Seismic Design Methods for Steel Concentric Braced Frame: State-of-the-art Review

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Abstract

Due the unexpected failures during past earthquakes, the seismic design methods for civil structure evolved significantly in the last century. Especially the introduction of performance based design methods enhance the force-based design and also lead to the development of displacement based design concept in structural engineering. On this regards, this paper critically reviews the current design methods for steel concentric braced frame (CBF) structures. The review process focuses on variation in the design methods and assumptions made in each methods. Further, the comparison of performance of the structures design based on each methods are discussed.

Keywords: seismic design, performance method, direct displacement based design, concentric braced frames

1. Introduction

In the last few decades seismic structural analysis and design has been given a special attention through research and governmental policies more than other load cases. At the beginning of 20th century, the analysis methods were simulating the seismic attack by terms of lateral forces proportion to mass (inertia forces), which are resisted by elastic forces that produced by structural elements (structural stiffness). In the 1940's and 1950's the effect of the natural frequency on the inertia forces was included in the analysis. Yet the structural analysis was still depending on the elastic behaviour. Twenty years later in 60's and 70's experimental studies have revealed that the well detailed structures could resist higher levels of earthquake intensities more than that are predicted by elastic response because of ductility characteristic (Priestley et al., 2007). More recently, force reduction factors for elastic values has been considered in the structural design as assessment of predicted performance. In 1980s and 1990's it was recognised that structural vulnerability would be linked to structural performance, and not strength which only helps to reduce displacements or strains (Priestley et al., 2007). Therefore, in 1993 the Direct Displacement Based Design (DDBD) was established by Priestley as the first design concept which adopts the structural performance philosophy, defined by strain or drift limits under a specified seismic intensity level. He suggested the latter method as a viable alternative to the strength-based design or acceleration spectra. Meanwhile, he presented many myths and fallacies in aspects of seismic analysis and design which were adopted by the current international codes that lead to conflict between the idealization and reality (Priestley, 1993). Then the Direct Based Displacement design method has been followed with intensive coordinated research efforts especially in Europe, New Zealand, and North America (Wijesundara and Rajeev, 2012).

2. Development and application of DDBD on Steel Structures

There are many studies aimed to improve and develop the design phase assumptions either by suggesting new conceptions or providing specific parameters of the new concepts. The current section reviews the main studies that could improve and develop DDBD aspects and assumptions to design CBFs.

2.1 First application of DDBD on steel structures

Medhekar and Kennedy (2000a) firstly applied the Direct Displacement Based Design (DDBD) approach to design steel concentrically braced frames. In this study the proposed procedure of DDBD by Medhekar and Kennedy (2000b) has been used to design two and eight steel frames as shown in figure (1) and figure (2) respectively.

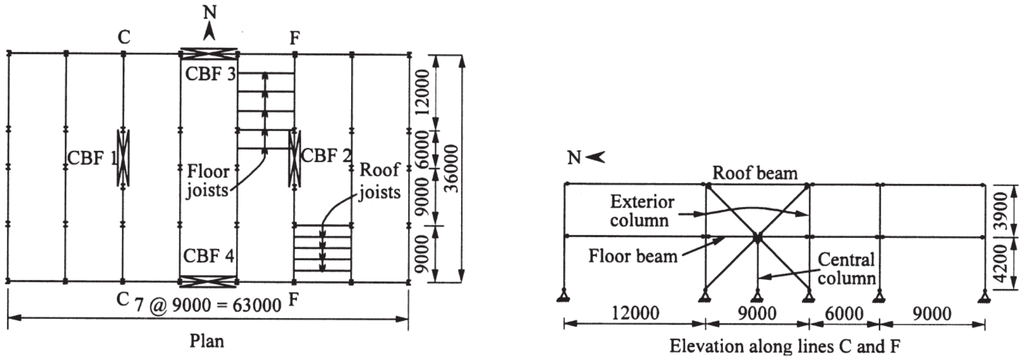


Figure 1 Symmetric layout of two storey building

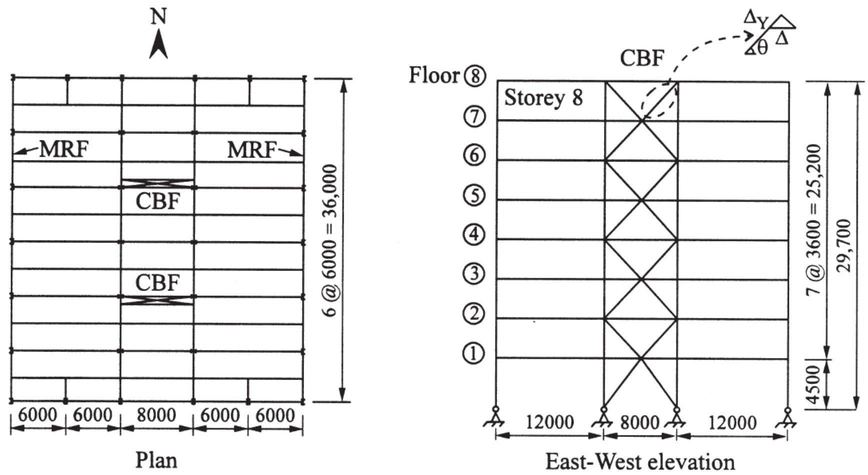


Figure 2 Symmetric layout of eight-storey building

For the two-storey building, 5% damped mean plus one standard deviation of twenty suitable earthquake records were used to generate the displacement response spectrum. The generated DRS is compared with the DRS specified by National Building Code of Canada (NBCC) in Figure 3. In current study the Comparison reveals good agreement between the calculated $\mu + 1\sigma$ DRS of the twenty accelerograms and DRS of NBCC. This due to the fact that the chosen records were suitable for the assumed regional area. Therefore, the DRS of NBCC is used for the design of the two storeys building and scaled the results of dynamic non-linear time history analyses to the $\mu + 1\sigma$ for evaluation when compared with the DRS of NBCC.

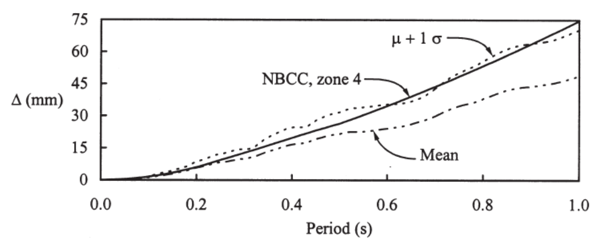


Figure 3 Displacement response spectra for two storeys building

On the other hand, for the eight storeys building two sets of accelerogram records are utilized. The first is an acceleration set consisting of twelve records with peak acceleration that is scaled to the horizontal peak acceleration. While another set consisting of velocity records which consists of seven accelerograms with peak velocity of ground modified to the horizontal peak of velocity. The mean plus one standard deviation ($\mu + 1\sigma$) of displacement response spectra (DRS) of the two sets of records are computed individually. The calculated $\mu + 1\sigma$ DRS of the two sets are compared with elastic DRS obtained from national building code of Canada (NBCC). Then the average value of the computed $\mu + 1\sigma$ DRS of the two sets has been used in design. Thus, in this study the response of non-linear time history analyses is assessed by comparing the RS of NBCC with $\mu + 1\sigma$ of the calculated responses.

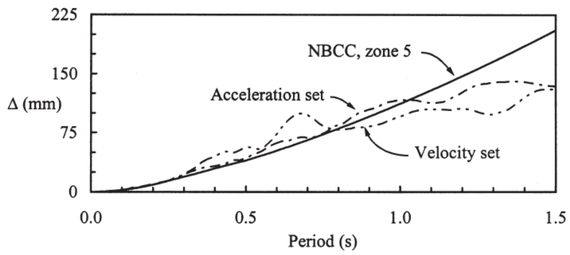


Figure 4 Displacement response spectra for eight storeys building

In designing process of the case study the yield displacement profile is based on the braces yielding only whilst other members remain elastic (Medhekar and Kennedy, 2000b). The two buildings a two and an eight storeys are investigated and many cases has been investigated in terms of elastic and inelastic responses with ductility demand of the values 1 and 2 respectively, uniform and non-uniform ductility demand over the height of the building, asymmetric of building layout (torsion effect) Figure 5 , column deformation, p–δ effects on the lateral displacement profile and how to reduce the ductility demand in the upper storey levels.

