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More Efficient Lateral Load Patterns for Seismic Design of Steel Moment Resisting Frames

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ABSTRACT:

The preliminary design of building structures is normally based on the equivalent lateral forces provided in seismic design guidelines. The height-wise distribution of these loads is predominantly based on elastic vibration modes. However, as structures exceed their elastic limits in severe earthquakes, these design load patterns may not necessarily lead to efficient distribution of strength within the structures. To address this issue, several alternative load patterns have been proposed for seismic design of nonlinear structures. However, due to the simplifications involved in the development of these design load patterns, their adequacy should be assessed for different structural systems and earthquake excitations before they can be used in common practice. This paper aims to identify the most suitable lateral load patterns for seismic design of steel moment resisting frames. The nonlinear seismic behaviour of 3, 5, 7, 10, and 15-storey frames designed with nine different lateral load patterns are compared under 20 real and synthetic spectrum-compatible earthquakes using performance parameters such as maximum inter-storey drift, maximum plastic hinge rotation and cumulative damage. It is shown that, for the same structural weight, structures designed with more efficient load patterns experience up to 68% less global damage compared to their code-based counterparts.

Keywords: Seismic Engineering, Steel Structures, Buildings Structure & Design
1. INTRODUCTION

The preliminary design of most buildings is normally based on equivalent static forces obtained from seismic design guidelines and codes of practice. The height-wise distribution of these static forces is implicitly based on the dynamic response of elastic structures (Chopra 2012). As structures exceed their elastic limits in severe earthquakes, the use of inertia forces corresponding to the elastic modes may not lead to the best design solutions.

The seismic performance of code-based designed lateral load resisting systems has been widely studied (e.g. Kato and Akiyama 1982; Moghaddam 1996; Mohammadi et al. 2004; Moghaddam and Mohammadi 2006). 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 within the structure. Therefore, the employment of such lateral load patterns does not guarantee the optimal distribution of structural materials throughout the structures in the nonlinear range of behaviour. It was also concluded that a uniform distribution of ductility demands can be obtained by using other lateral design load patterns.

Several researchers have attempted to develop more efficient lateral design load patterns for seismic design of multi-storey buildings. By conducting a trial-and-error dynamic response analyses, Kato and Akiyama (1982) determined the height-wise distribution of maximum storey shear forces of elastoplastic frames to develop uniform plastic deformations under the El Centro 1940 ground motion. They concluded that the shear distribution which leads to a uniform plastic deformation almost coincides with the distribution of maximum storey shear forces under the design earthquake.

By comparison between different lateral load distribution patterns, Moghaddam and Karami-Mohammadi (2006) introduced a new design load pattern for shear-building structures to obtain a more uniform distribution of inter-storey ductility demands. Using a similar approach, Park and Medina (2007) and Motamedi and Nateghi (2008) proposed new lateral design load patterns for seismic design of steel moment-resisting frames based on the concept of uniform damage and uniform earthquake energy distribution, respectively. Lee and Goel (2001) and Chao and Goel (2007) analysed a series of steel moment and braced frames subjected to earthquake excitations. They showed that, in
general, there is a discrepancy between the earthquake induced shear forces and those determined by assuming code-based design load distribution patterns. Based on their results, they suggested a shear distribution factor which was then validated for a wide range of structural systems such as Moment Resisting Frames, Eccentrically-Braced Frames and Special Truss Moment Frames (Goel et al. 2010). However, the effects of ground motion characteristics and the degree of nonlinearity are not directly considered in their suggested design force distribution. In another relevant study, Degochi et al. (2008) proposed a load pattern for seismic design of steel frames using the shear forces developed in an elastic bar with uniform stiffness and mass distributions under a velocity design response spectrum.

Moghaddam and Hajirasouliha (2006) and Moghaddam and Hajirasouliha (2008) developed an effective optimization method based on the concept of uniform damage distribution to find optimum lateral load patterns for seismic design of non-linear shear-building structures. They showed that, for the same target storey-ductility demand, structures that are designed with the optimum load pattern require considerably lower structural weight compared to those designed using conventional methods. In a follow-up study, Hajirasouliha and Moghaddam (2009) proposed a new design load pattern for seismic design of shear buildings that is a function of fundamental period of the structure and the target storey-ductility demand. This load pattern was further developed by Hajirasouliha and Pilakoutas (2012) to include the effects of height-wise irregularity and site soil classifications.

While the design lateral load pattern can play an important role in the seismic performance of structures, the load patterns proposed in the previous studies cannot be used directly in practical design of building structures, as they were mainly based on simple models and/or the utilized seismic records were not compatible with modern building code design spectra (such as EC8 and ASCE/SEI 7-10). To address this issue, this study investigates the efficiency of different design load patterns proposed in the previous studies at improving the seismic performance of 3, 5, 7, 10, and 15-storey steel moment resisting frames (SMRFs) under a wide range of real and synthetic design spectrum-compatible earthquakes. Using different seismic performance criteria, the most suitable design load patterns will be identified for practical design purposes.
2. SEISMIC DESIGN LATERAL LOAD PATTERNS

