ORIGINAL ARTICLE

Shake Table Testing of Self-Centring Concentrically Braced Frames

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Abstract

The self-centring system presented in this paper is a novel damage control technique designed to improve the resilience of concentrically braced frames (CBF) under seismic action. Namely, traditional CBFs can undergo large residual drifts following an earthquake event which can limit the opportunity for cost-effective repair of the structure. Additionally, the gusset plates connecting the brace members to beams and/or columns can experience substantial rotations as a result of the compression buckling of the bracing members. Through the utilisation of post-tensioning strands placed between flanges of beams, the novel self-centring concentrically braced frame (SC-CBF) system can return the frame to its original position after significant inelastic deformations experienced during large earthquakes, resulting in minimum residual drifts.

In this paper, shake table testing of the aforementioned SC-CBF system subjected to realistic earthquake loading is presented. The research is carried out as part of the H2020 "Seismology and earthquake engineering research infrastructure alliance for Europe" SERA project. Four sets of bracing configurations, incorporating varying square hollow section (SHS) braces and gusset plates were utilised in the shake table testing. Uniaxial loading with varying shake table accelerations was executed and the structural response evaluated using data from strain gauges (SG), load cells (LC), displacement transducers and accelerometers. The measured results provide information on the important parameters such as the tensile and compressive strength of the braces, post-buckling capacity, gusset plate strains and post-tensioning force. These findings are then presented and the crucial local and global response performance emphasised.

Keywords

Self-Centring System, Concentrically Braced Frame, Shake Table Test, Steel Structure, Earthquake Engineering, Residual Drifts, Experimental Study, Resilient Building, Buckling.

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1 Introduction

As one of the most efficient earthquake-resistant steel structural systems, the concentrically braced frames (CBFs) are widely used in seismic regions. In the past few decades, many studies [1-9], focusing on the seismic performance of CBFs, were conducted. As directly related to the structural damage, the peak inter-storey drift is of main concern in these studies, while less attention has been paid to the post-earthquake condition of CBFs. The structural residual displacement is important in seismic design, in terms of the re-occupancy of the building, monetary losses and the speed of repairs/modifications. To minimise the CBF residual deformations, the concept of a self-centring steel braced frame (SC-CBF) system was proposed by O'Reilly *et al.* [10, 11, 12]. This system can dissipate energy during earthquakes through braces, which are expected to yield in tension and buckle in compression during strong earthquakes, while all other elements such as columns, beams and connections are expected to behave elastically. Moreover, this innovative form has the advantage of a self-centring (SC) system, namely, returning to its original position after the seismic event. Thus, the application of a SC-CBF could make replacing damaged braces easier after large earthquake events, as the residual drift is minimised. Hence, immediately reoccupation of building comprising a SC-CBF after an earthquake could be possible. Therefore, monetary losses during downtime are reduced.

The concept of a single-storey SC-CBF, which was proposed by O'Reilly [12], is shown in Figure 1. The connection types and post-tensioned strands are the major differences between SC-CBFs and conventional CBFs. Columns of SC-CBFs are pinned at the base, while beams are connected to columns via rocking connections. The rocking-connections allow the beam to rock against the column and, thus, protect the beams and columns from yielding. By employing the post-tensioned strands, the gap-openings of rocking connections can be closed by the strand forces. Hence, the frame is centred back to its initial position. The combination of rocking connection and post-tensioned strands ensures that the braces are the only energy-dissipating components of the SC-CBF. As the residual displacement is minimised under earthquake loading, only the replacing of the damaged brace members is required after an earthquake event. Thus, it can reduce the economic losses caused by repairs and shortens the downtime. Theoretical, experimental and numerical studies were carried out by O'Reilly [12] to characterise the response of SC-CBFs, with different brace members installed, under lateral loading. With full-scale single-storey SC-CBF tests conducted, a flag-shaped lateral force versus drift ratio hysteretic curve, the typical response of a self-centring (SC) system, was observed. The tests also verified that all of the imposed energy was dissipated by the braces.