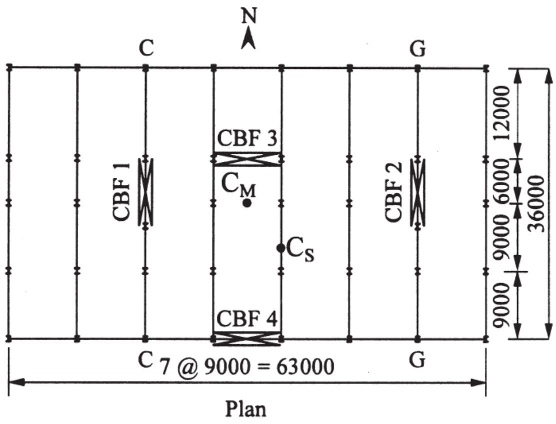


Figure 5 Asymmetric layout of a two storeys building

2.2 Developing a procedure of DDBD for steel structures

Della Corte and Mazzolani (2008) have developed procedures of the direct displacement-based design (DDBD) method and discussed many concepts of performance for steel braced frames as shown in Figure 6 and Figure 7. Their study depended on the basic concept of DBD that yield deformation is independent from structural members' strength. Hence the yield deformation can be computed earlier in the beginning of the design. Actually, the yield displacements of the whole structure can be calculated independently from cross section properties by using suitable sub-structuring techniques with appropriate limit state (Figure 6 and Figure 7).

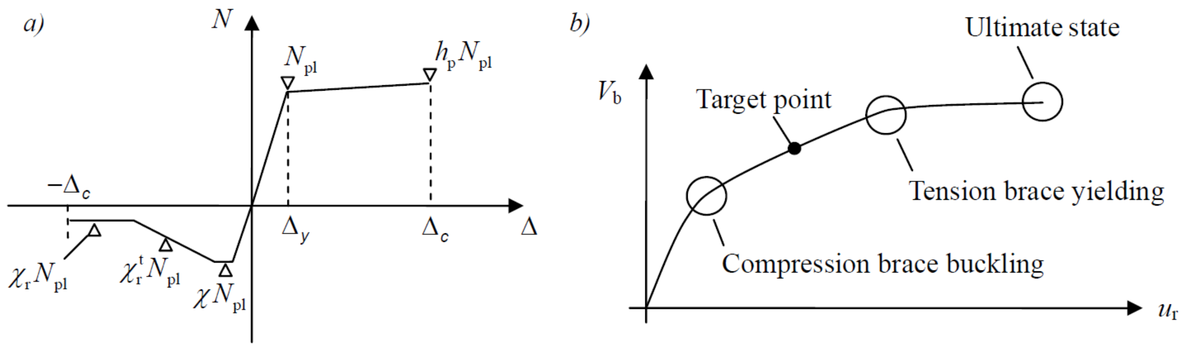


Figure 6 a) Schematic response of braces. b) A typical pushover response of a structure with slender braces

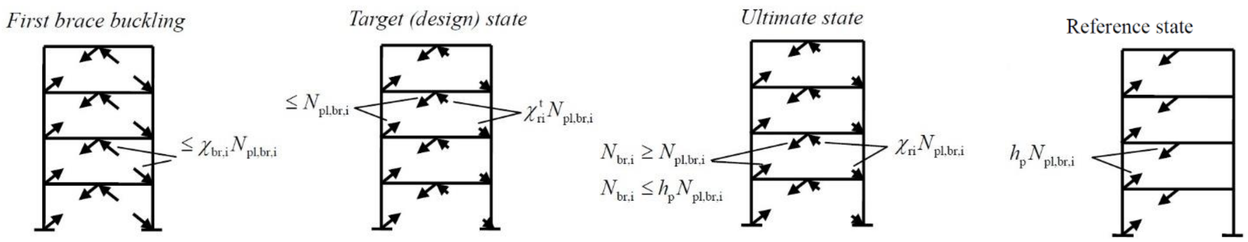


Figure 7 Buckling, target, ultimate and reference state in terms of brace forces

Where:
N_{pl} Axial yield force.
h_p Ultimate tension strength factor
x Compression strength reduction factor.
x_r , x_r^t Residual compression strength factors.

In this regard this study highlighted four limit states as illustrated in Figure 7 : first when the braces start to buckle (buckling state). Second the loads in braces reach the yield in tension while the compression load in braces is between buckling and residual strength (target or design state). Next the ultimate limit state is when the load in braces equal or less the ultimate load and in compression equal residual strength. Finally, the reference limit state which is the load in braces reaches the tension ultimate state and neglecting the residual strength of the compression braces.

The proposed procedure of this study can be summarized with: First the deformations at the buckling state were computed. Second deformations at the target or design state were computed. Then the base shear is calculated within target design state using the proposed viscous damping by Priestley et al. (2007). Revise the calculated base shear from the target state to the buckling state by using the proposed relationship in this study eq. (1) which connects between the base shear at design state and at the buckling state. Then design the braces according to the revised base shear. Finally, beams and columns are designed according to the reference state.

$$\frac{V_{b,d}}{V_{b,y}} = \frac{(\bar{\varepsilon} + \chi_r) N_{pl,br,1} \cos \alpha_1}{\frac{1}{\Omega_1} N_{pl,br,1} \cos \alpha_1} = (\bar{\varepsilon} + \chi_r)\Omega_1 \tag{1}$$

Where,
$V_{b,d}$ Base shear at design state.
$V_{b,y}$ Base shear at buckling state.

$\bar{\varepsilon}$ Design parameter governing the distance of the target point from the ultimate state (Figure 7). $\Omega_1 = \frac{N_{pl,br,1}}{2N_{br,1,y}} = \frac{\varepsilon_y}{2\varepsilon_{br,1,y}} = \frac{1}{2\chi_{br,1}}$

ε_y Axial yield strain

$\varepsilon_{br,1,y}$ The 1st-storey axial buckling strain ($\varepsilon_{br,y} = x_{br}\varepsilon_y$).

x_{br} Brace compression strength reduction factor.

$N_{br,1,y}$ The 1st-storey brace axial force at buckling.

The suitable sub-structuring technique also depends on the state of structure. For instance, pre-buckling drift is a function of the storey rigid rotation plus the braces deformations (Figure 8). On the other hand, the drift of the post buckling is a function of the storey rigid rotation plus beam and tension brace deformations (Figure 9).

