Modified Seismic Design Lateral Force Distribution for the Performance-Based Plastic Design (PBPD) of Steel Moment Structures Considering Soil Flexibility

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Abstract. It is well recognized that structures designed by conventional seismic design codes experience large inelastic deformations during strong ground motions. Realistic estimation of force distribution based on inelastic response is one of the important steps in a comprehensive seismic design methodology in order to represent expected structural response more accurately. This paper presents an extensive parametric study to investigate the structural damage distribution along the height of the steel moment-resisting frames (SMRFs) designed based on the stat-of-art constant-ductility performance-based plastic design (PBPD) approach considering soil flexibility effects when subjected to 20 strong ground motions. To this end, the effect of fundamental period, target ductility demand and base flexibility level are investigated and discussed. Based on the numerical results of this study, simplified equations are proposed for practical purpose to refine and modify the lateral force distribution pattern already suggested by researchers based on the study of inelastic behavior developed for fixed- and flexible-base structures by using relative distribution of maximum story shears of the selected structures subjected to various earthquake ground motions. It is demonstrated that the proposed equations can be adequately estimated the optimum values of shear proportioning factor for both fixed-based and soil-structure systems.

Keywords: Seismic design lateral force; Performance-based plastic design; Steel moment structures; Soil-structure interaction, optimum shear proportion factor

1. Introduction

The design lateral forces and design story shears from equivalent lateral force recommended by the current code-specified seismic design procedure (e.g., IBC 2009 [1], ASCE/SEI 7-10 [2], NEHRP 2009 [3], UBC [4]) are primarily based on elastic analysis. In this method the structural elements are designed based on equivalent static forces and the shape of fundamental mode of the structure is dominant to determine the height-wise distribution of these seismic design static forces. Establishing such code-compliant lateral load distributions patterns may not provide an accurate representation of the story shear strength demands and explicitly lead to seismic performance assessment criteria [5-12]. Chopra [13] conducted nonlinear dynamics analyses of several shear-building models subjected to the El-Centro Earthquake of 1940 to evaluate the ductility demands corresponding to each story. The Models were designed in accordance with the seismic force patterns specified by Uniform Building Code [4]. It was concluded that this distribution pattern does not lead to equal ductility demand in all stories. Moghaddam [14] conducted the same analysis for a number of shear buildings with the specified yield strength distributed by UBC-97 [4] suggestion pattern. They showed that code-compliant lateral load distribution does not lead to a uniform height-wise distribution of ductility. Moghaddam and Mohammadi [15] proposed design lateral load pattern for seismic design of shear-building structures to achieve uniform deformation distribution. In another investigation, they developed a new concept to optimize the distribution pattern for performance-based seismic design approach [11]. However, Their study were based on the results of shear-building structures that may not be applicable for more realistic building structures such as moment-resisting frames that are basically designed based on the “strong- column weak-beam” design philosophy. Several other studies focused on moment-resistant frame aimed to develop new lateral load patterns to control the amount of the global structural damage, and to achieve predefined performance objectives and finally to provide higher performance levels exposed to seismic ground motions. Leelataviwat et al. [8] proposed improved load distribution using the concept of energy balance applied to moment-resisting frames with an intended yield mechanism. Lee and Goel [16] primarily discussed the discrepancy between the earthquakes induced shear forces and the forces determined by lateral load distribution patterns. They used the same concept
to propose load pattern in accordance with the Uniform Building Code [4] which was a function of the mass and fundamental period of the structure. Goel et al. [17] applied the method successfully to a variety of common steel framing systems and Reinforced Concrete (RC) moment frames. Through the results of extensive inelastic static and dynamic analyses, they showed that the frames could be able to develop desired strong column-sway mechanisms, and the story drifts and ductility demands were well within the target values, thus meeting the desired performance objectives. Park and Medina [18], proposed a seismic design methodology for moment-resisting frames based on uniform structural damage distributed along the height. They concluded that based on the proposed approach designs are expected to provide increased protection against global collapse and loss of life during a strong earthquake event. Chao et al. [19] primarily reviewed the lateral force distributions used in the current seismic codes by performing nonlinear dynamic analyses of a number of frame structures. They demonstrated that code lateral force distributions do not represent the maximum force distributions that may be induced during nonlinear response of motions and may causes inaccurate predictions of deformation and force demands. Their comprehensive studies lead to the development of a new seismic design lateral load distribution based on inelastic behavior of a structure and also a new methodology called Performance-Based Plastic Design (PBPD) for seismic design of a wide ranges of frame systems including moment-resisting frames, eccentrically-braced frames, special truss-moment frames and reinforced concrete frames. In these investigations, performance limit states are pointed out by predictable global yield mechanism and pre-designated target drift limit. The design base shear for each performance level is derived from 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 [16, 17, 19]. However, they did not incorporate the target ductility demand in the design process directly.

