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Direct displacement based seismic design for single storey steel concentrically braced frames

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Abstract. The direct displacement based design (DDBD) approach is spreading in the field of seismic design for many types of structures. This paper is carried out to present a robust approach for the DDBD procedure for single degree of freedom (SDOF) concentrically braced frames (CBFs). Special attention is paid to the choice of an equivalent viscous damping (EVD) model that represents the behaviour of a series of full scale shake table tests. The performance of the DDBD methodology of the CBFs is verified by two ways. Firstly, by comparing the DDBD results with a series of full-scale shake table tests. Secondly, by comparing the DDBD results with a quantified nonlinear time history analysis (NLTHA). It is found that the DDBD works relatively well and could predict the base shear forces (F_b) and the required brace cross sectional sizes of the actual values obtained from shake table tests and NLTHA. In other words, when comparing the ratio of F_b estimated from the DDBD to the measured values in shake table tests, the mean and coefficient of variation (C_V) are found to be 1.09 and 0.12, respectively. Moreover, the mean and C_V of the ratios of F_b estimated from the DDBD to the values obtained from NLTHA are found to be 1.03 and 0.12, respectively. Thus, the DDBD methodology presented in this paper has been shown to give accurate and reliable results.

Keywords: concentrically braced frames; displacement based design; shake table tests; nonlinear time history analysis; seismic design; equivalent viscous damping

1. Introduction

Over the last decades extensive research was carried out on concentrically braced frames (CBFs), as they are simple to design and fabricate with low cost and showed good performance during earthquakes (Tremblay *et al.* 1995). In CBFs, the main source to absorb and dissipate energy demand during seismic actions is the inelastic behaviour of the bracing members, which can dissipate energy primarily through yielding in tension and through inelastic buckling in compression (Remennikov and Walpole 1997a).

EC8 (CEN 2004) prescribes the forced based design methodology for the seismic design of

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structures. In this approach, preliminary estimates of geometry and section sizes are carried out. Seismic forces are estimated from an elastic response spectrum. To account for dissipation of energy during the seismic design, these forces are reduced by a reduction factor dependent upon the type of the building. These seismic forces are then used to analyse the structure and determine member sizes by selecting those which have capacities larger than the estimated demand forces. Structural displacements of the frame are then estimated and checked against the code displacement limits. If the presented limits are exceeded, redesign is required.

Elghazouli (2010) assessed the fundamental approaches and main procedures adopted in the seismic design of steel frames, with emphasis on the provisions of EC8. He highlighted areas that require careful consideration with the force based design procedure and suggested a number of clarifications and modifications. Málaga-Chuquitaype and Elghazouli (2011) highlighted further considerations of the seismic demand in the design of braced frames to force based design methodologies, again with particular emphasis on European seismic provisions.

On the other hand, a new performance based seismic design methodology called the direct displacement based design (DDBD) procedure starts by considering a design displacement depending upon the drift limit chosen, then the strength required to achieve this displacement is calculated. For this procedure, a model code has been published by Calvi and Sullivan (2009). This approach was developed as a result of the shortcoming of the force based design approach identified by Priestley (1993, 2003), which led to Priestley *et al*. (2007) publishing a book on displacement based seismic design of structures. The provisions in this DDBD code have been well developed for reinforced concrete structures. However, the recommendation for steel concentrically braced frame (CBF) structures are limited in the draft model code (Calvi and Sullivan 2009). Several researchers (Medhekar and Kennedy 2000, Medhekar and Kennedy 2000, Della Corte 2006, Della Corte and Mazzolani 2008, Garcia *et al*. 2010, Maley *et al*. 2010) carried out research for DDBD procedure for steel structures. In particular, some work was carried out by Goggins and Sullivan (2009), Wijesundara (2009) and Della Corte *et al*. (2010) to verify the DDBD procedure for CBFs. This work was based on limited data from tests and numerical models. In this paper, the DDBD procedure will be developed for one-storey CBFs and compared with a series of full scale shake table tests and a large range of non-linear time history analysis (NLTHA) to assure its validity. The NLTHA used to validate the DDBD was calibrated using shake table tests (Goggins and Salawdeh 2013). Full details of the shake table tests, including observations and findings from these tests are presented elsewhere (Goggins 2004, Elghazouli *et al*. 2005, Broderick *et al*. 2008). Special attention is paid to the choice of the equivalent viscous damping (EVD) model for use in the DDBD of CBFs. EVD is an important element of DDBD methodology as it characterizes the non-linear response of the hysteretic system with the effective stiffness at maximum displacement.

2. Direct displacement based design

DDBD characterises the structure by an effective stiffness at maximum displacement and a level of EVD. In this section, complete design approach for DDBD of single degree of freedom (SDOF) structures is outlined. The latter sections will check the validity of the method using shake table tests and NLTHA.

2.1 Design displacement

For a SDOF structure, the lateral design displacement of the frame can be taken as the maximum lateral frame displacement that occurs based on the design drift limit chosen, where the maximum lateral frame displacement can be found by

$$\Delta_D = \theta_C h \tag{1}$$

where θ_C is the design drift limit and h is the height of the storey.

2.2 Yield displacement and ductility

The yield displacement of the CBF, Δ_y, is required to find the design displacement ductility of the frame, μ, in order to calculate the EVD, ξ_{eq}.

The design displacement ductility factor of the frame, μ, is found by dividing the design displacement, Δ_D, over the yield displacement, Δ_y, as shown in Eq. (2)

$$\mu = \frac{\Delta_D}{\Delta_y} \tag{2}$$

In order to limit the damage to the structural elements, the design ductility values should be less than the total ductility reached at fracture, μ_f, obtained from the expressions established by Nip *et al*. (2010) for hot-rolled and cold-formed steel shown in Eqs. (3) and (4).

Hot-rolled carbon steel

$$\mu_f = 3.69 + 6.97\bar{\lambda} - 0.05(b/t\varepsilon) - 0.19(\bar{\lambda})(b/t\varepsilon) \tag{3}$$

Cold-formed carbon steel

$$\mu_f = 6.45 + 2.28\bar{\lambda} - 0.11(b/t\varepsilon) - 0.06(\bar{\lambda})(b/t\varepsilon) \tag{4}$$

where $\bar{\lambda}$ is the normalised slenderness ratio, b is the width of the wider face of the section, t is the thickness of the section and $\varepsilon = \sqrt{235/f_y}$ where f_y is the yield strength. Several researchers (Tang and Goel 1989, Tremblay 2002, Shaback and Brown 2003, Goggins *et al*. 2006) have proposed empirical equations for predicting the fracture life of bracing members for different cross-sections and other CBF configurations.

