

PERFORMANCE-BASED SEISMIC DESIGN OF STEEL MOMENT FRAMES USING TARGET DRIFT AND YIELD MECHANISM

Soon-Sik LEE¹, Subhash C. GOEL², and Shih-Ho CHAO³

SUMMARY

A performance-based seismic design procedure for steel moment frames based on pre-selected yield mechanism and target drift is proposed in this paper. The design base shear is derived from a modified energy balance equation incorporating the concept of seismic force reduction factor and the displacement amplification factor. A new seismic lateral force distribution based on nonlinear dynamic analyses is also presented. Three steel moment frames were designed using the proposed lateral force distribution and the performance plastic design methodology with specified pre-selected target drifts. The inelastic seismic behaviors of the three frames were studied through nonlinear static and dynamic analyses. The results showed that all three frames performed very well under design level ground motions; in particular, all frames exhibited expected yield mechanism with story drifts within the target limit. That is, the structural performance can be well predicted and "controlled" by employing the proposed performance-based design procedure.

INTRODUCTION

Many studies have shown that building structures designed by modern seismic code procedures are expected to undergo large cyclic deformations in the inelastic range when subjected severe earthquake ground motions. Nevertheless, most seismic design codes are still based on elastic methods using equivalent static lateral design forces. This procedure can result in unpredictable and poor response during severe ground motions with inelastic activity unevenly distributed among structural members. To solve this problem, Leelataviwat [1] developed a new performance-based plastic design procedure for steel moment frames using the concept of energy balance applied to a pre-selected yield mechanism, with adequate strength and ductility. This study is an extension and modification of the previous study by Leelataviwat [1]. It is well known that force reduction and displacement amplification factors, intended to account for damping, energy dissipation capacity as well as overstrength, play important roles in seismic design. However, since the previous proposed design method (Leelataviwat, 1998) did not consider the above factors as influenced by the structural periods, the method could result in conservative design for

¹ Senior Bridge Engineer, URS Corporation, Roseville, USA. Email: soon_sik_lee@urscorp.com

² Professor, University of Michigan, Ann Arbor, USA. Email: subhash@engin.umich.edu

³ Ph.D. Candidate, University of Michigan, Ann Arbor, USA. Email: shchao@engin.umich.edu

long period structures and unconservative design for low-rise, short period structures. For simplicity, a linear distribution of lateral design forces has been generally used in the codes. However, many studies have shown that this distribution may not be applicable in the inelastic stage and may underestimate the story shears. It can also be too conservative for the design of columns in the performance based plastic design procedure. Moreover, this distribution does not satisfactorily recognize the higher mode effects for high-rise building structures. In this study, a new distribution of the lateral forces derived from nonlinear dynamic analysis is presented and applied to the proposed performance based plastic design procedure. The results of nonlinear static and dynamic analyses of three example steel moment frames (3, 9, and 20 story) designed by the proposed method are also presented and discussed.

MODIFIED ENERGY BALANCE EQUATION

Design base shears in current seismic codes are calculated by reducing the elastic strength demands to the inelastic strength demands using the response modification factors. These inelastic strength demands are further increased according to the importance of specific structures using occupancy importance factor. Generally, the design base shear is determined from the code prescribed design acceleration spectrum and expressed in the following form:

$$V = C_e \left(\frac{I}{R}\right) W \tag{1}$$

where C_e is the normalized design pseudo-acceleration; *I* is the occupancy importance factor; *R* is the response modification factor; and *W* is the total seismic weight. After selecting the member sizes for required strengths (which is generally done by elastic analysis) the calculated drift using elastic analysis is multiplied by deflection amplification factor, such as C_d given in the codes, and kept within specified drift limits (in the order of 2%).

It is noted that the response modification factors, R, specified in design codes for various structural systems are determined primarily based on engineering judgment. Moreover, as stated earlier, the conventional design procedures in the codes are based on elastic force-based analysis methods rather than displacement-based methods, thus the inelastic response beyond the elastic limit for a structure cannot be predicted with good precision. A more rational design approach to overcome the shortcomings in the conventional approach was proposed by Leelataviwat [1] and modified by Lee and Goel [2], which uses energy balance equation as the design basis with the structure pushed monotonically up to a target drift after the formation of a selected yield mechanism. The amount of external work needed to do that is assumed as a factor γ times the elastic input energy $E(=\frac{1}{2}MS_{\nu}^2)$. The modification factor γ is dependent on the natural period of the structure which has significant influence on the earthquake input energy, as observed by many investigators [3]. Thus, the modified energy balance equation can be written as:

$$\gamma E = (E_e + E_p) \tag{2}$$

where E_e and E_p are, respectively, the elastic and plastic components of the energy needed as the structure is pushed up to the target drift; S_v is the design pseudo-velocity; M is the total mass of the system. The modification factor, γ , depends on the structural ductility factor (μ_s) and the ductility reduction factor (R_{μ}), which is related to the structure's period. Fig. 1 shows the relationship between the base shear (*CW*) and the corresponding drift (Δ) of an elastic system and an elastic-plastic system and Eqn. 2 can be written as:

$$\gamma \frac{1}{2} C_{eu} W \Delta_e = \left(\frac{1}{2} C_y W (2\Delta_{\max} - \Delta_y) \right)$$
(3)

Using the expression for drifts (Δ), Eqn. 3 can be rewritten as:

$$\gamma \frac{\Delta_e}{\Delta_y} = \frac{(2\Delta_{\max} - \Delta_y)}{\Delta_e}$$
(4)

where Δ_e and Δ_{\max} from Fig. 1 are equal to $R_{\mu} \Delta_y$ and $\mu_s \Delta_y$, respectively. Substituting these terms in Eqn. 4, the energy modification factor γ can be determined as:

$$\gamma = \frac{2\mu_s - 1}{R_u^2} \tag{5}$$

According to Eqn. 5, the modification factor is a function of the ductility reduction factor and the structural ductility factor. Using different approaches, many investigators have studied the relationship between ductility reduction factor (R_{μ}) and structural ductility factor (μ_s) [4]. In this study, the relationship suggested by Newmark and Hall [5] is used. Fig. 2 shows the relationship between μ_s , R_{μ} , and the structural period, and Fig. 3 shows the resulting values of γ .

The design input energy can be determined from the elastic design pseudo-acceleration spectra as given in the building codes. In this study, the design is based on the UBC [6] design spectrum which, for elastic systems, is specified as:

$$A = C_e g \tag{6}$$

where A is the design pseudo-acceleration, g is the acceleration due to gravity, and C_e is the normalized design pseudo-acceleration as defined in Eqn. 1. Note that no occupancy importance factor is included in the design pseudo-acceleration for the approach proposed. The occupancy importance factor, I, raises the design force level in an attempt to decrease the drift and ductility demand for the structure for a given level of ground shaking. However, that cannot be considered as a direct method to achieve the intended purpose such as damage control. The reduction of potential damage should better be handled by using appropriate drift limitations. In this regard, the method of calculating the design base shear proposed in this study uses the target drift as an important parameter. It is assumed that the selected target drift will have the occupancy importance factor built into it.

The energy balance equation can be rewritten as:

$$(E_e + E_p) = \gamma \left(\frac{1}{2}MS_v^2\right) = \frac{1}{2}\gamma M \left(\frac{T}{2\pi}C_e g\right)^2$$
(7)

Akiyama [7] showed that the elastic vibrational energy can be calculated by assuming that the entire structure is reduced into a single-degree-of-freedom system, *i.e.*,

$$E_e = \frac{1}{2}M\left(\frac{T}{2\pi}\cdot\frac{V}{W}\cdot g\right)^2 \tag{8}$$

where V is the design yield base shear and W is the total seismic weight of the structure (W=Mg). Substituting Eqn. 8 into Eqn. 7 and rearranging the terms gives:

$$E_{p} = \frac{WT^{2}g}{8\pi^{2}} \left(\gamma C_{e}^{2} - \left(\frac{V}{W}\right)^{2} \right)$$
(9)



Figure 1. Structural idealized response and energy balance concept.



NEW LATERAL FORCE DISTRIBUTION FOR PROPOSED DESIGN PROCEDURE

Inelastic dynamic analyses were conducted to study the distribution of maximum story shears. The nonlinear analysis program SNAP-2DX [8] was used to perform the analyses. The study frames were subjected to a set of four earthquake records [9]. Fig. 4 shows an example 5-bay, 9 story steel frame. The relative distributions of maximum story shears due to four selected earthquake records and the UBC static story shears are compared in Fig. 5 and a considerable difference can be noticed between the UBC lateral force distribution and that obtained from the analyses. The ratio of the earthquake induced story shear at level i to that at the top level, n, is assumed to be of the form:

$$\beta_i = \left(\frac{V_i}{V_n}\right)^b \tag{10}$$

where V_i and V_n , respectively, are the static story shears at level *i* and at the top level as computed from the design forces given by the UBC lateral force equations. The static story shears, V_i and V_n , at level *i* and the top story were assumed as:

$$V_{i} = \left(\frac{\sum_{i}^{n} w_{i}h_{i}}{\sum_{j=1}^{n} w_{j}h_{j}}\right) \cdot V$$
(11)

$$V_n = \left(\frac{w_n h_n}{\sum_{j=1}^n w_j h_j}\right) \cdot V \tag{12}$$

Substituting Eqs. 11 and 12 into Eqn 10 gives:

$$\boldsymbol{\beta}_{i} = \left(\frac{\sum_{i=1}^{n} w_{i} h_{i}}{w_{n} h_{n}}\right)^{b}$$
(13)

where V is the base shear, w_i (or w_j) is the weight of the structure at level *i* (or *j*), h_i (or h_j) is the height of the structure at level *i* (or *j*); w_n is seismic weight of the structure at the top story and h_n is the height of top story. An equation of $0.5T^{-0.2}$ for the exponent *b* in Eqn. 13 was found to give a good fit with the analysis results as shown in Fig. 5 [9]. Thus, the proposed lateral force applied at the top story, F_n , is given by:

$$F_n = V \cdot \left(\frac{w_n h_n}{\sum_{j=1}^n w_j h_j}\right)^{0.5T^{-0.2}}$$
(14)

The force applied at level, F_i , can be written as:

$$F_i = (\beta_i - \beta_{i+1})F_n \tag{15}$$

where β_i and β_{i+1} , respectively, are the shear proportioning factors at level *i* and level *i*+1. Note when i = n, $\beta_{i+1} = 0$.



Figure 4. Example 5-bay, 9 story frame.

Figure 5. Distribution of maximum story shears due to selected ground motions.

DESIGN BASE SHEAR BASED ON TARGET DRIFT

By using a pre-selected yield mechanism as shown in Fig. 6 and equating the plastic energy term E_p to the external work done by the design lateral forces gives:

$$E_p = \sum_{i=1}^n F_i h_i \theta_p \tag{16}$$

where θ_p is the global inelastic drift of the structure, which is the difference between the target drift (θ_u) and yield drift (θ_y). Substituting Eqs. 9 and 15 into Eqn. 16, and solving for *V/W* gives:

$$\frac{V}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma C_e^2}}{2} \tag{17}$$

where V is the design base shear and α is a dimensionless parameter, which depends on the stiffness of the structure, the modal properties and the intended drift level, and is given by:

$$\alpha = \left(\sum_{i=1}^{n} (\beta_i - \beta_{i+1})h_i\right) \cdot \left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.5T^{-0.2}} \cdot \left(\frac{\theta_p 8\pi^2}{T^2 g}\right)$$
(18)

Eqn.17 gives the required design base shear corresponding to an intended inelastic drift level, θ_p , and pre-selected yield mechanism.



Figure 6. Steel moment frame in the target drift response state with the pre-selected yield mechanism.

PLASTIC DESIGN OF MOMENT FRAMES

It is desirable that the required distribution of beam strength along the height closely follow the distribution of story shears induced by design lateral forces. Therefore, the required beam strength at each level can be determined as follows:

$$\sum_{i=1}^{n} 2\beta_{i}M_{pbr} + 2M_{pc} = \sum_{i=1}^{n} F_{i}h_{i} = \sum_{i=1}^{n} (\beta_{i} - \beta_{i+1})h_{i}F_{n}$$
(19)

where M_{pbr} is the reference plastic moment of beams and the only unknown variable in the above equation. The required plastic moment capacity of the first-story columns to prevent the story mechanism in the first story can be taken as [1]:

$$M_{pc} = \frac{1.1Vh_1}{4}$$
(20)

where V is the total base shear, h_1 is the height of the first story, and the factor 1.1 is the overstrength factor to account for possible overloading due to strain hardening [1]. In order to ensure the selected strong column-weak beam plastic mechanism at the ultimate drift level, it is essential that columns are designed by assuming that all beam plastic hinges are fully yielded and strain-hardened when the drift is at the target ultimate level. The detailed procedure for designing the columns can be found in the previous study [1].

SEISMIC EVALUATION OF THE STUDY FRAME

The three example frames were designed using the proposed design procedure. Fig. 7 shows the resulting member sizes of these frames. In this study, the frames were designed by selecting an assumed 2% target drift, and all members to have specified yield strength of 50 ksi. The members were designed by using AISC-LRFD specification. Design parameters for the example frames are shown in Table 1.

