DIRECT DISPLACEMENT BASED DESIGN OF STEEL CONCENTRIC BRACED FRAME STRUCTURES

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ABSTRACT

The direct displacement based design (DDBD) procedure is well developed and used for designing reinforced concrete moment resisting frame structures, wall structures and bridges. However, there is limited number of studies available on designing steel concentric braced frame (CBF) structures using DDBD approach. Therefore, it is necessary to develop a DDBD procedure for CBF structures. On this regards, this paper proposes a DDBD procedure for steel CBF structures. The proposed procedure utilises yield displacement shape derived on the basis of tensile yielding of the braces, and equivalent viscous damping equation of the system proposed by Wijesundara *et al.* (2011) as a function of system ductility and non-dimensional slenderness ratio for steel CBF structures. Finally, the performance of four steel CBF structures designed according to the proposed DDBD procedure is studied using nonlinear dynamic response of the structures. The results show that the performance of CBF structures is in good agreement with the design considerations.

Key words**:** Direct displacement based design, Concentric braced frames, Ductility

1. INTRODUCTION

The direct displacement based design (DDBD) was first introduced by Priestley [5,6] and it has been subjected to a considerable research attention in Europe, New Zealand, and North America in the intervening years. The procedure is well developed for moment resisting frames, wall structures and bridges over the last decade. However, DDBD procedure for steel concentric braced frame (CBF) has not been developed fully and only very limited number of studies has been found in the literatures. Medhekar and Kennedy [4] have developed a displacement based design procedure for (CBF) structures. However, in that design procedure, the equivalent viscous damping (EVD) coefficient of the equivalent single degree of freedom (SDOF) system is taken as 5% of the critical damping. More recently, the DDBD procedure for CBF structures has been developed by Della Corte and Mazzolani [2], but in that procedure the reference is made to the Takeda-Thin EVD expression which was developed for reinforced concrete (RC) structures. Goggins and Sullivan [3] reviewed the apparent EVD of a number of CBF structures, with slender braces, subject to shake table testing. They found that the EVD coefficient should be less than that indicated by the Takeda-Thin model, and argued that there was a need for EVD expressions specific to CBF structures.

As consequences, this study describes a new DDBD procedure for steel CBF structures by using yield displacement profile derived using the tensile yielding of braces and the EVD coefficient equation developed by Wijesundara *et al.* [8]. The proposed procedure has been validated using the performance of two steel CBF structures, which were designed according to the proposed procedure. For the validation purpose, the average peak displacement profiles from nonlinear dynamic analyses (NDAs) are compared with the corresponding design profiles used in DDBD procedure.

2. DIRECT DISPLACEMENT BASED SEISMIC DESIGN

The fundamentals of DDBD have been presented in several publications [5,6]. The important features of DDBD are summarised in Figure 1. SDOF system representation of a frame building through the basic fundamentals applied to all structural types is shown in Figure 1(a), while Figure 1(b) shows the bi-linear lateral force-displacement curve of the multi degree of freedom (MDOF) system with the initial stiffness K_i and the post yield stiffness rK_i. According to the DDBD procedure, the structure is characterized by the secant stiffness K_{eff}, at the maximum design displacement Δ_d and a level of EVD coefficient ξ that combines the elastic and the hysteretic energy absorbed during the inelastic response, as shown in Figure 1(b) and (c). As shown in Figure 1(d), with knowledge of the design displacement and the EVD coefficient, the effective period T_{eff} required to limit the displacement to the design displacement can be read from the displacement response spectrum. The effective stiffness of the equivalent SDOF system at the maximum displacement can be then found by inverting the normal equation for the period of a SDOF oscillator and subsequently, the design base shear can be obtained.

