Comparing the seismic performance of concentrically braced frames with and without self-centering behaviour

G.J. O'Reilly & J. Goggins
National University of Ireland, Galway, Ireland.

ABSTRACT: Conventional concentrically braced frames (CBFs) are designed to provide lateral resistance to earthquake loading through axial resistance of the brace member, as well as providing energy dissipation during earthquake loading through tensile yielding and inelastic buckling of its bracing members. These cycles of inelastic deformation lead to the possibility of residual deformations being present in a structure that has been designed to current seismic code provisions. This paper presents a new self-centering concentrically braced frame (SC-CBF) where a conventional CBF is combined with a post-tensioning (PT) arrangement to give a system that will self-center after earthquake loading. The behaviour and layout of the SC-CBF is first described, followed by the numerical model used to analyse the performance of the system. An example SC-CBF is then designed for a set of performance goals, and their performance is analysed using a suite of design spectrum compatible ground motions. The same structure is then designed using a conventional CBF lateral resisting system and a similar analysis is carried out. The results of the two systems are then compared to show the benefits of using a SC-CBF over a conventional CBF.

1 INTRODUCTION

An investigation into the effects of residual drifts in Japan (McCormick *et al.*, 2008) concluded that once the residual drifts in a structure exceed 0.5%, the occupants of the building begin to experience nausea, headaches and an overall hindrance on daily life. It was concluded during this study that for 12 case study steel-framed buildings following the Hyogoken-Nanbu earthquake in 1995, where the residual interstorey drifts exceeded 0.5%, it was more financially viable to demolish and rebuild the structure rather than attempt to repair it due to the repair costs and economical losses that would be incurred with the building closed during the repair period. While attempts have been made to incorporate the occurrence of residual drifts in structures subjected to earthquakes into the design procedure to ensure excessive residual drifts do not occur, the use of self-centering systems is becoming more popular, which consists of a post-tensioning (PT) arrangement along with a energy dissipating mechanism to give a structure that will re-center after loading.

During the early 1990's, a joint initiative was undertaken in the US and Japan to develop a new innovative way of constructing concrete wall and frame systems subject to earthquake loading. This gave rise to the concept of post-tensioning these wall and frame systems in various ways to achieve a new self-centering behaviour to mitigate the unwanted residual displacements caused by ductile behaviour during the seismic event. This concept has been well developed for concrete systems throughout the 1990's and 2000's. In the early 2000's, the concept was beginning to be applied to steel systems and this led to the development of various self-centering mechanisms (Christopoulos *et al*. 2002; Clayton *et al*. 2012).

This paper presents a newly developed self-centering concentrically braced frame (SC-CBF) system, which consists of a conventional concentrically braced frame (CBF) together with a PT arrangement to give a CBF that will self-center after a seismic event. That is, it will return to its original vertical position after the occurrence of an earthquake. The behaviour and modelling of the SC-CBF are discussed, followed by the design and numerical analysis of a SC-CBF and a CBF to compare the performance of the two systems and to show the added benefits of using a self-centering system.

2 SELF-CENTERING CONCENTRICALLY BRACED FRAME

2.1 *Mechanics and behaviour*

This section presents the mechanics and behaviour of the SC-CBF. Figure 1 shows the general layout of a single storey SC-CBF. This consists of a conventional CBF combined with a PT element running between the flanges of the beams and anchored at the exterior columns. The system combines the hysteretic behaviour of the conventional CBFs with a bilinear elastic response of the PT system to give a ‘flag shaped’ hysteretic loop, which is characteristic of all self-centering systems, as shown in Figure 2. The achievement of this loop strongly depends on the slenderness of the braces and the level of post-tensioning force applied to the system. A detailed description of the mechanics and behaviour of the SC-CBF has been presented in O’Reilly *et al.* (2012a), where a set of analytical expressions are derived describing the effect of each parameter of the SC-CBF on the shape of the hysteretic loop.