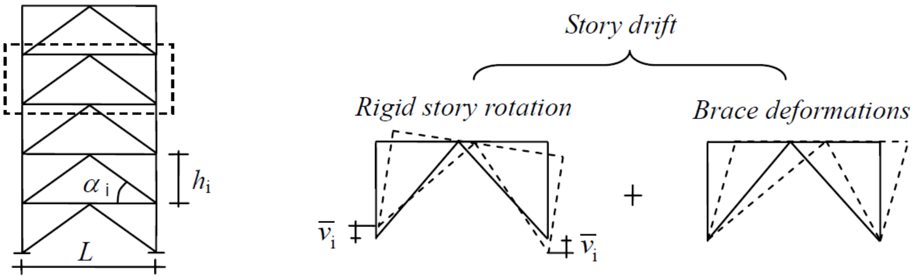


Figure 8 Pre-buckling displacements

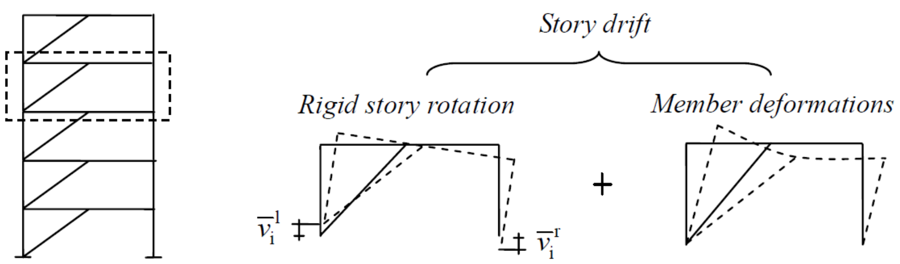


Figure 9 Post-buckling displacements

Basically, the current research developed procedure incorporates the braces and columns slenderness in the yield displacement profile of CBFs. In fact, it depends on the normalized slenderness of columns and braces. The slenderness was incorporated in such a way to ensure that buckling in the braces starts from the top storey and downward taking into account the initial strain in the braces resulted from gravity loads. At the time that brace buckles the column will reach to a fraction of its buckling load to avoid column buckling at the ultimate state. The fraction factor of the column buckling load depends on the axial force resulted from gravity loads which is unavailable in this stage. Therefore, the proposed procedure implies iterative computations. Beams and columns are designed according to loads of the reference state mentioned above (Figure 7).
The case study of this research is a ten-storey one-bay steel frame with inverted v-bracing has been designed by the developed assumptions of this study. V-bracing configuration is incorporated as the case study to deal with some problems could be occurred when apply the new methodology such as the unbalanced force imposed on the beam encountered after brace buckling in compression and brace yielding in tension. Thus it is essential to incorporate beam deformation in the building drift. EC8 design spectrum has been adopted in this study with the parameters: type 1, site condition type C and the peak ground acceleration (PGA) is 0.35g. For the steel sections European S 275 steel has been used with yield stress of 316 MPa. Wide flange of European I section has been used to design beams and columns, and circular hollow sections for braces. Subsequently, non-linear time history analyses

are performed to verify and assess the performance of the designed frame by the proposed procedure. Seven acceleration records are carefully selected to maintain the average displacement response spectrum closer to the design spectrum. The comparisons between numerical analyses and the analytical design phase predictions have been performed in terms of the first mode shape response in the pre-buckling and in the post buckling states, maximum displacement response and the ductility demand in compression and tension.

2.3 Validation of DDBD assumptions for steel structures

Goggins et al. (2009) investigated the validation of DDBD aspects for CBFs frames that are subjected to earthquakes. This work was part of ongoing investigation studies into the behaviour of CBFs. In this study, the design values are compared with experimental results which are obtained from the shaking table tests (Figure 10) and the numerical data that are provided from non-linear push over and non-linear time history analyses. Basically, El-Centro accelerogram has been used for excitation of eight single storey concentrically X-braced frames.

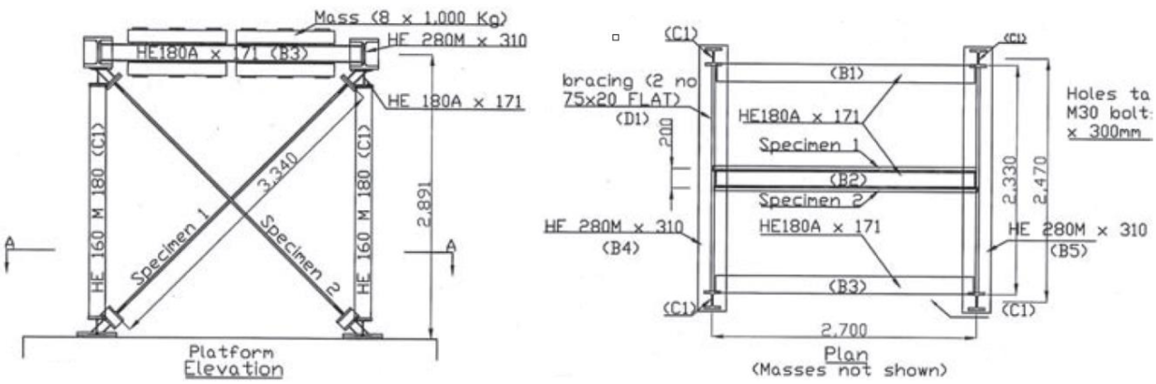


Figure 10 Shake table test model and set up

From experimental work dynamic responses and properties are predicted and recorded per each frame. In the trial design procedure, the strength of the tension brace only has been incorporated in the target displacement of DDBD, as it is claimed that it represents a good prediction of the total strength and stiffness of the CBFs. This also coincides with the recommendations of Eurocode8 (EC8) (2005). The energy dissipation which is represented by an equivalent viscous damping is reviewed in this study through referring to several models in the literature which were compared with the values that are computed from hysteretic loops which are recorded from the shake table tests. According to the comparison results in this study flag-shaped hysteretic model by Priestley et al. (2007) has been adopted to use in the DDBD procedure. P-Δ effects are incorporated in the trial design procedure of DDBD of the present study. For the numerical analyses verification, a two-dimensional model of the tested frames has been simulated by using ADAPTIC programme (Izzuddin, 1991) which utilizes geometric and material non-linearity. The inelastic tensile and slenderness of the braces have been included. The models are analyzed by non-linear pushover and time-history analyses.

Further, Goggins et al. (2009) examined the equivalent viscous damping (EVD) Coefficient for different CBF structures whose slender braces by using a shaking table.

2.4 Proposing a new damping expressions for CBFs

More Recently, Wijesundara et al. (2011) developed a new damping expressions for concentrically braced frame structures utilized the results of non-linear time history analyses of SDOF systems that

calibrates the area based EVD with the displacement convergence 5% based on the ductility and the non-dimensional slenderness ratio. Basically, the current study used fifteen pre-designed CBFs single storey in order to develop EVD for designated different ductility. Ten of the studied frames are decoupled diagonal bracing frames and the other frames are X bracing which is fully restrained at the crossing point of the braces (Figure 11).
The frames are 4m height and 7m bay width. Square hollow sections (SHS) are used for all braces. In this study high range of different section sizes are used so that the parametric study will cover a wide range of slenderness ratio. According to Eurocode3 (EC3) (2005) all braces sections which are used in this study are compact sections. The main design consideration that it is taken into account, the beams and columns behave elastically, meanwhile the braces are designed to dissipate energy through inelastic buckling and yielding in compression and tension respectively. The gravity load is neglected in this study. The out-of-plane-buckling of the braces is taken into consideration with initial camber L/360 out of the plane in the middle node of the bracing. The end of the brace connection with the gusset plate is also modelled to permit the out-of-plane-buckling by providing a free space equal two times the thickness of the guest plate ((American Institute of Steel Constructions (AISC), 2005) and (Canadian Standard Association (CSA), 2001)) (Figure 11 and Figure 12).

