16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° XXXX (ID: 2614)

Registration Code: S-XXXXXXXX

Displacement Profile for Displacement Based Seismic Design of Concentric Braced Frames

P.Rajeev [(1)], K. K. Wijesundara[(2)], M. Al-Mashaykhi [(3)], M. J. Hashemi[(4)], E. Gad[(5)]

[(1)] *Senior Lecturer, Swinburne University of Technology, Australia, prajeev@swin.edu.au*
[(2)] *Senior Lecturer, University of Peradeniya, Sri Lanka, kushan@civil.pdn.ac.lk*
[(3)] *PhD Candidate, Swinburne University of Technology, Australia, malmashaykhi@swin.edu.au*
[(4)] *Research Fellow, Swinburne University of Technology, Australia, jhasemi@swin.edu.au*
[(5)] *Professor, Swinburne University of Technology, Australia, egad@swin.edu.au*

Abstract

The direct displacement based design (DDBD) procedure is well developed and used for designing reinforced concrete and steel moment resisting frame structures, wall structures and bridges. However, limited number of studies is available on designing steel concentric braced frame (CBF) structures using DDBD approach and also those studies used the displacement profile developed for the reinforced concrete moment resisting frame structures by Priestley et al. (2007) as the design displacement profile for CBFs. Therefore, an appropriate design displacement profile for CBFs is required to enhance the seismic design and performance of CBFs. On this regards, an attempt is made to develop a design displacement profile based on the median maximum storey displacements obtained from the nonlinear time history analyses for a set of real ground accelerations. For this purpose, pre-determined four different steel CBF structures with varying brace configuration and storey height are selected. The nonlinear time history analyses are performed for the 3-D nonlinear finite element models of the selected frames using OpenSEES software. The developed models are capable of simulating the out-of-plane buckling and low cycle fatigue failure of braces. The models are subjected to a set of 30 real ground motion records with varying levels of spectral acceleration at the first model period of the structures. The maximum storey displacement at each storey levels is recorded for all the ground motion records. On the basis of the observed storey displacement and interstroey drift, the appropriate design displacement profile for CBFs is proposed.

Keywords: Concentric Braced Frames; Displacement Based Design; Out-of-Plane buckling; Displacement profile

1. Introduction

In the last century, seismic design of structures is conferred a special attention through research and governmental policies more than other load cases such as gravity, wind, traffic etc. Initially, Force Based Design (FBD) procedure is adopted in seismic design of structures based on strength as the most important design parameter. Later, it was recognised that structural vulnerability would be linked to the structural performance defined by a displacement level to which a structure is subjected under the specified seismic intensity, and not the strength which only helps to reduce displacements or strains. In 1993, the Direct Displacement Based Design (DDBD) was first introduced by Priestley [9] as the first design concept, which adopts the structural performance philosophy, defined by strain or drift limits under a specified seismic intensity level. Over the last decades it has been improved with intensive coordinated research efforts especially in Europe, New Zealand, and North America and applied for seismic designing of different structural configurations. In fact, now DDBD procedure is well developed and used for designing reinforced concrete and steel moment resisting frame structures, wall structures and bridges. However, there is still limited number of studies available on designing steel Concentric Braced Frame (CBF) structures using the DDBD approach. In 2000, as authors are concerned, it has been firstly applied and developed to design steel CBFs by Madhekar et al [7]. They also derived the yield displacement profile on the basis of the braces yielding. Nevertheless, their design procedure used 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. Della Corte and Mazzolani [3] has also discussed the concepts and developed procedures for the direct displacement-based design (DDBD) method for steel braced frames and has incorporated the braces slenderness in the yield displacement profile of CBFs. However, the equivalent viscous damping (EVD-Takeda-Thin) that is used in the presented procedure was for the reinforced concrete structures. Further, Goggins and Sullivan [4] examined the EVD Coefficient for different CBF structures whose slender braces by using a shaking table, they 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. More Recently, Wijesundara et al [18] developed a new damping expression for concentrically braced frame structures based on the ductility and the non-dimensional slenderness ratio. Moreover, Wijesundara and Rajeev [17] involved the braces tensile yielding and columns axial deformations in the yield displacement profile of the DDBD. However, the latter research used the displacement profile that is proposed for the moment resisting frame structures by Priestley et al. [10] as the design displacement profile for CBFs. However, it has been observed in the literature that the design displacement profile is yet to be developed for CBF structures.

