height, flange width and web thickness) are approximated by polynomial equations of order 6 with respect to cross section as the only effective parameter. The effect of gravity loads and the second-order deformations, P- Δ effects, were considered using the complete geometric stiffness matrix. Rayleigh damping model with a constant damping ratio of 0.05 was assigned to the first mode and to the mode in which the cumulative mass participation was at least 90%.

3. DETAILS OF EVALUATION PROCEDURE

The above-mentioned models were designed to comply with the requirements of ASCE/SEI 7-10 provisions. The buildings were assumed to be located on a soil type C of ASCE/SEI 7-10 category,

with the design spectral response acceleration at short and 1-sec periods equal to 1.1g and 0.64g, respectively.

Having nearly the same structural weight (constant sum of total required shear strength) they were re-designed by the other proposed lateral load patterns (P.2 to P.9). Finally subjected to the five selected ground motion records and nonlinear response history analyses were conducted using the OpenSees platform (Pacific Earthquake Engineering Research Center, 2012). Comparisons are made between the responses of models designed according to various proposed lateral load patterns.

4. RESULTS

Fig. 4.1. Shows considered lateral load patterns (P.1 to P.9) used for designing of 5-storey building and it also compares the storey drift ratios of example building structures designed according to these lateral load patterns by nearly the same structural weight at each case under Northridge 1994 earthquake. It shows that none of the suggested patterns led to a uniform distribution of storey drifts.

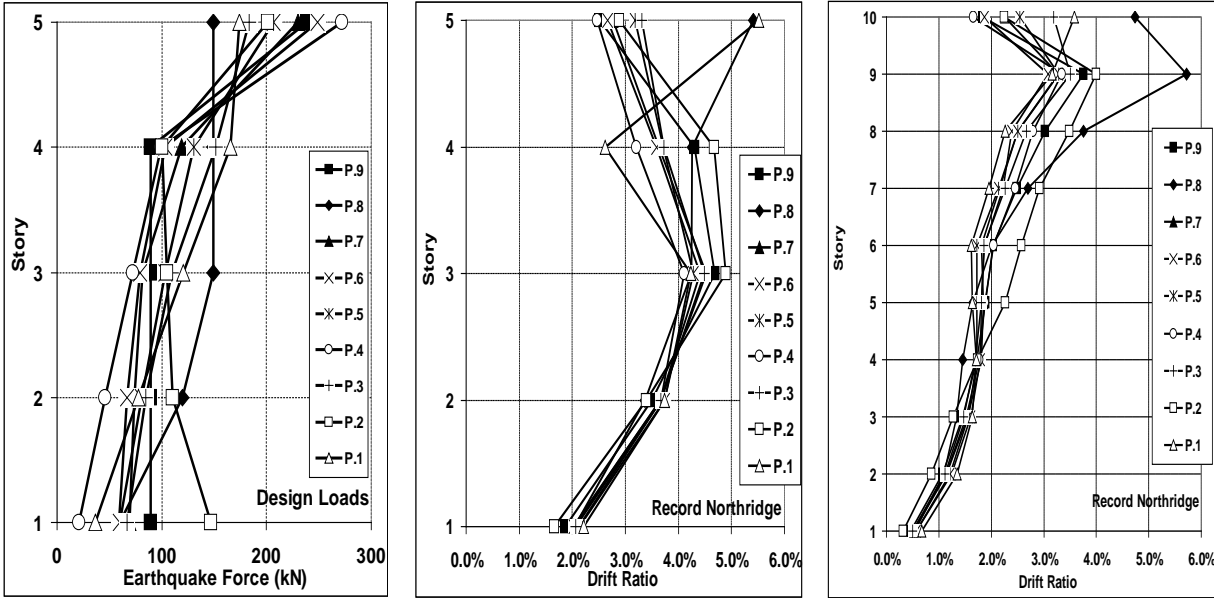


Figure 4.1. Considered lateral load patterns (P.1 to P.9) used for designing of 5-storey building (Left) and comparison of storey drift ratios of 5-storey (Middle) and 10-storey (Right) building structures designed according to these lateral load patterns by nearly the same structural weight at each case under Northridge 1994 earthquake.

Fig. 4.2. compares the maximum storey drift ratios of example building structures designed by various proposed lateral load patterns (P.1 to P.9) by nearly the same structural weight at each case under five selected earthquakes. It shows that, load patterns P.4 and P.6 represents a better performance compared to other patterns. Also, load patterns P.1 and P.8 represents a worse performance compared to other patterns.