All the above-mentioned researches were based on the fixed-base structures without considering the effect of soil flexibility i.e., soil-structure interaction (SSI). Several studies have been performed to investigate the effect of SSI on seismic responses of structures [20-24]. Results of these studies demonstrated that structures supported by soil-foundation may be affected by SSI significant roles due to wave propagation in the soil medium. Based on the concept developed for fixed-base shear structures, Ganjavi and Hao [25] proposed a new optimum design lateral loading patterns for seismic design of elastic soil-structure systems through an intensive dynamic analyses of multistory shear-building models subjected to a group of 21 artificial earthquakes adjusted to soft soil design. They also [26] parametrically investigated the adequacy of code-specified lateral loading patterns for seismic design of elastic and inelastic soil-structure systems through analyses of shear-buildings considering SSI effects. Due to code-specified lateral load distributions inabilities and to improve the seismic performance of flexible-base buildings on soft soils, Ganjavi et al. [26] proposed an optimum seismic design methodology for non-linear shear-buildings located on soft soils based on the concept of uniform damage distribution. However, their study was also based on the results of shear-building structures that may not be applicable for more realistic building structures such as SMRFs.

In this paper, lateral force distributions are evaluated by nonlinear dynamic analyses of constant-ductility SMRF structures designed according to PBPD procedure located on alluvium soil considering SSI effects. The aim of this study is to parametrically investigate height-wise structural damage (ductility demand) distribution designed based on conventional PBPD approach for constant-ductility fixed- and flexible-based SMRF structures. Also, the adequacy of load patterns already proposed in PBPD is investigated through height-wise distribution of inter-story ductility demand ratio subjected to various strong ground motions.

2. Analytical model based on PBPD approach

The PBPD method is based on two key performance limit sates including pre-selected target drift and yield mechanisms [8]. These two design parameters control the degree and distribution of structural damages directly. In this approach, determination of design base shear, lateral force distribution and plastic design corresponding to the specified performance level are the three main components of design. For a specified hazard the design base shear is calculated by equating the work needed to push the structure monotonically up to the target drift to the energy required by an equivalent elastic-plastic single-degree-of-freedom (EP-SDOF) system to achieve the same state. Also, the height-wise distribution of lateral design forces is developed based on the concept of the relative story shear distributions and is consistent with inelastic dynamic response results [19]. Finally, proposed plastic design procedure is performed to detail the frame members in order to achieve the intended yield mechanism.

2.1. Design base shear

As explained, the design base-shear as a key element in the PBPD method is calculated by equating the work needed to push the structure monotonically up to the target drift to that required by an equivalent EP-SDOF
system to achieve the same state. For idealized Elastic-perfect Plastic (EP) behavior and using the value of pseudo-velocity or substituting pseudo-acceleration, the work-energy can be calculated as Eq. (1) [27]:

\[ E_e + E_p = \gamma E = \gamma \left( \frac{1}{2} MS_i^2 \right) = \frac{1}{2} \gamma M \left( \frac{T}{2\pi} S_a g \right)^2 \]  

(1)

where \( E_e \) and \( E_p \) are the elastic and plastic components of the energy needed to push the structure up to the target drift respectively. \( S_i \) is the design pseudo-spectral velocity; \( S_a \) is the pseudo-spectral acceleration; \( T \) and \( M \) are respectively, the natural period and total mass of the system. \( \gamma \) is the energy modification factor which is related to the structural ductility factor (\( \mu_s \)) and the ductility reduction factor (\( R_{\mu_s} \)), and can be obtained by the following equation (Eq. (2)):

\[ \gamma = \frac{2\mu_s - 1}{R_{\mu_s}^2} \]  

(2)

Using the spectra proposed by Newmark and Hall (1982) [27] the energy modification factor (\( \gamma \)) can be obtained from equation 2 and plotted in Figure 1 [16].