As a consequence, this study developed a suitable design displacement profile for CBFs. On this regards, four steel CBFs with different brace configurations and storey heights are pre-designed. By using OpenSees program, the 3-D nonlinear finite element models of the pre-designed frames are developed. Braces are modeled using inelastic beam-column elements taking in to account inelastic bucking either in in-plane or out of plane as described by Wijesundara et al 2014 [18]. The nonlinear time history analyses are performed for the pre-determined frame models for a set of 30 real ground motion records with variable levels of spectral acceleration in order to obtain the absolute maximum displacement and inter-storey drift at each storey levels recorded during an each ground motion records. Then, based on this results, 16% fractile, median and 84% fractile maximum displacement and drift profiles for each CBFs are developed. Finally, the appropriate design displacement profile for CBFs is proposed based on the median displacement profiles of the four CBFs.

2. Steel CBF considered in this study

Four CBF structures are selected for this study varying the type of concentric bracing configuration and the number of storeys. Two CBF structures are four and eight storeys with inverted-V bracing configuration and continuous middle column. It links the brace-to-beam intersection points at each floor level directly to the foundation. From this point onwards, the inverted-V bracing configuration with middle column is called as IVMC bracing configuration. Other two frames are also four and eight storeys but with X bracing configuration. The floor plan and the elevation of the buildings were predetermined as shown in Fig. 1. The building geometry is symmetric in plan and elevation. The height of each storey is 3.5m and the bay width of each of braced and unbraced bays is 7m. The locations of the concentric braced frames are shown by the bold lines in Fig. 1.

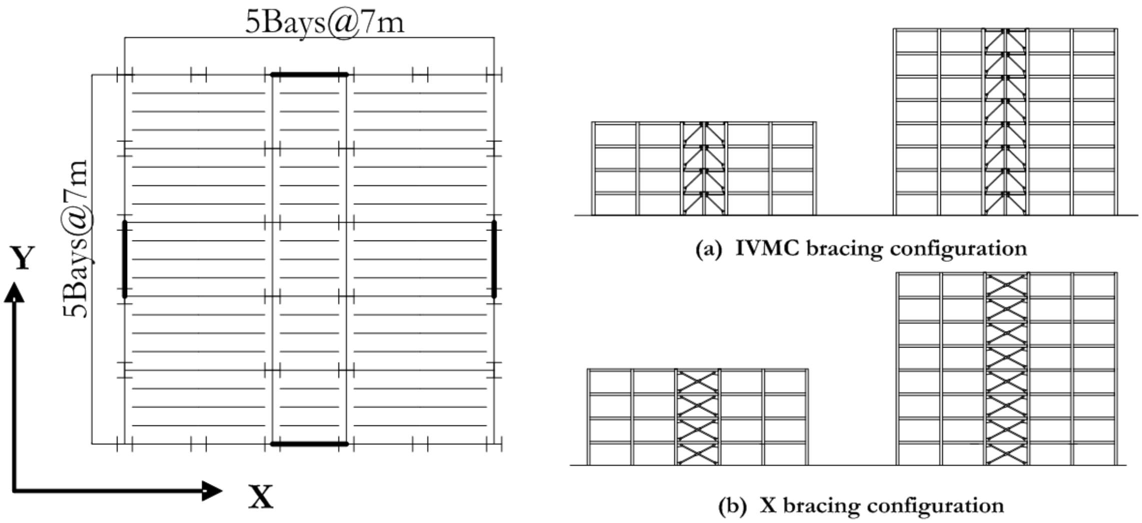


Fig. 1 – Plan view and elevation of CBF structures with (a) IVMC (b) X bracing configuration

As shown in Fig. 1, IVMC bracing configuration resists the vertical unbalanced forces following the brace bucking at the brace-to-beam intersection. It helps to redistribute the drift demand more uniformly over the height, rather than to concentrate the drift demand at the first storey level as noticed in the ordinary inverted-V braced frames. The advantages of such a configuration identified by Khatib et al. [5] include that the tension braces can develop their yield strength, additional axial loads in outer columns due to the vertical unbalanced forces are avoided and try-linear hysteretic response is achieved. Therefore, such configuration, which may behaves in different than the conventional bracing configuration such X- bracing configuration, are also included in this study together with conventional configuration of X braces.

All the four steel CBF structures are designed according to the DDBD procedure described in study by Wijesundara and Rajeev [17] 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 0.3g. 5% damped displacement spectrum as defined in Eurocode (EC)8 (2005) is used for the design with the corner period (T_c) of 4s. The displacement spectrum for other damping levels are obtained by multiplying the 5% design displacement spectral ordinates with the damping correction factor $\eta = (7/(2+\xi))^{0.5}$ as given in Priestley et al. [9]. The CBFs were designed as if located on stiff soil (i.e., ground motion of magnitude 5.5 or above and class-A according to EC-8 (2005) definitions). The important factor (γ_I) was assigned equal to 1 for the buildings according to the classification of important classes for the buildings in EC8 (2005). It is important to note that some assumptions are made in designing and modelling the braced frames such that the accidental torsion, the stiffness and the strength contributions of external cladding and interior partitions are ignored and columns are continuous over the height in order to reduce the drift concentration at the lower storeys [6]. Braces are designed and detailed to have an out-of-plane deformation by providing a free space of two times the gusset plate thickness in single gusset plate at each end of the brace in order to reduce additional demand to the welds when the brace buckles [2, 15].

