Available online at www.sciencedirect.com

ScienceDirect

Procedia Engineering 14 (2011) 3354–3361

Procedia Engineering

www.elsevier.com/locate/procedia

The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction

Direct Displacement Based Design of Regular Steel Moment Resisting Frames

S. MALEKPOUR[1a], H. GHAFFARZADEH[2b], and F. DASHTI[3]

[1] *Structural Engineering Department, School of Civil Engineering, University of Tabriz, Tabriz, Iran*
[2] *Structural Engineering Department, School of Civil Engineering, University of Tabriz, Tabriz, Iran*
[3] *University College of Science and Technology, Oroumieh, Iran*

Abstract

Displacement Based Design method represents a new approach to performance-based design. This research tries to assess the Direct Displacement Based Design (DDBD) method for regular steel moment resisting frames and develop a reliable design method for them so that they withstand various seismic levels within certain performance levels. For this purpose, regular steel frames with 4, 8, 12, 16 stories are designed based on DDBD approach utilizing displacement spectrum of the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800). In order to evaluate seismic response of the designed structures, a series of non-linear time-history analyses have been performed under different records compatible with Standard No. 2800. All the non-linear analyses were carried out using the fiber-element models developed in Seismostruct computer program. According to the results, inter-story drift profile of the structure which is corresponding to its damage was less than the allowable value in most cases. Also, Maximum displacement profile of the structure along its height is completely matched with the primary assumed design profile. The structures have mostly experienced similar residual drift values under different records. In summary, the method performed quite satisfactorily in terms of story maximum displacements, maximum inter-story drifts and story ductility demands, even for tall models.

Keywords: Direct displacement based design, Steel moment resisting frames, Design method, Seismic response

[a] Corresponding author: Email: saleh_malekpour@yahoo.com
[b] Presenter: Email: ghaffar@tabrizu.ac.ir

doi:10.1016/j.proeng.2011.07.424

Direct Displacement Based Design method is one of the most recently methods proposed for performance-based design of structures. During last decade, different methods have been proposed based on displacement-based design of structures but only a few are appropriately applicable within modern design codes.

This study concentrates on direct displacement based design of regular steel moment resisting structures. The concepts of Gulkan & Sozen (1974) which are recently developed by Priestley & Kowalski (2000) for direct displacement based design of regular and ductile RC structures are used to estimate nonlinear response of elastic models with equivalent damping. This paper tries to propose reliable design procedures for seismic design of regular steel moment-resisting structures within certain performance levels.

For this purpose, four 2D regular steel frames with 4, 8, 12, 16 stories are designed based on DDBD approach, and utilizing displacement design spectrum of the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800). Assuming that only beams are to be yield through estimating story yield drift, capacity design method is used to design frame elements under lateral loads. Finally, applying different design spectrums compatible ground motions nonlinear time history analysis is performed for the designed structures under DDBD method. The results are almost in agreement with the seismic provisions of current codes. DDBD method is expected to be an alternative for the current design methods of steel moment resisting frames.

2. DDBD METHODOLOGY OF REGULAR STEEL MOMENT RESISTING FRAMES

2.1 Displacement based design basis

As also mentioned by Priestly (2003) it is proven that the damage limit can be related to strain which can be transformed into equivalent displacements. But it is not practical to directly relate the damage limit to force-level.

In DDBD methodology, the original MDOF structure is substituted with an equivalent SDOF system. This equivalent system is represented by a secant stiffness K_e at maximum displacement Δ_d and an equivalent viscous damping including both the viscous and hysteretic damping of structure.

With the design displacement Δ_d determined (Eq. 14), and the damping estimated from the expected ductility demand ξ_{eq} (Eq. 19), the effective period T_e at maximum displacement response can be read from a set of design displacement spectra. Representing the structure as an equivalent SDOF oscillator stiffness K_e at maximum response spectrum, displacement can be found by inverting the equation for natural period of the SDOF oscillator:

$$T_e = 2\pi\sqrt{\frac{m_e}{k_e}} \tag{1}$$

$$K_e = 4\pi^2 m_e / T_e^2 \tag{2}$$

where m_e and K_e are respectively effective mass and effective stiffness of SDOF structure.