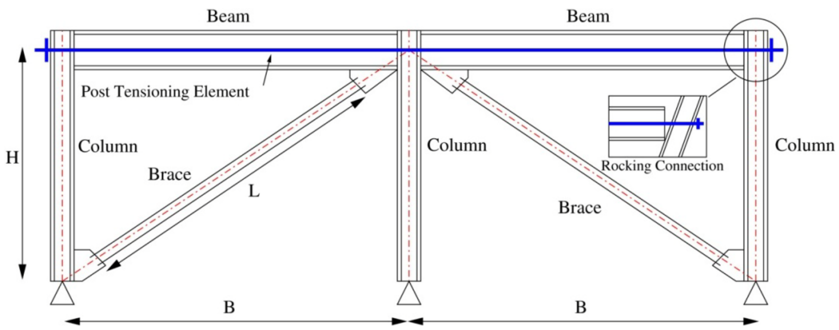


Figure 1. General arrangement of a SC-CBF.

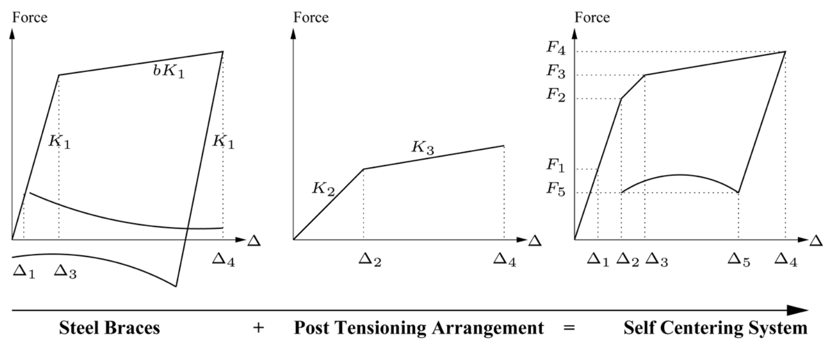


Figure 2. Combination of hysteresis rules for the SC-CBF.

2.2 *Numerical modelling of the SC-CBF*

A numerical model developed using the OpenSees framework (McKenna *et al.*, 2000) is used to represent the behaviour of the SC-CBF. The development of the model used for the SC-CBF is described in O’Reilly *et al.* (2012a), where the analytical expressions presented in this paper are compared with results from a pushover of a single storey SC-CBF numerical model. The model uses the same parameters as for conventional CBFs for the bracing members such as initial camber to encourage the brace members to buckle during loading, but uses a different connection model to capture the rocking beam-column connection employed in the SC-CBF. This connection model is illustrated in Figure 3.

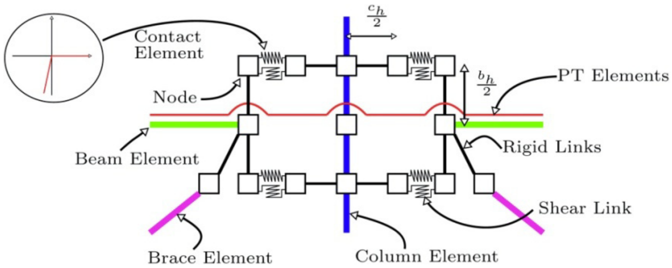


Figure 3. Rocking connection model used for the SC-CBF.

3 PERFORMANCE-BASED DESIGN OF A SC-CBF AND CBF

3.1 *Introduction*

This section presents the design of a SC-CBF to a specified level of hazard, and using the numerical model of the SC-CBF described in Section 2.2, the performance of the SC-CBF is assessed through a series of nonlinear time-history (NLTH) analyses using a set of spectrum compatible ground motions. A similar conventional CBF system is then designed for the same hazard level and the performance of this design is assessed in the same way.

The design method used to design these frames is first described as this method, known as direct displacement-based design (DDBD) (Priestley *et al.*, 2007), is a relatively new method for designing braced frame systems for seismic loading. This method is first presented generally, followed by the details that apply specifically to the CBF and SC-CBF systems. The design goals are then presented for each system for the given level of seismic hazard and these design goals are then used to evaluate the performance of the two systems from the NLTH analyses.

3.2 *Direct displacement-based design*

DDBD has been the subject of much research throughout the past decade and has become a more prominent tool in seismic design. The design method differs to the force based design (FBD) approaches, which are employed in many seismic design codes internationally, in that a target displacement is set at the start of the design and governs the design process rather than being a check at the end of the design process, as with FBD. The method is presented generally here so that the key concepts are understood, with the more specific details for the SC-CBF and CBF systems also included. For a more in-depth description of this design approach, Priestley *et al.* (2007) provides a detailed description of the design of many different structure types.

