EXPERIMENTAL TESTING OF A NOVEL SELF-CENTRING STEEL BRACED FRAME ON THE SHAKE TABLE IN DYNLAB-IZIIS

J. Goggins[1], Y. Jiang[2], B.M. Broderick[3], S. Salawdeh[4], G.J. O'Reilly[5], A.Y. Elghazouli[6], H. Alwahsh[7], A. Bogdanovic[8], Z. Rakicevic[9], I. Gjorgjiev [10], A. Poposka[11], B. Petreski [12], I. Markovski [13]

[1] *Jamie Goggins, Principal Investigator and Senior Lecturer, MaREI Centre, Ryan Institute & School of Engineering, National University of Ireland Galway, jamie.goggins@nuigalway.ie*

[2] *Yadong Jiang, Postdoctoral Researcher, MaREI Centre, Ryan Institute & School of Engineering, National University of Ireland Galway, yadong.jiang@nuigalway.ie*

[3] *Brian M. Broderick, Professor, Civil Structural & Environmental Engineering, Trinity College Dublin, bbrodrck@tcd.ie*

[4] *Suhaib Salawdeh, Lecturer, Galway Mayo Institute of Technology, suhaib.salawdeh@gmit.ie*

[5] *Gerard J. O'Reilly, Assistant Professor, Scuola Universitaria Superiore IUSS di Pavia, Pavia, gerard.oreilly@iusspavia.it*

[6] *Ahmed Y. Elghazouli, Professor of Structural Engineering, Department of Civil and Environmental Engineering, Imperial College London, a.elghazouli@imperial.ac.uk*

[7] *Hatim AlWahsh, PhD Researcher, National University of Ireland Galway, H.Alwahsh1@nuigalway.ie*

[8] *Aleksandra Bogdanovic, Assoc. Prof. Dr. Institute of Earthquake Engineering and Engineering Seismology – IZIIS University "Ss. Cyril and Methodius", saska@iziis.ukim.edu.mk*

[9] *Zoran Rakicevic, Prof. Dr. Institute of Earthquake Engineering and Engineering Seismology – IZIIS University "Ss. Cyril and Methodius", zoran_r@iziis.ukim.edu.mk*

[10] *Igor Gjorgjiev, Assoc. Prof. Dr. Institute of Earthquake Engineering and Engineering Seismology – IZIIS University "Ss. Cyril and Methodius", igorg@iziis.ukim.edu.mk*

[11] *Angela Poposka, MSc, Institute of Earthquake Engineering and Engineering Seismology – IZIIS University "Ss. Cyril and Methodius", angela@iziis.ukim.edu.mk*

[12] *Borjan Petreski, MSc, Institute of Earthquake Engineering and Engineering Seismology – IZIIS University "Ss. Cyril and Methodius", borjan@iziis.ukim.edu.mk*

[13] *Igor Markovski, Electrical Engineer, Institute of Earthquake Engineering and Engineering Seismology – IZIIS University "Ss. Cyril and Methodius", igorm@iziis.ukim.edu.mk*

Abstract

Seismic design codes seek to limit structural displacements in order to minimise damage, but no explicit consideration is usually given to the state of the structure after the earthquake, which can be critical for re-occupancy, monetary losses and the rapidity of repair. In this paper, shake table testing of single-storey self-centring concentrically braced frame (SC-CBF) models subjected to a variety of seismic actions is presented. The aim of these tests is to assess the effectiveness of a novel self-centring system in controlling global deformation demand in CBFs, including the sensitivity of the SC-CBF response to ground motion characteristics. This system employs a post-tensioning arrangement that ensures the structure self-centres following an earthquake to minimise residual drift. A pair of brace members act as an energy dissipation system through ductile axial deformation in tension and compression. Brace member cross-section dimensions are varied between experiments to investigate a range of structural properties relevant for European design practice. A bolted connection detail facilitated the replacement of the damaged brace members after each experiment. In each experiment, table excitations are scaled to achieve increasing displacement demand and the peak drifts and residual drift are measured. The test results, including residual frame/brace deformations and rocking connection gap-openings, demonstrate the favourable self-centring characteristics of the SC-CBF under realistic earthquake excitations.

Keywords: Self-Centring System; Concentrically Braced Frame; Shake Table Test; Steel Structures; Residual Drifts

1 Introduction

In the past few decades, the seismic performance of concentrically braced frames (CBFs) has been investigated in many studies [1-11]. Among these studies, the peak inter-storey drift of the structure, which is directly related to the damage, is of main concern. However, less attention has been given to the CBF residual displacement after earthquake events. The post-earthquake condition of a structure is important for the re-occupancy of the building, monetary losses and the speed of repairs/modifications. To minimise the residual deformations of CBFs, the concept of a self-centring steel braced frame (SC-CBF) system was proposed by O'Reilly *et al.* [12, 13, 14]. This innovative form has the advantage of a self-centring (SC) system, namely, that the structure returns back to its original position after the seismic event, while using brace members as the primary energy-dissipating components during the earthquake. Hence, the application of a SC-CBF could reduce the residual drifts following large earthquake events and make it easier to replace damaged braces. Therefore, a building comprising a SC-CBF as its lateral load-resisting system may be reoccupied immediately after an earthquake and lead to reduced indirect monetary losses resulting from prolonged downtime.