The design base shear V_B, at maximum response can be expressed as below:

$$V_B = F_u = K_e.\Delta_d \tag{3}$$

Determined base shear in accordance with the Eq. 3 is vertically distributed in proportion to vertical mass and displacement profiles. Thus:

$$F_i = V_B \frac{m_i \Delta_i}{\sum_{i=1}^{n} (m_i \Delta_i)} \tag{4}$$

Where m_i, Δ_i are respectively related mass and design displacement at different storey's (i).

2.2 Determination of design displacement Δ_d

In many cases, the design displacement will be dictated by code drift limit (2~2.5 % of drift related to life safety performance level). However, generally maximum design drift θ_d, can be expressed as:

$$\theta_d = \theta_y + \theta_p \leq \theta_c \tag{5}$$

where $\theta_y, \theta_p, \theta_c$ are respectively inter-story yield drift, plastic drift and code proposed drift.

Priestly has proposed equation below to determine yield drift of steel frames, θ_y:

$$\theta_y = 0.6 \varepsilon_y l_b / h_b \tag{6}$$

where ε_y is the steel yield strain $(\varepsilon_y = F_y/E)$, l_b is the beam length, F_y is the steel yield strength, E Young's modulus and h_b is the beam height.

As Gupta & Krawinkler (2002) have proposed, yield drift of steel moment resisting structures can be expressed as:

$$\theta_y = (\theta_{yb} + \theta_{yc}) = \left[\frac{M_{pb} l_b}{6EI_b} + \frac{M_{pb} H}{6EI_c} \right] \tag{7}$$

where M_{pb} is the plastic moment of beam, H is the story height, I is the cross-sectional moment of inertia, subscript b denotes a beam, and subscript c denotes a column. Plastic moment of the beams is obtained from:

$$M_{pb} = Z_b F_y = 1.14 S_b F_y \leq \theta_c \tag{8}$$

where S_b is the section modulus. Substituting $I_b = S_b h_b / 2$, we get:

$$\theta_{yb} = \frac{1.14 F_y l_b}{3 E h_b} = 0.38 \frac{F_y l_b}{E h_b} \tag{9}$$

The relative contribution of column to story drift will vary over a range of values depending on the values of I_c/H and I_b/l_b. Assuming that the contribution of column is 40% that of the beam we get:

$$\theta_y = 0.532 \frac{F_y l_b}{E d_b} \tag{10}$$

Plastic drift can be determined as below:

$$\theta_p = (\phi_m - \phi_y) l_p . \left[\frac{l_c}{l_b} \right] \tag{11}$$

where ϕ_m is critical curvature and l_c and l_b are the clear beam length between column faces and the beam length from column center to center respectively.

ϕ_y is the yield curvature proposed by priestly (2003) for steel sections as:

$$\phi_y = 2.10\varepsilon_y / h_b \tag{12}$$

2.3 Determination of maximum displacement profile

Kravasilis et al. (2006) determined the maximum displacement profile of regular steel moment resisting frames using statistical analysis within elastic and inelastic ranges and in case of 3 column to beam capacity ratios (1.1, 1.3 and 1.5) and various story numbers.

$$\Delta_i = P_1 . \theta_d . h_i . (1 - P_2 . \frac{h_i}{H}) \tag{13}$$

The calculation of the parameters P_1 and P_2 is done with the aid of Table 1, as a function of the number of stories of the frame and the desired response range (elastic or inelastic). It should be noted that the three values of P_1 separated by a comma in the first column of the inelastic response case of Table 3 correspond to the three values of the joint capacity design factor α_{cd} (column to beam strength ratio), namely 1.1, 1.3 and 1.5.

Table 1. Values of the parameters of the proposed maximum displacement profile

Stories	Elastic response		Inelastic response	
	*P*1	*P*2	*P*1	*P*2
1	1.00	0.00	1.00	0.00
3	1.00	0.18	1.00	0.10
6	0.85	0.20	0.90	0.20
9	0.70	0.21	0.75, 0.80, 0.85	0.30
12	0.62	0.22	0.70, 0.75, 0.80	0.35
15	0.55	0.24	0.65, 0.70, 0.75	0.40
18	0.52	0.25	0.60, 0.65, 0.70	0.40
20	0.50	0.25	0.55, 0.60, 0.65	0.40