Figure 4 shows the basic concept of the DDBD method. First the multi-storey structure is modelled as an equivalent single degree of freedom (SDOF) system with an effective stiffness (K_e) as in Figure 4(b). The design displacement (Δ_d) is then specified and from knowledge of the buildings layout and material type, the yield displacement (Δ_y) can be determined. This gives a target displacement ductility (μ_Δ), and knowing the type of system being used, the equivalent viscous damping (EVD) (ξ) can be determined from Figure 4(c). Using Δ_d and ξ, the effective period of the system (T_e) can be determined from the elastic spectral displacement, as per Figure 4(d). From this, the design base shear can be determined and the structure designed and detailed accordingly.

The DDBD of CBFs has been investigated previously by other researchers (Goggins & Salawdeh, 2012; Wijesundara, 2009), and the details for the DDBD are hence not discussed in detail here. What is of importance is the expression used to calculate the EVD in a CBF system. Goggins & Sullivan (2009) also noted that at the time of publication, no expression existed for the EVD of CBF systems. Since then, Wijesundara (2009) has proposed an expression which relates the amount of EVD provided to the normalised slenderness and ductility of the braces used. Goggins & Salawdeh (2012) has since used this expression in the design of CBFs as part of a separate study and demonstrated the satisfactory prediction of EVD using this expression. The expression is as shown in Equation 1, where λ is the normalised slenderness of the brace. This expression can then be used in the DDBD of the CBFs, but for DDBD of the SC-CBF, a slight modification needs to be made. Since the SC-CBF is composed of two separate systems, a yielding CBF system and a PT system, the EVD expression needs to be adjusted to account for this. This is achieved by combining the EVD expression for CBFs, given in Equation 1, with an expression for the PT system. Since there is no energy dissipation in the PT, there is only an elastic damping term, which is assumed to be 5%. The combined EVD for the system is then weighted by the ratio of storey shear resisted by each system, and the total EVD for the equivalent SDOF is given in Equation 2, where V_i and Δ_i are the storey shear and displacement, and κ is the percentage of the base shear resisted by the PT system.

$$\xi = \begin{cases} 0.03 + (0.23 - \lambda/15)(\mu - 1) & if \quad \mu \le 2 \\ 0.03 + (0.23 - \lambda/15) & if \quad \mu > 2 \end{cases} \tag{1}$$

$$\xi_{SC-CBF} = \frac{\sum_{i=1}^{N} V_i \Delta_i (1-\kappa) \xi_{CBF,i} + \sum_{i=1}^{N} V_i \Delta_i \kappa \xi_{PT,i}}{\sum_{i=1}^{N} V_i \Delta_i} \tag{2}$$