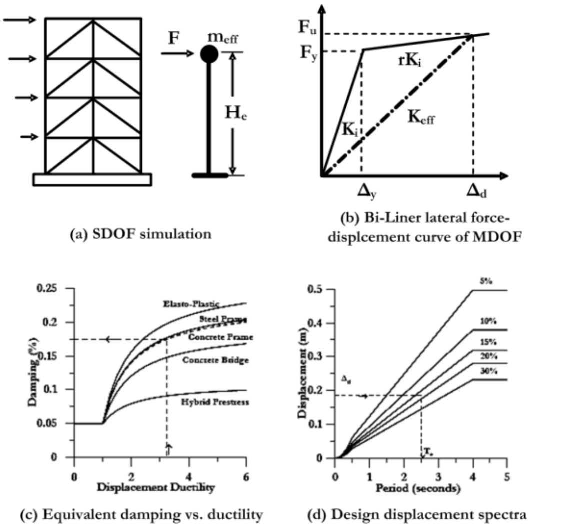


Figure 1: Fundamentals of Direct Displacement-Based Design (Priestley, 2007).

The yield displacement profile is developed in this study on the basis of following two assumptions: (1) buckling of the compression braces and yielding of the tension braces at all the storey levels occur simultaneously; and (2) the force-deformation curve is approximated to be bi-linear. Then, the lateral displacement at each storey level is basically induced due to *storey sway mechanism* resulting in the brace elongation in tension and shortening in compression and the *rigid body rotation* of the storey resulting in the axial deformations of the outer columns in the braced bay as shown in Figure 2 (a) and (b), respectively.

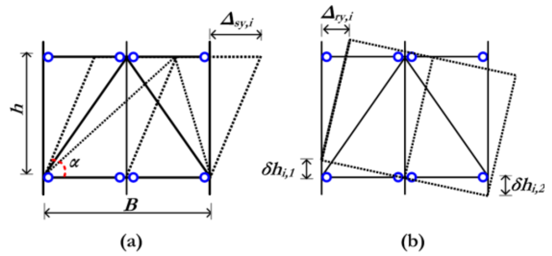


Figure 2: (a) Filtered velocity time history at 4th floor in E-W direction and (b) its fast Fourier transformation (FFT) plot.

Based on the deformed geometry shown in Figure 2(a), $\Delta_{sy,i}$ the lateral displacement induced by the sway mechanism at yielding of the i^{th} storey can be expressed as in Eq 1 neglecting the second order terms $(\Delta_{sy,i})^2$ and $(\varepsilon_y L_{ud,i})^2$.

$$\Delta_{sy,i} = \frac{2\varepsilon_y L_{ud,i}^2}{B} = \left(\frac{\varepsilon_y}{\sin\alpha\cos\alpha}\right)h_i \qquad (1)$$

where α is the angle of the brace to the horizontal line, B is the bay width, $L_{ud,i}$ is the undeformed and the brace at i^{th} storey, ε_y is the yield strain of the brace steel material and h_i is the storey height.

Tension and compression forces developed in outer columns in the braced bay, resulting in brace buckling in compression and yielding in tension, are significantly different to each other for the intermediate and slender braces. However, the gravity loads diminish the difference by decreasing the tension force and conversely increase the compression force. As consequence of that, it is reasonable to assume that the axial elongation and shorting of the outer column in tension and compression, respectively are approximately equal. Thus, $\Delta_{ry,i}$ the lateral displacement induced by the rigid rotation at yielding of the i^{th} storey can be expressed in the following form in Eq. 2.

$$\Delta_{ry,i} = \left(\frac{2\delta h_i}{B}\right)h_i = \left(\beta\varepsilon_{yc}h_i\right)\tan\alpha \qquad (2)$$

where ε_{yc} is the yield strain of the column steel material and β is the ratio of the design axial force to the yielding force of the column section at i^{th} storey. Finally, the total interstorey yield displacement at the i^{th} storey Δ_{yi} is:

$$\Delta_{y,i} = \left(\frac{\varepsilon_y}{\sin\alpha\cos\alpha}\right)h_i + \left(\beta\varepsilon_{yc}h_i\right)\tan\alpha \qquad (3)$$