Maximum displacement profile of stories determined, design displacement Δ_d, effective mass m_e and effective height h_e of equivalent SDOF system are calculated as the following:

$$\Delta_d = \sum_{i=1}^{n} (m_i \Delta_i^2) / \sum_{i=1}^{n} (m_i \Delta_i) \tag{14}$$

$$m_e = \sum_{i=1}^{n} m_i \Delta_i / \Delta_d \tag{15}$$

$$h_e = \sum_{i=1}^{n} m_i \Delta_i h_i / \sum_{i=1}^{n} m_i \Delta_i \tag{16}$$

In addition, design ductility is:

$$\mu_s = \frac{\Delta_d}{\Delta_y} \tag{17}$$

Design displacement and design yield displacement can be determined as the following:

$$\Delta_y = 0.523 \varepsilon_y \left(\frac{l_b}{h_b} \right) (h_e) \tag{18}$$

2.4 Equivalent viscous damping

Estimation of equivalent viscous damping factor (EVDF) is an important step in the methodology of the DDBD. Blandon (2005), For Ramberg-Osgood model (efficient for steel structure) the modified equivalent viscous damping factor for using in DDBD method is as the following:

$$\xi_{eq} = \frac{125.25}{\pi} \left(1 - \frac{1}{\mu^{0.45}} \right) . \left(1 + \frac{1}{(T_e + 1)^4} \right) \tag{19}$$

2.5 Design displacement spectra

Since the structural period of the substitute structure is longer than that for the elastic structure (i.e. $T_e = \sqrt{\mu} T_i$, where T_i is the initial, elastic period), it is necessary for the displacement spectra to continue to longer periods than commonly plotted for acceleration spectra.

Displacement spectra for other than 5% damping have been determined using the European seismic code (EC8) modification factor of:

$$\Delta(T, \xi) = \Delta(T, 5) \left(\frac{7}{2 + \xi} \right)^{\frac{1}{2}} \tag{20}$$

2.6 Building analysis for design moment

In order to determine the design moments, the lateral force analysis of the structure should incorporate member stiffnes representative of conditions at maximum displacement response. This is an essential component of the substitute structure approach (Shibata & Sozen, 1976). With a weak-beam/strong column design, beam members will be subjected to inelastic actions, and the appropriate stiffness will be:

$$I_b = \frac{I_b}{\mu} \tag{21}$$

Since the columns will be protected against inelastic action by capacity design procedures, their stiffness should be I_c, with no reduction for ductility.

3. DESIGN DETAILS

In this study, four two-dimensional steel frames with 4, 8, 12 and 16 stories have been designed using displacement response spectrum of 2800 seismic code. All the models have three bays with bay length of 5m and story height of 3m. Reference design acceleration assumed to be 0.5g and related drift of life safety performance level is considered 2.5%. In addition, assumed steel yield strength is 2400 kg/cm^2.

4. VERIFICATION OF THE DESIGN METHOD

In order to evaluate the seismic response of the designed structures, a series of non-linear time-history analyses have been performed under different 2800 code spectrum compatible records. All non-linear analyses were carried out using fiber-element models developed in Seismostruct computer program. Six near-field and far-field records (Chichi, Duzge, Erzincan, Imperial vallev, Kobe, and Tabas) have been utilized in the time history analysis. All records were downloaded from http:\\www. peerberkeley.edu. (Peer strong ground motion database). We have tried to select records of the identical soil type letting shear wave velocity of soil be within ($375 \leq V_s \leq 750$ m/s^2) range. Rascal is used to match records based on the design spectrum, which uses random vibration theory and considers frequency content of records in addition to displacement domain to match selected records within certain levels.