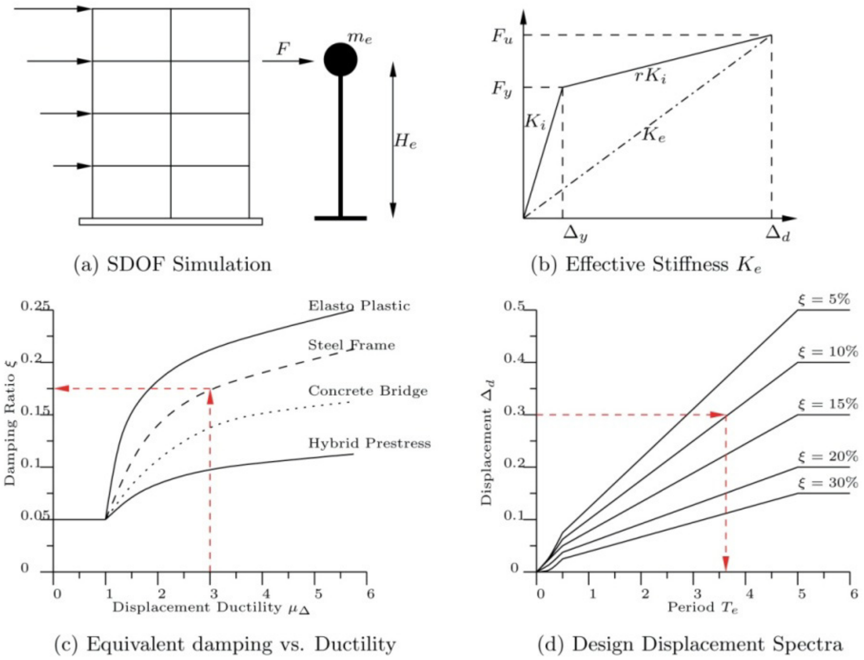


Figure 4. Fundamentals of DDBD (Adopted from Priestley *et al.* (2007)).

3.3 *Design goals*

In order to evaluate the performance of the system when conducting the NLTH analyses, we need to establish some design goals and performance criteria. This consists of setting out acceptable limits for the SC-CBF to be designed to, such as maximum interstorey drift and brace ductility. The performance limit states for both the SC-CBF and the CBF are shown in Table 1.

The acceptable residual drift limit is based on the construction tolerance for out of plumbness, as discussed in O'Reilly *et al*. (2012). The condition that the beams and columns remain elastic is self-explanatory as these elements are not intended to be a part of the energy dissipating mechanism, and are required at all times for resisting gravity loads. The brace ductility parameter is adopted from experimental and analytical work carried out by Nip *et al*. (2010), where the fracture ductility (μ_f) of a brace is related to its global and local slenderness. The condition set here is for the maximum brace ductility to be less than the fracture ductility to ensure ductile response of the braces. The interstorey drift parameters for the two systems were initially set at 2.5%, but due to the brace ductility exceeding the brace fracture ductility, this was reduced to 2.0%.

Table 1. Performance goals.

Parameter	SC-CBF	CBF
Interstorey drift	2.0%	2.0%
Residual drift	0.2%	0.2%
Beams	Elastic	Elastic
Columns	Elastic	Elastic
PT Elements	Elastic	N/A
Braces	$\mu_d < \mu_f$	$\mu_d < \mu_f$

3.4 *Building description and ground motions*

The structure that is to be designed using the above design method and performance goals is a 3 storey structure. Figure 5 shows a layout for the SC-CBF, with two SC-CBFs resisting the seismic force in each direction. The CBF structure has an identical building structure topography.

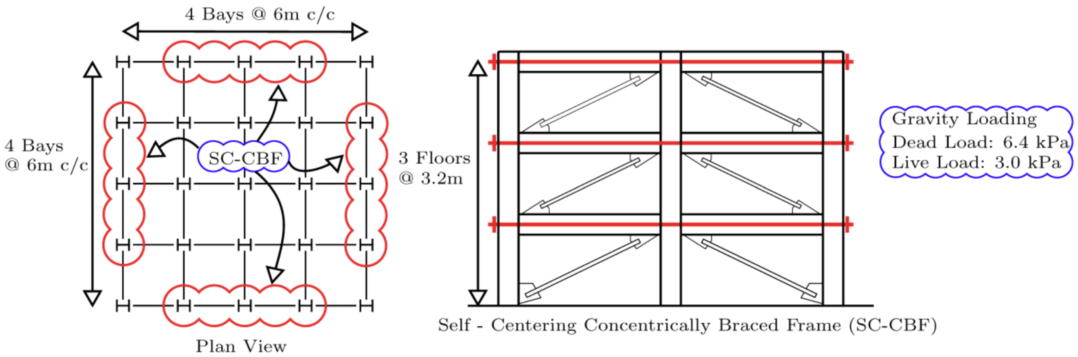


Figure 5. Layout of 3-storey building.

The design spectrum employed in the design of these systems is based on the example parameters outlined in Priestley *et al.* (2007) for a probability of exceedence of 10% in 50 years. The reason this spectrum was chosen was so that the other ordinates corresponding to the 2% probability of exceedence in 50 year design level could also be used and compared in a performance based design investigation. The resulting design displacement spectrum is shown in Figure 6 along with the chosen ground motions used in the NLTH analyses, where the average of these ground motions fits relatively well with the design spectrum. These ground motions were obtained using the PEER NGA database for the design spectrum specified, and the details of these ground motions are listed in Table 2.