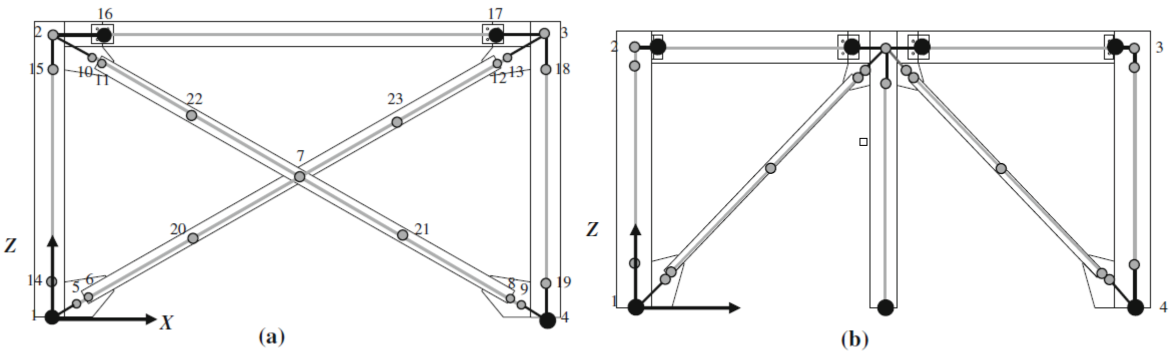


Figure 11 a) X brace model b) Decoupled diagonal brace

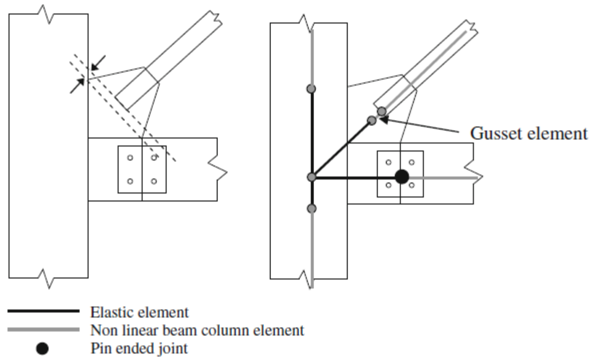


Figure 12 Connection detail and model

In fact the out-of-plane-buckling is detailed to share with energy dissipation through the inelastic rotation of the guest plate. The connections of beam-column and column-base are bolts connections. The brace effective length KL is calculated in the plane of buckling, L is the length of the bracing between plastic hinges which are developed in the gusset plate, considering the restraint condition of the gusset plate. The wide range of the non-dimensional slenderness ratio of the braces is considered between 0.44 and 1.6 to cover the most range specified in the Eurocode8 (EC8) (2005). Actually, because of the compression brace buckling, there is some energy dissipation could be expected when the ductility less than one. All frames are modelled by using OpenSees Programme (2006). The Corotational theory was utilized to represent the large and moderate deformation effects of the brace inelastic buckling (McKenna et al., 2014). On the other hand, small deformation theory has been used for the local stresses and strains of the inelastic beam and column elements. Menegotto-Pinto model was used to represent the steel material response of the braces with the assumed hardening ratio 0.8% (OpenSees Programme, 2006). It is worth to mention that in the present study the top lateral displacement was calculated depending on elongation and shortening of the braces without considering the rigid rotation which is induced by shortening and elongation of the columns. In the same way the base shear was computed from the horizontal components of the axial forces of the braces in tension and compression. Then the hysteretic response of the base shear and the top lateral displacement is obtained for all frames by applying a symmetric lateral displacement history (quasi-static cyclic displacement) at the top to each frame. Six different values of top lateral displacement: 20, 30, 40, 50, 60 and 70 mm are used. Consequently, six ductility values are also considered to compute the hysteretic EVD by using the area based method which is mainly attributed to Jacobsen (1960) (Figure 13 and equation 2).

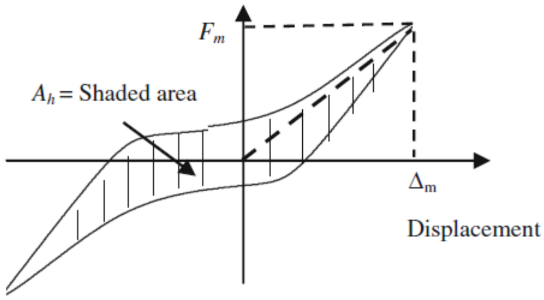


Figure 13 Hysteretic area for damping calculation

$$\xi_{hys} = \frac{A_h}{2\pi F_m \Delta_m} \tag{2}$$

Where,
A_h is the area of a complete cycle of force-displacement response.
F_m and Δ_m are the maximum force and the displacement occurred in the complete cycle respectively.

The non-linear time history analyses of a set of recorded fourteen actual accelerograms are utilized to calibrate or correct the area based hysteretic EVD expression by using the OpenSees programme. The effective period for the selected displacement per each frame is determined by the 5% damped effective spectrum which is computed by an almost linear average of displacement spectrum of the fourteen natural (without scaling) accelerograms. All tested periods within the range of 0.5-3.5s. To make the results of the current study more reliable the developed 5% average damped spectrum of displacement is compared with the design displacement spectrum of Eurocode8 (EC8) (2005) in the period range from 0s to 4s (Figure 14).

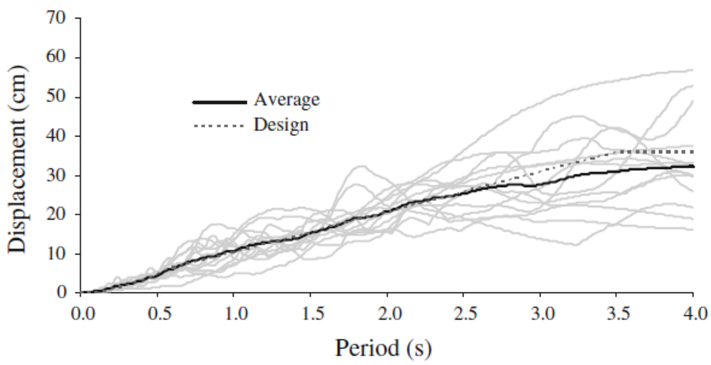


Figure 14 Design, average and fourteen displacement spectra at 5% damping level

The above figure obviously shows well agreement in the range of 0-2.5s and maximum deviation is not more than 10%. Moreover, the damping correction factor of equation 3 suggested by Priestley et al. (2007) was examined in the present study by comparing the average displacement spectra for different damping ratios with Eurocode8 (EC8) (2005) design spectra. The comparison was reasonably matched for the used records (Figure 15). The expression of equation 3 is utilized in the DDBD approach to find different damped spectra. However, the damping correction coefficient is not required in the current study because it is computed directly through the correction process of the EVD.

$$\eta = \sqrt{\frac{7}{2+\xi}} \geq 0.55 \tag{3}$$

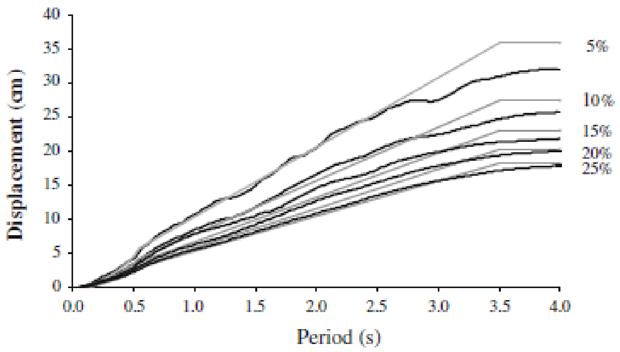


Figure 15 Comparison of average and design displacement spectra at different damping levels

One of the strengths of this study also is that it represents a comprehensive examination of the whole numerical calibration procedure by comparing analytical predictions with the experimental findings by Archambault et al. (1995) for twelve different frames. The comparison has been pursued to validate the numerical hysteretic response in terms of base shear-top displacement. It showed that the whole response is predicted well. This could emphasize that the use of L/350 as initial comber is suitable even though it is higher than the suggestions of the current codes (Figure 16).

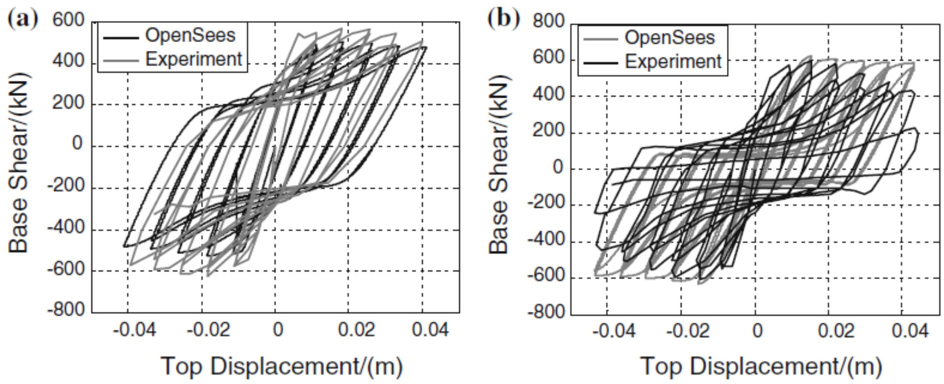


Figure 16 Comparison of experimental hysteretic responses and numerical results for two specimens

2.5 Suggesting a new procedure of DDBD for CBFs

Wijesundara and Rajeev (2012) have developed a procedure for CBFs design. This procedure included a developed yield displacement profile which depends on two assumptions: all the braces of the frame are buckled and yielded at the same time and the material behaviour of braces are assumed bi-linear. Basically, the lateral displacement of each storey is induced by the braces elongation and shortening in tension and compression respectively, in addition to the axial deformation of the outer columns of the braced bay due to the rigid rotation as illustrated in Figure 17 and equations 4 and 5.