The yield displacement of CBFs is governed by the conditions to cause yielding of the bracing elements. Tremblay (2002) suggested that the resistance of the frame can be estimated from the yield strength of the tension brace plus 80% of the compression brace buckling capacity for braces with $\bar{\lambda} \leq 1$. Goggins *et al*. (2006) found in their physical testing that the resistance provided by compression members with $1 < \bar{\lambda} \leq 2.4$ was, on average, 30% of their maximum buckling capacity. On the other hand, for the very slender brace members ($\bar{\lambda} \geq 2.4$) they found that the post-buckling resistance of the compression brace was very small and can be ignored finding that the resistance of the tension brace only, for a CBF with concentric braced members with high ductility, represents a good estimation of the overall resistance and the initial stiffness of the braced frame. This is in agreement with the provisions of EC8. Therefore, assuming the tension diagonals only participates in the lateral resistance of the structure and that strains in the beams and columns are negligible with respect to strains in the brace for a single storey structure, then from geometry shown in Fig. 1, the yield displacement can be found using Pythagoras' theorem as the following

$$(L_b + \varepsilon_y L_b)^2 = h^2 + (B + \Delta_y)^2 \tag{5}$$

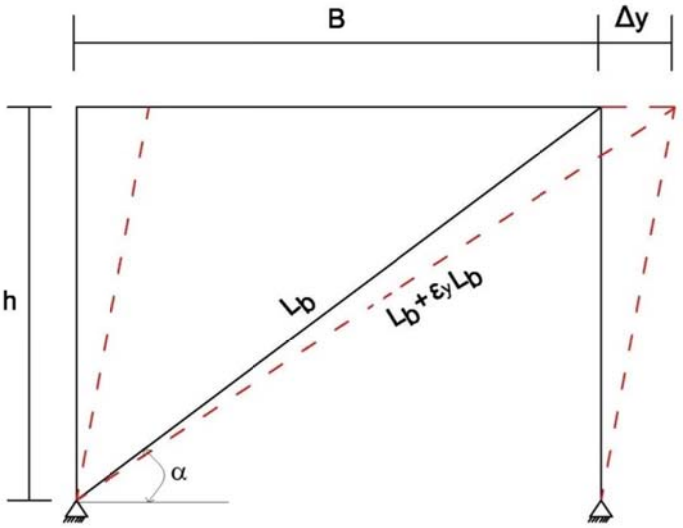


Fig. 1 Yield deformation of the CBF due to brace elongation

where L_b is the brace length, B is the bay width, h is the storey height, ε_y is the yield strain of the brace and Δ_y is the yield displacement of the frame. Eq. (5) can be rearranged as

$$\Delta_y = \frac{\varepsilon_y {L_b}^2}{B} \tag{6}$$

However, cosα=B/L_b, sinα=h/L_b and (sin2α=2cosα sinα), where α is the brace angle with the horizontal. Thus, Eq. (6) can be written as

$$\Delta_y = \frac{2\varepsilon_y h}{\sin 2\alpha} \tag{7}$$

From Eq. (7) it is evident that the yield displacement is dependent upon the material strength and the geometry, but is independent of the cross section or strength of the members.

2.3 Equivalent viscous damping

The EVD, ξ_{eq}, is the sum of the elastic and the hysteretic damping. The elastic damping of steel structures varies as a function of several parameters such as the type of connections, the non-structural elements attached to the main structure and the scope of the model and analysis where elastic damping is used to represent damping not captured by the hysteretic model adopted. For modern seismically designed structures a separation between structural and non-structural elements is required to reduce their contribution to damping. Wijesundara *et al.* (2011) suggested using 0.03 for the elastic component for EVD. The hysteresis damping depends on the hysteresis rule appropriate for the structure being designed and accounts for the effect of energy dissipated through nonlinear inelastic response. Priestley and Grant (2005) discussed the characterisation of viscous damping in time history analysis. They suggested that tangent-stiffness proportional damping is the realistic assumption for NLTHA and proposed the addition of a modification factor

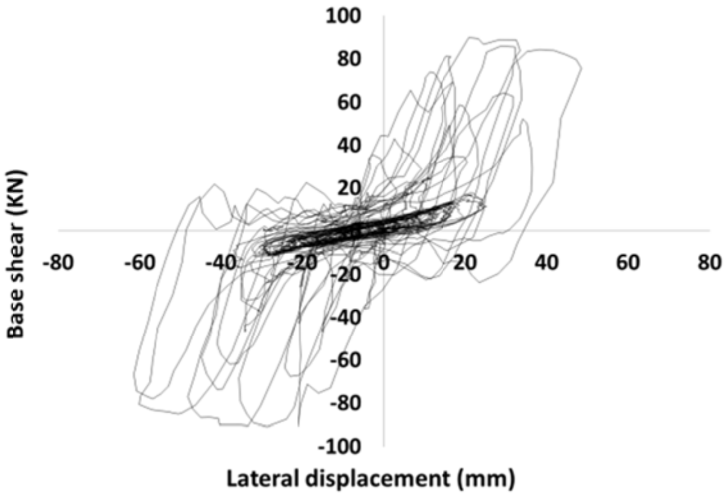


Fig. 2 Base shear-lateral displacement hysteretic response of CBF structure (Goggins and Salawdeh 2013)

to the elastic viscous damping added to the hysteretic damping in DDBD.

An example of the hysteretic behaviour of a CBF structure is shown in Fig. 2, where the behaviour is described by the base shear plotted against lateral displacement. The area under the curve indicates the amount of hysteretic energy the structure dissipates.