2.1. Code-compliant lateral load pattern (P-1)

Both Eurocode 8 (EC8, EN 1998-1:2004) and ASCE/SEI 7-10 (2010) use the following equation to calculate the lateral seismic force \( F_i \) at each storey level:

\[
F_i = \frac{w_i h_i^k}{\sum_{j=1}^{n} w_j h_j^k} V; \quad i = 1, 2, ..., n; \quad k = \begin{cases} 
1 & ; T \leq 0.5 \\
2 & ; 0.5 < T < 2.5 \\
\frac{1}{2} & ; T \geq 2.5 
\end{cases} \tag{1}
\]

where \( w_i \) is the weight of the structure at the \( i \)th level, \( h_i \) is the height from the base to level \( i \), \( n \) is the total number of storeys, \( V \) is the design lateral force at the base of the structure (base shear), \( k \) is an exponent related to the structure period, and \( T \) is the fundamental period of the structure.

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

Goel et al. (2010) proposed the following load pattern for seismic design of structures:

\[
F_i = C_{iv} V
\]

\[
C_{iv} = (\beta_i - \beta_{i+1}) \left( \frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j} \right) \alpha T^{-0.2}; \quad \beta_{n+1} = 0 \tag{2}
\]

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

where \( \beta_i \) and \( V_i \) are the shear distribution factor and the storey shear force at level \( i \), respectively. The parameter \( \alpha \) was originally proposed as 0.5 by Lee and Goel (2001), which was later modified to 0.75 by considering a wider range of steel framing systems (Goel et al., 2010).

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

The load pattern suggested by Hajirasouliha and Moghaddam (2009) and Hajirasouliha and Pilakoutas (2012) can be expressed as follows:

\[
F_i = \frac{w_i \phi_i}{\sum_{j=1}^{n} w_j \phi_j} V; \quad \phi_i = (a_i T + b_i) \mu_T \frac{c_i T + d_i}{100}; \quad i = 1, 2, ..., n \tag{3}
\]

where \( \mu_T \) is the target ductility demand, and \( a_i, b_i, c_i, d_i \) are constant coefficients at level \( i \) that can be calculated for each set of design earthquakes. The constant coefficients corresponding to different soil profiles are given in Hajirasouliha and Pilakoutas (2012) as a function of relative height.

2.4. Lateral load pattern proposed by Park and Medina (P-4)
The lateral load pattern proposed by Park and Medina (2007) for regular steel moment-resisting frames is given by the following formula:

\[
F_i = \left(\frac{1-\xi}{V_y}w_i h_i^k + \delta_{in} \frac{F_s}{V_y}\right)V_y; \quad i = 1, 2, \ldots, n; \quad \delta_{in} = \begin{cases} 0; & \text{if } i \neq n \\ 1; & \text{if } i = n \end{cases}
\]

\[
k = 0.56 - 0.17\mu_T; \quad 1 \leq \mu_T \leq 5
\]

\[
\frac{F_s}{V_y} = 0.32 - 0.0016H - 0.13k; \quad 22m \leq H \leq 66m
\]

where \(F_t\) is a concentrated force at the top of the structure, \(w_i\) is the weight at level \(i\), and \(V_y\) is the base shear strength to achieve a specified target storey ductility ratio of \(\mu_T\). \(H\) is the total height of the structure from the base.

### 2.5. Lateral load pattern proposed by Building Center of Japan (P-5)

The seismic code of Japan (BCJ, 1997) suggests the following shear strength distribution pattern:

\[
V_{yi} = C_B A_i \alpha_i W_i; \quad \alpha_i = \frac{W_i}{W_s}; \quad A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T}; \quad T = 0.03H
\]

where \(V_{yi}\) is the shear strength of the \(i^{th}\) storey, \(C_B\) is the base shear coefficient, \(A_i\) is the shear coefficient distribution which represents the vertical distribution of the seismic load, and \(W_i\) is the total weight of the structure.

### 2.6. Lateral load pattern proposed by Deguchi et al. (P-6)

Deguchi et al. (2008) proposed the following storey shear strength distribution for seismic design of steel frames:

\[
V_{yi} = C_B A_i \alpha_i W_i; \quad \alpha_i = \frac{W_i}{W_s}; \quad A_i = \frac{1}{\sqrt{\alpha_i}}
\]

All parameters are defined in previous equations.

### 2.7. Lateral load pattern proposed by Kato et al. (P-7)

The following load pattern is suggested by Kato et al. (1982) for elastoplastic frames:

\[
V_{yi} = C_B A_i \alpha_i W_i; \quad \alpha_i = \frac{W_i}{W_s}
\]

\[
A_i = 1 + 1.5927\xi_i - 11.8519\xi_i^2 + 42.5833\xi_i^3 - 59.4827\xi_i^4 + 30.1586\xi_i^5
\]

\[
\xi_i = 1 - \alpha_i
\]
2.8. Lateral load pattern proposed by Motamedi and Nateghi (P-8)

Motamedi and Nateghi (2008) proposed the following a triangular-rectangular load pattern for seismic design of steel moment-resisting frames:

\[
F_i = \begin{cases} 
\frac{B'}{2b} & \text{if } i \leq n/2 \\
B' & \text{if } i > n/2 
\end{cases} ; \quad B' = \frac{w_s}{3H} ; \quad i = 1, 2, ..., n \tag{8}
\]

In the above equation, \(B'\) is equal to \(\frac{2b}{3}\), where \(b\) is the altitude of the triangular load in Iranian seismic code (BHRC, 2005).