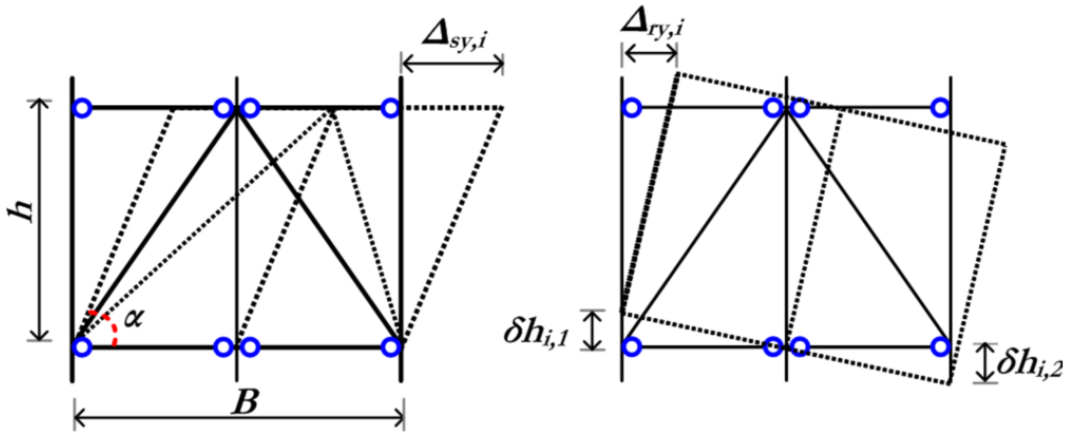


Figure 17 (a) Sway Mechanism Due to Yielding of Tension Brace, (b) Rigid Rotation of Floor Due to Outer Columns Deformation.

$$\Delta_{yi} = \Delta_{syi} + \Delta_{ryi} \tag{4}$$

$$\Delta_{yi} = \left(\frac{\varepsilon_y}{\sin \propto \cos \propto}\right) h_i + \left(\beta \; \varepsilon_{yc} \; h_i\right) \tan \propto \tag{5}$$

Where:

$\delta h_{i,1}$ Tension deformation of outer column of the braced bay due to floor rigid rotation.
$\delta h_{i,2}$ Compression deformation of outer column of the braced bay due to floor rigid rotation.
Δ_{yi} Lateral yield displacement at the i^{th} storey due to yielding of tension brace and storey rigid rotation.
Δ_{syi} Horizontal displacement due to yielding of tension brace at the i^{th} floor.
Δ_{syi} Horizontal displacement due to tension and compression deformation (rigid rotation) of outer columns of the braced bay at the i^{th} storey accompanying yielding of tension brace.
ε_y Yield strain of braces steel material.
α Angle between brace and horizontal line.
h_i Storey height.
β Ratio of design axial force to yielding force of column section at the i^{th} floor.
ε_{yc} Yield strain of column steel material.

The suggested procedure in this study also utilized the proposed EVD expressions by Wijesundara et al. (2011) (equations 11 and 12). And the slenderness ratio of the SDOF system is assumed to be equal to the average slenderness ratios of the braces in MDOF system. The developed procedure has been used to design four concentric steel frames: Two frames are four and eight storeys height with inverted V bracing configuration (Figure 18a). Other two frames are also four and eight storey height but with X bracing configuration (Figure 18b).

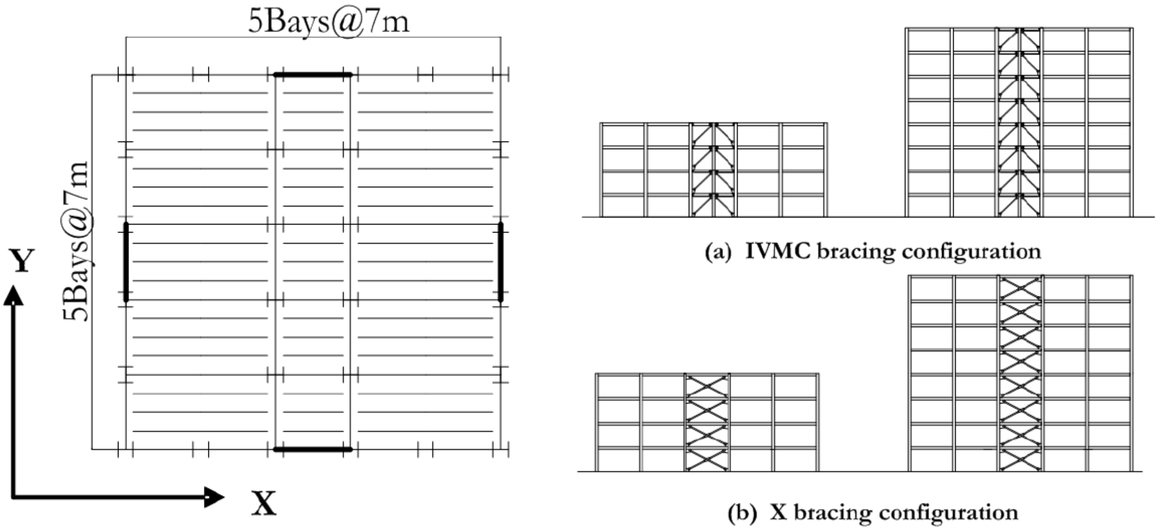


Figure 18 Plan view and elevation of CBF structures with (a) IVMC (b) X bracing configuration

The main assumptions that are used in this study are: exceedance probability 10% in 50 years, the peak ground acceleration 0.3g and 5% damped displacement spectrum according to Eurocode8 (EC8) (2005). The corner period (Tc) is 4s. The Priestley et al. (2007) damping scaling factor is also used (equation 3). It is assumed that the buildings are located on site of 5.5 or above earthquake magnitude with class A as defined in Eurocode8 (EC8) (2005) and the importance factor is one. It is worth to note that some assumptions are made in this study such as the accidental torsion and the stiffness of non-structural elements are neglected. Further, the out of plane buckling is allowed for the braces by providing two times the gusset plate thickness as free space at each end of the braces (Figure 12). The braces are designed according to earthquake loading only with cold formed HSS shapes. The strain hardening effect and overstrength are ignored in estimating the tension strength of the braces which are considered more conservative. Wijesundara et al. (2009) proposal relationship is adopted in this study which reduces the compressive strength as a function of the slenderness and ductility (equations 6-10).

$$V_i \leq V_R = \left(N_{ut}^i + N_{uc}^i\right)\cos\alpha \tag{6}$$

$$N_{ut} = A_g f_y \tag{7}$$

$$N_{uc} = A_g F_y \lfloor a\,(2\mu - 1)^b \rfloor \qquad \leq N_b \tag{8}$$

$$a = 0.369\lambda^{-1.819} \tag{9}$$

$$b = 0.390\lambda - 0.805 \tag{10}$$

Where:

V_i	Storey design shear.
V_R	Horizontal resultant components of ultimate tension and maximum compression forces of the braces.
N_{ut}	Ultimate tensile force of brace.
N_{uc}	Ultimate compressive force of brace (Wijesundara et al, 2009).
A_g	Brace gross section area.
f_y	Nominal yield tensile stress.
μ	Design ductility.
N_b	Initial buckling load.
a & b	slenderness ratio parameters.

It is important to note that the pre-designed braces are within the slenderness range and the limited width to thickness ratio that are specified in Eurocode3 (EC3) (2005) and Eurocode8 (EC8) (2005) for better energy dissipation. Beams and columns are designed elastically under the load combination of the gravity loads and the inelastic response of the braces. Moreover, it is assumed for the calculated forces of the beams and columns that buckling and yielding in the braces happen simultaneously.
To verify the proposed procedure of DDBD in the current study non-linear dynamic analyses are performed to investigate the designed frames performance by using the OpenSees programme. Then 3D numerical modelling of the CBFs has been done to take into account the out-of-plane buckling of the braces. However, the behaviour of the whole frame except the braces is taken in 2D. The columns are modelled as continuous members with column-base and column-beam connections are pinned connections. The inelastic beam column model of Uriz (2005) and Uriz et al. (2008) are used to model the braces, beams and columns which are available in the OpenSees framework. In this model two elements of the non-linear beam-column with five integration points are used to model one brace. The Corotational theory has been utilized to simulate the large to moderate deformation of the braces with the inelastic buckling. Tangent stiffness with damping 3% has been adopted. Seven real accelerograms have been used to simulate the ground motions which are corrected to match the 5% design displacement spectrum as defined in Eurocode8 (EC8) (2005) with corner periods TB, TC, TD, TE and TF are 0.15, 0.55, 2, 4.5 and 10s respectively. The amplitudes of real accelerograms are scaled to match the average with design spectrum within the range between 0-4s (Figure 19).