The characteristic values of imposed loads on the floors are taken according to the category B defined in EC-1 (2002). Finally, the total dead load including the self weights of the slab, the finishing, and the dead weight of the exterior cladding is 6.4 kPa while the imposed load is 3 kPa. The columns and beams are assigned to wide flange sections made in EN 10025-2 grade S355 hot rolled structural steel with nominal yield strength of 355 N/mm2 while the brace sections are assigned to hollow square sections (HSS) made in EN 10025-1 grade S355H cold formed steel with the nominal yield strength of 355 N/mm^2. Table 5 provides the selected wide flange sections for the columns and the beams and HSS sections for the braces in four CBF structures selected in this study.

Table 1. Wide Flange Column and Beam sections details at braced bays

Storey No.	4 Storey Frames					
	IVMC-Configuration			X-Configuration		
	Braces	Columns	Beams	Braces	Columns	Beams
1	HSS152.4X152.4X12.7	W360X287	W360X122	HSS203.2X203.2X9.5	W360X314	W360X122
2	HSS139.7X139.7X12.7	W360X287	W360X122	HSS203.2X203.2X9.5	W360X314	W360X122
3	HSS114.3X114.3X12.7	W360X134	W360X101	HSS177.8X177.8X7.9	W360X122	W360X101
4	HSS101.6X101.6X7.9	W360X134	W360X101	HSS139.7X139.7X6.4	W360X122	W360X101
Storey No.	**8 Storey Frames**					
	IVMC-Configuration			**X-Configuration**		
	Braces	**Columns**	**Beams**	**Braces**	**Columns**	**Beams**
1	228.6x228.6x15.9	W360X818	W360X134	177.8X177.8X15.9	W360X818	W360X134
2	228.6x228.6x15.9	W360X818	W360X134	177.8X177.8X15.9	W360X818	W360X134
3	203.2X203.2X15.9	W360X551	W360X134	177.8X177.8X15.9	W360X551	W360X134
4	203.2X203.2X15.9	W360X551	W360X134	177.8X177.8X12.7	W360X551	W360X134
5	203.2X203.2X12.7	W360X314	W360X134	177.8X177.8X12.7	W360X314	W360X134
6	203.2X203.2X9.5	W360X314	W360X134	177.8X177.8X9.5	W360X314	W360X134
7	177.8X177.8X7.9	W360X110	W360X134	152.4X152.4X7.9	W360X110	W360X134
8	139.7X139.7X6.4	W360X110	W360X134	114.3X114.3X6.4	W360X110	W360X134

3 Numerical modelling

In order to investigate the performance of the steel CBF models designed according to the DDBD procedure, nonlinear dynamic analyses are performed using the OpenSEES finite element computer program [8]. 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. Tremblay et al. [12] and Tremblay et al. [13] pointed out that the out-of-plane buckling response of a brace in concentric configuration can occur during a severe shaking. 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 et al. [14]. In this model, each brace is modelled using two nonlinear beam-column elements with five integration points Wijesundara et al [18]. 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 of inelastic buckling of braces. Newmark acceleration time integration scheme with beta and gamma 0.25 and 0.5, respectively and tangent stiffness proportional damping equal to 3% of critical damping are adopted for the analyses.

4 Earthquake ground motions

In order to perform time-history analyses, a suite of 30 recorded ground motion are selected as three groups of ten records from the European Strong Motion Data Base (ESD) maintained at Imperial College [1]. Each group matches a different target uniform hazard spectrum of average return periods of 100, 500, and 1000 years, respectively. Fig. 2 shows the target uniform hazard spectrum and the average elastic 5% damped response spectra of ten records. The records have been selected from stiff to soft soil sites, free-field, including large and distant, large and close, moderate and close as well as intermediate earthquake records in an attempt to cover a sufficiently representative range of distances and the expected range of magnitude in Europe. The magnitude (M_w) range from 5.5 to 7.9 and source-to-site distances (r) of $0 \le r \le 97$ km. Details about the records employed can be found in Rajeev [11].