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

Moghaddam and Karami Mohammadi (2006) load pattern is defined as a concentrated load at the top level \((F_t)\) accompanied by uniform distribution of the rest of base shear \((V)\) along the height of the structure:

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

\[
F_t = \vartheta TV
\]

\[
\vartheta = (0.9 - 0.04\mu_f)e^{-(0.6 + 0.03\mu_f)T}
\]

3. MODELLING AND ASSUMPTIONS

3.1. Case Study Frames

To investigate the efficiency of different seismic design load patterns, 3, 5, 7, 10, and 15-storey ordinary moment resisting frames (with the typical geometry shown in Figure 1) were examined. The bay-width and the storey-height of the frames were 6 m and 3 m, respectively, and all supports were considered to be fixed. The uniformly distributed dead and live loads were assumed as 6 kN/m\(^2\) and 2 kN/m\(^2\) for interior stories, and 6 kN/m\(^2\) and 1.5 kN/m\(^2\) for the roof level, respectively. The frames were assumed to be located on a soil type D category of ASCE/SEI 7-10, with the design spectral response acceleration at short periods and 1-sec period equal to 0.40g and 0.64g, respectively (see Figure 2).

The structural elements were designed to support gravity loads and lateral loads in accordance with the minimum requirements of ANSI/AISC 360-10. The P-Delta affects were taken into account in the
design process of the MRFs. The seismic force-resisting system was considered to be steel intermediate moment frame with the response modification coefficient (R) and over strength factor \( \Omega_0 \) equal to 4.5 and 3, respectively. The yielding stress and Young's modulus of the steel material were taken to be 235.4 MPa and 200 GPa, respectively. Strain hardening of steel was taken into account by considering the tangent modulus \( E_T \) equal to \( E/50 \), where \( E \) is the modulus of elasticity of steel material (Mazzolani and Gioncu 1995).

IPB and IPE sections, according to DIN-1025 (1995) standard, were chosen for columns and beams, respectively. To obtain the best design solutions, conceptual auxiliary sections were artificially developed by assuming a continuous variation of section properties. The optimum size of each structural element was calculated based on the required capacity under the design loads. For example, Table 1 shows the beam and column cross sections of the 5-storey frames designed based on the nine different load patterns. The numbers in this table show the area of the sections in cm\(^2\).

In this study, the drift limitations suggested by the design guidelines (e.g. EC8 and ASCE/SEI 7-10) were not controlled during the design process in order to provide a fair means of comparison between different load patterns using the same amount of structural weight. While the effects of uncertainties on the seismic performance of the structures is not in the scope of this paper, previous studies by Hajirasouliha et al. (2016) showed that typical uncertainties in conventional steel frames do not significantly influence the efficiency of the optimum design frames.

Nonlinear time history analyses were conducted using OpenSees (2015). The Rayleigh damping model with a constant damping ratio of 0.05 was assigned to the first mode and to the modes at which the cumulative mass participation exceeds 95%. A distributed plasticity fibre-based model in OpenSees (2015) was employed to model the nonlinear behaviour of the beam and column elements. The model can take into account the change in the plastic hinge length and variation of the stiffness under single or double curvature conditions. The P-Delta second order effects were also included in the non-linear dynamic analyses. The contribution of the panel zone deformation was considered to be insignificant in the MRFs with rigid connections (Mazzolani and Gioncu 2000; Medina and Krawinkler 2005).
Table 1. Beam and column cross sections of the 5-storey frames designed based on the nine different load patterns (numbers are the area of the sections in cm²).

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<td>IPE 85</td>
<td>IPE 86</td>
<td>IPE 86</td>
<td>IPE 86</td>
<td>IPE 86</td>
<td>IPE 85</td>
</tr>
</tbody>
</table>

3.2. Selected Ground Motions

Five medium-to-strong natural ground motion records were selected from PEER ground motion database (PEER, 2010) as listed in Table 2. All of these ground motions correspond to soil class D of ASCE/SEI 7-10 and are recorded in low to moderate distances from the epicentre (less than 45 km) with high local magnitudes (i.e. M>6.7). On average, the selected ground motions provide a reasonably close approximation to the design response spectra of ASCE/SEI 7-10 for the site class D.
in high seismic zones (i.e. PGA=0.4g), especially at the first mode periods of the designed frames. Therefore, in this study these earthquake records were used directly without being normalized (Nabid et al. 2017). The acceleration spectrum of these ground motions are compared with ASCE/SEI 7-10 design spectrum in Figure 2.

Fifteen synthetic records were also generated using SIMQKE program (Vanmarke, 1976) to be compatible with the soil type C of ASCE/SEI 7-10 design spectrum. Figure 2 demonstrates a good compatibility between the average of these synthetic earthquakes and the code design spectrum. Therefore, these synthetic earthquakes can be considered to be representatives of the response spectrum used in the design process.