The design displacement profile proposed by Priestley *et al.* [6] referring the first inelastic mode shape of RC moment resisting frame structures is used in this study as the design displacement profile for CBFs as well. The design displacement profile is obtained from a normalised inelastic mode shape δ_i, and the displacement of the lowest floor Δ_1 as given in Eqs. 4,5 and 6:

$$\Delta_i = \delta_i\left(\frac{\Delta_1}{\delta_1}\right) \qquad (4)$$

$$\delta_i = \frac{H_i}{H_n} \qquad n \le 4 \qquad (5)$$

$$\delta_i = \frac{4}{3}\left(\frac{H_i}{H_n}\right)\left(1 - \frac{H_i}{4H_n}\right) \qquad n \ge 4 \qquad (6)$$

where the normalised inelastic mode shape depends on the height H_i, and roof height H_n.
All the steps in the DDBD procedure for steel CBF structures are summarized in the flow chart shown in Figure 3

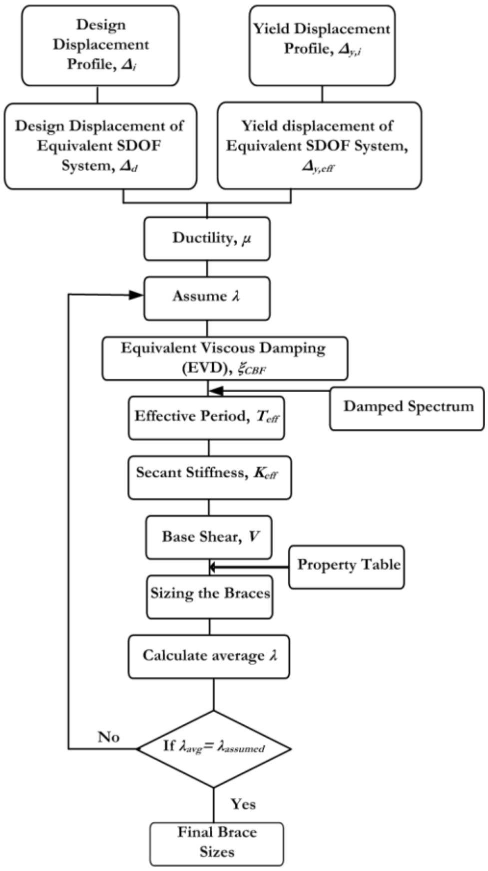


Figure 3: (a) Flow chart on DDBD procedure of CBF structures

3. NONLINEAR DYNAMIC ANALYSES

Two CBF structures are four and eight storeys with inverted-V bracing configuration and continuous middle column that links the brace-to-beam intersection points at each floor level directly to the foundation. The height of each storey is 3.5*m* and the bay width of each of braced and unbraced bays is 7*m*. All the frames are designed for the ground motion which has the probability of exceedance equal to 10% in 50 years (i.e., return period of 475 years) with the peak ground acceleration of 0.3*g*. 5% damped displacement spectrum as defined in EC8 [1] is used for the design with the corner period (T_c) of 4*s*. In order to investigate the performance of the buildings designed according to the DDBD procedure, nonlinear dynamic analyses are performed using the OpenSEES finite element computer program. The steel CBFs are modelled in 3-D rather than in 2-D to permit the braces to buckle in the out-of-plane direction of the frame since all the braces are designed and detailed to develop the out-of-plane buckling. The behaviour of all the frame elements except the braces is limited to in-plane displacement by restraining the translational degree of freedom in the perpendicular direction to the plane of the frame and the rotational degrees of freedom in the out-plane directions. The column-to-base and the beam-to-column connections are modelled as pinned connections while the columns are modelled as continuous members. All the braces are modelled using the inelastic beam-column brace model proposed by Uriz [7]. All the columns and beams are also modelled using nonlinear beam-column elements available in OpenSEES frame work. The corotational theory was used to represent the moderate to large deformation effects.

Seven real accelerograms are selected from PEER data base in order to carry out the nonlinear dynamic analyses and scaled to match with the design displacement spectrum. Figure 4 shows the 5% damped displacement spectra of the individual accelerogram and average of the individual accelerogram together with the design displacement spectrum, in the period range of 0 to 4s. The average displacement spectrum of the individual accelerogram is matched well with the design displacement spectrum.