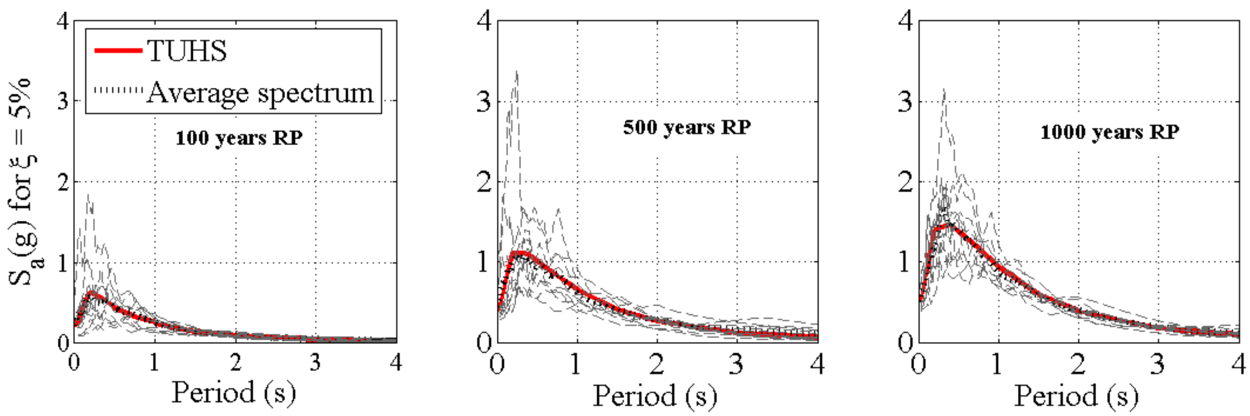
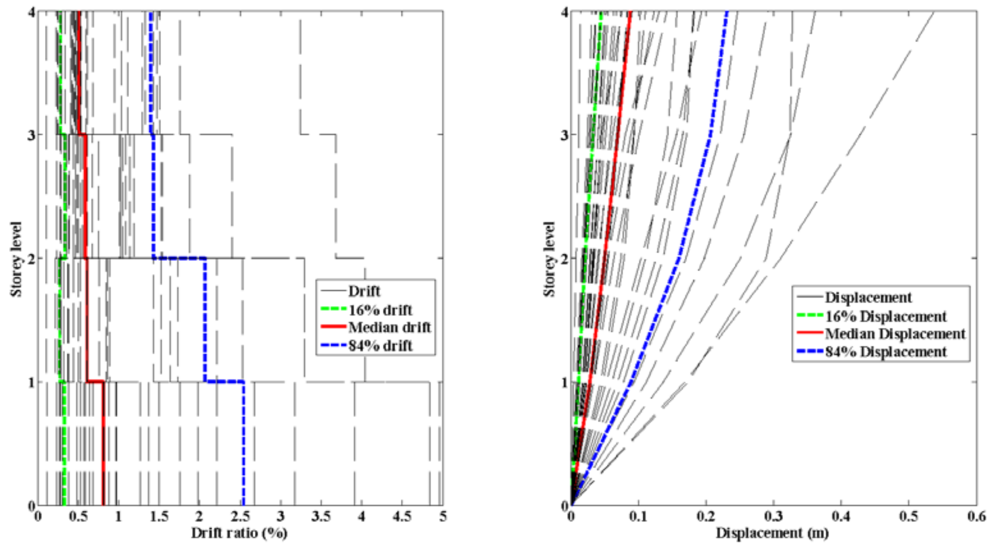