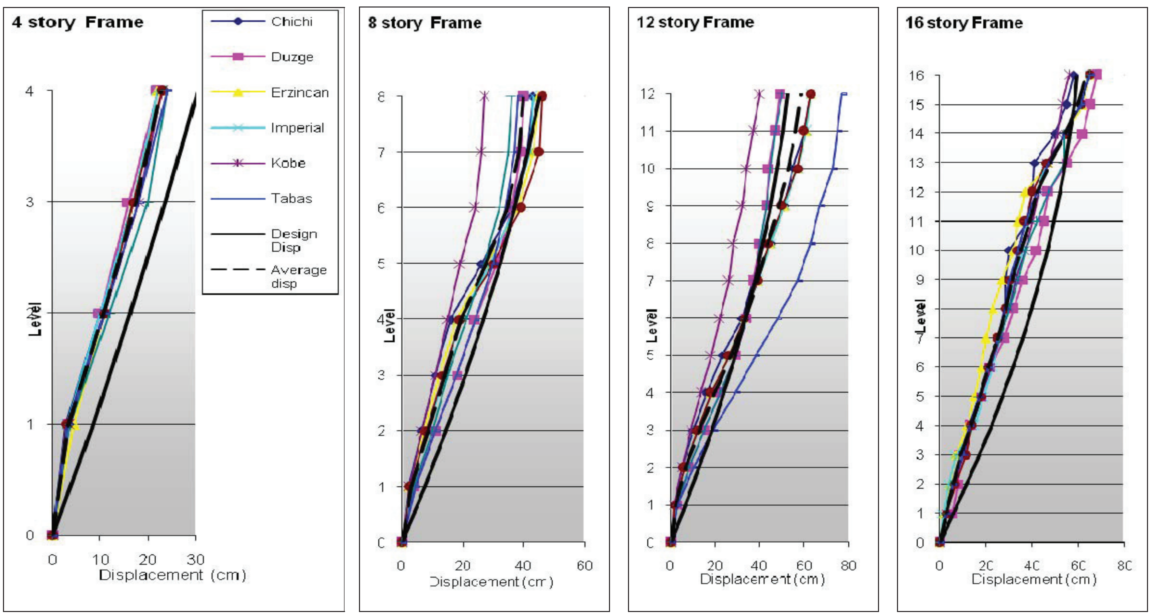

Figure 1. Absolute maximum story displacement.

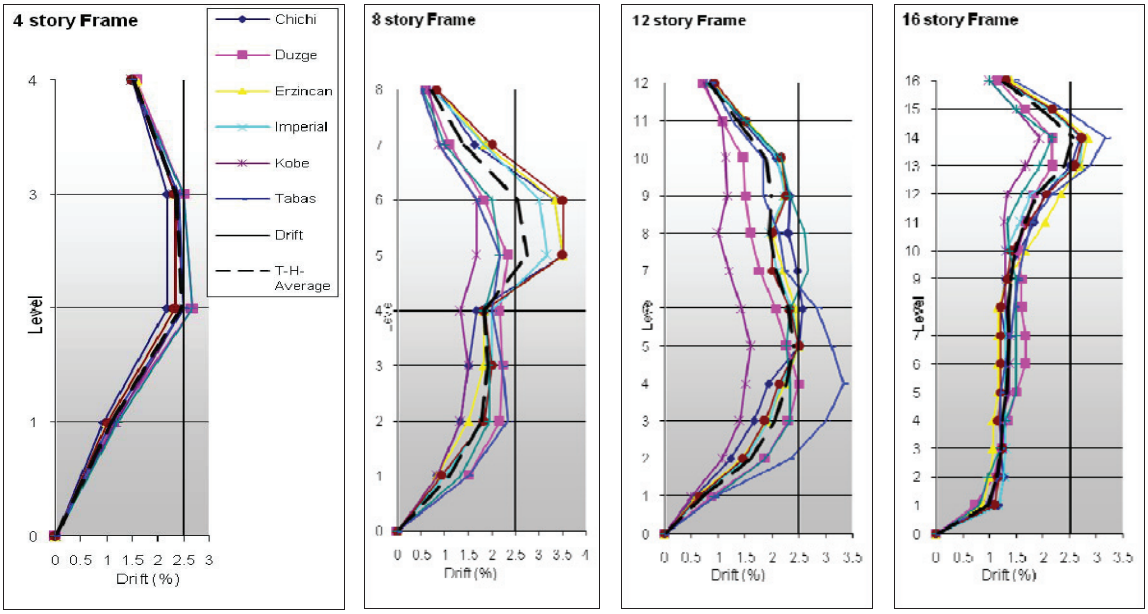

Figure 2. Maximum inter-story drift profiles.