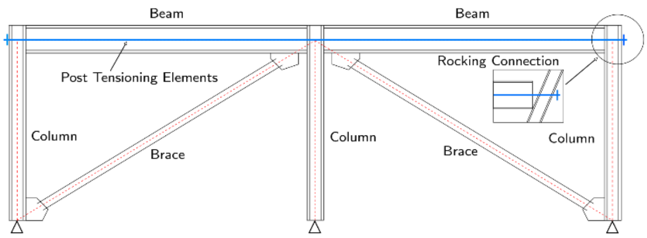


Figure 1 Concept of SC-CBF (adapted from O'Reilly [12])

However, in the pushover tests, the secondary effects were not considered as no roof weight was employed or simulated. Moreover, the quasi-static pushover tests could not directly validate the self-centring behaviour of the SC-CBF. Thus, the research conducted here focuses on the self-centring behaviour of SC-CBFs under real earthquake excitations. The SC-CBF structure designed and tested in this study was modified based on the frame proposed by O'Reilly [12]. Seven pairs of SHS braces were prepared and two real earthquake records were selected. Instrumentations, including accelerometers, displacement transducers, load cells and strain gauges were installed to monitor the response of the SC-CBF. Seven series of shake table tests were conducted sequentially under various ground motion intensities. The seismic behaviour and self-centring performance of the SC-CBF structure are evaluated and discussed in this paper.

2 Test Campaign

2.1 Structural Details

As shown in Figure 2, the tested structure comprises of one SC-CBF and two gravity frames. The two gravity frames were simple one-bay frames with all beams and columns employing pinned connected. Thus, the two gravity frames bore part of the roof weight, but contributed zero lateral stiffness to the structure. The SC-CBF, which was designed to resist all the inertial force from the roof weight, was located in the middle of the structure. As detailed in Figure 3, there were three types of connections, namely pinned connections, slotted connections and rocking connections, used in the SC-CBF. The frame middle column was connected to the roof and table via the pinned connections, which can transfer both the roof lateral inertial load and the gravity load. The north and south columns were constrained by the slotted connections to model that the frame tested is part of a multi-bay structure. With its vertical movement fixed, the roller of the slotted connection was allowed to have a maximum horizontal movement of ±30 mm. Thus, the slotted connection can only transfer vertical loads. The rocking connection, used to connect the beams and columns, is one of the key components of the system. Figure 4 shows the schematic of a rocking connection, which was proposed based on the connections used by Christopoulos [13] and Clayton *et al.* [14]. As the rocking connection has no moment resistance capacity, post-tensioned strands, installed along the centre line of the beams, were employed to provide lateral stiffness to the frame. The initial compressive forces in the strands can keep the rocking connection closed to make the SC-CBF behave essentially as a moment-resisting frame (MRF). When the SC-CBF is laterally deformed, the rocking connection opens a gap, which causes the elongation of the post-tensioned strands. The compressive forces provided by the strands closes the opened gap and, thus, centre the SC-CBF back to its original position. Through the rocking mechanism, the beams and columns are prevented from damaging. However, large local stresses are induced in the contact surfaces of beams and columns due to rocking, which could potentially damage the components. Therefore, steel plates and stiffeners were welded to enhance the strength of the rocking connection. As an extensive study, the middle frame, namely the SC-CBF, was designed to maintain consistency with what was designed by O'Reilly [12]. But there were still some modifications on the structural details of the SC-CBF, which are summarised as follows:

1. The steel sections were selected from European H and IPE sections but the profile geometries were kept consistent with the UK sections used by O'Reilly [12].
2. The nominal diameter of the strands was changed from 12.3 mm to 15.3 mm due to availability of strands in R. North Macedonia. However, the strand pre-tensile forces remained as 80 kN.
3. The centre pin diameter was increased from 40 mm to 48 mm, which improved the maximum lateral resistance of the SC-CBF.

The two gravity frames were located symmetrically about the SC-CBF. To ensure that the three frames had similar lateral movements under excitations, beams and braces were used to create a diaphragm linking the external frames to the middle frame. The mass block on the roof was made up with 48 steel ingots, which totalled an approximate mass of 20 ton. As the gravity frame has no lateral stiffness, the roof inertial force induced under earthquake loading was transferred to the SC-CBF through the pin joint of the SC-CBF. The structure was mounted on the 5×5 m shake table in the DYNLAB, Institute of Earthquake Engineering and Engineering Seismology (IZIIS), Skopje, R. North Macedonia.