Fig. 1 shows the concept of a single-storey two-bay SC-CBF. The major differences between SC-CBFs and conventional CBFs are the connections and post-tensioned strands. Different from the beam-column connections used in CBFs, where a non-negligible flexural resistance is typical, columns of SC-CBFs are pinned at the base, while beams are connected to columns via rocking connections. Fig. 2 shows the schematic of a rocking connection, which is similar to the connections used by Christopoulos [15] and Clayton *et al.* [16], among others. With the local vertical and horizontal movement locked, the beam is allowed to rock around the panel zone, and thus, damage to beam and column members is avoided. The lateral stiffness of the rocking connection is mainly provided by the post-tensioned strands, which are installed along the centre line of the beams. The initial compressive forces in the post-tensioned strands restrains the rocking connection to make the SC-CBF initially behave essentially as a moment resisting frame (MRF). After an earthquake event, the strands cause the gap of the rocking connection to close and, thus, centre the frame back to its initial position. The beams, columns and post-tensioned strands of SC-CBFs are capacity designed to remain in the elastic range and undamaged under rocking, and the brace members are the only energy-dissipating components. With proper brace design, gap-opening in the rocking connections will cause plastic deformation of the braces and, thus, dissipate hysteretic energy. Benefiting from the combination of rocking connections and post-tensioned strands, the beams and columns are protected, and the residual displacement is minimised under earthquake loading. After an earthquake event, only the replacement of the damaged brace members is required, which can reduce the economic losses caused by repairs and shortens expected downtime.

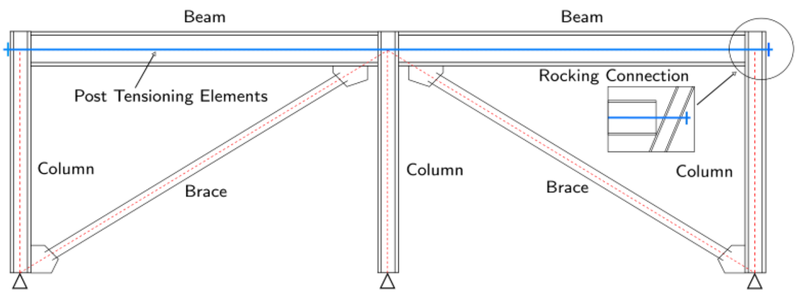

Fig. 1 – Schematic of a single storey SC-CBF (adapted from O'Reilly [14])

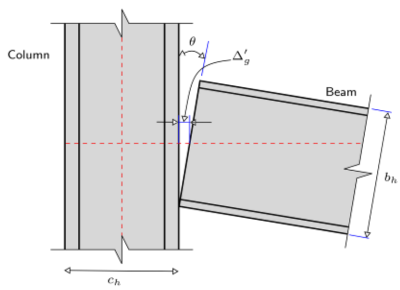

Fig. 2 – Beam-column rocking connection (adapted from O'Reilly [14])

In the research work of O'Reilly [14], a single-storey SC-CBF (Fig. 3) was designed and tested under cyclic lateral loading. The design involved the careful selection of combinations of brace sizes and post-tensioning forces necessary to create an effective self-centring system. Four types of square hollow section (SHS) braces, with different cross-section dimensions, were considered as the SC-CBF energy-dissipating members. The study revealed the SC-CBF flag-shaped lateral force versus drift ratio hysteretic curve (Fig. 4), which generally aligned well with the theoretical behaviour. Most of the imposed energy was shown to be dissipated by the braces.

Fig. 3 – SC-CBF tested under cyclic static lateral loading (adapted from O'Reilly [14])

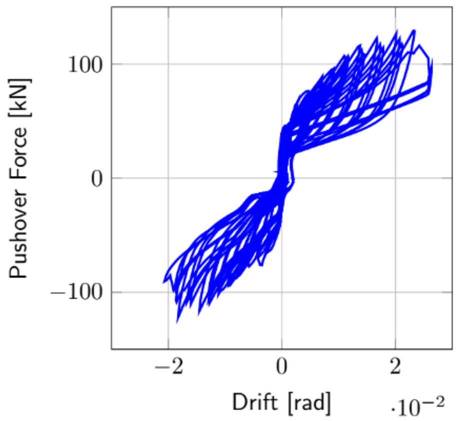

Fig. 4 –Lateral force versus drift ratio plot of test B3A (adapted from O'Reilly [14])

The pushover tests characterised the cyclic behaviour of the SC-CBF under static lateral loading, but no axial column loading was applied. Moreover, the pushover tests were carried out under displacement control, which could not directly monitor the frame self-centring behaviour under earthquake excitation. Therefore, an extensive study was initiated in order to characterise the dynamic seismic behaviour of SC-CBFs under real earthquake events. In this research, the SC-CBF designed by O'Reilly *et al.* [12, 13, 14] was modified to be incorporated into a 3D model placed on the shake table in the Institute of Earthquake Engineering and Engineering Seismology (IZIIS), R. North Macedonia. With two ground motions selected, a series of shake table tests were carried out under different scale factors. The seismic behaviour and self-centring performance of the SC-CBF structure are investigated and discussed in this paper.