The work-energy equation can be rewritten in the following form (Eq. (3)):

\[ \frac{1}{2} \left( \frac{W}{g} \right) \left( \frac{T}{2\pi} \frac{V_y}{W} g \right)^2 + V_y \left( \sum_{i=1}^{N} \lambda_i h_i \right) \theta_p = \frac{1}{2} \gamma \left( \frac{W}{g} \right) \left( \frac{T}{2\pi} S_a g \right)^2 \]  

(3)

By simplifying the Eq. (3), the ratio of \( (V_y/W) \) can be written as Eq. (4):

\[ \frac{V_y}{W} = \alpha + \sqrt{\alpha^2 + 4\gamma S_a^2} \]  

(4)

where \( \alpha \) is a dimensionless parameter given by Eq. (5):

\[ \alpha = \left( h_i^* \times \frac{\theta_p 8\pi^2}{T^2 g} \right) \]  

(5)

In which the \( \theta_p \) represents the plastic rotation at target drift ratio, and \( h_i^* \) stands for \( \sum_{i=1}^{N} \lambda_i h_i \) and where \( \lambda_i \) is the proportioning factor of the equivalent lateral force at level \( i \).

### 2.2. Lateral force distribution

A new design lateral force distribution for the plastic design was obtained based on the results of inelastic dynamic responses and maximum story shears along the height of structural systems, which is defined as Eq. (6) [19]:

\[ V_y = \left( \frac{\sum_{i=1}^{N} w_i / h_i}{\sum_{i=1}^{N} w_i / h_i} \right)^{0.75 \beta \alpha - 0.1} \]  

(6)

where the \( w \) and \( h \) are the seismic weight and height above the base, respectively, also \( T \) is the fundamental period and \( V_y \) represents the design base shear. The equation is more consistent to the results of inelastic analysis compared to the code-specified seismic load pattern. It can be shown that the ratio \( V/V_w \), designated as shear distribution factor, \( \beta_i \), can be obtained using Eq. (7) [19]:

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\[
V_i = \frac{\beta_i}{V_n} = \left(\sum_{j=1}^{n} \frac{w_j h_j}{w_n h_n}\right)^{0.75}\]

where \( V_i \) and \( V_n \) are the story shear at level \( i \) and top level, respectively and \( \beta_i \) is the shear proportioning factor at level \( i \). Hence, the lateral force at level \( i \), \( F_i \), can be expressed as Eq. (8):

\[
F_i = (\beta_i - \beta_{n+1}) V_n
\]

2.3. Plastic design procedure

The provided design approach is capable of achieving satisfactory performance of structures under a severe earthquake by means of a pre-defined controlled mechanism. The procedure develops the strong-column-weak beam mechanism and a stable hysteretic response within an acceptable margin of target drift [8]. Applying the principle of virtual work for the beam mechanism (Figure 2) the required beam strength at each level can be obtained from Eq. (9):

\[
\sum_{i=1}^{n} 2\beta_i M_{phi} + 2M_{pc} = \sum_{i=1}^{n} F_i h_i = \sum_{i=1}^{n} (\beta_i - \beta_{n+1}) h_i F_n
\]

where \( M_{phi} \) and \( M_{pc} \) are the plastic moment of beams and the required plastic moment of columns in the first story, respectively (Figure 2). Leelataviwat et al. [8] proposed the plastic moment of the first-story columns to avoid the pre-defined mechanism as Eq. (10):

\[
M_{pc} = \frac{1.1 V h_i}{4}
\]

in which \( V \) is the total base shear, \( h_i \) is the height of the first story and the coefficient 1.1 is the overstrength factor to account for possible overloading due to strain hardening. However, as a main goal of this approach, it is attempted to prevent formation of plastic hinges in the columns except at the column bases of the structure. Hence, column should be designed for the flexural moment greater than the sum of the flexural strength of the beams at the same joint. To ensure that the strong column-weak beam mechanism is achieved columns should be designed for sum of nominal plastic moment of beams multiplied by the over-strength factor (\( \xi \)). Also, to include the beams yielding over-strength, the applied force at each level, \( F_i \), must be updated as (Eq. (11)):

\[
F_i = (\beta_i - \beta_{n+1}) F_{nu}
\]

where \( F_{nu} \) is the updated force at the roof level and can be determined by the equilibrium equation for one column as Eq. (12):

\[
\sum_{i=1}^{n} (\beta_i - \beta_{n+1}) h_i F_{nu} = M_{pc} + \sum_{i=1}^{n} \xi_i M_{phi}
\]

where \( M_{pc} \) is the plastic moment at the base of the frame (equation 10), \( \xi \) and \( M_{phi} \) are the overstrength factor and the nominal plastic moment of beam at level \( i \), respectively. After updating the lateral forces, design moments of the column can be determined by developing the column as a cantilever based on Eq. (13):