Fig. 2 – Target and average demand spectra

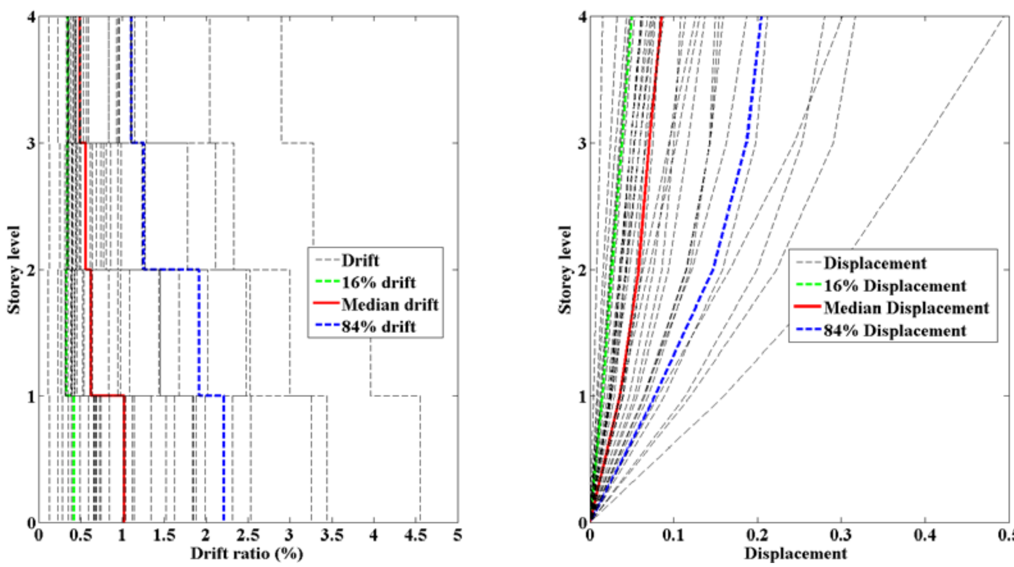
5. Results and discussion

A nonlinear time history analysis of all the four buildings was performed for the 30 ground motion records. The storey displacements and the interstorey drift ratios were recorded from the analysis. Storey displacement and interstorey drift ratio profiles of the median and 16 and 84 percentiles were computed to develop a suitable design profile. Figure 3 (a and b) illustrates the interstorey drift ratio and storey displacement profiles for the individual records together with the median and, 16 and 84 percentiles envelopes. The interstorey drift ratio is highest at the first storey level and then it decreases with the storey level for both 4-stroey IVMC and X braced frames. As shown in Figure 4 a and 4b, the interstorey drift ratio is again highest at the first storey level similarly to the 4-storey frames but it decreases with storey level only up to 4th storey and then increases from 5th to 8th storeys. This observation is valid for the entire median and 16 and 84 percentiles envelopes of interstorey drift ratio.

In the direct displacement based design, the design displacement profile should be selected at the beginning the of the design stage. As stated above, the design displacement profile proposed by Priestley et al. [9] referring the first inelastic mode shape of moment resisting frame structures is commonly adopted as a design displacement profile of CBFs. Khatib et al. [5] reported that higher drift demands at lower storeys are resulted in the CBFs when they were subjected to a severe ground shaking. Thus, it is reasonable to use the design displacement profile proposed by Priestley [9] for steel CBFs.



(a)



(b)

Fig. 3 – Interstorey drift ratio and displacement profiles: (a) 4IVMC and (b) 4X

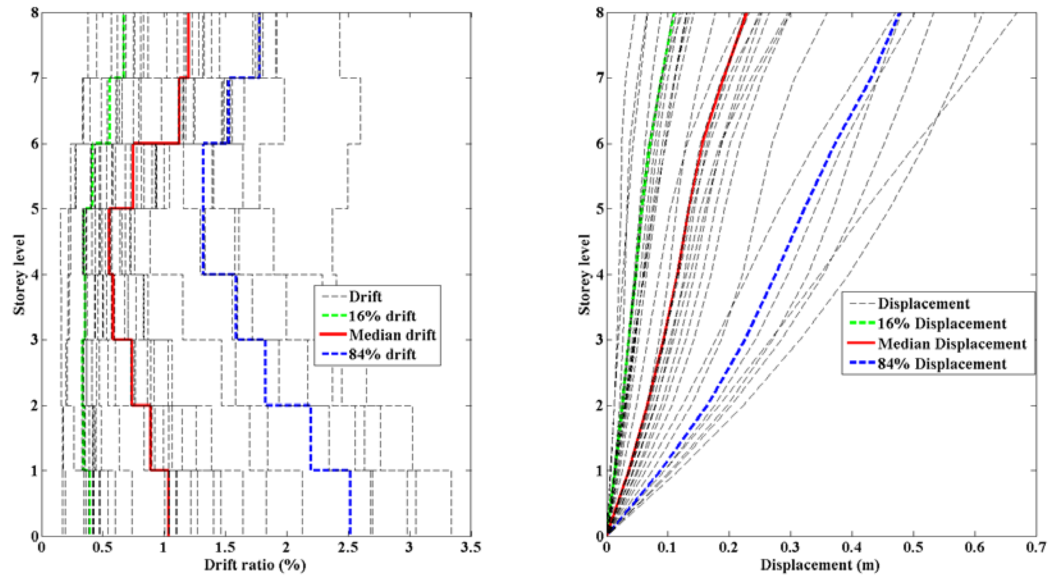
The design displacement profile proposed in Priestley et al. [10] using normalized inelastic mode shape δ_i, and the displacement of the lowest storey Δ_1 is summarized as given below:

$$\Delta_i = \delta_i \left(\frac{\Delta_1}{\delta_1} \right) \tag{1}$$

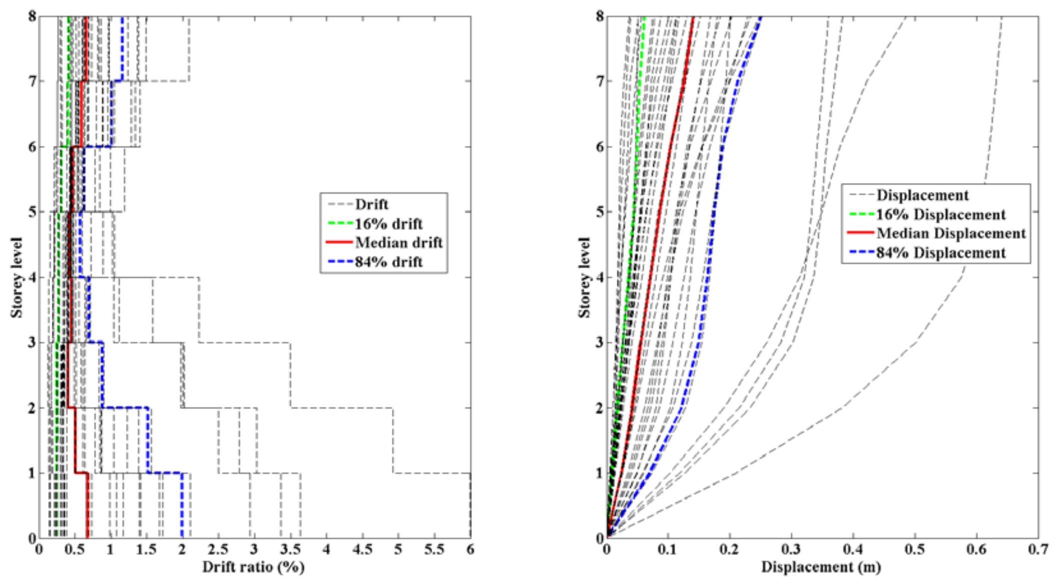
$$\delta_i = \frac{H_i}{H_n} \quad n \le 4 \tag{2}$$

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

where the normalized inelastic mode shape depends on the height H_i, the roof height H_n and the number of storeys n.



(a)



(b)

Fig. 4 – Interstorey drift ratio and displacement profiles: (a) 8IVMC and (b) 8X

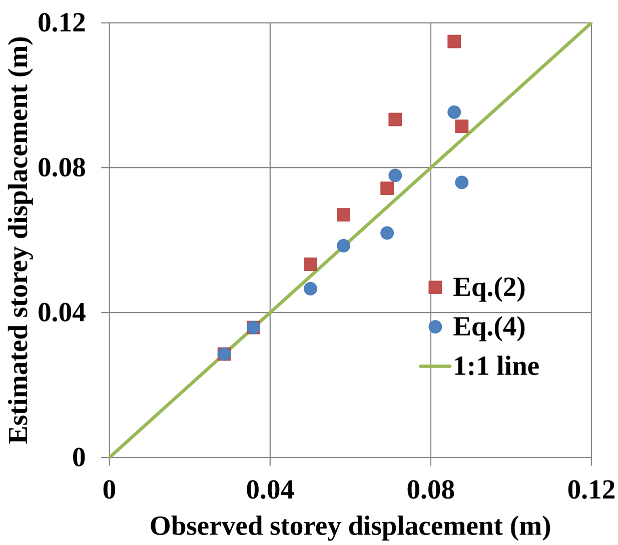
According to Eqs. (1) and (2), the displacement profile for a building having number of storeys below four is linearly increasing with height. Therefore, the interstorey drift ratio is constant along the height of the building. However, it is clear that the intersorey drift ratios of 4-storey frames decrease with height as illustrated in Fig. 3. On the other hand, the proposed displacement profile of CBF buildings having number of storey more than four is assumed to be parabolic following the first mode shape. Therefore, the interstorey drift ratio is decreasing with height of the building. The observation made in this study (Fig. 4) shows that the interstorey drift decreases with up to the middle height of the building and then increases. Hence, the displacement profile proposed by Priestley et al. [10] is required to be modified.

In order to make modification to original Priestley's displacement profile, the median displacement profiles were used. As the interstorey drift ratio decreasing with the height for the 4-story frame, the normalized inelastic mode shape given in Eq. (2) can be modified as given in Eq. (4):

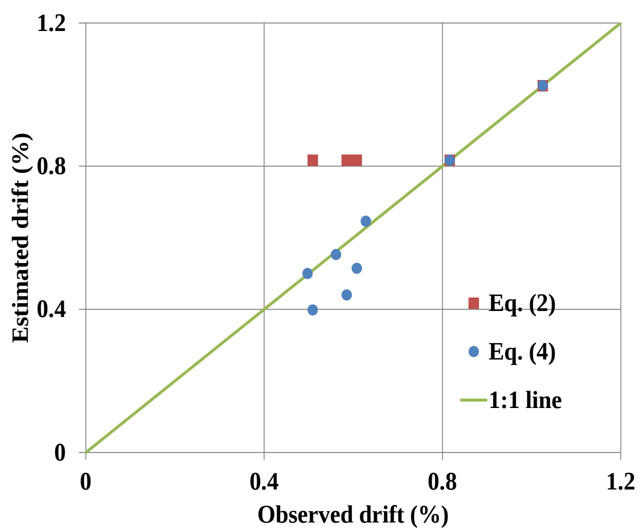
$$\delta_i = \left(\frac{H_i}{H_n}\right)^{\alpha} \quad n \leq 4 \tag{4}$$

where α is a constant that need to be calibrated. The proposed modification can provide the decreasing interstorey drift ratio profile along the height of the building. The suitable value for α was estimated by minimising the error between the observed median displacement profile from the nonlinear time history analyses and the computed displacement profiles using the proposed Eqs. (4) and (1). The median displacement profile estimated for both 4 storey IVMC and X braced frames was combined together to find a value for α that is around 0.7 on the basis of the frames considered in this study.