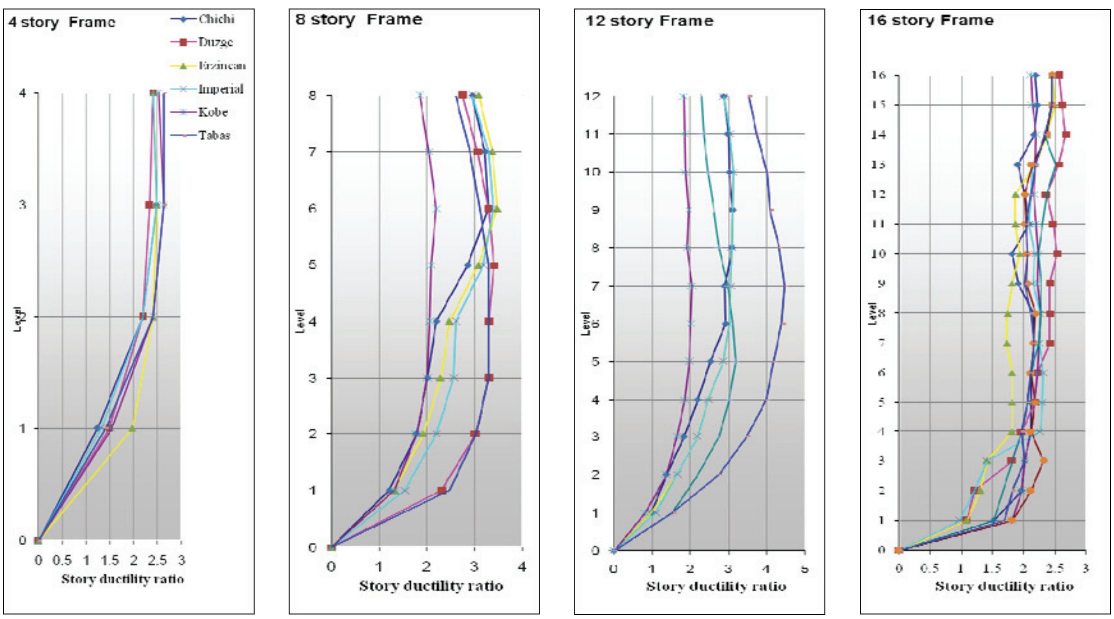

Figure 3. Story ductility demand profiles.

Absolute maximum story displacement, Inter-story drifts and story ductility demands were selected as the controlling parameters for performance assessment of the structures (Figures 1, 2 and 3). According to Figures 1, 2 and 3, in terms of absolute maximum story displacement, maximum inter-story drifts and story ductility demands performed quite satisfactorily. Many studies (e.g. Priestley & Krawinkler) have shown that inter-story drift has a key role in damage potential of structures. Story ductility demands were calculated as the ratio of maximum inelastic story displacement and story yield displacement. Story displacement ductility factors are shown in Figure 3. This figure shows that a high degree of similarity exists between inter-story drift profiles and ductility demand profiles. Such similarity shows a direct relationship between displacement and ductility demands of the structures. Maximum story ductility demands occur when inter-story drift is at its maximum point.

5. CONCLUSIONS

The present study focuses on seismic behavior of structures designed using a new performance-based design tool called the Direct Displacement-Based Besign. Performance verification studies show that the method can be regarded as an appropriate alternative to current erroneous force-based seismic design of structures. The method, in terms of absolute maximum story displacement, maximum inter-story drifts and story ductility demands performed quite satisfactorily, even for tall models. The DDBD methodology is able to design structures with quite controlled residual behavior.

References

[1] Gulkan, P. & Sozen M.A. 1974. Inelastic responses of reinforced concrete structures to earthquakes motions. *Proceedings of the* ACI 71 (12).

[2] Gupta, A. & Krawinkler, H. 2002. Relating the seismic drift demands of SMRFs to element deformation demands. *Engineering Journal* 39(2), 100-108.

[3] Priestley, M.J.N. & Kowalski, M.J. 2000. Direct displacement-based design of concrete buildings. *Bull. New Zealand Nat. Soc. Earthq. Engrg.* 33(4).

[4] Priestley, M.J.N. 2003. Myths and fallacies in earthquake engineering, revisited. *The Mallet Milne Lecture.* IUSS Press, Pavia, Italy.

[5] Kin, W. Lee . 1985. Rascal Program: A Computer Program for Generated Synthetices Time History and Computes Peak Acceleration, Peak Velocity Response Spectral.

[6] SeismoSoft. 2006. SeismoStruct- A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures", *Available online at http://www.seismosoft.com*