\[
M_c(h) = \sum_{i=1}^{n} \delta_i \xi_i M_{phi} - \sum_{i=1}^{n} \delta_i F_{nu}(h_i - h)
\]

where \( M_c(h) \) is the moment in the column at a height \( h \) above the ground, and \( \delta_i \) is equal to 1 for \( h \leq h_i \) and zero for else. The axial force in the column at the height \( h \) above the ground, \( P_c(h) \), can be obtained by Eq. (14):

\[
P_c(h) = \sum_{i=1}^{n} \frac{2 \xi_i M_{phi}}{L} + P_{cg}(h)
\]

where \( L \) is the span length of the beams, \( P_{cg}(h) \) is the gravity axial force acting at height \( h \). By applying the explained approach the values of \( M_c(h) \) and \( P_c(h) \) of the column element can be obtained according to plastic analysis procedure and then it can be designed as a beam-column elements by appropriate design provisions.
Finally, it should be mentioned that as a well-known numerical modeling strategies the superstructure frame elements followed the lumped-plasticity modeling approach for computing their nonlinear response regarding the Rayleigh damping and rigid diaphragm assumption. For computer modeling beams and columns were modeled as non-degrading quasi-elastic-plastic (i.e. with strain-hardening ratio 2%). Moment-curvature relationship that considers axial load-flexural bending interaction was considered to model the hysteretic behavior of the steel columns. However, slab contribution to the beam’s bending capacity was neglected in this study. The design parameters and elevation views of designed MF frames based on PBPD approaches are listed in Table 1. The sizes of beams and columns were selected using AISC-LRFD Specifications [28] assuming A572 GR.50 steel for all members and shown in Figure 3.

3. Base flexibility model

In this study, the cone model was utilized to simulate the dynamic behavior of an elastic homogeneous soil half-space as shown in Figure 4 [29]. The model is developed using one-dimensional wave propagation theory and can represent a circular rigid foundation with mass \( m_s \) and mass moment of inertia \( I_s \) resting on a homogeneous half-space soil. The cone model has been widely used for modelling both surface and embedded foundations and, in lieu of the rigorous elasto-dynamical approach, can provide sufficient accuracy for engineering design purposes [30]. The soil-foundation system is modelled by an equivalent linear discrete model based on the cone model approach with frequency-dependent coefficients [29]. The foundation is considered as a circular rigid disk (the flexibility of the foundation is not taken in to account). The components of motions for half-space below were modeled through the two transitional and rotational DOFs. The coefficient of sway and rocking springs and dashpots representing the associated motions are summarized as equation (15) and (16) [29]:

\[
\begin{align*}
    k_h &= \frac{8\rho V^2 r^3}{2(1-\nu)} , \\
    c_h &= \rho V A , \\
    k_v &= \frac{8\rho V^2 r^3}{3(1-\nu)} , \\
    c_v &= \rho V I , \\
    M_{\phi} &= \frac{9\pi r^5}{128} (1-\nu) \left( \frac{V}{V_s} \right)^2
\end{align*}
\]

where \( k_h, k_v, c_h, \) and \( c_v \) are sway stiffness, sway viscous damping, rocking stiffness, and rocking viscous damping, respectively, \( \rho, \nu, V, \) and \( V_s \) stand for the density, Poisson’s ratio, and the dilatational and shear wave velocities of soil, respectively and \( r \) is the radius of the equivalent circular foundation. Also, for the vertical and rocking motions in the case of nearly-incompressibility soil (\( 1/3 < \nu < 1/2 \)), an additional trapped mass moment of inertia \( \Delta M_{\phi} \) equal to \( \Delta M_{\phi} = 0.3\pi(\nu-1/3)\rho r^4 \) is added to \( I_j \), which is connected to the foundation and moves as a rigid body in the phase with the foundation for the rocking degree of freedom. An internal rotational degree of freedom \( \phi \), with a mass moment of inertia \( m_\phi \) was defined to incorporate frequency dependency of soil dynamic-stiffness. It is worth noting that based on the current seismic provisions such as NEHRP 2003 [3] and FEMA 440 [31], the soil strain level related to the degraded shear wave velocity to approximate the soil nonlinearity effects on soil-foundation elements is considered [32].