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake</th>
<th>Record/Component</th>
<th>Station</th>
<th>Magnitude (Ms)</th>
<th>PGA (g)</th>
<th>PGV (cm/s)</th>
<th>PGD (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Duzce, Turkey 1999</td>
<td>DUZCE/DZC270</td>
<td>Duzce</td>
<td>7.3</td>
<td>0.535</td>
<td>83.5</td>
<td>51.59</td>
</tr>
<tr>
<td>2</td>
<td>Imperial Valley 1979</td>
<td>IMPVALL/HE04140</td>
<td>955 El Centro Array #4</td>
<td>6.9</td>
<td>0.485</td>
<td>37.4</td>
<td>20.23</td>
</tr>
<tr>
<td>3</td>
<td>Loma Prieta 1989</td>
<td>LOMAP/G03000</td>
<td>47381 Gilroy Array #3</td>
<td>7.1</td>
<td>0.555</td>
<td>35.7</td>
<td>8.21</td>
</tr>
<tr>
<td>4</td>
<td>Cape Mendocino 1992</td>
<td>CAPEMEND/PET090</td>
<td>89156 Petrolia</td>
<td>7.1</td>
<td>0.662</td>
<td>89.7</td>
<td>29.55</td>
</tr>
<tr>
<td>5</td>
<td>Northridge 1994</td>
<td>NORTHR/NWH360</td>
<td>24279 Newhall - Fire Sta</td>
<td>6.7</td>
<td>0.59</td>
<td>97.2</td>
<td>38.05</td>
</tr>
</tbody>
</table>

### Table 2. Characteristics of the selected ground motions

#### 3.3. Design Load Patterns

The 3, 5, 7, 10, and 15-storey moment resisting frames were designed using the nine different load patterns discussed in section 2 (45 frames in total). For the frames designed with the load patterns P-2 to P-9, the sum of inter-storey shear forces was scaled so that the total structural weight becomes equal to that of the reference models designed with ASCE load pattern (P-1). By using this adjustment, the fundamental period of the frames designed with different load patterns were very close to the fundamental period of the corresponding ASCE frame.

While almost all of the proposed load patterns depend on the fundamental period of the structure $T$, load patterns P-3, P-4 and P-9 are also a function of the target ductility demand $\mu_T$. Calculating these load patterns is not straightforward as $T$ and $\mu_T$ are also affected by the seismic design loads.
Therefore, in this study an iterative method was used to update the values of $T$ and $\mu_T$ for more accurate calculation of the seismic design loads. In the first step, the target ductility demand was considered to be 2 and the fundamental period of the structure was calculated based on the ASCE suggested equation. Subsequently, the structure was designed based on the calculated loads and the accurate fundamental period was obtained from the FE model. If the target ductility is also required to calculate the design load pattern, the structure was subjected to the 15 synthetic spectrum-compatible earthquakes and the average storey ductility (maximum displacement of each storey divided by the yield displacement of the storey) was calculated. The yield displacements were obtained from a non-linear pushover analysis (Hajirasouliha and Doostan, 2009). The iteration process was repeated until the difference between $T$ and $\mu_T$ values in two subsequent steps became negligible. For example, Figure 3 shows the lateral load patterns obtained for the seismic design of the 10-storey frame.

It is shown in Figure 4(a) that the models designed with different load patterns have similar total structural weight, while their design base shear can be very different. In general, for high-rise buildings (10 and 15 storey) the design load patterns P-3 and P-9 led to maximum base shear, while P-1 and P-2 resulted in lower design base shear values compared to the other load patterns (see Figure 4(b)). For low- to medium-rise buildings, the design load pattern P-4 led to the highest base shear values.

Figs-3-4

4. ADEQUACY OF THE DESIGN LOAD PATTERNS

In this study, the following performance parameters were used to identify the most suitable design load patterns for SMRFs with different number of storeys.

4.1. Inter-Storey Drift
Maximum inter-storey drift has been widely used to evaluate the level of damage to both structural
and non-structural elements in building structures. ASCE 41-13 (2013) limits the maximum inter-
storey drift ratio to 0.7%, 2.5% and 5% for immediate occupancy (IO), life safety (LS) and collapse
prevention (CP) performance levels, respectively. Figure 5 compares the average of the inter-storey
drift ratios in the 15 synthetic earthquakes spectrum-compatible earthquakes for the 3, 5, 7, 10 and 15-
storey frames designed with different load patterns (675 different cases in total). The results indicate
that the design lateral load pattern can play an important role in controlling the maximum inter-storey
drift and also some design load patterns can be considerably more efficient than the conventional
code-compliant lateral loads.

It is shown in Figure 5 that, for the same structural weight, the load patterns P-2, P-6 and P-7 in
general led to the design solutions with lower maximum inter-storey drift ratios compared to the other
load patterns. While the load pattern P-4 could not control the maximum inter-storey drift ratios at the
top floors of low to medium rise buildings (3, 5 and 7-storey frames), it was considerably more
efficient for high-rise buildings (10 and 15-storey frames). The load pattern P-8 resulted in structures
with high maximum inter-storey drift ratios in the top floors, especially in high-rise buildings.

It is shown in Figure 5 that the top floors of SMRFs usually exhibit higher inter-storey drift ratios
when compared to the bottom floors. Also it can be noted that in general the maximum inter-storey
drift distribution of low-rise buildings follow the first mode shape of the structure, while for the high-rise buildings the effect of higher modes is evident. The effect of higher modes is especially dominant
when the design load patterns P-4 and P-8 are utilized.