2 Shake Table Testing

2.1 Structural Details

The single-storey structure tested in this research consists of one middle frame and two external frames. As shown in Fig. 5, the middle frame is a two-bay SC-CBF, which is a modified version of the frame designed and tested by O'Reilly [14] consisting of two braced bays with post-tensioned strands installed at the upper and lower beam levels and three column members extended beyond the beam levels to facilitate connection to the test mass and shake table. The beam and column members used in the middle frame were selected from European H and IPE sections, with the aim to maintain consistency with the sections used in O'Reilly [14], in terms of dimensions and steel grades. The frame middle column was connected to the storey mass and shake table via two pinned connections, whilst the north and south columns were connected using slot connections. With a roller plugged in the slot, this connection could fix the roller vertical displacement, but allow a relative horizontal movement of ±30 mm. This combination of two connection types ensured that the inertial force from the mass block is only transferred to the SC-CBF through the centre pin during an earthquake excitation. As indicated in Fig. 6, the beams were connected to the columns via rocking connections. The beam end could rock against the face of the column flange via two slotted connections.

During rocking action, large local stresses were anticipated in the contact surfaces of the beam and column flanges. Therefore, it was necessary to enhance the strength of the steel sections around the rocking connection. For this purpose, steel plates and stiffeners were welded to the steel profiles webs and flanges. The quantity and the location of post-tensioned strands used in O'Reilly [14] were followed. Regarding the nominal diameter of the strands, the size used in the shake table tests increased from the 12.3 mm diameter strands used in the push-over tests to 15.3 mm due to availability of post-tensioned strands. However, the initial post-tension forces of the strands were kept as 80 kN.

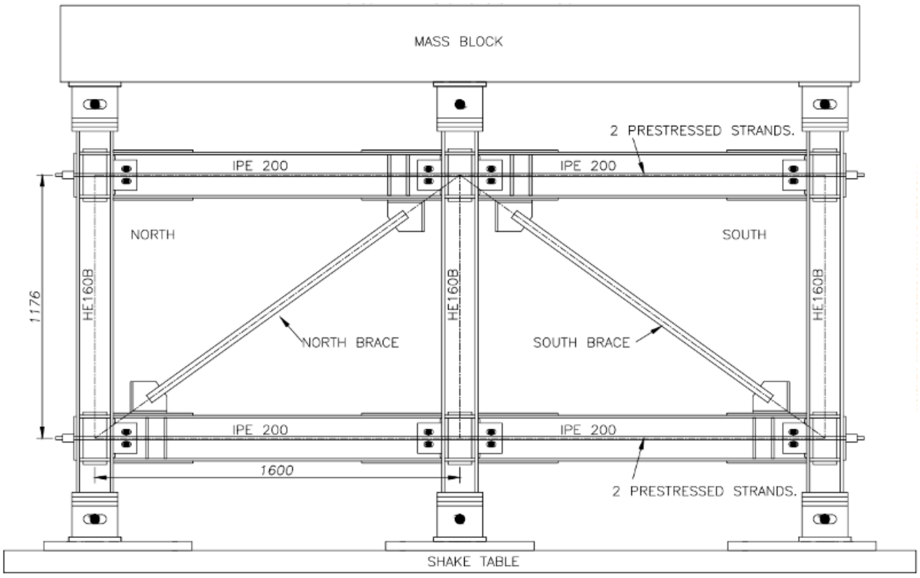

Fig. 5 – Schematic of the SC-CBF system utilised in the shake table tests

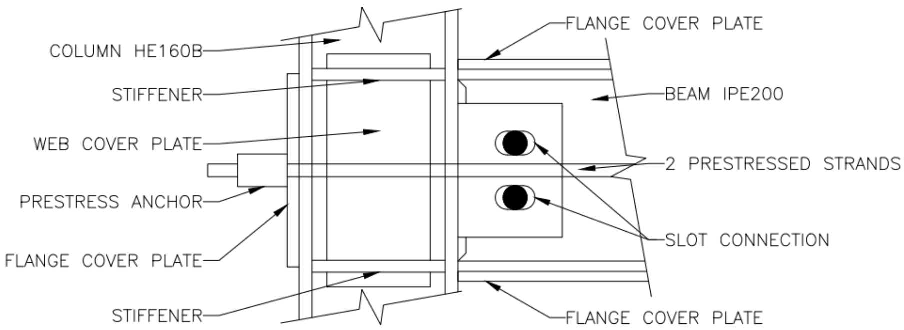

Fig. 6 – Detail of rocking connection

Fig. 7 shows the geometry of the external frames. As indicated, all components were connected through pinned connections. Thus, the external frame can provide gravity resistance to the roof weight, but does not contribute to the lateral stiffness of the structure. The two external frames were connected to the middle frame via beam and brace members, which formed a rigid diaphragm at roof level and ensured the three frames experience similar lateral displacements. As shown in Fig. 8, the structure was mounted on the 5×5 m shake table in the Institute of Earthquake Engineering and Engineering Seismology (IZIIS), R. North Macedonia. Steel ingots, with an approximate total mass of 20 ton, were placed and fixed on the roof. Under an earthquake excitation, all of the inertial force due to the motion of this mass was transferred to the SC-CBF through the pin joint at the top of the central column. It should be noted that the temporary braces installed on the external frames (Fig. 8) were removed before testing started.