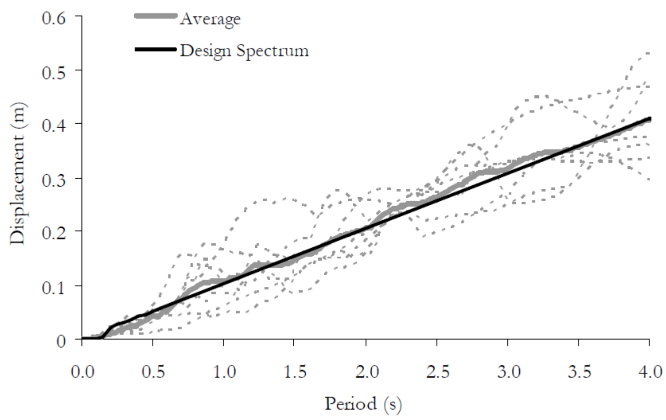


Figure 19 Displacement spectra from the scaled natural accelerogram at 5% damping

The above figure shows that the adjusted average displacement spectrum of the real accelerograms is well matched with the design displacement spectrum.

3. Results and suggestions of the studies

The main issues and suggestions of the reviewed studies of DDBD for steel frames that would develop the application of DDBD on CBFs can be summarized to;

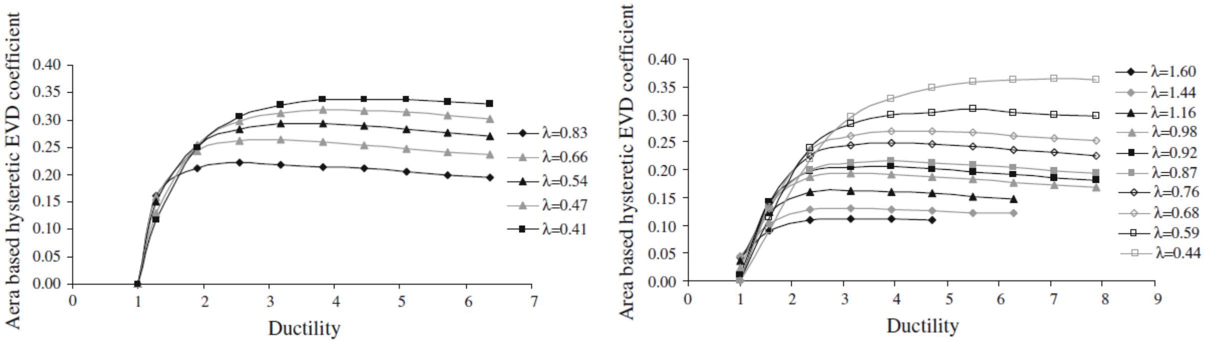
The evidence from Medhekar and Kennedy (2000a) study suggests that: first, design displacement spectra could be generated numerically by integrating appropriate accelerograms of SDOF system. For the periods less than 1.0s the mean plus one standard deviation ($\mu + 1\sigma$) of displacement spectra (DRS) that adopted in this study reveals good agreement with the obtained DRS of NBCC for the same damping, and for longer periods the results show lower response, thus NBCC includes a correction factor for impacts of higher modes (Figure 3 and Figure 4). Further when the hysteretic response occur the effective damping should be raised otherwise the obtained effective period will be less and the design base shear will be greater. Second base shear, distribution of static force profile and members' strength are largely affected by the initial displacement shape. A greater displacement profile leads to lower base shear, however ductility will be bias toward upper storeys and the larger amplitude of the assumed displacement results in increasing the lateral force at the top levels. It has been also evident in this study modifying the lateral force distribution over the height decreases localisation of ductility demand by increasing the stiffness of the higher storeys. Moreover, displacement profile of the first mode provides reasonable results due to the fact that MDOF system performs elastically in the first mode. They stated also some effects such as column deformation and P-Δ effect could be represented approximately within the pre-design stage. Hence, the column deformations would be estimated using eq. (2) of this study or approximately by decreases the calculated period of the equivalent SDOF system to produce stiffer braces which compensate the deformation in columns. While the P-Δ effect is a function of the assumed displacement shape and the computed gravity loads and it could be implied during preliminary design stage. Finally, it seems from the findings that the proposed displacement shape of the design process agrees fairly well with that obtained from the dynamic analyses. Nevertheless, in some cases the ductility demand is greater than the assumed displacement shape due to the effect of higher mode shapes in the top storeys. It can also adjust the lateral load profile without changing the base shear by applying 15% of the base shear on the roof level and distribute the remainder over the building height, which could make top storeys stiffer and reduce the ductility demand. In fact, it is essential to evaluate the brace ductility demand which is the more important dynamic property and directly affect the inter-storey drift to avoid damage to structural and non-structural components.

Della Corte and Mazzolani (2008) applied the proposed assumptions of their study to design a ten-storey one-bay steel frame with inverted v-bracing. The findings of their study indicate that: first it reveals that the post buckling response is in well agreement and shows closer response to the numerical analyses than pre-buckling response. Second the design displacement is larger than the numerical average displacement response. This could result from design values are less than actual strength and stiffness of the structural members and the viscous damping modelling adopts initial stiffness instead of the tangent stiffness. Next scattering of the numerical displacement responses resulted from differences in displacement spectra of records. Moreover, higher strength of structure may reduce local ductility demand which is resulted from higher mode actions. In general, the comparison between analytical predictions and the numerical results shows an optimistic agreement.

Further, the following conclusions can be drawn from Goggins et al. (2009) research which compared the proposed DDBD procedure suggested by authors with the both of shake table tests and numerical analyses: The evidence from experimental findings suggests that: first the proposed procedure of DDBD significantly conservative and overestimates the base shear. It may be contributed to the following reasons: first neglecting compression braces strength. Second from the comparison of different damping factors in the literature as it is mentioned above in Goggins et al. (2009) study, it seems that the effective damping ratio of CBFs considerably higher than the adopted approach in the trial design procedure of DDBD of this study. Finally the residual drifts which could be an important performance factor have been not taken into account. While the evidence from numerical analyses suggests that: first the non-linear pushover analysis shows that the model which contains only the tension brace strength will underestimate the response. Thus, the both actions of the braces in compression and in tension should be represented. Second it is evident from non-linear time history analysis that the acceleration response of CBFs could be well estimated. While the amplitude of the displacement response is underestimate. Further the numerical model could not capture the asymmetry drift which is showed by shake table tests. Generally, the results of this study indicate that further studies should be pursued to predict a suitable damping model for CBFs and to develop material models in non-linear time history analysis for CBFs that can precisely capture the asymmetric displacement effect.

Goggins et al. (2009) also concluded that the EVD (Takeda-Thin) model is larger than the actual EVD coefficient. Hence they suggested further study work to develop a specified EVD coefficient for CBFs.

Therefore, the study of Wijesundara et al. (2011) concerned to produce a new damping expression for CBFs. This research shows that; The EVD that is calculated from the area based approach depends significantly on the bracing slenderness and the ductility. But it is independent on the bracing configuration (Figure 20).



a *b*

Figure 20 Variation of area based hysteretic EVD against ductility a) X braced frame b) decoupled diagonals braced frames (λ, non-dimensional slenderness ratio)

In addition, the corrected values of the hysteretic EVD are greatly functioned with the bracing slenderness (Figure 21).