Fig. 4.3. compares the mean value (average values for five selected earthquakes) of $\frac{(\theta_p)_{max}}{\theta_y}$ of beams and columns of example building structures designed by various proposed lateral load patterns (P.1 to P.9) by nearly the same structural weight at each case. It shows that, load patterns P.4 to P.7 represents a better performance compared to other patterns. Also, load patterns P.1 and P.8 represents a worse performance compared to other patterns.

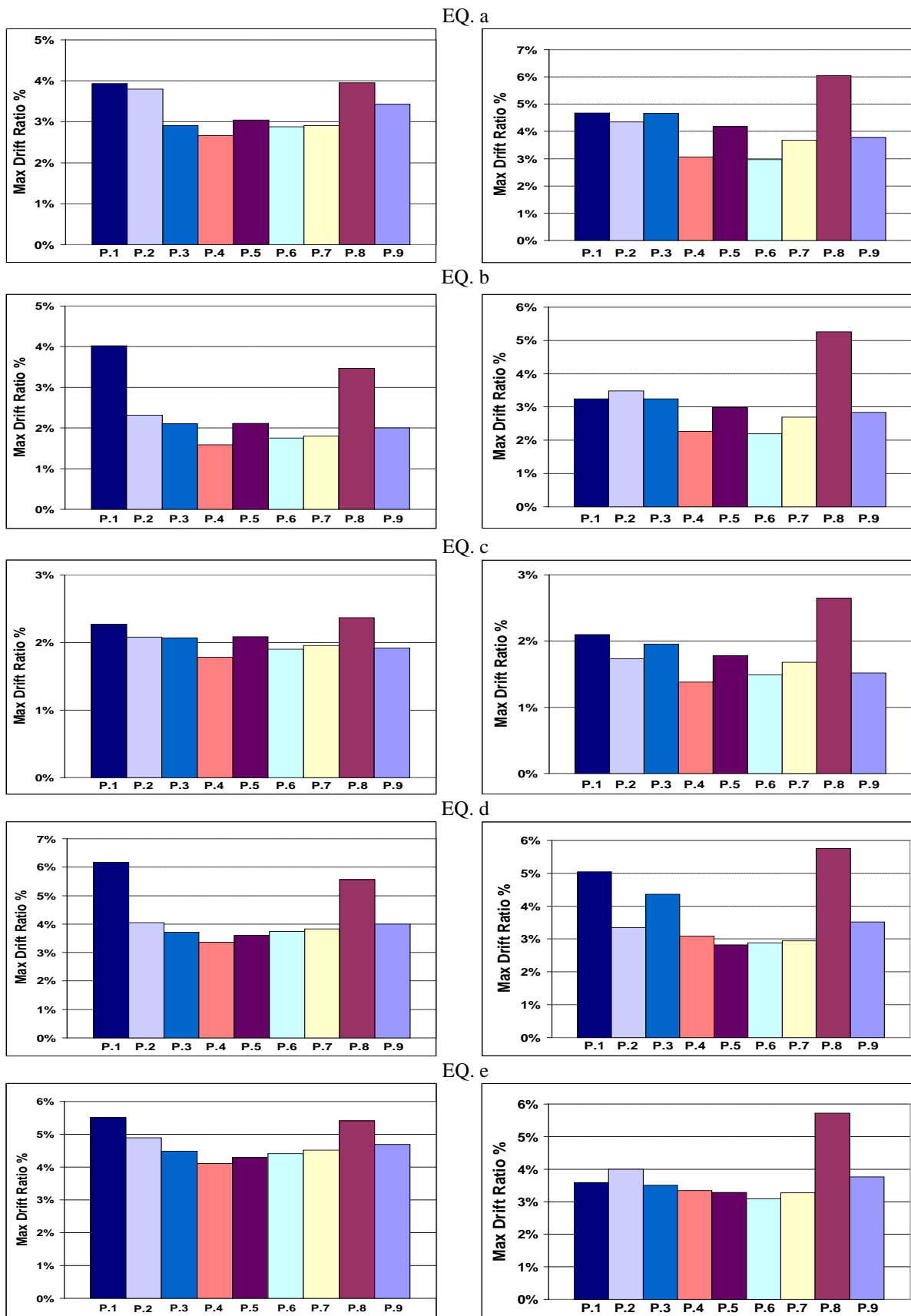


Figure 4.2. Comparison of maximum storey drift ratios of 5-storey (Left) and 10-storey (Right) building structures designed by various proposed lateral load patterns (P.1 to P.9) by nearly the same structural weight at each case under five selected earthquake.