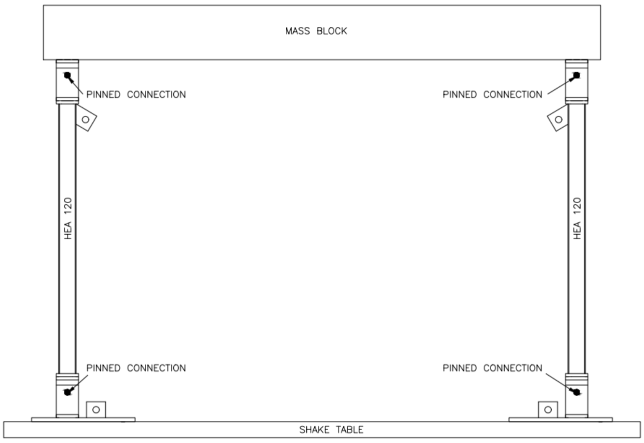

Fig. 7 – Schematic of the external frames

Fig. 8 – Overall view of the SC-CBF system (north direction is from left to right)

2.2 Brace Specimens

Two braces, the major energy-dissipating members of the structure, were concentrically installed in the SC-CBF. As shown in Fig 9, each brace specimen consisted of one SHS member and two gusset plates. The gusset plate was connected to the beam flange via four bolts, which facilitated the replacement of the damaged braces after each experiment. An additional 10 mm thick steel plate was welded to the beam flange to avoid flange local failure. Four sizes of SHS braces, manufactured in accordance with EN 10025:2004 [17], were selected for testing based on the properties of the brace members investigated by O'Reilly [14]. Table 1 summarises the brace details. The normalised brace slenderness ranged from 1.03 to 2.21, which covers the range of slenderness values permitted in Eurocode 3 [18]. The gusset plates were designed using conventional design methods and a vertical stiffener was provided for each plate to replicate the boundary condition of a conventional gusset plate, as outlined by O'Reilly [14].

Table 1 – Brace geometries

ID	*b* [mm]	*t* [mm]	*L* [mm]	Number
B40×40	40	4	1395	2
B30×30	30	3	1433	4
B25×25	25	2.5	1435	4
B20×20	20	3	1438	4

b is the nominal cross-section width of the SHS member
t is the steel wall nominal thickness of the SHS member
L is the length of the SHS member (excluding the gusset plates)

Fig. 9 – SHS brace and gusset plates

2.3 Ground Motions

Two real ground motion records with distinctive characteristics were selected for testing. The time-history plots and their elastic acceleration response spectra are displayed in Fig. 10 and Fig. 11, respectively. The time history of ground motion 1 (‘GM 1’) is roughly symmetric distributed around 12 s. Under the excitation of GM 1, the structure could be considered to suffer two similar events consecutively. Thus, under the same PGA, GM 1 will introduce more energy to the structure compared to ground motion 2 (‘GM 2’). However, GM 2 has a sudden acceleration drop at 10 s, which can give the structure a pulse excitation. The frequency content of GM 1 matches the expected natural frequency of the test frame better than GM2, but it is also more narrow-banded. The broader bandwidth of GM2 will lead to greater force and displacement demands after brace yielding during strong motion. A primary objective of the experimental programme is to validate the performance of the self-centring system under for these contrasting ground motion conditions. It should be noted that the high-frequency and low-frequency components of the two selected ground motions were filtered according to the shake table technical specifications.