For better comparison, Table 3 presents the maximum inter-storey drift ratio and the performance
limit of the frames designed with different load patterns (average of 15 synthetic earthquakes). It is
shown that the SMRFs designed with the code suggested load pattern (P-1) always satisfied the
collapse prevention (CP) performance level under the design earthquakes, while using P-4 and P-8
design load patterns led to the structural collapse in low to medium-rise and high-rise buildings, respectively. For the same structural weigh, structures designed based on P-2, P-6 and P-7 remained in the life safety (LS) performance level, which confirms the efficiency of these load patterns. The results in Table 3 also indicate that the design load pattern P-2 suggested by Goel et al. (2010) was the most efficient pattern to control maximum inter-storey drifts in almost all cases.

Table 3. Maximum inter-storey drift ratio and performance limit of the SMRFs designed with different load patterns (average of 15 synthetic earthquakes)

<table>
<thead>
<tr>
<th>Design Load Pattern</th>
<th>3-Storey</th>
<th>5-Storey</th>
<th>7-Storey</th>
<th>10-Storey</th>
<th>15-Storey</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drift Ratio (%)</td>
<td>Performance Level</td>
<td>Drift Ratio (%)</td>
<td>Performance Level</td>
<td>Drift Ratio (%)</td>
<td>Performance Level</td>
</tr>
<tr>
<td>P-1</td>
<td>2.73 CP</td>
<td>2.78 CP</td>
<td>3.30 CP</td>
<td>3.58 CP</td>
<td>3.95 CP</td>
</tr>
<tr>
<td>P-2</td>
<td>1.93 LS</td>
<td>1.73 LS</td>
<td>1.90 LS</td>
<td>2.15 LS</td>
<td>2.28 LS</td>
</tr>
<tr>
<td>P-3</td>
<td>2.55 CP</td>
<td>2.50 LS</td>
<td>3.20 CP</td>
<td>3.53 CP</td>
<td>4.20 CP</td>
</tr>
<tr>
<td>P-4</td>
<td>5.15 Collapse</td>
<td>5.10 Collapse</td>
<td>5.20 Collapse</td>
<td>2.38 LS</td>
<td>2.28 LS</td>
</tr>
<tr>
<td>P-5</td>
<td>2.85 CP</td>
<td>2.35 LS</td>
<td>2.50 LS</td>
<td>2.68 CP</td>
<td>2.88 CP</td>
</tr>
<tr>
<td>P-6</td>
<td>2.00 LS</td>
<td>1.93 LS</td>
<td>2.00 LS</td>
<td>2.18 LS</td>
<td>2.25 LS</td>
</tr>
<tr>
<td>P-7</td>
<td>2.15 LS</td>
<td>1.88 LS</td>
<td>2.20 LS</td>
<td>2.45 LS</td>
<td>2.50 LS</td>
</tr>
<tr>
<td>P-8</td>
<td>3.60 CP</td>
<td>4.13 CP</td>
<td>4.30 CP</td>
<td>4.95 CP</td>
<td>5.50 Collapse</td>
</tr>
<tr>
<td>P-9</td>
<td>2.15 LS</td>
<td>2.15 LS</td>
<td>2.23 LS</td>
<td>2.53 CP</td>
<td>3.00 CP</td>
</tr>
</tbody>
</table>

4.2. Plastic Hinge Rotation

To evaluate the seismic performance of the frames, the maximum plastic rotation of each beam and column element ($\theta_{pl}$) was compared with the allowable plastic rotation ($\theta_{all}$) under the design earthquake. In this study, $\theta_{all}$ was assumed to be the maximum allowable rotation of each element in the life safety (LS) performance level specified in ASCE41-13 (2013). For steel beams, $\theta_{all}$ is a function of the yield rotation ($\theta_y$) and the section dimensions, while for steel columns the maximum axial force of the column is also considered. According to ASCE41-13 (2013), the yield rotation of beam and column elements can be calculated by using Equations 9 and 10, respectively.

$$\theta_{pl} = \frac{ZF_{pl}I_y}{6EI_y}$$

(9)
\[
\theta_{ye} = \frac{ZE_{ye}l}{6EI_c}(1 - \frac{P}{P_{ye}})
\]

where \(E\) and \(F_{ye}\) are the elastic modulus of the material and the expected yield stress. \(I_b, l_b, I_c, l_c\) represent the moment of inertia and the effective length of the beam and column elements, respectively. \(Z\) is the plastic modulus of each cross-section, and \(P\) and \(P_{ye}\) denote the axial force of the column and the expected axial yield force of the column, respectively.