Table 2. Ground motion details.

EQ No.	Earthquake Record	Station	PEER ID	Scale
EQ1	Imperial Valley-06	El Centro Diff. Array	184	1.170
EQ2	Imperial Valley-06	Delta	169	1.614
EQ3	Loma Prieta	Saratoga - Aloha Ave	802	1.465
EQ4	Superstition Hills-02	El Centro Imp. Co. Cent	721	1.751
EQ5	Gazli- USSR	Karakyr	126	0.947
EQ6	Chi-Chi- Taiwan	TCU089	1521	1.777
EQ7	Chi-Chi- Taiwan	TCU076	1511	1.341

3.5 *Resulting designs for SC-CBF and CBF systems*

Using the design method and design spectrum described in Sections 3.3 and 3.4, the two structures are designed to meet the performance criteria set out in Table 1. The resulting designs are shown in Table 3 and Table 4 for the SC-CBF and CBF, respectively. Steel grade S275 was used for brace members and S355 was used for the beams and columns. The seismic mass used in design consisted of the total dead load plus 24% of the imposed load, as per Eurocode 8 (Wijesundara, 2009). Initial eigenvalue analyses gave the first mode period to be 0.467s for the SC-CBF and 0.543s for the CBF.

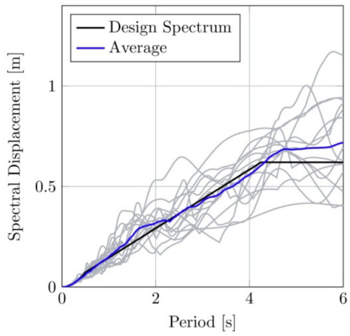


Figure 6. Design displacement spectrum and ground motions.

Table 3. SC-CBF design details.

Storey	Brace	Beams	Columns	No. Strands	PT Force [kN]
3	90x90x8SHS	W24x9x76	W10x10x68	16	312
2	120x120x10SHS	W24x9x94	W12x12x120	24	577
1	140x140x10SHS	W24x9x128	W12x12x152	26	675

Table 4. CBF design details.

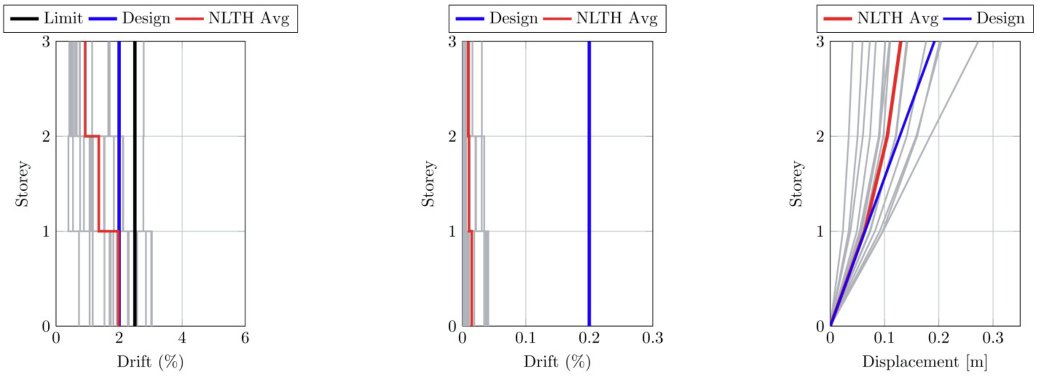
Storey	Brace	Beam	Column
3	90x90x8SHS	W16x7x36	W8x8x58
2	120x120x10SHS	W16x7x40	W10x10x60
1	140x140x10SHS	W16x7x45	W10x10x77

3.6 *NLTH analysis results and discussion*

The resulting design of the SC-CBF and CBFs was then evaluated using a series of NLTH analyses as previously outlined. This was achieved using the OpenSees framework for which the connection model presented in Section 2.2 was employed. The use of 3% tangent stiffness proportional damping was assumed for the dynamic analysis. In addition, the PDelta effects were modelled using a "PDelta column" to take into account the gravity loading acting during the earthquake. Other specific details have been omitted here due to space constraints.