It shown that for the specific earthquake, the seismic response of soil-structure system depends on the dynamic characteristics of the structure and soil beneath it. The SSI effective parameters which are known as non-dimensional key parameters, and can best describe the seismic response of super-structure in complex soil-structure system are defined as follows [33]:

A dimensionless frequency as an index for the structure-to-soil stiffness ratio defined as \( \alpha_0 = \omega_0 \bar{H} / v_s \), where \( \omega_0 \) denotes the natural frequency of the fixed-base structure. \( \bar{H} \) is the effective height of the structure corresponding to the fundamental mode properties of MDOF building and can be obtained from Eq. (17):

\[
\bar{H} = \frac{\sum_{i=1}^{n} m_i \phi_{j,i} \left( \sum_{i=1}^{n} h_i \right)}{\sum_{i=1}^{n} m_i \phi_{j,i}}
\]

(17)

Where \( j \) is the number of story, \( m_j \) is the mass of the \( j^{th} \) storey; \( h_i \) is the height from the base level to level \( j \); and \( \phi_{j,i} \) is the amplitude at \( j^{th} \) storey of the first mode.

It is shown that the \( \alpha_0 \) have the most significant effects on the seismic response of soil-structure system [34]. For conventional steel moment building structures, it takes the value between zero for fixed-base structures to about 2 for the very flexible model [35].
• Aspect ratio of the building $\frac{H}{r}$ defined as a second interacting parameter with various influences on SSI practices [35].
• Ductility demand of structure defined as $\mu = \frac{u_m}{u_y}$, where $u_m$ and $u_y$ are the maximum inter-story displacement and the yield inter-story displacement, respectively [36].
• Structure-to-soil mass ratio index defined as $\bar{m} = \frac{m_{so}}{\rho r^2 H}$, where $m_{so}$ and $H$ are the total weight and total height of the structure, respectively.
• The ratio of foundation-to-structure mass $m_f / m_{tot}$, where $m_f$ is the mass of the rigid foundation.
• Material damping ratio of the soil ($\zeta_0$).

The first two interacting parameters are the key parameters that define the main SSI effect [31]. The third one controls the level of nonlinearity in the structure. Other parameters with less importance are assigned to some typical values for conventional buildings structures [35]. Hence, in this study the foundation-to-structure mass ratio is considered as 0.5 and the foundation mass ratio was set to %10 of total mass of the superstructure. Poisson’s ratio is set to 0.4 for alluvium soil. In addition, the damping ratio of the soil material is set to 5%.

4. Nonlinear dynamics analysis procedure and selected ground motions

A series of 4-, 8-, 12- and 16-story steel moment-resisting frames (SMRFs) are considered to investigate the effect of various design load patterns on height-wise distribution of story shear forces. All models are incrementally subjected to a group of strong ground motions and the step-by-step solution scheme is applied during time history analysis. The steel frames were analyzed with the nonlinear dynamic analysis computer software OpenSees [37]. Rayleigh type damping was considered for the analysis in which a 5% of critical damping was assigned to the first two modes of vibration of the frames. Also, nonlinear static (pushover) analyses for each frame were performed to obtain relevant dynamic characteristics such as base shear and roof drift at first plastic hinge and yielding. It should be noted that pushover analysis was conducted by using the OpenSees software [37] considering a slow ramp loading function assuming a triangular-inverted loading distribution as prescribed in the Mexican seismic design standards, which is similar to that of the ASCE-7-10 load pattern (2010) and utilized by Ruiz-García and González for steel moment frame structures [38]. Since the objective of this study is to achieve a pre-specified constant-ductility demand, it is necessary to evaluate the nonlinear maximum story drift demand for various building frames. Hence, it was decided to scale up the acceleration spectral ordinates in such a manner that the maximum inter-story ductility demand among all stories reaches the specified target value [38]. For the inelastic dynamic analysis, an ensemble of 20 earthquake ground motions with different characteristics were utilized to incorporate the effect of ground motion characteristics recorded on alluvium (NEHRP site class D [39]). All the selected ground motions are obtained from earthquakes with magnitude greater than 6.5 having closest distance to fault rupture more than 10 km without pulse type characteristics. The main parameters of the selected ground motions are given in Table 2. For each record, the horizontal component with larger peak ground velocity is defined as the strong component. A flow chart is provided to show the iterative analysis procedures for constant-ductility PBPD approach as illustrated in Figure 5 [40].