The \(\theta_p/\theta_{all}\) ratio was calculated for all structural elements of the 3, 5, 7, 10 and 15-storey frames designed with different load patterns. Figure 6 compares the maximum and the average of the results in the 15 synthetic spectrum-compatible earthquakes. Overall, the maximum \(\theta_p/\theta_{all}\) ratios (in Figure 6 (a)) are in good agreement with the performance levels obtained based on the inter-storey drift ratios (in Table 3). It is shown that load patterns P-2, P-6 and P-7 in general provided the best design solutions with lower Max \(\theta_p/\theta_{all}\) ratios. In most cases, these three load patterns led to lower Mean \(\theta_p/\theta_{all}\) ratios as well, which implies they could efficiently reduce the overall displacement demands of the designed structures. It is clear from Figure 6 (a) that using the load pattern P-8 resulted in very high plastic rotations in the structures with more than 5 storeys. As shown in Figure 5, it is mainly due to the soft storey failure of the top floors. However, this design load patterns seems to be acceptable for 3 and 5-storey buildings.

Fig-6

To provide more details on the height-wise inter-storey drift distributions ratios, Figure 7 (a) compares the “average”, “average + standard deviation” and “average – standard deviation” of the results of the 5-storey frame under the 15 synthetic spectrum-compatible earthquakes. The results indicate that the average values could efficiently show the general trend of the results in the individual earthquakes. For better comparisons, the probability density of the maximum inter-storey drifts at different storey levels are also calculated as depicted in Figure 7 (b). It is shown that, in general, the
utilised records led to maximum inter-storey drift ratios with a relatively wide normal distribution at all storey levels.

Fig-7

4.3. Cumulative damage

To investigate the extent of damage within the designed structures, the damage index proposed by Baik et al. (1988) based on the classical low-cycle fatigue approach is used in this study (Equation 11).

The inter-storey inelastic deformation is chosen as the basic damage quantity, and the cumulative damage index after $N$ cycles of plastic deformation is calculated as:

$$D_i = \sum_{j=1}^{N} \left( \frac{\Delta\delta_{pj}}{\delta_{yi}} \right)^c$$

where $D_i$ is the cumulative damage index at $i^{th}$ storey, ranging from 0 for undamaged to 1 for severely damaged storeys. $N$ is the number of plastic excursions. $\Delta\delta_{pj}$ and $\delta_{yi}$ denote the plastic deformation of $i^{th}$ storey in $j^{th}$ cycle and the nominal yield deformation of $i^{th}$ storey, respectively. $c$ is a parameter to account for the plastic deformation magnitude which is taken to be 1.5 as suggested by Krawinkler and Zohrei (1984).

To evaluate the total damage of the structure, the global damage index $D_t$ was calculated as a weighted average of the damage indices at the storey levels by using the dissipated energy as the weighting function:

$$D_t = \frac{\sum_{i=1}^{n} D_i W_{pi}}{\sum_{i=1}^{n} W_{pi}}$$

In the above equation, $W_{pi}$ and $D_i$ are the dissipated energy and the damage index at $i^{th}$ storey, respectively, and $n$ is the total number of storeys. The global damage index $D_t$ was calculated for the designed 3, 5, 7, 10 and 15-storey frames under the 15 synthetic spectrum-compatible earthquakes. For example, Figure 8 shows the global damage index $D_t$ of the 5-storey frames designed with different
load patterns under the synthetic spectrum-compatible earthquakes number 4, 8 and 12. While there are some discrepancy between the global damage indices in different synthetic earthquakes, the results show a very similar trend. Therefore, the average of the results can provide reliable information to assess the efficiency of each design load pattern.

Figure 9 compares the average of the results in 15 synthetic earthquakes for the 3, 5, 7, 10 and 15-storey frames designed with the nine different load patterns. It should be mentioned that the damage indices above 1 represent extensive damage and failure of the structure.

In general, the global damage results are in very good agreement with the maximum inter-storey drift ratios and performance limits discussed in previous sections (e.g. Table 3). It is shown in Figure 9 that the load patterns P-2, P-6 and P-7 led to design solutions with lower global indices compared to the other load patents. The load patterns P-5 and P-9 could also control the global damage of the structures with different number of storeys within an acceptable range. However, the load patterns P-3 and P-4 were suitable only for low to mid-rise structures (less than 7 storeys) and high-rise structures (more than 10 storeys), respectively. Using the load pattern P-8 resulted in high global damage indices especially in high-rise buildings, which confirms the previous conclusion that this load pattern is not suitable for seismic design of SMRFs. Also it can be noted from Figure 9 that the efficiency of the EC8 and ASCE/SEI 7-10 design load distribution pattern (P-1) decreases by increasing the number storeys. This can be due to the fact that the code-based design patterns cannot accurately take into account the higher mode effects in non-linear multi-storey structures under strong earthquakes as was also reported by Hajirasouliha and Moghaddam (2009) and Hajirasouliha and Pilakoutas (2012).

By calculating the average of the results for the 3, 5, 7, 10 and 15-storey frames (red dotted line in Figure 9), it can be concluded that, for the same structural weight, structures designed with the load patterns P-2, P-6 and P-7 on average exhibit 68%, 66% and 54% less structural damage compared to
their code-based counterparts. This implies that the seismic performance of the non-linear SMRFs can be significantly improved by using more appropriate design load patterns.