Figure 2 Overview of SC-CBF structure (on shake table)

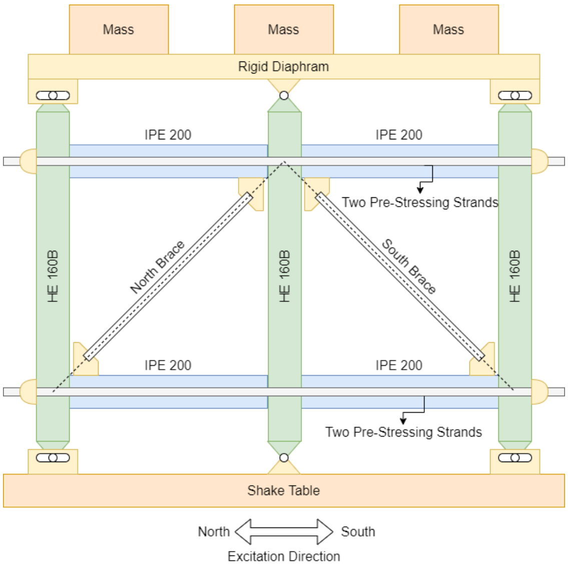


Figure 3 Schematic of the middle frame (SC-CBF)

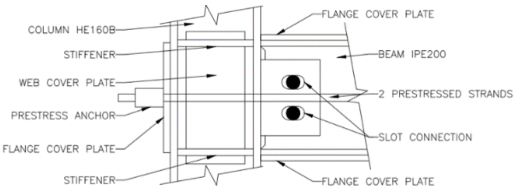


Figure 4 Schematic of rocking connection

2.2 Brace Specimens

There are four types of square hollow section (SHS) structural steel braces tested in the study, with details summarised in Table 1. The SHS braces were manufactured in accordance with EN 10025:2004 [15]. The brace slenderness, ranging from 1.03 to 2.21, covers the slenderness values permitted in Eurocode 3 [16]. As shown in Figure 5, the braces were connected to the frame through two gusset plates. The gusset plates, designed according to conventional design methods, were constrained to the beam flange via four bolts. To strengthen the connection and avoid the local beam failure, the beam flanges and webs were reinforced with steel plates and stiffeners (Figure 5). Aiming to replicate the boundary condition of a conventional gusset plate, gusset plates were provided with a vertical stiffener, as illustrated in O'Reilly [12].

Table 1 Brace geometries

ID	*b* [mm]	*t* [mm]	*L* [mm]	Number
B40x40	40	4	1395.3	2
B30x30	30	3	1432.6	4
B25x25	25	2.5	1435.1	4
B20x20	20	3	1437.6	4

b is the section nominal 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)

2.3 Data Acquisition

Instrumentations, including accelerometers, displacement transducers and load cells, were utilised to monitor the responses of the SC-CBF. There were nine accelerometers installed at roof level, beam levels and table level. The measured accelerations were used to obtain the natural frequency, the damping ratio and roof acceleration amplification factor of the structure. Besides the installed accelerometers, the shake table acceleration and the corresponding displacement were recorded by the table internal sensors. As the residual drifts and peak drifts were of concern in this study, two linear string potentiometers were used to capture the relative displacement between the upper and lower beams. As the only energy-dissipating components, the responses of the braces under excitations were captured by strain gauges and linear variation displacement transducers (LVDTs). There were 20 strain gauges installed at the mid-span, the two ends and the two gusset plates of the north specimen. Regarding the south brace, due to the limited number of channels, only four strain gauges were installed at the mid-span. Besides the strain values, the brace elongations were measured by LVDTs. To track the rocking connection behaviour, the gap-openings were recorded by two LVDTs attached to the flanges (Figure 6). The roller movements of the four slotted connections were monitored by the LVDTS to check that the rollers did not hit slot edge and cause connection damages. The compressive force provided by the post-tensioned strands were measured by four load cells (Figure 7).

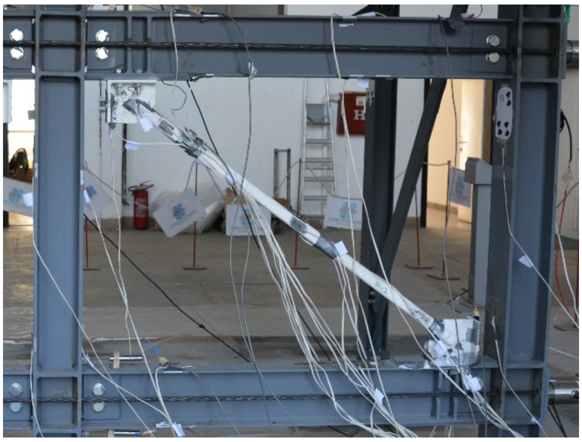


Figure 5 SHS brace specimen and gusset plates (B40x40, north bay)