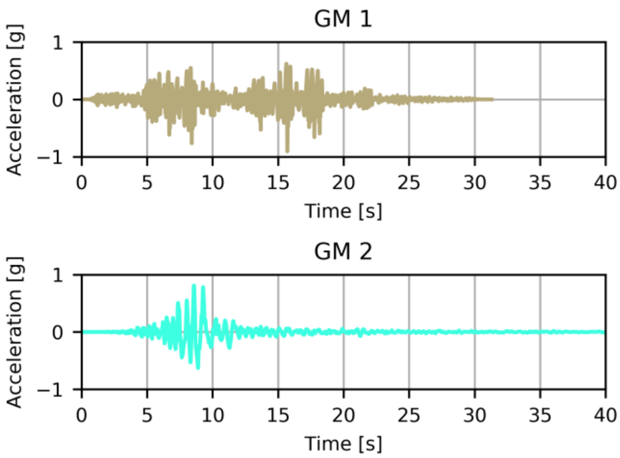

Fig. 10 – Acceleration time-history of the selected ground motions

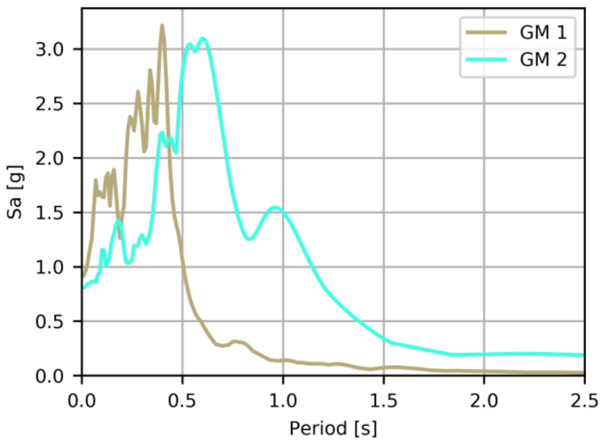

Fig. 11 – Elastic response spectrums of the two selected ground motions (at 5% damping)

2.4 Instrumentation

One main objective of using SC-CBFs in a seismic zone is to avoid damage of the beams and columns. Therefore, strain gauges were placed on the beam and column flanges to monitor the member yielding, but also to estimate the force distribution in the structure during the earthquake. The braces are the only energy-dissipating members of the structure and their responses, in terms of strain values and elongation, were monitored by strain gauges and displacement transducers. Regarding the north brace, strain gauges were installed at mid-span, at the two ends and on the two gusset plates. For the south brace, there were only four strain gauges installed at mid-span. The elongation of each brace was measured by a linear variation displacement transducer (LVDT) (Fig. 9). The accelerations at roof level, post-tensioned strand levels and table level were measured by accelerometers connected to the beam or column. The absolute displacements at the upper/lower beam levels were measured by linear string potentiometers. The recorded data can hence capture the structural storey drift. To measure the rocking mechanism performance, two LVDTs were installed at the top and bottom beam flanges to measure the gap-opening (Fig. 12). LVDTs were also used to monitor the roller movements to ensure that the rollers do not hit the slot edge and cause connection damage. The tensile forces in each of the four post-tensioned strands were directly monitored by load cells. The shake table displacements and acceleration time-histories were also measured by displacement transducers and accelerometers in 3 directions that are mounted on the shake table.

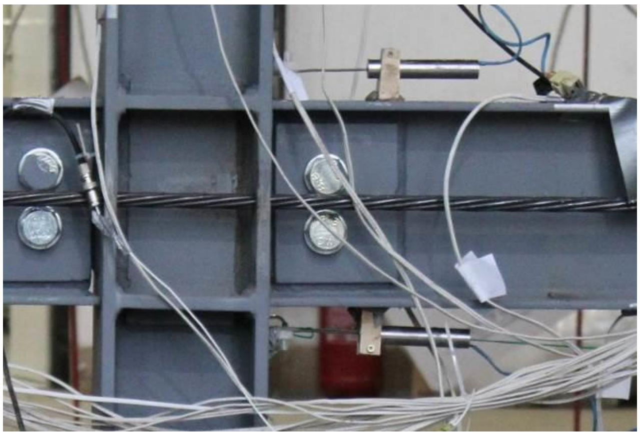

Fig. 12 – LVDTs for measuring the gap-opening of the rocking connection

2.5 Testing Programme

The shake table tests were carried out in the DYNLAB, IZIIS, Skopje, R. North Macedonia. The testing programme details, including the brace pair, the ground motion and the corresponding PGA, are listed in Table 2. The tests were divided into 7 series according to the brace pair installed. Limited by the shear capacity of the centre pin and the shake table overturning moment, the brace B40×40, which was the strongest specimen of the tests, was not yielded during testing. Thus, in the first series with braces B40×40, two tests, corresponding to the two ground motions, were performed to validate the feasibility of the structure boundary conditions, the connections and the instrumentation.

Regarding Test Series 1 ~ 6, the first two or three tests were carried out with ascending ground motion scale factors until the braces approached yielding. To be more specific, each test series started with a relatively small earthquake excitation (PGA ≈ 0.1 g). This test would not cause any brace damage and, thus, could be used to obtain the roof acceleration amplification factor for the initial structural response and validate the brace instrumentation. Following the first test of each series, more tests were performed with amplitude scale factors carefully adjusted until the brace reached its yield limit state. The ground motion amplitude scale factors were calculated based on the peak strain values measured in the previous test and the brace yield strain. After the yield displacement and the corresponding scale factor were identified, the last test, aiming to impose permanent deformation into the braces, was conducted. The scale factor used in this test was determined according to the target lateral displacement (nominally four times the yielding displacement), but could also be limited by the overturning moment capacity of the shake table.