5. EFFECT OF EARTHQUAKE EXCITATION

To investigate the efficiency of different design load patterns under real earthquake excitations, the designed frames were also subjected to the five selected strong earthquake records listed in Table 2. Overall, the results confirmed the general conclusions made based on the synthetic spectrum-compatible earthquakes in previous sections. For example, Figures 10 and 11 compare the maximum inter-storey drift distribution and the global damage index of 5-storey frames designed with different load patterns under the 15 synthetic and the 5 selected earthquake records. It is shown that, on average, the global damage indices and the maximum inter-storey drift ratios corresponding to different design load patterns exhibit a very similar trend in real and synthetic earthquake records. For example, in both cases the load patterns P-2, P-6 and P-7 provided the best design solutions with minimum global damage and inter-storey drift ratios, while using the load patterns P-4 and P-8 led to a very poor seismic performance. Therefore, it can be concluded that the general results based on the average of synthetic spectrum-compatible earthquakes are also valid for real earthquake records.

6. DESIGN VERSUS INDUCED SEISMIC FORCES

In general, lateral seismic design load patterns aim to represent the height-wise distribution of maximum storey shear forces during design earthquakes. Hence, the agreement between the design and the induced storey shear forces can be used as a measure to assess the adequacy of each seismic design load pattern. In this section, the relative distribution of design storey shear forces \( V_i/V_n \) in 3,
5, 7, 10, and 15-storey frames designed with different load patterns are compared with the average relative distribution of maximum storey shear forces \((V_i/V_n)_E\) occurred during the 15 synthetic spectrum-compatible earthquakes. Figure 12 shows the results for the load patterns P-1 to P-9. For a better comparison, the mean absolute error (MAE) between \((V_i/V_n)_D\) and \((V_i/V_n)_E\) ratios are also calculated for each design load pattern by using the following equation:

\[
MEA = \frac{1}{n} \sum_{i=1}^{n} \left| \frac{V_i}{V_n}_D - \frac{V_i}{V_n}_E \right|
\]

The \(MEA\) factor would tend to zero if the height-wise distribution of the storey shear forces used during the design process is similar to the average distribution of maximum storey shear forces under the design spectrum-compatible earthquakes. Figure 13 compares the \(MEA\) factor for the frames designed with different load patterns. In general, the \(MEA\) results follow a similar trend as the global damage indices, and the design load patterns with lower damage indices (i.e. P-2, P-6 and P-7) show a better agreement with the maximum storey shear forces observed during the design earthquakes.

Figs-12-13

It should be mentioned that the selected seismic design load pattern can have a significant effect on the sizing of the structural members, which in turn affects the dynamic characteristics of the structural system both in the linear and nonlinear regions. Therefore, the maximum storey shear forces that are exhibited by the structure during design earthquakes may not represent the best design lateral load pattern as it was shown in previous studies (e.g. Moghaddam 1996, Hajirasouliha and Moghaddam 2009). However, it is shown in this paper that this criterion can be used to assess the overall adequacy of the selected design load pattern, as the results are in agreement with other performance parameters such as maximum inter-storey drift, plastic hinge rotation and global damage.

In this study, the average response of each group of earthquakes was used to evaluate the seismic performance of the moment resisting frames. While 84th, 50th and 16th percentiles of the results (as suggested by Longo et al. 2009; Tenchini et al. 2014; and Silva et al. 2016) can be also used to provide
more in-depth information, this will not affect the general conclusions of this article. In general, the outcomes of this study should provide very helpful information for structural designers, especially those involved in the seismic design of buildings, to obtain more efficient and resilient multi-storey steel moment resisting frames suitable for seismic regions.

6. CONCLUSIONS

An extensive analytical study was conducted to investigate the effects of lateral design load pattern on the seismic performance of SMRFs. The nonlinear seismic behaviour of 3, 5, 7, 10, and 15-storey frames designed with nine different design load patterns (from design guidelines and literature) were compared under 15 synthetic spectrum-compatible earthquakes and 5 real strong earthquake ground motions. Different performance parameters such as maximum inter-storey drift, plastic hinge rotation and cumulative damage were used to identify the most suitable load patterns for practical applications. Based on the results presented in this paper, the following conclusions can be drawn:

- Overall, the efficiency of Eurocode 8 (EC8, EN 1998-1:2004) and ASCE/SEI 7-10 (2010) lateral design load pattern decreases by increasing the number of storeys. This may imply that the code suggested load pattern cannot accurately take into account the higher mode effects in non-linear multi-storey frames under strong earthquakes. The design load pattern proposed by Moghaddam and Karami Mohammadi (2006) and the seismic code of Japan (BCJ, 1997), on average, led to the design solutions with slightly lower global damage indices compared to Eurocode 8 (EC8, EN 1998-1:2004) and ASCE/SEI 7-10 (2010).

- The Park and Medina (2007) proposed load pattern could not control the maximum inter-storey drift ratios at the top floors of low to medium-rise SMRFs, which led to very high global damage indices and collapse in these structures. However, this load pattern could provide acceptable design solutions for high-rise buildings. In contrast, the Hajirasouliha and Moghaddam (2009) proposed pattern was only suitable for low to medium-rise buildings. The load pattern proposed by Motamedi and Nateghi (2008) was shown to be inappropriate in most cases.
The SMRFs designed with the load patterns proposed by Goel et al. (2010), Deguchi et al. (2008) and Kato et al. (1982) exhibited considerably (up to 68%) lower inter-storey drift ratios, plastic hinge rotations and global damage indices compared to their code-based counterparts with the same structural weight. While all of these load patterns are suitable for the seismic design of SMRFs, the Goel et al. (2010) load pattern seems to be the most efficient one.