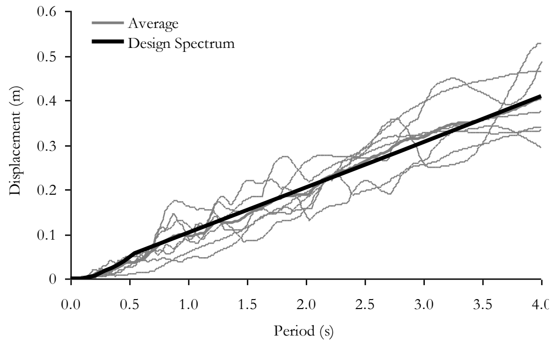
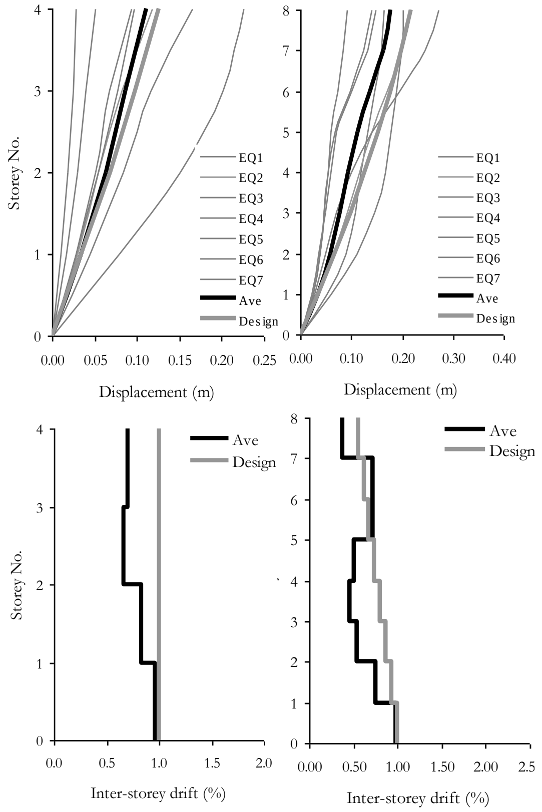


Figure 4: Displacement spectra from the scaled natural accelerogram at 5% damping

4. RESULTS AND DISCUSSION

The average profiles of peak inter-storey drift ratios of the two buildings resulting from NDA are compared with the corresponding design profiles.
Figure 5(a) and (b) illustrate the average peak displacement and drift ratio profiles for 4 and 8 storeys CBFs with IVMC configuration, respectively. It is clear that the resultant average displacement profile of 4 storey frame is almost linear and well matched to the design displacement profile. The average drift ratio is 4% below the design drift ratio at the 1st storey while 30% below at the top storey.

In the case of 8 storey frame, the average displacement profile is fairly matched with the design displacement profiles ensuring that average displacements do not exceed the design displacements corresponding to the presumed displacement shape significantly as shown in Figure 5(b). The average drifts at storey levels 5, 6 and 7 are slightly higher than the design drifts, and the maximum of 16% higher drift is observed at 7th storey level.



(a) (b)

Figure 5: Average peak displacement and drift ratio profiles of (a) 4 storey (b) 8 storey CBFs

5. CONCLUSION

In this study, a DDBD procedure is developed to design steel CBF structures. The yield displacement profile is evaluated on the basis of tensile yielding of braces. The procedure uses assumed first mode displacement shape proposed by Preistley *et al.* [6] for MRFs as the design displacement profile with the EVD equation proposed by Wijesundara *et al.* [8]. The proposed procedure is validated using NDA results of 4 and 8 steel CBFs.

The results of NDA prove that presumed linear displacement shape proposed by Preistley *et al.* [6] for low-rise MRFs is reasonably valid for the low-rise CBF structures.

Furthermore, the NDA results of the medium-rise CBF structures also prove that the presumed inelastic first mode displacement shape with higher drift concentrations at lower storeys is a reasonably good estimation for the displacement shape for medium-rise CBF structures.

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