The energy dissipated and, therefore, the equivalent viscous damping values for CBF systems with slender braces is relatively low compared to other structures. Goggins and Sullivan (2009) compared several EVD models from the literature (for example, models presented in Kowalsky 1994, Kwan and Billington 2003, Priestley *et al.* 2007) with equivalent viscous damping values computed from measured hysteretic loops obtained from shake table tests using Jacobsen's approach (Jacobsen 1960). They noted that, at the time, there was no equivalent viscous damping model developed specifically for use in the DDBD of CBFs. EVD expression proposed by Priestley *et al.* (2007) for the flag-shaped hysteretic rule with β=0.35 was selected for use within the DDBD approach, as this gave the closest estimates to the EVD values obtained from shake table tests. However, Goggins and Sullivan (2009) emphasised that this model was not developed for steel CBF structures and a new relationship should be developed as part of future work. A study carried out by Wijesundara (2009) proposed an EVD, ξ_{eq}, model for concentrically braced frames as a function of non-dimensional slenderness ratio, $\bar{\lambda}$, and the ductility, μ, as shown in the following equations

$$\xi_{eq} = 0.03 + \left(0.23 - \frac{\bar{\lambda}}{15}\right)(\mu - 1) \qquad \mu \leq 2 \tag{8}$$

$$\xi_{eq} = 0.03 + \left(0.23 - \frac{\bar{\lambda}}{15}\right) \qquad \mu \geq 2 \tag{9}$$

These expressions developed by Wijesundara (2009) were initially developed based on cyclic displacement pushover simulations using the software tool, Opensees, (McKenna *et al.* 2000). Fifteen SDOF CBF systems with different brace configurations were used. The area based approach proposed by Jacobsen (1960) was used to estimate the EVD value from the hysteretic loops. The normalised slenderness ratio, $\bar{\lambda}$, of brace members ranged from 0.44 to 1.6. All brace members had compact class 1 cross sections. The model was correlated to the results of twelve single storey braced frames which were obtained from an experimental programme conducted by Archambault (1995) on brace members subjected to displacement histories to replicate the behaviour of single storey CBFs. The brace member slenderness ratios ranged from 0.8 to 1.57. To validate the EVD values obtained from the area based approach, for each target relative lateral frame displacement, Wijesundara (2009) carried out NLHTA using seven real earthquakes scaled to an appropriate displacement spectrum for each frame.

Wijesundara (2009) recommended that if the brace normalised slenderness ratio was either below 0.4 or above 1.6, then these limits should be used in replace of the actual normalised slenderness ratio in Eqs. (8) and (9). In the current study, for which brace normalised slenderness ratios ranged from 1.5 to 2.9, no such limits were imposed. More details about the rationale behind this decision can be found in Goggins and Salawdeh (2013).

Table 1 and Fig. 3 compares various EVD models with equivalent viscous damping values computed from measured hysteretic loops obtained from shake table tests conducted by Elghazouli *et al*. (2005) using Jacobsen's area based approach (Jacobsen 1960). In particular, EVD values for a given measured ductility are obtained using the EVD expression proposed by Priestley *et al*. (2007) for the flag-shaped hysteretic rule with β=0.35 which was used for CBFs by Goggins and Sullivan (2009), EVD equations for CBFs proposed by Wijesundara (2009), which are given in Eqs. (8) and (9), and EVD values estimated by Goggins and Salawdeh (2013) from NLTHA simulations of the frames tested in the aforementioned shake table tests. It is evident from Fig. 3 that both the values obtained from the NLTHA (Goggins and Salawdeh 2013) and Eqs. (8) and (9) (Wijesundara 2009) give reasonable predictions of the measured equivalent viscous damping values. It is recommended to use the expressions in Eqs. (8) and (9) proposed by Wijesundara (2009), but without imposing upper limits on the normalised slenderness ratio, as these have been independently developed specifically for CBFs and match relatively well with values computed from shake table tests carried out by Elghazouli *et al*. (2005). For the shake table tests with braces having slenderness ratios of 1.49 and 1.58, there was a scatter in the results, as they had experienced full brace fracture during the shake table tests.

Table 1 Comparison of equivalent viscous damping values for single storey concentrically braced frames

Test ID	Slenderness ratio, $\bar{\lambda}$	Ductility μ	Equivalent viscous damping, ξ_{eq}%			
			Measured	NLTHA	Flag-shaped (β=0.35)	Eq. (9) (Wijesundara 2009)
ST1-R50H	2.21	2.39	9.09	4.23	6.4	11.27
ST2-E50H	2.23	7.71	11.21	6.42	8.39	11.13
ST4-R20H	2.78	4.72	3.59	2.89	8.08	7.47
ST5-E20H	2.87	6.59	7.34	6.04	7.69	6.87
ST5E20HB	2.87	8.02	5.51	3.96	8.63	6.87
ST7-R40H	1.49	14.93	6.18	4.67	8.82	16.07
ST8-E40H	1.58	13.83	7.08	6.12	7.9	15.47

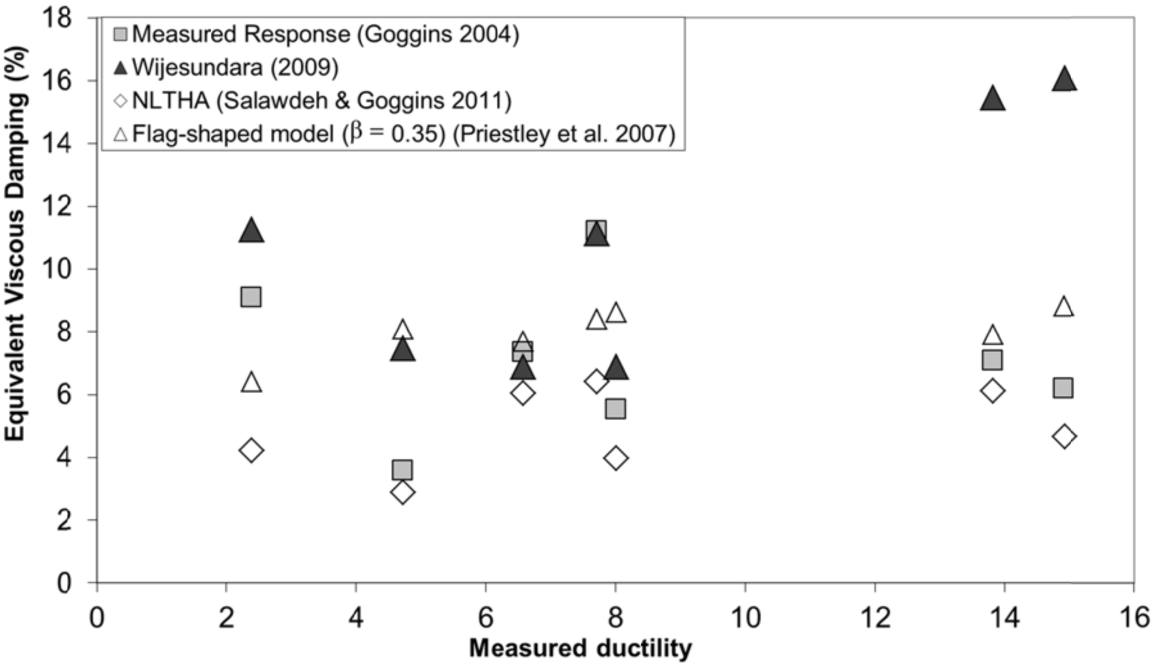


Fig. 3 Comparison of equivalent viscous damping values

2.4 Effective period of the structure

To find the effective period, the design displacement spectrum should be developed at the design level of damping. This can be achieved by applying a damping modifier, R_ξ, to the elastic displacement spectrum. In this work, the damping modifier expression used in the 1998 edition of EC8 has been adopted, as the DDBD methodology was carried out using this expression