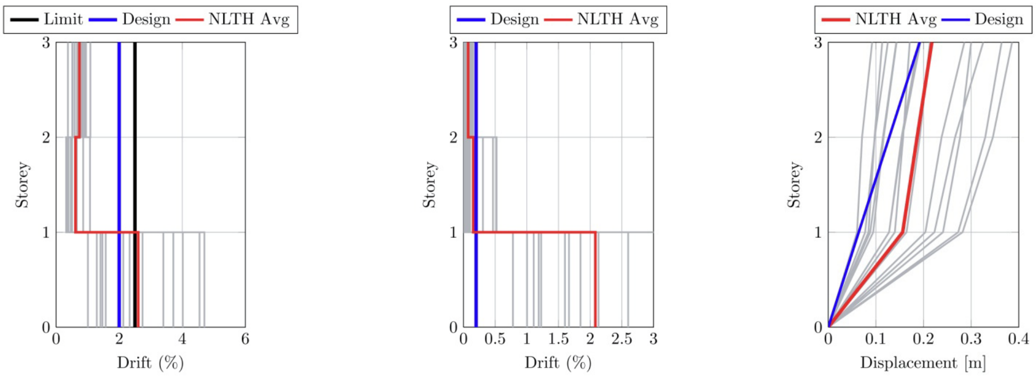
Findings from the NLTH analyses for the SC-CBF are shown in Figure 7. Table 5 also gives the utilisation ratios for all the other elements showing the compliance with the initial performance goals, where the utilisation ration (η) represents the ratio of the demand to the capacity of the member, and the subscripts N, M and V represent the axial, moment and shear forces, respectively. It can be seen from Figure 7 and Table 5 that the performance limits set out in Table 1 have all been met with the drifts less that maximum allowed and all beams, column and PT elements remaining elastic.

The NLTH results for the corresponding CBF system are shown in Figure 8. Comparing the response of the CBF to the performance goals, it can be seen that the CBF exceeds the design drift in the first storey of the structure and also has a residual drift far beyond the acceptable limit in the same storey. This is due to the formation of a soft storey and the influence of PDelta effects on that storey. Residual displacements have been shown to be a direct function of the post yield stiffness of the system (Kawashima *et al*., 2008), and it can be seen in the case of the CBF that the relatively low post-yield stiffness leads to the occurrence of such residual displacements. Comparing these observations in the CBF with the corresponding SC-CBF, it can be seen no such soft-storeys formed and no residual drifts were present in the SC-CBF. This is due to the fact that the SC-CBF has a flag shaped loop, which will re-center the structure after loading, but also that the post-yield stiffness is much higher in the SC-CBF due to the PT system. An added benefit of the SC-CBF is that this post-yield stiffness, denoted K_3 in Figure 2, can be easily increased or decreased by adding or removing more PT strands to the system. The details of how this occurs can be found in O'Reilly *et al*. (2012b). It should also be noted that as a soft storey mechanism formed in the CBF, the brace ductility exceeded the predicted fracture ductility given by the expressions in Nip *et al*. (2010). This indicated that brace fracture is very probable in this storey, which could possibly lead to structural collapse. For this study, the design process was carried out once, whereas in reality, the CBF structure would be redesigned with larger brace members to prevent this soft storey from forming.



(a) Interstorey drift (b) Residual drift (c) Displaced shape

Figure 7. SC-CBF NLTH results.



(a) Interstorey drift (b) Residual drift (c) Displaced shape

Figure 8. CBF NLTH results.

4 CONCLUSIONS

The mechanics and behaviour of a new self-centering CBF have been introduced, along with the modelling and behaviour under cyclic loading. An example 3 storey SC-CBF was then presented and designed to specific performance limits under a design levels earthquake. The performance of this system was then assessed through the NLTH of a numerical model of the designed SC-CBF. These NLTH results were then compared with the performance limits to demonstrate the satisfactory performance of the SC-CBF under the design level earthquake. A similar structure was then designed using a conventional CBF system and analysed in the same way as the SC-CBF. These NLTH results showed the forming of large residual drifts and possibility of brace fracture in the bottom storey of the conventional CBF system. This is clearly unsatisfactory performance of the CBF when comparing with the initial performance goals. Comparing the results of the two systems demonstrates the superior performance of the SC-CBF to the CBF. This is also important when considering the post-earthquake occupancy of the structure, as since no residual deformations are present, no re-straightening of the structure is necessary, hence the downtime is minimal compared with the CBF.