5. Relative story shear distribution

The relative story shear distributions is defined as the ratio of maximum earthquake-induced story shear force at level $i$ to that at top level $n$ (i.e., $\beta_i = \frac{V_i}{V_n}$). The mean responses are obtained by averaging the results of the structures to each record. It is notable that story shear $V_i$ in any story is the sum of the lateral forces above that story, and thus the story shear distribution and lateral force distribution have a direct relationship defined as Eq. (18):

$$V_i = \sum_{j=i}^{n} F_j$$  \hspace{1cm} (18)

In addition, the proposed equation [16] tends to somewhat overestimate the optimal $b$ value obtained from the analyses. The equation (19) is recommended and used for both cases of fixed- and flexible-based SMRFs as:

$$\beta_i = \left(\frac{V_i}{V_n}\right)^\delta$$  \hspace{1cm} (19)
Figure 6 is plotted to show relative story shear distributions obtained from nonlinear dynamics analyses of 4-, 8- and 16-story fixed-base SMRFs designed by conventional PBPD subjected to 20 individual earthquake ground motions. The results are provided for two levels of low and high inelastic behavior. In addition, the average results of all the selected ground motions along with shear force pattern having different shear proportional values are provided and compared. AS seen, the distribution power b is functions of fundamental period of vibration and the level of inelastic behavior such that it respectively decreases and increases with increasing the period and ductility demand value. The dependency to the fundamental period is also reported and consistent to the previous studies [16, 19]. However, nothing has been yet reported for the effect of the target ductility demand on the b value. Results of this figure show that the optimum b value could be varied from 0.67 to 1.1 depending on the values of $T_{fix}$ and $\mu$. It will be more discussed in the upcoming section.

6. Effect of shear proportional factor

Frames with 4-, 8-, 12-, 16-story levels were used to find an optimal distribution of shear proportioning factor through nonlinear dynamic analysis. Each frame was designed using four possible functions for the shear proportioning factor. These functions were assumed by the values of b taken as 1, 0.75, 0.5, and 0.25. The member sizes of the frame structures designed by selecting 2% target drift and using the design parameters presented in previous section. It was assumed that the distribution of beam strengths follows the distribution of shear proportioning factors. The results present the distributions of the relative distributions of maximum story shears of the selected frames and the upper bound distributions of shear proportioning factors of the nonlinear dynamic analysis results for various ductility demands.

For the 4-story frame structure with shorter period ($T=0.72s$) and regardless of the values of shear proportioning factor the relative distributions of maximum story shears are close together. The relative story shear distribution using various values of b ranging from 0.7 to 0.9 represents an upper bound of the nonlinear dynamic analysis results for low to high ductility ratio. As shown for short period frame structure, the required value of b increases considerably as the level of inelasticity increases. To achieve more realistic distributions of shear proportioning factor according to the period of the structures under strong motions, results of 8-, 12-, 16-story building structure are provided. The relative distributions of maximum earthquake-induced story shears for each case of 8- to 16-story frames are shown in Figure 3. As shown, the distributions of maximum earthquake-induced story shears of frames are related to the period and can be approximated using optimal b values of shear proportioning factor for various level of inelasticity. As illustrated, the optimal b value decreases as the period increases and it increases as the target ductility demand increases. Based on the mean results of the distribution of maximum story shears of frames obtained from 20 strong earthquakes the optimal b value for each frame can be determined using trend line for upper bound of story shear distribution for various ductility demand. For 8-story ($T=1.44s$) building the optimal b value obtain ranging from 0.65 to 0.7 for low to high ductility demand by calculating the least square fit values. Further analyses by Chao and Goel (2005 and 2006a) [41- 42] show that relative story shear distribution using $b =0.75$ represents an upper bound of the nonlinear dynamic analysis results. The same analysis conducted for the 12- and ($T=2.1s$) 16-story ($T=2.5s$) building frames with longer period and the corresponding values of b were then obtained for various ductility ratios. Consequently, the optimal b value for these cases and the recommended trend line for selected frames are presented in Figure 7.

7. Effect of ductility

The effects of ductility demand on relative story shear demand distribution are investigated using three SMRF systems with 4, 8, 12 and 16 stories as representative of low- to high-rise buildings. Results of relative distribution of story shear for various low to high level of inelasticity ($\mu=2,4,6$) are plotted in Figure 8. As demonstrated, generally by increasing the level of inelasticity the relative story shear increases for all models from shorter to long period cases. In the selected frames with various stories a small difference of story shear can be seen between moderate and high level of inelasticity ($\mu=4.6$) when compared with the corresponding value of low ductility demand. Also, it is revealed that this trend is followed for low- and high-rise SMRF buildings. The b value corresponding to lower and higher ductility demand are obtained based on the relative distributions of maximum story shears.