$$R_\xi = \left(\frac{0.07}{(0.02+\xi)}\right)^{0.5} \tag{10}$$

The relative lateral frame design displacement, Δ_D, is then used to read off the required effective period, T_e, from the displacement spectrum developed at the design level of damping as shown in Fig. 4. T_c is the corner period of the elastic displacement response spectrum and Δc is the elastic displacement at the corner period of the spectrum.

2.5 Effective stiffness of substitute structure

With the effective period, T_e, established, the effective stiffness, K_e, is determined as

$$K_e = \frac{4\pi^2 m_e}{T_e^2} \tag{11}$$

where m_e is the floor mass and T_e is the effective period.

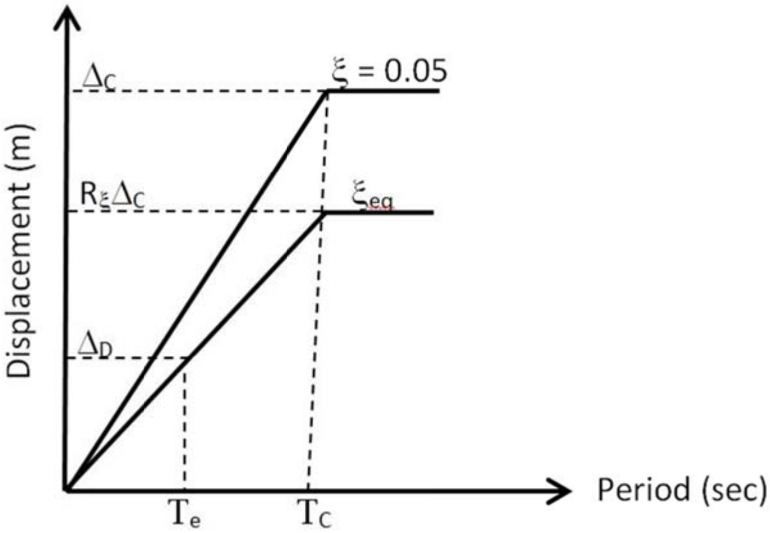


Fig. 4 Displacement response spectrum

2.6 Design base shear force

In the DDBD approach, the design base shear, F_b, required to limit the response to the target relative lateral frame displacement is obtained by multiplying the required effective stiffness, K_e, (Eq. (11)) by the design displacement, Δ_D, as shown

$$F_b = K_e \Delta_D \tag{12}$$

In the DDBD approach, it is recommended that the design base shear be increased to account for the reduction in effective lateral stiffness due to P-Δ effects (Priestley *et al.* 2007). The increase in the lateral force required to account for P-Δ effects in steel structures can be estimated as

$$F_{P-\Delta} = \frac{mg}{H_e} \Delta_D \tag{13}$$

where g is acceleration due to gravity and H_e is the effective height of the SDOF system. Thus, the base shear can be found by

$$F_b = \left(K_e + \frac{m_e g}{H_e}\right) \Delta_D \tag{14}$$

2.7 Brace cross-section size

EC8 (CEN 2004) suggested for frames with concentric bracings that tension diagonal bracings should be designed to resist the shear and no contribution in resistance is assumed by compression braces. In contrast, during the real-time shake table tests (Elghazouli *et al.* 2005), a percentage of the shear was found to be resisted by the compression member.

Assuming tension only members are resisting the base shear, Goggins and Sullivan (2009) estimated the cross-section of the brace members by using

$$A_b = \frac{F_b}{C f_y \cos\alpha} \tag{15}$$

where f_y is the actual yield stress obtained from monotonic tensile tests on the sections, α is the slope of the brace members to the horizontal, and C is an over strength ratio which accounts for the expected maximum brace resistance being higher than the yield stress due to strain hardening and the rate of strain. Goggins and Sullivan (2009) suggested to take C as 1.24 as they found that the maximum measured resistance of brace members in the shake table tests was on average 24% higher than the measured yield strength. Goggins and Sullivan (2009) mentioned that this factor can be modified to take into account the contribution of the compression brace to the lateral resistance of the structure. In this paper, the results from shake table tests (explained in the next Section) were re-investigated and the factor C from the different sources of overstrengths and the contribution of the compression members was found from the tests to range from 1.18 to 1.47 with an average value of 1.28.

For the frames tested in this paper, a trial to assign a percentage of the base shear to be resisted by the compression members was investigated. In this approach, full fracture of brace members occurred and caused collapse of the frame in physical tests ST7-E40H and ST8-E40H and in many frames during NLTHA while checking the sensitivity of the shake table tests to different earthquakes as discussed in Sections 3 and 4. For comparison reasons, the factor C that will be used for the design methodology in the following sections will be taken similar to values measured in the physical tests, to take into account the strain hardening and the contribution of the compression brace members, even though collapse of the test frame occurred during some shake table tests. Values of the factor C were different for each test as it depends mainly upon the slenderness ratio for brace members. However, for designing new structures, the authors suggest using the factor C as unity, which means designing the tension diagonal bracings to resist the shear assuming no contribution in resistance by compression braces, as suggested by EC8 (CEN 2004).

3. Comparison of DDBD with shake table tests

3.1 Shake table test set-up

Elghazouli *et al*. (2005) carried out shaking table tests in the Laboratory for Earthquake Engineering of the National Technical University of Athens (NTUA) on full-scale structures that represented an idealisation of a single-storey within a typical form of concentrically braced frame type of construction, in which the load-sharing between the tension and compression braces is accounted for. The test frame had an overall height of 2.89 m and plan dimensions of 2.70 m×2.47 m, as shown in Fig. 5.