A series of non-linear analyses including inelastic static analysis and inelastic time history analysis was conducted to investigate the response of these frames. All analyses were carried out by using the computer program SNAP-2DX [8]. Strain-hardening and viscous damping values of 2 % were used for all members. Fig. 8 shows the base shear versus roof displacement plot for the frames obtained from the static pushover analysis. As can be seen, the yield drift and the design base shear of the frame are very close to the values assumed in the design stage as shown in Table 1. Fig. 9 shows the envelopes of maximum story drifts of the three frames due to four selected ground motions. It can be noticed that the story drifts are generally within the target design limit of 2%, as expected.

The locations of inelastic activity and the rotational ductility demands at plastic hinges are shown in Fig. 10. As expected, the plastic hinge in the frames formed only in the beams and at the column bases without developing any undesirable mechanism, such as a soft story mechanism. It was observed that the plastic hinges at the column bases formed later than those in the beams in most cases, and the rotational ductility demands at the column bases are much smaller than those in the beams. This suggests that the chances of

premature failure at the column base can be significantly reduced by designing the structure according to the proposed methodology.

Number of	Period	Ce	Assumed	$ heta_ ho$	γ	α	V/W	
Stories	(sec.)		Yield Drift					
3	0.546	1.100	0.01	0.01	0.820	2.775	0.325	
9	1.285	0.635	0.01	0.01	0.750	1.505	0.179	
20	2.299	0.431	0.0075	0.0125	0.609	1.272	0.083	

Table 1. Design Parameters (2% Target Drift Limit) of the Three Structures.





_			W30x90		W	/30x90			W30x90			W30x90			W30x90	
8 stories @ 130"	W14x257		W30x116	W14x426	w	'30x116	W14x426		W30x116	W14x426		W30x116	W14x426		W30x116	W14x257
		ļ Ŧ	W33x130	H	=	/33x130		=	W33x130	=	F	W33x130	:	 T	W33x130	ļ
	W14x 342		W36x135	W14x550	w	/36x135	W14x550		W36x135	W14x550		W36x135	W14x550		W36x135	W14x342
		F	W40x149	-	- w	40x149		=	W40x149	-	-	W40x149	-	Ē	W40x149	
	9CFx42K		W40x149	W14x665	w	40x149	W14x665		W40x149	W14x665		W40x149	W14x665		W40x149	W14x426
		Ŧ	W40x149	11	= w	40x149		=	W40x149	=	-	W40x149	-	Ī	W40x149	Ţ
	W14x455		W40x149	W14x665	W	40x149	W14x665		W40x149	W14x665		W40x149	W14x665		W40x149	W14x455
		Ī	W40x149	1	=w	40x149		=	W40x149	=	-	W40x149	:	L T	W40x149	Ţ
-18'-0''	W14x500			W14x665			W14x665			W14x665			W14x665			W14x500
1				77	<i>]]i</i>		7//	<i>"//</i>		7//	<i> </i>		7			
	•5 equal bays @ 30'-0"•															



(c) 20-Story Frame

Figure 7. Member sizes of the 3, 9, and 20-Story frames designed for 2% target drift.



Figure 8. Base shear versus roof drift responses from nonlinear pushover analysis.



Figure 9. Maximum story drifts of the frames due to selected earthquake records.











Rotational Ductility Demands: 1.0-1.5, 1.5-2.0, 2.0-2.5, 2.5-3.0



Rotational Ductility Demands: **•**1.0-1.5, **•**1.5-2.0, **•**2.0-2.5, **•**2.5-3.0

Figure 10. Location of inelastic activity in 3, 9, and 20-Story frames under the four selected earthquakes.

CONCLUSION

A new performance-based seismic design procedure using modified energy balance concept and plastic design methodology is proposed. The modified energy balance equation accounts for the structural ductility factor (μ_s) and the ductility reduction factor (R_{μ}), which is related to a structure's period. A new design lateral force distribution based on nonlinear inelastic dynamic analysis results was also developed. It is noted that, in the proposed design method, the target story drift is specified as a key design parameter and, therefore, no further check for drift is required.

In order to validate the proposed method, three steel moment frames with 3, 9, and 20 stories were designed. Nonlinear static and dynamic analyses were conducted and the results show that the proposed method can produce structures that meet preselected performance objectives in terms of yield mechanism and target drift.

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