ACKNOWLEDGEMENTS

The first author would like to acknowledge the funding provided by the Irish Research Council.

Table 5. Utilisation ratios (η) for NLTH analyses.

	Storey	CBF				SC-CBF			
		η_M	η_N	η_V	$\eta_M + \eta_N$	η_M	η_N	η_V	$\eta_M + \eta_N$
Beams	1	0.44	0.39	0.08	0.83	0.09	0.27	0.00	0.36
	2	0.50	0.13	0.08	0.63	0.12	0.36	0.01	0.48
	3	0.57	0.05	0.07	0.62	0.09	0.30	0.01	0.39
Columns	1	0.36	0.39	0.01	0.74	0.20	0.18	0.13	0.38
	2	0.46	0.27	0.02	0.71	0.26	0.23	0.18	0.49
	3	0.14	0.14	0.00	0.28	0.55	0.24	0.15	0.79
		η				η			
PT	1	N/A				0.82			
	2	N/A				0.70			
	3	N/A				0.74			
		μ_f		μ_d		μ_f		μ_d	
Braces	1	7.12		8.26		7.23		7.75	
	2	7.85		1.95		8.19		5.30	
	3	8.53		2.34		9.45		3.62	

REFERENCES

Clayton, P., Berman, J. & Lowes, L. (2012). Seismic design and performance of self-centering steel plate shear walls. *Journal of Structural Engineering*. 138(1):22-30.

Christopoulos, C., Filiatrault, A., Uang, C.M., & Folz, B. (2002) Posttensioned energy dissipating connections for moment-resisting steel frames. *Journal of Structural Engineering*, 128 (9):1111-1120.

Goggins, J. & Sullivan, T. 2009. Displacement-based seismic design of SDOF concentrically braced frames. *In Proceedings of 6th International Conference on Behaviour of Steel Structures in Seismic Areas*. Philadelphia, Pennsylvania, USA.

Goggins, J. & Salawdeh, S. 2012. Validation of nonlinear time history analysis models for single-storey concentrically braced frames using full-scale shake table tests. *Earthquake Engineering & Structural Dynamics*, DOI: 10.1002/eqe.2264.

Kawashima, K., MacRae, G.A., Hoshikuma, J. & Nagaya, K. 1998. Residual displacement response spectrum. *Journal of Structural Engineering*, 124(5):523-530.

McCormick, J., Aburano, H., Ikenaga, M. & Nakashima, M. 2008. Permissible residual deformation levels for building structures considering both safety and human elements. *In Proceedings of the 14th World Conference on Earthquake Engineering.* Beijing, China.

McKenna, F., Fenves, G., Filippou, F. & Mazzoni, S. 2000. Open system for earthquake engineering simulation (OpenSees).

Nip, K.H., Gardner, L. & Elghazouli, A.Y. 2010. Cyclic testing and numerical modelling of carbon steel and stainless steel tubular bracing members. *Engineering Structures*, 32(2).

O'Reilly, G.J., Goggins, J. & Mahin, S.A. 2012a. Behaviour and design of a self-centering concentrically braced steel frame system. *In Proceedings of the 15th World Conference on Earthquake Engineering*. Lisbon, Portugal.

O'Reilly, G.J., Goggins, J. & Mahin, S.A. 2012b. Performance-based design of a self-centering concentrically braced frame using the direct displacement-based design procedure. *In Proceedings of the 15th World Conference on Earthquake Engineering*. Lisbon, Portugal.

Priestley, M.J.N., Calvi, G.M. & Kowalsky, M.J. 2007. *Displacement-Based Seismic Design of Structures*. IUSS Press, Pavia, Italy.

Wijesundara, K.K. 2009. Design of Concentrically Braced Steel Frames with RHS Shape Braces. PhD thesis, European Centre for Training and Research in Earthquake Engineering (EUCENTRE), Pavia, Italy.