The one-storey one-bay frames tested supported a mass of approximately 10,000 kg. Columns and beams were designed to behave elastically with pinned end connections in the direction of the earthquake. For each test, two cold-form structural steel hollow brace specimens were rigidly connected at their top ends to the bottom flange of the transverse beams, and at their lower ends to the table platform. The two braces are not connected at mid-length, and are separated in plan by an appropriate spacing to avoid contact during out-of-plane buckling. Slender brace specimens were used with different section sizes with a normalised slenderness ratio between 1.49 and 2.87.

Eight shake table tests are studied in this paper. Test ID's assigned to describe the tests were the same used by Goggins (2004) for ease of referencing. The first two letters 'ST' are abbreviation of shake table followed by the test number then a number which represents the nominal depth of the hollow steel section represented by 'H' when it is hollow or 'F' if the steel tube is filled with mortar. The frames, except ST5-20H, were subjected to scaled acceleration history from the Imperial Valley record of the El Centro earthquake. The original record, with a peak ground acceleration of about 0.34 g was appropriately scaled for each frame depending on the expected strength. Test frame ST5-20H was subjected to a synthetic record representing the range of dominant frequencies in an idealized EC8 spectrum. Detailed discussions of the results and observations from the shake table tests are found elsewhere (Goggins 2004, Elghazouli *et al.* 2005, Broderick *et al.* 2008). Data obtained from the tests are used to validate the DDBD procedure presented in this paper.

The shaking table test specimens provided in this paper are idealizations of CBFs, where the very important effect of brace-beam-column connections is not included in the test model. To consider real-world constructions as the final objective of the design methodology, additional shake table tests with a realistic gusset-plate connections connecting bracing members to the beams and columns should be carried out to validate the theoretical predictions.

3.2 Comparison with shake table tests

The DDBD methodology is performed for the shake table tests by using the target displacement as the maximum lateral relative frame displacement obtained during the tests. For each test, a unique design displacement spectrum is developed from the measured ground (shake table) motions for the design level of damping, as shown for example in Fig. 6 for ST2-E50H test. The design spectrum was found by applying a damping modifier, R_ξ, calculated using Eq. (10).

From this spectrum, the effective period corresponding to the design displacement can be obtained. Then, the effective stiffness and the base shear can be calculated. The results and a comparison between the base shear F_b and brace cross-sectional areas A_b obtained from the DDBD and the physical experiments are given in Table 2, where the estimated F_b and A_b were obtained

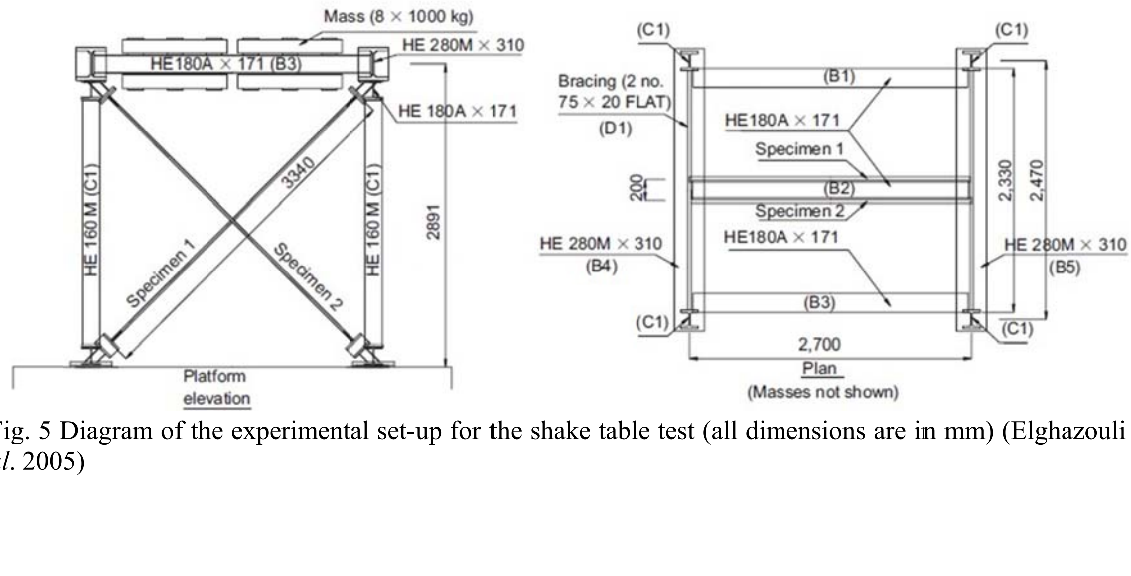


Fig. 5 Diagram of the experimental set-up for the shake table test (all dimensions are in mm) (Elghazouli *et al.* 2005)