Table 2 – Testing programme and summary of results

Series #	Test #	Brace	Pair #	GM #	PGA [g]	Max. Drift Ratio [%]	Residual Drift Ratio [%]
Series 0	1	B40×40	1	GM 1	0.36	0.36	0.022
	2	B40×40	1	GM 2	0.43	N.A.	0.004
Series 1	1-1	B20×20	1	GM 1	0.10	0.15	0.013
	1-2	B20×20	1	GM 1	0.18	0.27	0.004
	1-3	B20×20	1	GM 1	0.26	0.36	0.015
	1-4[1,2]	B20×20	1	GM 1	0.57	0.76	0.078
Series 2	2-1	B20×20	2	GM 1	0.10	0.10	0.016
	2-2	B20×20	2	GM 1	0.21	0.22	0.016
	2-3[2]	B20×20	2	GM 1	0.45	0.59	0.022
	2-4[3]	B20×20	2	GM 1	0.41	0.93	0.035
Series 3	3-1	B25×25	1	GM 1	0.10	0.10	0.001
	3-2	B25×25	1	GM 1	0.25	0.24	0.038
	3-3	B25×25	1	GM 1	0.48	1.19	0.002
Series 4	4-1	B25×25	2	GM 2	0.09	0.09	0.005
	4-2	B25×25	2	GM 2	0.22	0.17	0.016
	4-3	B25×25	2	GM 2	0.43	0.40	0.007
	4-4	B25×25	2	GM 2	0.84	2.51	0.027
Series 5	5-1	B30×30	1	GM 1	0.09	0.07	0.006
	5-2	B30×30	1	GM 1	0.25	0.24	0.012
	5-3[4]	B30×30	1	GM 1	0.50	1.10	0.0004
Series 6	6-1	B30×30	2	GM 2	0.09	0.09	0.003
	6-2	B30×30	2	GM 2	0.24	0.17	0.014
	6-3[4]	B30×30	2	GM 2	0.62	0.61	0.002
	6-4[4]	B30×30	2	GM 2	0.68	1.23	0.061

[1]Test was terminated at 14s
[2]Slight slippage was observed between the mass block and the SC-CBF
[3]Slight slippage between the mass block and the SC-CBF was prevented
[4]Damage was observed at the upper south rocking connection

3 Shake Table Testing Results

3.1 General Observations

During testing, the structure produced significant sounds, which were caused by joint movements during the severe seismic excitations. In Tests 1-4 and 2-2, slight slippage between the roof and the central pin of the SC-CBF was observed. To address this, additional welds were placed onto the connection. In Tests 5-3, 6-3 and 6-4, the south upper beam was observed to have slight vertical movement at its connection to the column flange, which indicated some damage to this rocking connection. Due to the locations of the post-tensioned strands, this damage to the rocking connection could not be fixed within the test window available. Consequently the global lateral stiffness of the south bay of the SC-CBF was reduced and hence the north brace dissipated most of the energy. Visual checks and the measurements of the LVDTs verified that none of the rollers in the slot connections exceeded their movement allowance in any test. Benefiting from the post-tensioned strands, the SC-CBF structure was observed to position itself back to its vertical position after each test, which verified the self-centring behaviour of the SC-CBF.

3.2 Brace Failure and Energy Dissipation

As shown in Fig. 13, significant buckling was observed at the end of the final test in each series except Series 0. It should be noted that due to the self-centring behaviour of SC-CBF, the frame brings the gusset plates back to their original position and reduces global brace buckling deformations. Regarding the beam and column members, flange yielding was not observed and the measured strains values did not exceed the yield strain. Therefore, it can be concluded that the SC-CBF behaved as designed and the braces dissipated most of the earthquake energy as expected.

(a) Test 2-4 (b) Test 4-4 (c) Test 5-3

Fig. 13 – Global brace buckling deformations

3.3 Rocking Connection

Rocking behaviour was observed at the beam-column connections in the last one or two tests of Series 1 ~ 6, which demonstrated that the designed rocking connections behaved efficiently. This was further verified by the scratches observed on the column flange and the slot connections (Fig. 14). Rocking connection gap-opening measurements are plotted in Fig. 15. For Test 4-4, with a maximum drift ratio of 2.51%, the gap-opening was up to 4 mm. No post-tensioned strand damage was observed in any of the tests. However, the strand force had a reduction of around 7 kN after Test 4-4, which was mainly attributable to the temporary strand anchors used in the model SC-CBF. From Fig. 15, it can also be observed that the gaps closed at the end each excitation, which demonstrated that the forces offered by the post-tensioned stands could elastically restrain the rocking connection and, thus, shift the SC-CBF back to its original vertical position.

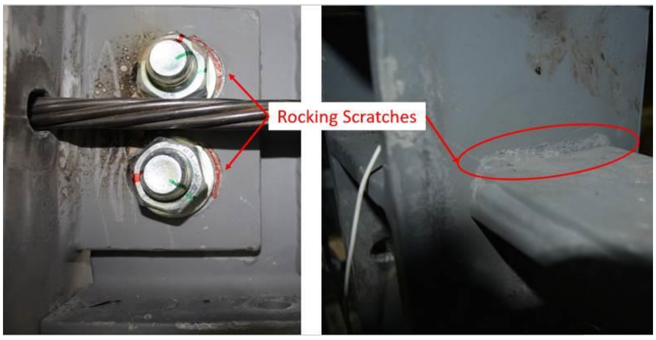

Fig. 14 – Scratches caused by rocking

3.4 Inter-Storey Drift Ratio

The maximum inter-storey drift ratio and residual drift ratio observed in each test are summarised in Table 2 and the inter-storey drift ratio plots from three selected tests are shown in Fig. 16. For all the tests, the residual inter-storey drift ratios were less than 0.1%, which is less than the residual drift ratio limit of 0.2%, suggested by Calvi and Sullivan [19]. This indicates that even when the peak drift ratio reached 2.5% and the braces were seriously damaged, the residual displacement of the SC-CBF structure could be effectively controlled by the combination of rocking connections and the post-tensioned strands.