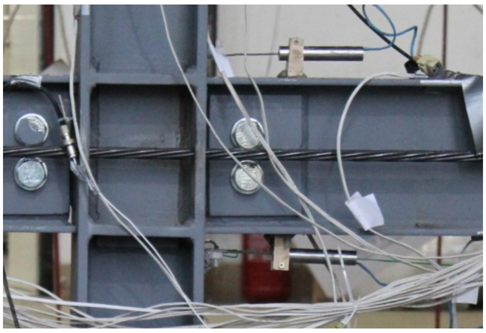


Figure 6 LVDTs for measuring rocking connection gap-opening

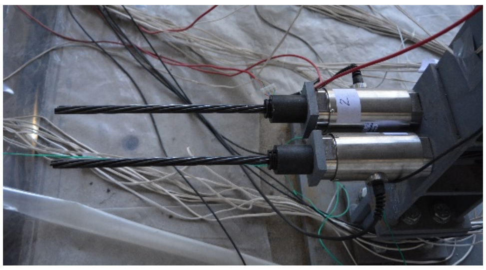
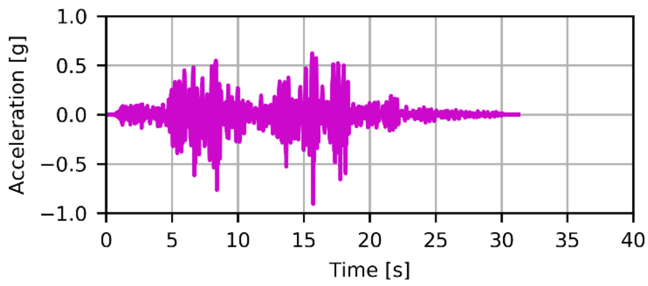


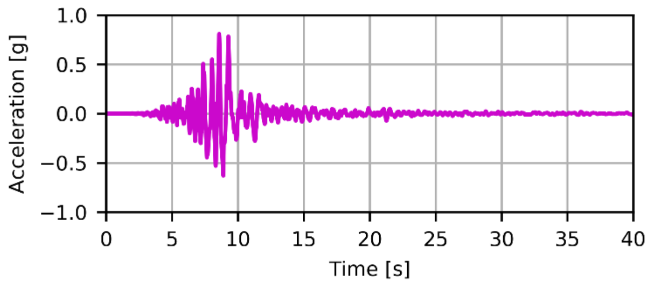
Figure 7 Two load cells installed at lower beam level

2.4 Ground Motions

In this study, the table motion was limited to thenorth-south direction (marked in Figure 3). One main objective of the experimental programme was to validate the performance of the self-centring system under ground motions with various characteristics. There were two ground motions (GMs) selected from real earthquake events, namely Duzce 1999 (M7.3) and Central Italy 2016 (M6.5), respectively. Figure 8 (a) and (b) display the acceleration time-history plots of the GMs. The frequency content of GM 1 is more narrow-banded compared to that of GM 2, but it matches the expected natural frequency better. The broader bandwidth of GM 2 was expected to lead to greater force and displacement demands after brace yielding during strong motion.



(a) Ground Motion 1 (GM 1)



(b) Ground Motion 2 (GM 2)

Figure 8 Time-histories of the two selected ground motions

2.5 Testing Procedure

The tests conducted in this study were grouped as S0 ~ S6 according to the brace pairs. For each testing series, a number of tests with different PGAs were conducted. The brace details, GM and corresponding PGA of each test are summarised in Table 2. It should be noted that the lateral resistance provided by the two B40×40 braces was larger than the shear capacity of the centre pin. Hence, the aim of the two tests conducted in S0 was to validate the feasibility of the boundary conditions, the connections and the data acquisition system.