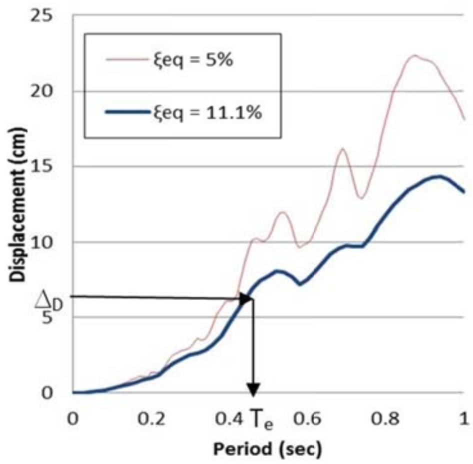


Fig. 6 Displacement spectrum for design of frame ST2-50H

Table 2 Results from DDBD and comparison to physical experimental results

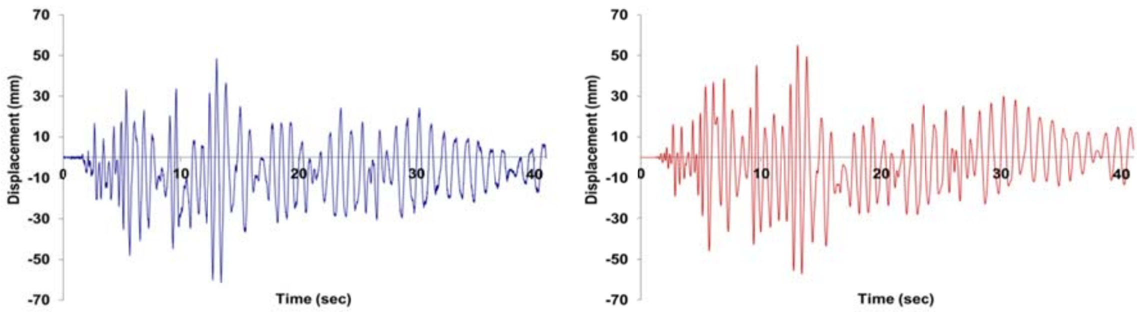
							Estimated		Measured		*Estimated/* Measured	
Test ID	f_y (N/mm^2)	Δ_y (mm)	$\bar{\lambda}$	Δ_d (mm)	μ	ξ_{eq} (%)	F_b (kN)	A_b (mm^2)	F_b (kN)	A_b (mm^2)	F_b	A_b
ST2-50H	333	7.56	2.23	61.5	8.14	11.1	123	430	96	334	1.29	1.29
ST3-50F	333	7.55	2.33	47.6	6.30	10.5	109	389	94	334	1.16	1.16
ST5-20H	377	8.56	2.87	59.9	7.00	6.9	52	133	53	135	0.99	0.99
ST5-20HB	435	9.87	2.87	83.5	8.46	6.9	59	151	54	138	1.09	1.09
ST6-20F	292	6.62	2.92	90.5	13.7	6.5	50	169	40	136	1.24	1.24
ST7-40H	358	8.14	1.49	128	15.8	16.1	109	338	113	351	0.96	0.96
ST8-40H	396	8.99	1.58	131	14.6	15.5	115	335	122	357	0.94	0.94
ST9-40F	507	11.5	1.83	33.9	2.94	13.8	156	369	151	357	1.04	1.04
Mean											1.09	1.09
C_v											0.12	0.12

using Eqs. (14) and (15). As can be seen from Table 2, the DDBD methodology proposed here gives a very good estimates of the required base shear strength and, hence, the required cross-sectional area. In fact, the mean and coefficient of variation (C_v) for the ratio of base shear (F_b) estimated from the DDBD to the measured values in shake table tests are 1.09 and 0.12, respectively.

4. Verification of DDBD methodology using NLTHA

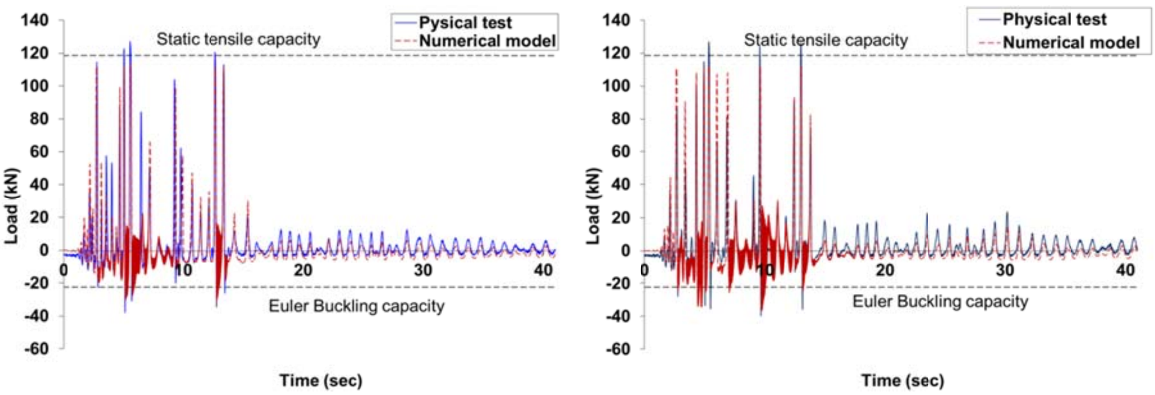
Another method to verify the DDBD methodology is the NLTHA. It is chosen here because it is one of the most accurate numerical methods to represent the inelastic performance of structures. The software used is OpenSees (McKenna *et al.* 2000). To assure that the numerical model is representing the real behaviour of CBFs, it was validated using physical tests. First, a study of the behaviour of brace members using pseudo-static cyclic tests was carried out to get a robust nonlinear beam column element model for steel structural rectangular hollow sections representing the cyclic behaviour of the brace elements (Salawdeh and Goggins 2013). This model is based on fibre elements incorporating a fatigue model that detect fracture due to low cyclic fatigue.

This model was advanced to get a robust numerical model for concentrically braced frames representing the shake table tests. Two-dimensional numerical models are carried out, in which columns and beams are modelled to behave elastically with a pinned end conditions. The hollow brace specimens are modelled as nonlinear beam column elements with fixed connections with the beam at their top ends, and to the ground at their lower ends. Tangent stiffness viscous damping model is adopted for the numerical simulations. This model is found to represent very well the behaviour of the shake table tests (for more details about the numerical model see (Goggins and Salawdeh 2013). Fig. 7 and Fig. 8 show an example of the comparison between displacement and axial force time history response of the physical test ST2-E50H and the numerical model results. As shown in Fig. 7, the numerical model could predict the displacement accurately for the first phase of the response. However, a significant difference is found in the second phase. It can be due



(a) physical test (b) numerical model

Fig. 7 Measured displacement time-history response of test ST2-E50H (Goggins and Salawdeh 2013)



(a) first Brace (b) second Brace

Fig. 8 Comparison for the axial load time-history between the physical test ST2-E50H and the numerical model (Goggins and Salawdeh 2013) to elongation that occurred to a brace member causing the frame to lean to one side which couldn't be captured in the numerical model. In general, these results are fairly accurate and assure that the NLTHA model can be used for different CBFs with different earthquakes with confidence.

For this study, frames ST2-50H, ST5-20H, ST5-20HB, ST7-40H and ST8-40H designed using the DDBD procedure described in Section 2 and having hollow steel braced sections are subjected to different earthquakes scaled to have displacement response spectrums compatible with the displacement spectrums used for the shake table tests. For example the displacement spectrum used for ST2-50H is shown in Fig. 9. As the numerical model was developed for steel hollow sections, NLTHA was not applied for the frames with steel filled sections in frames ST3-50F, ST6-20F and ST9-40F.