8. Effect of base flexibility

In order to examine the effect of soil flexibility on nonlinear response of SMRFs designed based on PBPD approach Figure 9 is illustrated. To this end, the individual, mean, mean plus standard deviation (mean + $\sigma$) and different shear proportion factors are shown for low ($a_0=1$) and high ($a_0=2$) levels of base flexibility. As seen, by increasing the a0 value the amount of required parameter b decreases; however, for the case of predominant SSI effect ($a_0=2$) the relative story shear demands of top stories increase. In addition, as can be seen, the mean
Envelope curves of the maximum responses can be adequately predict the required relative shear force demands of all the selected records which are obtained by changing the b values. In fact, for each soil-structure systems the optimum b values were computed from nonlinear dynamic analyses. It is also observed that the envelope curves corresponding to the optimum values are very close to the curves of mean +σ. The results show that similar to the fixed-base systems, the shear proportion factor b is dependent on both fundamental period and level of inelastic demands, which the latter effect has not been taken into account for the conventional PBPD approach. It can be observed that by increasing the structural period and ductility demand the b values respectively decrease and increase. Based on the optimum values of b, practical expression are proposed in the next section.

9. Dispersion of results and proposed practical equation

The results presented in the previous sections are obtained from the mean responses of an ensemble of 20 strong ground motions listed in table 2. It is clear that utilizing the mean value provides an average that the efficiency of it depends on the dispersion of computed results. Based on observations, it is believed that the structural response of a structural system with the certain dynamic characteristics is strongly related to the selected ground motion record. Also, the dispersion of results shows the impact of record-to-record variability on the nonlinear responses of systems. Hence, as an effective tool to evaluate the dispersion of obtained results, the coefficient of variation (COV) is utilized that is defined as the ratio of the standard deviation to the mean value. As illustrated in Figure 10 the COV distribution are provided for the 8- and 16- story SMRF structures with various ductility demands. The derived spectra present the dispersion along the height of selected models. It is observed that increasing the fundamental period of structures is accompanied by an increase in COV of maximum story shear distribution for both fixed-base and flexible cases. The COV values of the results exhibit low dependency on the ductility demand for structures with shorter periods. This is probably because the structural response is governed primarily by the fundamental vibration mode in short period range; hence the ductility demands along the building height also follow the fundamental mode pattern.

The format for this design lateral force distribution based on inelastic state of a structure was originally proposed by Lee and Goel [16] by using shear distribution factor derived from the relative distribution of maximum story shears of a large number of SMRFs subjected to four selected earthquake records. They applied the least square fit method and proposed practical equation 19 in which b is defined as Eq. (20):

\[ b = aT^{-0.2} \]  

(20)

where \( \beta_i \) is the shear distribution factor at level \( i \); \( V_i \) and \( V_n \) respectively, are the story shear forces at level \( i \) and at the top (nth) level, T is the fundamental period. The value of parameter \( \alpha \) was originally proposed as 0.5 by Lee and Goel (2001) [16], which was later revised to 0.75 based on more extensive nonlinear dynamic analyses on eccentrically braced frames (EBFs) and special truss moment frames (STMFs) by Chao and Goel [41, 42]. However, the level of inelastic behavior (i.e., ductility demand) has not been considered in their proposed equation. In this study, extensive nonlinear dynamic analyses were carried out on SMRF systems designed based on PBPD approach and with different levels of inelasticity (i.e., constant ductility demand) to modify the suggested lateral force distribution for various ductility demands. Based on the proposed equation for various ductility demands, more realistic design lateral force distribution is obtained accounting for inelastic behavior of structures when subjected to strong ground motions. The suggested optimum \( b \) value for lateral force distribution that can be applied to most of the conventional frame types is defined as Eq. (21):

\[ b = aT^{-c} \]  

(21)

where the parameters \( a \) and \( c \) are computed from nonlinear regression analyses of numerical data for both fixed-base and flexible-base structures having two levels of low and high inelastic behaviour as shown in Figure 11. As can be seen, the proposed equations can be adequately estimated the optimum values of \( b \) for both fixed-and SSI systems. In addition, the results clearly show that optimum \( b \) value increase with increasing the ductility demand. Conversely, the \( b \) value decreases with increasing the \( a_0 \) parameter while similar to the fixed-base systems descends with increasing the fundamental period.