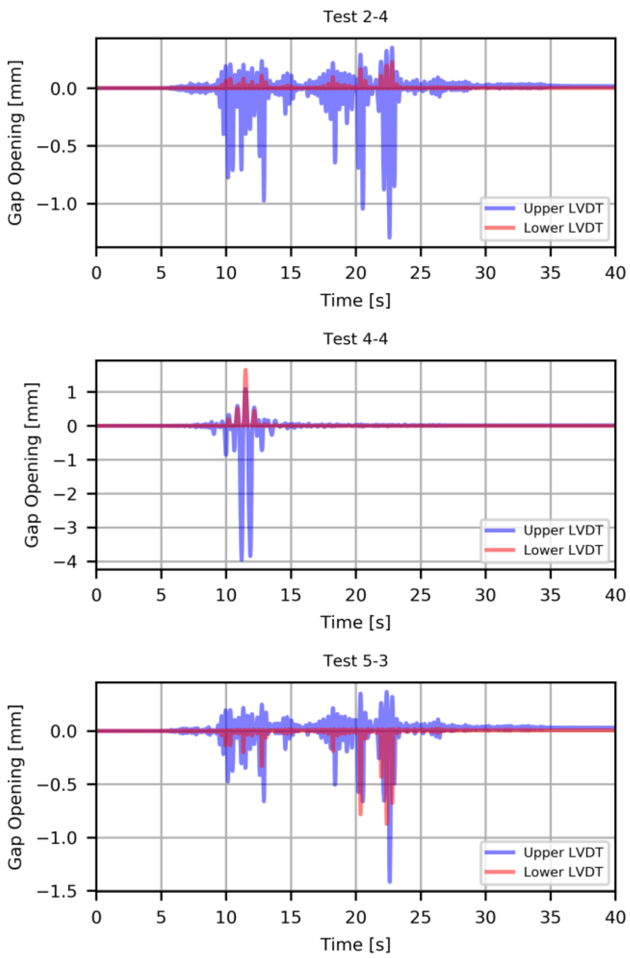

Fig. 15 – Gap opening of Tests 2-4, 4-4 and 5-3

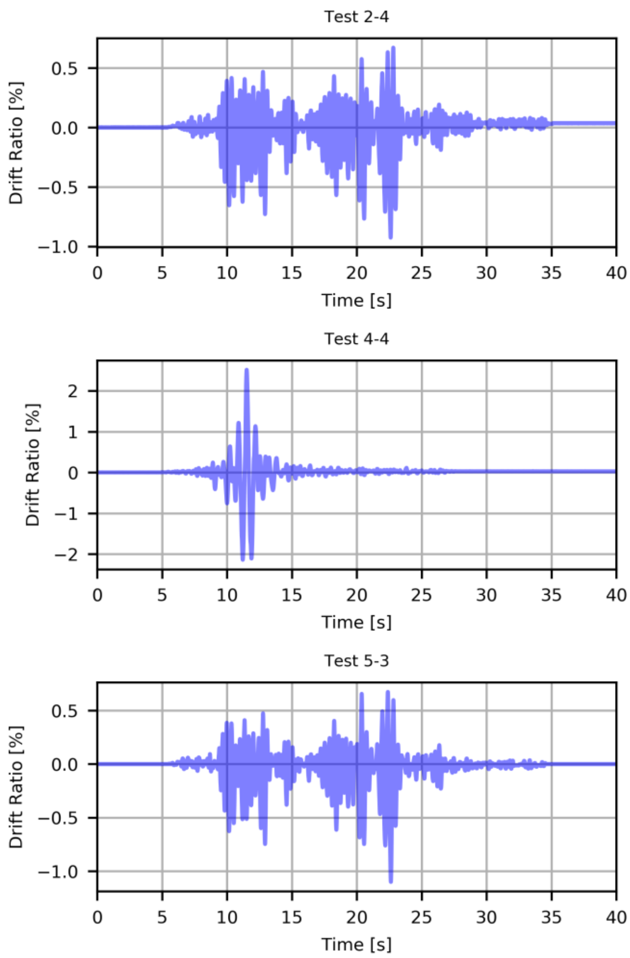

Fig. 16 – Drift ratio of Tests 2-4, 4-4 and 5-3

4 Conclusion and Future Work

In this research, a series of shake table tests, aiming to characterise the seismic behaviour of SC-CBF structures, were carried out. Based on the research conducted by O'Reilly *et al.* [12, 13, 14], a single-storey SC-CBF structure was designed and manufactured for shake table testing. Seven pairs of SHS braces with four cross-section sizes were prepared. Two real earthquake records with very different time-history and frequency characteristics were selected and filtered. Instrumentations, including accelerometers, displacement transducers, load cells and strain gauges were installed to monitor the structural behaviour of the SC-CBF. In total, seven series of shake table tests were conducted sequentially with different ground motion scale factors. Based on the experimental observations and results presented in this paper, the following main conclusions can be made:

- The proposed rocking connection effectively protected the beams and columns from notable damage under earthquake loading. Consequently, most of the imposed energy was dissipated by the braces of the SC-CBF.
- By utilising the post-tensioned strands and rocking connections, the SC-CBF exhibited excellent self-centring behaviour under earthquake excitations and eliminated residual drifts even for large peak inter-storey drift demands.
- The damaged brace members were easily replaced between experiments, restoring the full resistance of the SC-CBF after strong earthquake loading with large displacement demands.

5 Acknowledgements

This research was funded by the SERA - Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (SERA-H2020-INFRAIA-2016-2017/H2020-INFRAIA-2016-1) under grant agreement No. 730900 for the project 'Investigation of Seismic Deformation Demand, Capacity and Control in a Novel Self-Centring Steel Braced Frame (SC-CBF)'. The first author would like to acknowledge the support of Science Foundation Ireland through the Career Development Award programme (Grant No. 13/CDA/2200) and the MaREI Centre (Grant No. 12/RC/2302_2). Additional thanks are given to the technical staff at IZIIS, Skopje, R. North Macedonia.

6 References

[1] Goggins J (2004): Earthquake resistant hollow and filled steel braces. *PhD thesis*, University of Dublin, Trinity College, Dublin, Ireland.

[2] Goggins J, Broderick BM , Elghazouli AY, Lucas AS (2005): Behaviour of tubular steel members under cyclic axial loading. *Journal of Constructional Steel Research*, 62 (1-2), 121–131.

[3] Goggins J, Broderick BM , Elghazouli AY, Lucas AS (2005): Experimental cyclic response of cold-formed hollow steel bracing members. *Engineering Structures*, 27 (7):977–989.

[4] Elghazouli AY, Broderick BM, Goggins J, Mouzakis H, Carydis P, Bouwkamp J, Plumier A (2005): Shake table testing of tubular steel bracing members. *Institute of Civil Engineers: Structures and Buildings*, 158(SB4):229 – 241.

[5] 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*, 42(8), 1151-1170.

[6] Salawdeh S, Goggins J (2013): Numerical simulation for steel brace members incorporating a fatigue model. *Engineering Structures*, 46, 332-349.

[7] Salawdeh S, Goggins J (2016): Performance based design approach for multi-storey concentrically braced steel frames. *Steel and Composite Structures*, 20(4): 749-776.

[8] Salawdeh S, Goggins J (2016): Direct displacement based seismic design for single storey steel concentrically braced frames. *Earthquakes and Structures*, 10(5): 1125-1141.

[9] Salawdeh S, English J, Goggins J, Elghazouli AY, Hunt A, Broderick BM (2017). Shake table assessment of gusset plate connection behaviour in concentrically braced frames. *Journal of Constructional Steel Research*, 138: 432-448.

[10] Goggins J, Broderick BM, Elghazouli AY, Salawdeh S, Hunt A, Mongabure P, English J (2018): Shake table testing of concentrically braced steel structures with realistic connection details subjected to earthquakes. *Structures*, 12: 102-118.

[11] Salawdeh S, Ryan T, Broderick BM, Goggins J (2019): DDBD assessment of steel CBFs using full scale shake table tests with realistic connections. *Journal of Constructional Steel Research*, 154: 14-26.

[12] O'Reilly GJ, Goggins J, Mahin SA (2012): Behaviour and design of a self-centering concentrically braced steel frame system. *15th World Conference on Earthquake Engineering*, Lisbon, Portugal, 2012.

[13] O'Reilly GJ, Goggins J, Mahin SA (2012): Performance-based design of a self-centering concentrically braced frame using the direct displacement-based design procedure. *15th World Conference on Earthquake Engineering*, Lisbon, Portugal.

[14] O'Reilly GJ (2013): Development of a novel self-centering concentrically braced steel frame system. *MSc thesis*, National University of Ireland Galway, Galway, Ireland.

[15] Christopoulos C (2002): Self-centering post-tensioned energy dissipating (PTED) steel frames for seismic regions. PhD thesis, University of California, San Diego, CA, USA.

[16] 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.

[17] CEN (2004): European structural steel standard. *European Standard EN 10025:2004*, Comite Europeen de Normalisation, Brussels, Belgium.

[18] CEN (2005): Eurocode 3: Design of steel structures - part 1-1: General rules and rules for buildings. *European Standard EN 1993-1-1:2005*, Comite Europeen de Normalisation, Brussels, Belgium.

[19] Calvi GM and Sullivan TJ (2009): Development of a Model Code for Direct Displacement Based Seismic Design. *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2005-2008 Project*, Napoli, Italy.