Eight accelerograms from four different earthquakes (2 components in orthogonal direction for each earthquake) are used. The records were selected by using a computer programme that first filters the whole PEER record database (PEER 2011) on user-defined selection criteria, then calculates an optimum linear scaling factor for all filtered records to best fit the target spectrum across a period range of interest, and finally ranks these records in order of root-mean-square error (Grant *et al*. 2008). For this study the earthquakes selected are shown in Table 3. It gives the Earthquakes with the ID's used, date of occurrence, PEER ID, the magnitude, M, and the epicentre distance, r.

Two alternatives are given in codes (CEN 2004, IBC 2012) for the number of accelerograms to be used in design verifications. The first involves using three spectrum-compatible accelerograms, with the design response being taken as the maximum from the three records for the given response parameter (e.g., displacement). The second approach uses a minimum of seven spectrum-compatible accelerograms, with the average value being adopted for the response parameter considered. This approach is almost always adopted and there appears to be a tendency to increase the number of records above the minimum of seven, to insure a more representative average (Priestley *et al*. 2007). Different numerical results for the response parameters are found for each accelerogram as the records are selected focusing on compatibility with the response spectrum following code guidance but not on seismological parameters. Accuracy in the prediction of mean and C_V of response parameters can be improved by adding more numerical analyses.

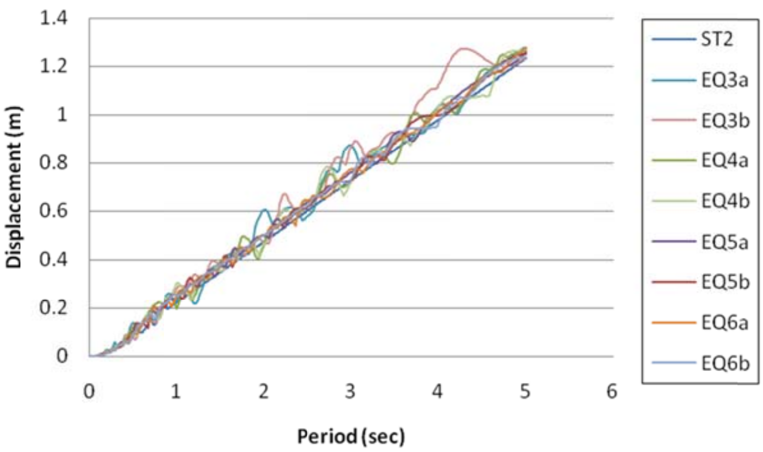


Fig. 9 Displacement response spectrum for eight accelerograms scaled to be compatible with the displacement spectrum for the earthquake used for ST2-50H test

Table 3 Properties of ground motions used

Earthquake	ID used	Date	PEER ID	M	r (km)
Northridge	EQ3a, EQ3b	Jan. 17, 1994	959	6.7	5
Imperial Valley	EQ4a, EQ4b	Oct. 15, 1979	169	6.5	34
Hector	EQ5a, EQ5b	Oct. 16, 1999	1762	7.13	48
Landers	EQ6a, EQ6b	Jun. 28, 1992	900	7.28	86

Table 4 Comparison of displacements and base shear obtained from NLTHA and DDBD

	Average value from NLTHA for 8 different accelerograms		DDBD		NLTHA/DDBD	
Test ID	Δ_{max} (mm)	F_b (KN)	Δ_{max} (mm)	F_b (KN)	Δ_{max}	F_b
ST2-50H	66.8	103	61.5	123	1.09	0.83
ST5-20H	43.5	60	41.7	52	1.04	1.16
ST5-20H-B	85.5	62	83.5	59	1.02	1.06
ST7-40H	135	110	128.0	109	1.05	1.01
ST8-40H	140.9	130	131.0	115	1.08	1.12
Mean					1.06	1.03
C_v					0.02	0.12

After carrying out the NLTHA for the frames, it is found that the average of the maximum displacements obtained from the time history analysis are very close to the values obtained from DDBD as seen in Table 4. Furthermore, the average values of the maximum base shear of the frames subjected to the eight accelerograms are close to the base shear values obtained from DDBD. The mean and coefficient of variation (C_v) for the ratio of the maximum displacement (Δ_{max}) estimated from the DDBD to values obtained from NLTHA are 1.06 and 0.02, respectively. Furthermore, the mean and C_v for the ratio of F_b estimated from the DDBD to the values obtained from NLTHA are 1.03 and 0.12, respectively. Accuracy in the prediction of mean and C_V of response parameters can be improved by adding more numerical analyses.

5. Conclusions

A direct displacement based design (DDBD) procedure for single-storey concentrically braced frames (CBFs) was presented. An equivalent viscous damping model developed specifically for CBFs by Wijesundara (2009) was validated using full scale shake table tests.

A DDBD methodology for single storey CBFs was validated using shake table tests and non-linear time history analysis (NLTHA). DDBD was performed for all the shake table tests with a target displacement equal to the maximum displacement from the shake table tests. It was found that estimated base shear forces and required brace cross sectional sizes predicted by the DDBD methodology match very closely to the actual values obtained from shake table tests.

NLTHA is used to verify the DDBD methodology using eight different accelerograms scaled to have displacement spectrum equal to displacement spectrums used for the DDBD. It was found that the average of the displacements and base shear obtained from NLTHA and the ones obtained from DDBD are very similar.

Thus, the DDBD methodology presented in this paper has been shown to give accurate and reliable results, which was proven by comparing predictions to data obtained from shake table tests and large range of NLTHA.

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References

Archambault, M.H. (1995), *Etude du comportement séismique des contreventements ductiles en X avec profiles tubulaires en acier*, EPM/GCS-1995-09, Department of Civil Engineering, école Polytechnique, Montréal, Que.

Broderick, B.M., A.Y., Elghazouli and J., Goggins (2008), "Earthquake testing and response analysis of concentrically-braced sub-frames", *J. Constr. Steel Res.*, **64**(9), 997-1007.

Calvi, G.M. and T.J. Sullivan (2009), "A model code for the displacement-based seismic design of structures", Pavia, Italy, IUSS Press.

CEN (2004), *Eurocode 8, design of structures for earthquake resistance* - Part 1: General rules, seismic actions and rules for buildings, EN 1998-1:2004/AC:2009.

Della Corte, G. (2006), "Vibration mode vs. collapse mechanism control for steel frames", *Proceeding of the Fourth International Conference on Behaviour of Steel Structures in Seismic Area (STESSA 2006)*, Yokohama, Japan.