Fig. 5 (a) shows the comparison between the estimated displacement profiles using Eq. (2) and (4) with the observed median displacement profiles of 4 storey frames. The coefficient of determination (R^2) for Eq. (2) prediction is 0.56 while it is for the Eq. (4) close to 0.90. The coefficients of determination (R^2) estimated for the distribution of interstroey drift ratios of 4-storey IVMC and X braced frames is 0.81 for Eq. (4) and no correction is very low for Eq. (2).



(a)



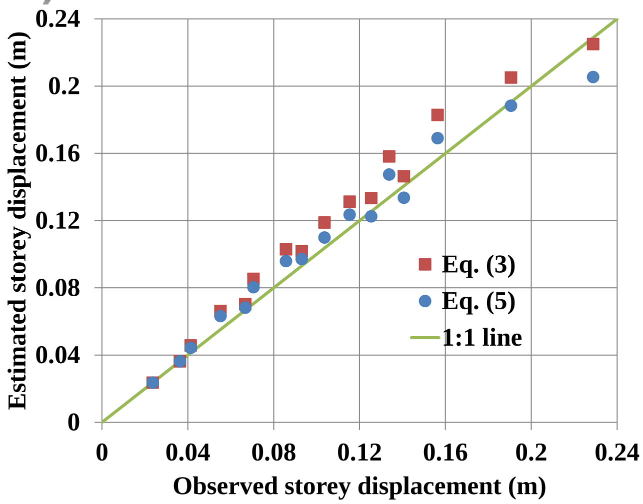
(b)

Fig. 5 – (a) Estimated displacement profile Priestley's model (Eq. 2) and the proposed model (Eq. 4) and (b) Estimated drift profile Priestley's model (Eq. 2) and the proposed model (Eq. 4)

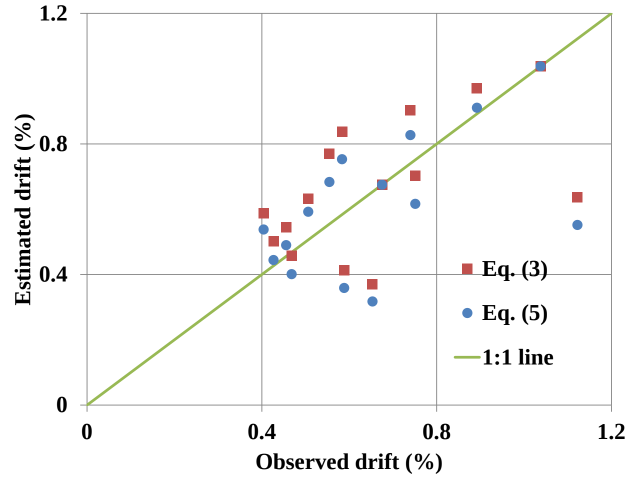
The modification to original Priestley's displacement profile was carried out using the median displacement computed for the 8 storey frames as follows:

$$\delta_i = \frac{4}{3}\left(\frac{H_i}{H_n}\right)^{\alpha}\left(1 - \frac{H_i}{4H_n}\right) \quad n \geq 4 \tag{5}$$

Again the suitable value for α was estimated by minimising the error between the observed median displacement profile from the nonlinear time history analyses and the computed displacement profiles using the proposed Eqs. (4) and (1). The median displacement profile estimated for both 8 storey IVMC and X braced frames was combined together to find a value for α that is around 0.96 (i.e., close to 1) on the basis of the frames considered in this study. Therefore, it is reasonable use the Priestley's displacement profile for the design of concentric braced frames having number of storeys above 4. However, the comparison of observed and estimated displacement and drift profiles is shown in Fig. (6).



(a)



(b)

Fig. 6 – (a) Estimated displacement profile Priestley's model (Eq. 3) and the proposed model (Eq. 5) and (b) Estimated drift profile Priestley's model (Eq. 3) and the proposed model (Eq. 5)

Fig. 6 (a) shows the comparison between the estimated displacement profiles using Eq. (3) and (5) with the observed median displacement profiles of 8 storey frames. The coefficient of determination (R^2) for Eq. (3) prediction is 0.94 while it is for the Eq. (5) close to 0.97. However, the coefficients of determination (R^2) estimated for the distribution of interstroey drift ratios of 8 storey IVMC and X braced frames is negative for the both models. Thus, further study is required to find a better design displacement model for building having number of storey more than 4.