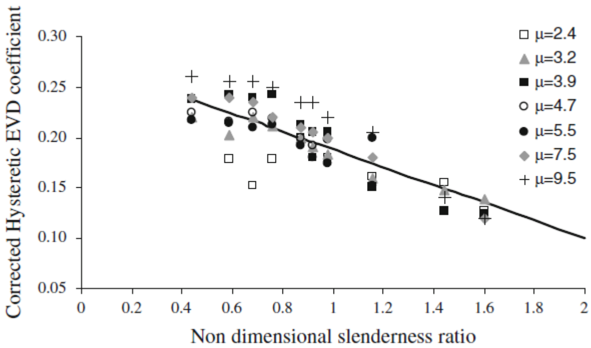


Figure 21 Variation of corrected EVD coefficient against the non-dimensional slenderness ratio (μ, ductility)

Generally, the developed equation (equation 11 and equation12) of EVD in this study for CBFs predicts higher energy dissipation for the short braces than that was estimated by Ramberg Osgood of MRF system, the difference is higher for low ductility, but it reduces for higher ductility. While the EVD of CBFs for the slender braces shows lower values particularly for higher ductility (Figure 22), which agrees with what was observed by Goggins et al. (2009). This could be attributed to significant pinching hysteretic response of slender braces after buckling.

$$\xi_{CBF} = 0.03 + \left(0.23 - \frac{\lambda}{15}\right)(\mu - 1) \quad \mu \leq 2 \tag{11}$$

$$\xi_{CBF} = 0.03 + \left(0.23 - \frac{\lambda}{15}\right) \quad \mu \geq 2 \tag{12}$$

Finally, the developed equations of the EVD show a bi-linear relation with the variation of the ductility and depends linearly on the slenderness (Figure 21). This could imply a simplified calculation of EVD and with the high ductility demand levels the precise estimation of the yield displacement is not essential.

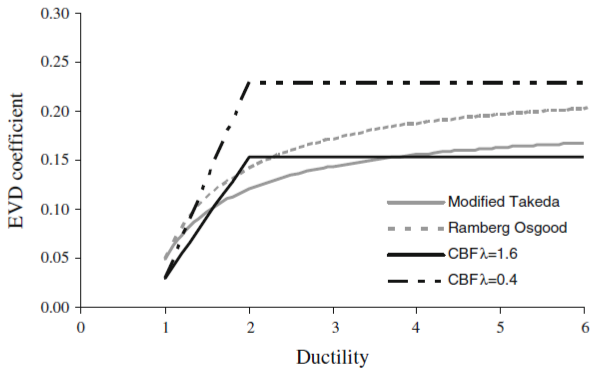


Figure 22 Comparison of EVD expressions proposed for different structural systems

Moreover, The results of numerical investigation of Wijesundara and Rajeev (2012) which was used to validate their proposed procedure show that;
For the 4IVMC building the computed average drift is less than the design drift around 4% but the difference becomes 30% less for the top storey (Figure 23). Furthermore, numerical analyses of the 8IVMC building show that the average displacement profile is properly matched with the design displacement profile (Figure 24). In general, the average numerical displacement values are less than the displacement profile of the design phase. While the calculated drift is marginally higher than the proposed drift for the storey floors 5, 6 and 7 with maximum difference of 16 % higher at level 7.

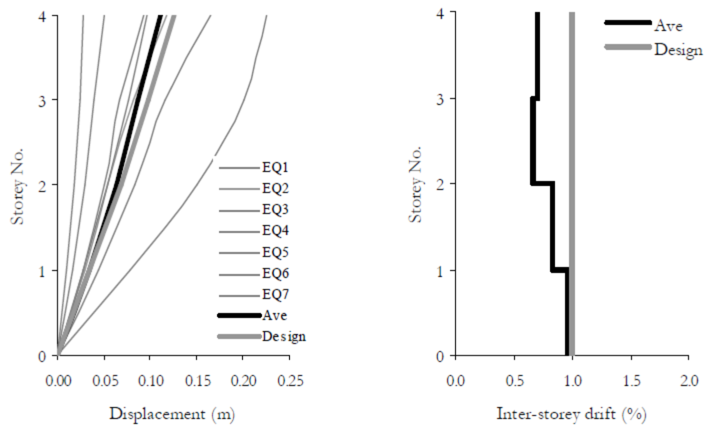


Figure 23 Average time-history response of 4 storey braced frame with IVMC configuration

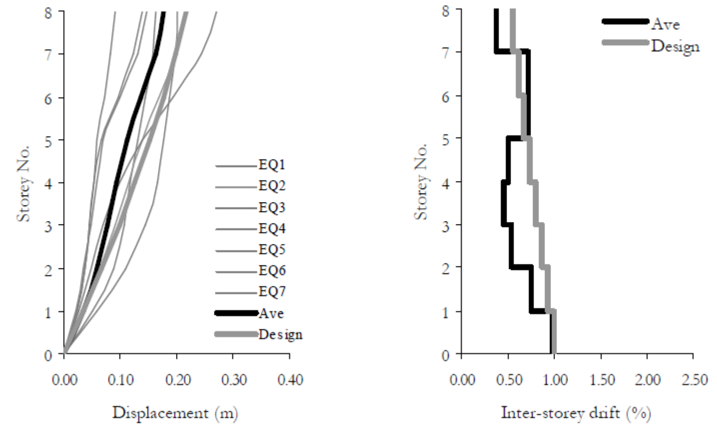


Figure 24 Average time-history response of 8 storey braced frame with IVMC configuration

The comparisons in terms of shear forces at each floor between the numerical results and the design phase of the both 4 and 8 IVMC storey frames show that the design shear is slightly lower than the numerical average shear of the all records. While they are in good match for the higher floors.

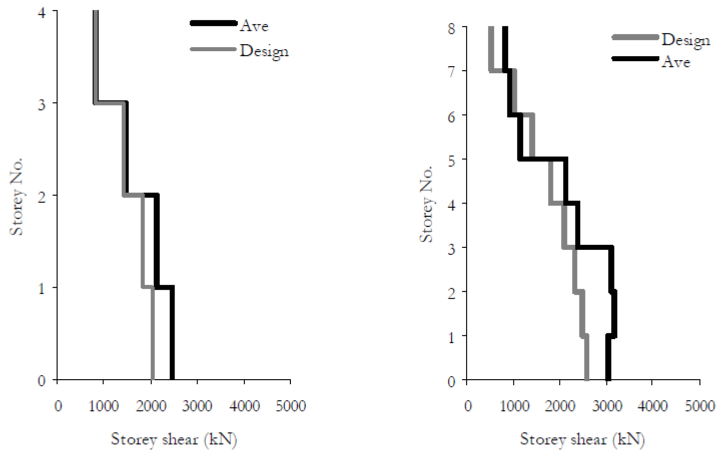


Figure 25 Storey shear distributions of 4 and 8 storey frames with IVMC configuration

It shows also from the numerical analyses for X bracing frame configuration, in the case of 4 storeys there is a marginal deviation in displacement profile comparing with the pre-assumed profile which resulted in drift concentration at the first floor that is 17% higher at 1st storey and 80% lower at the top level (Figure 26). Moreover, for 8 storeys frame the calculated displacement design profile is fairly matched with the design profile which ensures that there is no significant exceedance in the design drifts (Figure 27). And the numerical shear forces at each storey are significantly higher than the design values, while they match at upper floors (Figure 28).