Della Corte, G. and F.M., Mazzolani (2008), "Theoretical developments and numerical verification of a displacement-based design procedure for steel braced structures", *14th World Conference on Earthquake Engineering*, Beijing, China.

Della Corte, G., R., Landolfo and F.M., Mazzolani (2010), "Displacement-based seismic design of braced steel structures", *Steel Constr.*, **3**(3), 134-139.

Elghazouli, A.Y. (2010), "Assessment of European seismic design procedures for steel framed structures", *Bull. Earthq. Eng.*, **8**(1), 65-89.

Elghazouli, A.Y., B.M., Broderick, J., Goggins, H., Mouzakis, P., Carydis, J., Bouwkamp and A., Plumier (2005), "Shake table testing of tubular steel bracing members", *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, **158**(4), 229-241.

Garcia, R., T.J., Sullivan and G., Della Corte (2010), "Development of a displacement-based design method for steel frame-RC wall buildings", *J. Earthq. Eng.*, **14**(2), 252-277.

Goggins, J. (2004), "Earthquake resistant hollow and filled steel braces", Doctoral dissertation, Ph.D. thesis, Trinity College, University of Dublin.

Goggins, J. and S., Salawdeh (2013), "Validation of nonlinear time history analysis models for single-storey concentrically braced frames using full-scale shake table tests", *Earthq. Eng. Struct. Dyn.*, **42**(8), 1151-1170.

Goggins, J. and T., Sullivan (2009), "Displacement-based seismic design of SDOF concentrically braced frames", *Proceeding of STESSA 2009*, Philadelphia, USA.

Goggins, J.M., B.M., Broderick, A.Y., Elghazouli and A.S., Lucas (2006), "Behaviour of tubular steel

members under cyclic axial loading", *J. Constr. Steel Res.*, **62**(1-2), 121-131.
Goggins, J., B.M., Broderick and A.Y., Elghazouli (2006), "Recommendations for the earthquake resistant design of braced steel frames", *Proceeding of First European Conference on Earthquake Engineering and Seismology*, Geneva, Switzerland.
Grant, D.N., P.D., Greening, M., Taylor and B., Gosh (2008), "Seed record selection for spectral matching with RSPMatch", *The 14th World Conference on Earthquake Engineering*, Bejing, China.
IBC (2012), *2012 International Building Code*, Falls Church, VA, USA.
Jacobsen, L.S. (1960), "Damping in composite structures", *2nd World Conference on Earthquake Engineering*, Japan.
Kowalsky, M.J. (1994), "Displacement-based design-a methodology for seismic design applied to RC bridge columns", Master's thesis, University of California at San Diego.
Kwan, W.-P. and S.L., Billington (2003), "Influence of hysteretic behavior on equivalent period and damping of structural systems", *J. Struct. Eng.*, **129**(5), 576-585.
Málaga-Chuquitaype, C. and A.Y., Elghazouli (2011), "Consideration of seismic demand in the design of braced frames", *Steel Constr.*, **4**(2), 65-72.
Maley, T.J., T.J., Sullivan and G., Della Corte (2010), "Development of a displacement-based design method for steel dual systems with buckling-restrained braces and moment-resisting frames", *J. Earthq. Eng.*, **14**(S1), 106-140.
McKenna, F., G.L., Fenves and M.H., Scott (2000), "Object oriented program", OpenSees, Open system for earthquake engineering simulation, http//opensees.berkeley.edu.
Medhekar, M.S. and D.J.L., Kennedy (2000), "Displacement-based seismic design of buildings-application", *Eng. Struct.*, **22**(3), 210-221.
Medhekar, M.S. and D.J.L., Kennedy (2000), "Displacement-based seismic design of buildings-theory", *Eng. Struct*., **22**(3), 201-209.
Nip, K.H., L., Gardner and A.Y., Elghazouli (2010), "Cyclic testing and numerical modelling of carbon steel and stainless steel tubular bracing members", *Eng. Struct.*, **32**(2), 424-441.
PEER (2011), *Pacific Earthquake Engineering Research. PEER Strong Motion Database*, Available at http://peer.berkeley.edu/smcat/.
Priestley, M.J.N. (1993), "Myths and fallacies in earthquake engineering.-conflicts between design and reality", *Bull. NZ. Nat. Soc. Earthq. Eng.*, **26**(3), 329-334.
Priestley, M.J.N. (2003), "Myths and fallacies in earthquake engineering, revisited", Mallet Milne lecture, IUSS Press. Pavia, Italy.
Priestley, M.J.N. and D.N., Grant (2005), "Viscous damping in seismic design and analysis", *J. Earthq. Eng.*, **9**(sup2), 229-255.
Priestley, M.J.N., G.M., Calvi and M.J., Kowalsky (2007), "Displacement-based seismic design of structures", IUSS Press, Pavia, Italy.
Remennikov, A.M. and W.R., Walpole (1997a), "Analytical prediction of seismic behaviour for concentrically-braced steel systems", *Earthq. Eng. Struct. Dyn.*, **26**(8), 859-874.
Salawdeh, S. and J., Goggins (2013), "Numerical simulation for steel brace members incorporating a fatigue model", *Eng. Struct.*, **46**, 332-349.
Shaback, B. and T., Brown (2003), "Behaviour of square hollow structural steel braces with end connections under reversed cyclic axial loading", *Can. J. Civ. Eng.*, **30**(4), 745-753.
Tang, X. and S.C., Goel (1989), "Brace fractures and analysis of phase I structures", *J. Struct. Eng*., **115**(8), 1960-1976.
Tremblay, R. (2002), "Inelastic seismic response of steel bracing members", *J. Constr. Steel Res.*, **58**(5-8), 665-701.
Tremblay, R., Timler, P., Bruneau, M. and A., Filiatrault (1995), "Performance of steel structures during the 1994 Northridge earthquake", *Can. J. Civ. Eng.*, **22**(2), 338-360.
Wijesundara, K.K. (2009), "Design of concentrically braced steel frames with RHS shape braces", Doctoral dissertation, Ph.D. thesis, European Centre for Training and Research in Earthquake Engineering (EUCENTRE).

Wijesundara, K.K., R., Nascimbene and T.J., Sullivan (2011), "Equivalent viscous damping for steel concentrically braced frame structures", *Bull. Earthq. Eng.*, **9**(5), 1535-1558.

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