8. Conclusions

In this study, extensive nonlinear dynamic analyses were carried out on fixed- and flexible-based SMRF systems designed based on new constant-ductility PBPD approach with different levels of inelasticity under a group of 20 strong earthquake ground motions to modify the suggested lateral force distribution for various
ductility demands. Effect of inertial soil-structure systems is parametrically investigated on height-wise distribution of relative shear force demands. In addition, the adequacy of various lateral load patterns in ductility distribution of PBPD frame buildings has been investigated. Based on the results, it is revealed that the height-wise distribution of story ductility demands tend to more uniform as the fundamental period of the structure increases. However, by increasing the ductility ratios the distribution of the ductility demand becomes more non-uniform along the height of structure.

Based on the proposed equation for various ductility demands, more realistic design lateral force distribution was proposed accounting for inelastic behavior of structures when subjected to strong ground motions. The suggested lateral force distribution can be applied to the most of the conventional steel moment frame systems. The results show that the suggested modification for lateral force distribution, for the types of steel framed structures investigated in this study, is more rational and gives a better prediction of inelastic seismic demands at various levels of inelasticity. It is demonstrated that the proposed equations can be adequately estimated the optimum values of shear proportioning factor b for both fixed- and SSI systems. In addition, the results clearly show that optimum b value increase with increasing the ductility demand. Conversely, the b value decreases with increasing the $a_0$ parameter while similar to the fixed-base systems descends with increasing the fundamental period.

References


Figure 1. Energy modification factor, $\gamma$, versus period

Figure 2. One-Bay Frame with (a) Selected Mechanism (b) Frame with Soft-Story Mechanism

Figure 3. Selected 4, 8, 12, 16-story moment frame

Figure 4. Typical multi-story SMRF building models (a) fixed-base model and (b) flexible-base model

Figure 5. Flowchart showing the general procedure for PBPP approach for constant-ductility nonlinear dynamic analysis of fixed- and flexible-base building

Figure 6. Relative distribution of story shear $V_i/V_n$ for fixed-base structures under strong ground motions

Figure 7. Comparison of relative shear distributions by various shear proportioning factors- Fixed-base system

Figure 8. Relative distribution of story shear $V_i/V_n$ for designed MFs with various ductility

Figure 9. Relative distribution of story shear $V_i/V_n$ for designed MFs for flexible base SMRFs

Figure 10. The COV of relative distribution of story shear $V_i/V_n$ for various ductility demands

Figure 11. Optimum $b$ Values versus Structure Period for fixed and flexible SMRFs, average of 20 earthquakes

Table 1. Design parameters for 4-, 8-, 12-, 16-storey steel PBPD frame used to various calibrate $\beta_i$

Table 2. Characteristics of strong ground motions used in this study
Figure 2.

(a) \[ F_i = (\beta_i - \beta_{i1}) F_{\text{br}} \]

(b) \[ F_{\text{br}} = \beta_i M_{\text{pc}} \]

Figure 3.
Figure 4.
Select the design parameters for steel moment building model \((T_{fix}, \mu, \theta_{p}, \theta_{u}, a_{0})\)

Calculate the \(\theta_p, \gamma\), required design base shear coefficient for the target ductility ratio \((\mu)\) and lateral load distribution

Analyze the steel frame based on PBPD approach

Design steel frame based on PBPD approach applying AISC Code

Conducting nonlinear dynamic analysis and calculate the maximum story ductility demand \((\mu_{max})\)

Scale up the acceleration spectral

NO

Error \(\mu_{max} \leq 0.5\%\)

YES

End

Story shear distribution

Figure 5.
Figure 6.
Figure 7.
Figure 8.
Figure 9.
Figure 10.

Figure 11.
Table 1.

<table>
<thead>
<tr>
<th>Design parameters</th>
<th>10% in 50 years hazard</th>
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<td>$\gamma$</td>
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<td>$\beta_i = \left(\frac{V_i}{V_s}\right)^b$</td>
<td>$b = 0.75\beta_i^{-0.2}$</td>
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Table 2.

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<table>
<thead>
<tr>
<th>Name</th>
<th>Degree, Institution, Year, CurrentPosition, Research Interests</th>
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<tbody>
<tr>
<td>Behnoud Ganjavi</td>
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<td>PhD in Earthquake Engineering, McGill University, 1997, Associate Professor at School of Civil Engineering, Iran University of Science &amp; Technology, Tehran, Iran, Reliability of Structures, Retrofitting and Repairing of Structures, Buildings and Bridges</td>
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