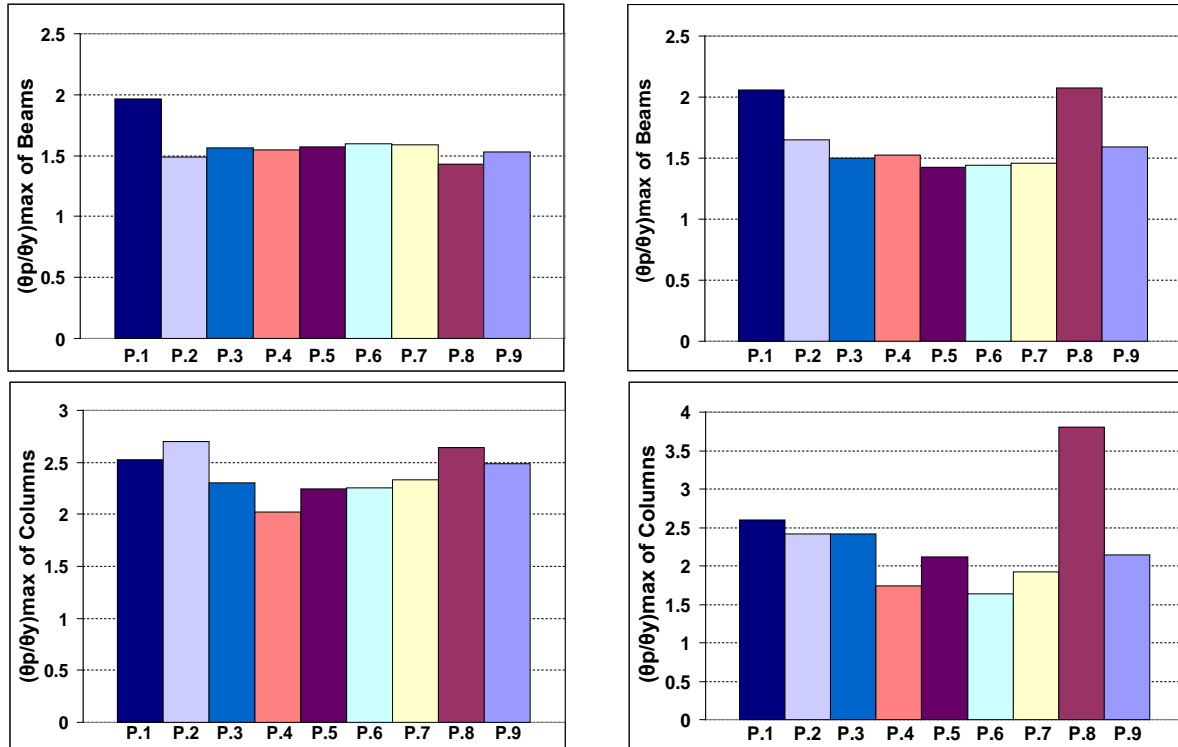


Figure 4.3. Comparison of mean value (average values for five selected earthquakes) of $\frac{(\theta_p)_{\max}}{\theta_y}$ of beams and columns of 5-storey (Left) and 10-storey (Right) building structures designed by various proposed lateral load patterns (P.1 to P.9) by nearly the same structural weight at each case.

5. CONCLUSIONS

1. In the present study, various proposed lateral load patterns are reviewed and the adequacy of them for seismic design of steel moment resisting frames are evaluated for two design examples under five different strong earthquake records.
2. Amongst the 9 considered lateral load patterns, only the proposed load patterns by Hajirasouliha and Moghaddam (P.3), Park and Medina (P.4) and Moghaddam and Karami Mohammadi (P.9) depend on structural performance parameters (i.e. target storey ductility), and therefore, are suitable for performance-based seismic design of structures. However, the results indicate that these load patterns cannot be used directly in the practical design of structures due to the assumptions associated with them.
3. In general, none of the suggested patterns led to a uniform distribution of lateral storey drifts and flexural damage in beam and column elements. It is shown that Park and Medina (P.4) and Deguchi et al. (P.6) load patterns usually lead to a relatively lower deformation demands compared to other considered load patterns. In contrast, ASCE/SEI 7-10 (P.1) and Motamedi et al. (P.8) load patterns generally resulted in a relatively higher deformation demands.
4. The seismic behaviour of the steel moment resisting frames designed with Hajirasouliha and Moghaddam load pattern (P.3) was usually better than those designed with ASCE/SEI 7-10 (P.1). However, this load pattern is mainly based on the shear-building structures, and therefore, does not lead to a uniform damage distribution in the steel moment resisting frames.

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