It was shown that the storey shear distribution of the design load patterns with better seismic performance had a better agreement with the distribution of maximum storey shear forces during the design earthquakes. Therefore, this criterion can be used as a simple measure to assess the adequacy of the design load pattern in practice.

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List of Notations

530  $D_t$  Global damage index

531  $h_i$  Height from the base to level $i$

532  $A_i$  Shear distribution factor used in P.5 load pattern

533  $C_B$  Base shear coefficient used in P.5 load pattern

534  $C_{iw}$  Coefficient used in P.2 load pattern.

535  $F_i$  Lateral seismic force at $i^{th}$ storey level

536  $F_t$  The portion of the base shear that is applied as a concentrated force at the top of the structure

537  $V_i$  Storey shear force at level $i$

538  $V_y$  Base shear strength

539  $V_{yi}$  Shear strength of the $i^{th}$ storey

540  $W_t$  Total weight of the structure.

541  $a_i, b_i, c_i, d_i$  Constant coefficients corresponding to the soil profile used in P.3 load pattern

542  $w_i$  Total effective seismic weight of the structure located at $i^{th}$ level

543  $\alpha_i$  Coefficient used in P.5 load pattern

544  $\beta_i$  Shear distribution factor at level $i$

545  $\delta_{i,n}$  Coefficient used in P.4 and P.9 load patterns

546  $\mu_t$  Target ductility demand of structure

547  $\xi_i$  Coefficient used in P.7 load pattern

548  $\phi_i$  Dimensional less parameter used in P.3 load pattern

549  $b$  Altitude of triangular load used in Iranian seismic code (BHRC, 2005)

550  $c$  Parameter to account for the plastic deformation magnitude

551  CP  Collapse prevention performance level of ASCE/SEI41-13

552  $D_i$  Cumulative damage index at $i^{th}$ storey

553  $E$  Elastic Modulus

554  $E_T$  Tangent Modulus

555  $F_{ye}$  Expected yield stress

556  $I_b$  Moment of inertia of the beam elements

557  $I_c$  Moment of inertia of the column elements
Immediate occupancy performance level of ASCE/SEI41-13

Effective length of the beam elements

Effective length of the column elements

Life safety performance level of ASCE/SEI41-13

Mean absolute error

Local magnitude of the earthquake

Number of plastic excursions

Axial force of the column

Peak ground acceleration of the earthquake

Peak ground displacement of the earthquake

Peak ground velocity of the earthquake

Expected axial yield force of the column

Response modification coefficient

Total design lateral force at the base of the structure (base shear)

Dissipated energy at \( i^{th} \) storey

Plastic modulus of each cross-section

Nominal yield deformation of \( i^{th} \) storey

Plastic deformation of \( i^{th} \) storey in \( j^{th} \) cycle

Maximum allowable rotation for life safety (LS) performance level

Maximum plastic rotation of \( i^{th} \) element

Yield rotation specified in ASCE/SEI41-13

Overstrength factor

Total height of the structure from the base

Fundamental period of the structure

Coefficient used in P.4 load pattern

Exponent parameter related to the fundamental period of the structure

Number of storeys

Exponent parameter used in P.2 load pattern.

Coefficient used in P.9 load pattern
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Figure 3. Lateral load patterns (P-1 to P-9) used to design 10-storey SMRFs

Figure 4. (a) Structural weight and (b) Design base shear of 3, 5, 7, 10 and 15-storey frames designed with different load patterns
Figure 5. Average storey drift ratio of 3, 5, 7, 10 and 15-storey frames designed with different load patterns in 15 synthetic spectrum compatible earthquakes.
Figure 6. (a) Maximum ($\Theta_p/\Theta_{all}$), and (b) Mean ($\Theta_p/\Theta_{all}$) for the structural elements of the frames designed with different load patterns, average in 15 synthetic spectrum compatible earthquakes.

Figure 7. (a) Height-wise distribution of inter-storey drift ratios, (b) Probability density of maximum inter-storey drift ratios, 5-storey frame designed with P-6 load pattern under 15 synthetic earthquakes.

Figure 8. Global damage index of 5-storey frames designed with different load patterns under synthetic spectrum compatible earthquakes (a) No. 4, (b) No. 8, and (c) No. 12.
Figure 9. Global damage index of 3, 5, 7, 10 and 15-storey frames designed with the different load patterns, average of 15 synthetic spectrum compatible earthquakes.

Figure 10. Maximum inter-storey drift distribution of 5-storey frames designed with different load patterns (a) average of the 15 synthetic earthquakes, (b) average of the 5 real earthquakes.

Figure 11. Global damage index of 5-storey frames designed with the different load patterns (a) average of the 15 synthetic earthquakes, (b) average of the 5 real earthquakes.
Figure 12. Comparison between the relative distribution of design storey shear forces and the average of storey shear forces exhibited in the 15 synthetic spectrum compatible earthquakes.
Figure 13. The mean absolute error (MAE) between ($V_i/V_n)_D$ and ($V_i/V_n)_E$ ratios for the 3, 5, 7, 10 and 15-storey frames designed with different load patterns.