6. Summary and conclusion

In this study, an attempt has been made to develop a suitable design displacement profile for the direct displacement based design of steel concentric braced frame structures. Four different steel structures with varying the storey height and the braced configuration were considered for the analysis. The nonlinear time history analysis was carried for 30 ground motion records having the average acceleration spectrum matching three different uniform hazard spectrums.

On the basis of the analysis, the median and 16 and 84 percentile displacement and drift profiles were estimated. The observed displacement profile was compared with the commonly used design displacement profile proposed in Priestley et al. (2007). The accuracy of using the Priestley's displacement profile was discussed. An improved design displacement model particularly for steel concentric braced frame structures was proposed and the accuracy of the proposed model in terms of displacement and drift was compared with the observed response and the Priestley's model. The Priestley's model can be directly applicable to the steel concentric braced frames having number of storey more than 4.

Although, the proposed model shows the better prediction for the observed response in comparison to the Priestley's model, further studies are recommended with varying number of storyes, member sizes and different ground motion records to improve and validate the model parameters.

5. References

[1] Ambraseys, N., Smit, P., Sigbjornsson, R., Suhadolc, P., & Margaris, B. 2002. ¨ Internet-Site for European Strong-Motion Data. http://www.isesd.cv.ic.ac.uk. European Commission, DirectorateGeneral XII, Environmental and Climate Programme, Brussels, Belgium.

[2] AISC. Seismic Provisions for Structural Steel Buildings. American Institute of Steel Construction; 2005.

[3] Della Corte, G. and F. Mazzolani. Theoretical developments and numerical verification of a displacement-based design procedure for steel braced structures. in Proceedings of the 14th world conference on earthquake engineering, Beijing. 2008.

[4] Goggins JG, Sullivan TJ (2009) Displacement-based seismic design of SDOF concentrically braced frames. In: Mazzolani, Ricles, Sause (eds) STESSA 2009. Taylor & Francis Group, pp 685–692

[5] Khatib I.F., Mahin S.A. and Pister K.S. 1988. Seismic behavior of concentrically braced steel frames. *Report no. UCB/EERC-88/01. Berkeley: Earthquake Engineering Research Center*, University of California.

[6] MacRae, G.A., Kimura, Y. and Roeder, C. (2004). Effect of column stiffness on braced frame seismic behaviour. *Journal of Structural Engineering*, **130**, 381-391.

[7] Medhekar, M. and D. Kennedy, *Displacement-based seismic design of buildings—application.* Engineering structures, 2000. **22**(3): p. 210-221.

[8] OpenSees. Open System for Earthquake Engineering Simulation, Pacific Earthquake Engineering Research Center. Berkeley, CA: University of California; 2013.

[9] Priestly, M.J.N. (2003). *Myths and Fallacies in Earthquake Engineering, Revisited*. The Mallet Milne Lecture, IUSS Press, Pavia, Italy.

[10] Priestly, M.J.N., Calvi, G.M., Kowalsky, M.J. (2007). Displacement-Based Design of Structures. IUSS Press, Pavia.

[11] Rajeev, P. (2007) Vector-Valued intensity measures in probabilistic seismic risk assessment, Individual Study Report, ROSE School, University of Pavia, Italy.

[12] Tremblay, R. Timler, P. Bruneau, M., and Filiatrault, A. [1995] ‘‘Performance of steel structures during the January 17, 1994 Northridge earthquake,’’ Canadian Journal of Civil Engineering 22, 338–360.

[13] Tremblay, R., Bruneau, M., Nakashima, M., Prion, H. G. L., Filiatrault, A., and DeVall, R. [1996] ‘‘Seismic design of steel buildings: lessons from the 1995 Hyogo-ken Nanbu earthquake,’’ Canadian Journal of Civil Engineering 23(3), 727–756.

[14] Uriz P, Filippou FC, Mahin SA. Model for cyclic inelastic buckling for steel member. J Struct Eng ASCE 2008;134(4):619–28.

[15] Whitmore, R. E. [1952] ‘‘Experimental investigation of stresses in gusset plates,’’ Bulletin No. 16, Engineering Experiment Station, University of Tennessee, Knoxville, May.

[16] Wijesundara, K., R. Nascimbene, and T. Sullivan, *Equivalent viscous damping for steel concentrically braced frame structures.* Bulletin of Earthquake Engineering, 2011. **9**(5): p. 1535-1558.

[17] Wijesundara, K.K. and P. Rajeev, *Direct Displacement-Based Seismic Design of Steel Concentric Braced Frame Structures.* Australian Journal of Structural Engineering, 2012. **13**(3): p. 243-257.

[18] Wijesundara K.K., Nascimbene R., Rassati G.A., [2014] “Modelling of Different Bracing Configuration in Multi-Storey Concentrically Braced Frame using a Force Based Approach”, Journal of Constructional Steel Research, 103, 241-250.