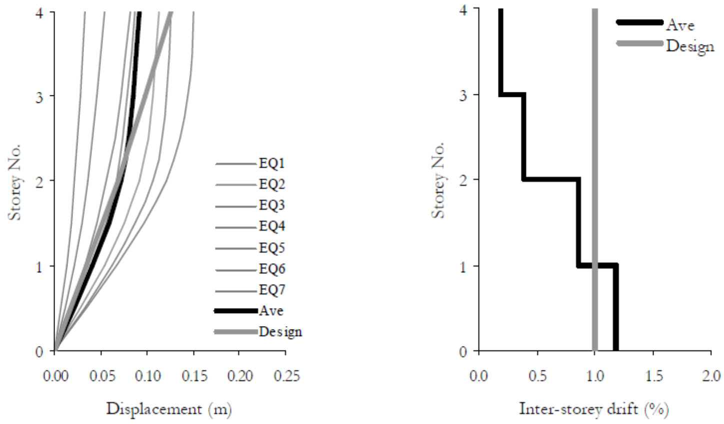


Figure 26 Average time-history response of 4 storey braced frame with X configuration

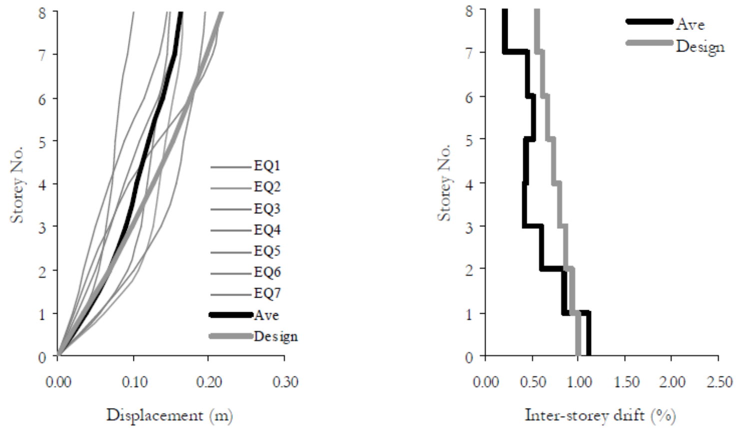


Figure 27 Average time-history response of 8 storey braced frame with X configuration

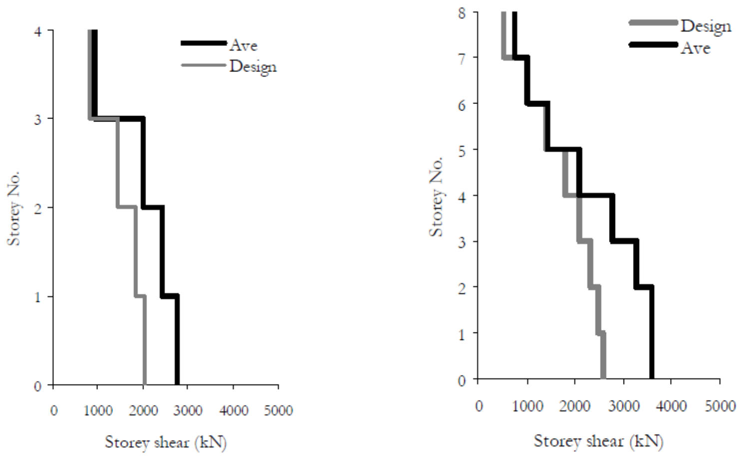


Figure 28 Storey shear distributions of 4 and 8 storey frames with X configuration

Further, it is revealed from the hysteretic response of axial force-displacement of the 4 IVMC braces that all braces are buckled and some are significantly yielded under the shaking of EQ4. It is also noted that there is a significant strength degradation of the buckled braces. This investigation also shows that the design profile slightly depends on bracing configuration.

4. Conclusion

The recent seismic design method is direct displacement based design (DDBD) methodology which has been well developed by many studies and successfully utilized to design concrete structures and bridges. However, the literature has limited research concerning steel structures especially for CBFs. In this study, an attempt has been made to highlight and review studies of direct displacement based design methodology concerning concentric steel braced frames (CBF).

In the current investigation it is recognized many limitations in the reviewed studies of steel frames in DDBD which could be summarized as, firstly although the study of Medhekar and Kennedy (2000a) has successfully applied the Direct Displacement Based Design (DDBD) approach for steel frames, it has certain drawbacks in terms of their analysis procedure which adopted the nominal viscous damping 5% as a critical damping in the both elastic and inelastic actions and they neglected the axial deformation of columns when they developed the yield displacement profile. Further, Della Corte and Mazzolani (2008) performed the comparison between analytical predictions and the numerical results and show an optimistic agreement. However, the latter research was limited by using the equivalent viscous damping (EVD-Takeda-Thin) which is used for the reinforced concrete structures (Priestley et al., 2007). Finally, the proposed procedure of Wijesundara and Rajeev (2012) study covers many gaps in the DDBD of CBFs. But this procedure has used the displacement shape of inelastic first mode shape which is suggested by Priestley et al. (2007) for the reinforced concrete moment resisting frame structures MRFs.

Further work is required to develop a displacement profile for CBFs by incorporating different heights, bracing configuration and changing braces slenderness ratios over the height of building to confirm and expand the findings. There are also still many unanswered questions about the redistribution adequacy of the base shear over the height in tall CBFs and the impacts of the higher modes of vibration of the top storeys on the ductility demand and lateral force localization.

References

AMERICAN INSTITUTE OF STEEL CONSTRUCTIONS (AISC) 2005. Seismic provisions for structural steel buildings, Chicago.

ARCHAMBAULT, M.-H., FILIATRAULT, A. & TREMBLAY, R. 1995. *Étude du comportement séismique des contreventements ductiles en X avec profiles tubulaires en acier*, Département de génie civil, Section structures, École polytechnique de Montréal.

CANADIAN STANDARD ASSOCIATION (CSA) 2001. Limit state design of structures, CAN/CSA-S16.1-M94, Rexdale.

DELLA CORTE, G. & MAZZOLANI, F. Theoretical developments and numerical verification of a displacement-based design procedure for steel braced structures. Proceedings of the 14th world conference on earthquake engineering, Beijing, 2008. 12-17.

EUROCODE3 (EC3) 2005. ENV 1993-1-1: Design of Steel Structures - Part 1.1: General rules and rules for buildings. *European Committee for Standardisation, Brussels*.

EUROCODE8 (EC8) 2005. EN 1998-1: design provisions for earthquake resistance of structures, part 1: general rules, seismic actions and rules for buildings. *European Committee for Standardisation, Brussels*

GOGGINS, J., SULLIVAN, T., SAUSE, R., MAZZOLANI, F. & RICLES, J. 2009. Displacement-based seismic design of SDOF concentrically braced frames. *Mazzolani, Ricles, Sause (eds) STESSA*, 685-692.

IZZUDDIN, B. A. 1991. *Nonlinear dynamic analysis of framed structures.* PhD thesis, University of London.

JACOBSEN, L. S. 1960. Damping in composite structures. *Proc., 2nd WCEE,* vol 2**,** pp 1029–1044.

MCKENNA, F., MCGANN, C., ARDUINO, P. & HARMON, J. A. 2014. OpenSees Laboratory.

MEDHEKAR, M. & KENNEDY, D. 2000a. Displacement-based seismic design of buildings—application. *Engineering structures,* 22**,** 210-221.

MEDHEKAR, M. S. & KENNEDY, D. J. L. 2000b. Displacement-based seismic design of buildings—theory. *Engineering Structures,* 22**,** 201-209.

OPENSEES PROGRAMME 2006. Open system for earthquake engineering simulation. Pacific Earthquake Engineering Research Center, University of California, Berkeley.

PRIESTLEY, M., CALVI, G. & KOWALSKY, M. Direct displacement-based seismic design of structures. 5th New Zealand Society for Earthquake Engineering Conference, 2007.

PRIESTLEY, M. N. 1993. Myths and fallacies in earthquake engineering—conflicts between design and reality. *Bulletin of the New Zealand National Society for Earthquake Engineering,* 26**,** 329-341.

URIZ, P. 2005. *Towards earthquake resistant design of concentrically braced steel structures.* University of California, Berkeley.

URIZ, P., FILIPPOU, F. C. & MAHIN, S. A. 2008. Model for cyclic inelastic buckling of steel braces. *Journal of structural engineering,* 134**,** 619-628.

WIJESUNDARA, K., BOLOGNINI, D., NASCIMBENE, R. & CALVI, G. 2009. Review of design parameters of concentrically braced frames with RHS shape braces. *Journal of Earthquake Engineering,* 13**,** 109-131.

WIJESUNDARA, K., NASCIMBENE, R. & SULLIVAN, T. 2011. Equivalent viscous damping for steel concentrically braced frame structures. *Bulletin of Earthquake Engineering,* 9**,** 1535-1558.

WIJESUNDARA, K. K. & RAJEEV, P. 2012. Direct Displacement-Based Seismic Design of Steel Concentric Braced Frame Structures. *Australian Journal of Structural Engineering,* 13**,** 243-257.