For each testing series of S1 ~ S6, tests with ascending PGAs were carried out until extensive yielding and buckling of braces. The testing procedures of the six series are listed as follows:

- The first test was carried out with PGA ≈ 0.1 g, with aims to validate the whole testing system and to obtain the initial structural responses.
- The following one or two tests were performed with ground motion amplification carefully adjusted until the brace approached yielding. As the whole system behaved elastically here, the ground motion PGAs were scaled based on the peak strain values of the previous test and the brace yield strain.
- In the last test, aiming to validate the structural self-centring behaviour and the feasibility of the rocking connections, the imposed GM was expected to impose permanent deformation into the braces. The GM intensity used in this test was addressed according to the target lateral displacement (nominally four times the yielding displacement). It should be highlighted that the addressed PGA was limited by the shake table overturning moment in tests of braces S1.
- The fundamental periods and damping ratios of the structure between the tests were measured by white noise tests. The acquired fundamental periods could identify that the structural system, especially the braces, was not damaged until the last test was performed.

Table 2 Test programme and summary of results

	Brace	Test #	GM #	PGA [g]	DR_{max} [%]	$DR_{residual}$ [%]
S0	B40x40	1	GM 1	0.36	0.36	0.022
		2	GM 2	0.43	N.A.	0.004
S1	B20x20 (Pair 1)	1-1	GM 1	0.10	0.15	0.013
		1-2		0.18	0.27	0.004
		1-3		0.26	0.36	0.015
		1-4*		0.57	0.76	0.078
S2	B20x20 (Pair 2)	2-1	GM 1	0.10	0.10	0.016
		2-2		0.21	0.22	0.016
		2-3		0.45	0.59	0.022
		2-4		0.41	0.93	0.035
S3	B25x25 (Pair 1)	3-1	GM 1	0.10	0.10	0.001
		3-2		0.25	0.24	0.038
		3-3		0.48	1.19	0.002
S4	B25x25 (Pair 2)	4-1	GM 2	0.09	0.09	0.005
		4-2		0.22	0.17	0.016
		4-3		0.43	0.40	0.007
		4-4		0.84	2.51	0.027
S5	B30x30 (Pair 1)	5-1	GM 1	0.09	0.07	0.006
		5-2		0.25	0.24	0.012
		5-3		0.50	1.10	0.0004
S6	B30x30 (Pair 2)	6-1	GM 2	0.09	0.09	0.003
		6-2		0.24	0.17	0.014
		6-3		0.62	0.61	0.002
		6-4		0.68	1.23	0.061

* Test was terminated at 14s

3 Test Results

3.1 General Observations

Generally, the SC-CBF behaved as expected in all the tests. Based on visual checks and LVDT measurements, the rollers of the slotted connections didn't reach their movement limits in any of the tests. No relative displacement was observed between the SC-CBF and the two gravity frames. 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. Owing to rocking and friction between steel components, loud sounds, related to the GM intensities, were heard during testing. This acoustic effect introduced spikes to the recorded acceleration values, especially at the roof level. As the sounds had higher frequency components compared to the structural response signals, the recorded accelerations were corrected by low-pass signal filtering (< 50 Hz) in post-processing. In tests 1-4 and 2-3, relative movement between the roof and the SC-CBF was observed. Additional welds were added to the connection to solve this issue. A replicated test 2-4 was carried out after the strengthening and the slippage issue was proved to be fixed. In tests 5-3, 6-3 and 6-4, the south upper beam end was noted to slide slightly along the column flange, which indicated the damage of this rocking connection. Due to the locations of post-tensioned strands, the slightly damaged rocking connection could not be fixed within the available test window. Consequently, the global lateral stiffness of the south bay of the SC-CBF was reduced, and thus, the north brace dissipated most of the energy.

As shown in Figure 9, significant buckling was observed at the end of each series except the first series. It should be noted that the gusset plates were brought back to their original position due to the excellent self-centring behaviour of the SC-CBF. Thus, the global brace buckling deformations were weakened at the end of each test. Between the series, the damaged braces were easily replaced, as there was no need to re-centre the frame.

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

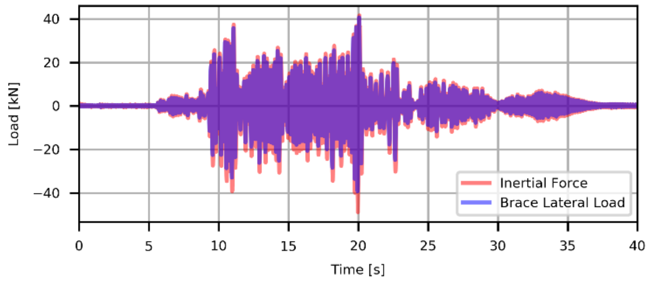
Figure 9 Global brace buckled deformations

3.2 Instrumentation Validation

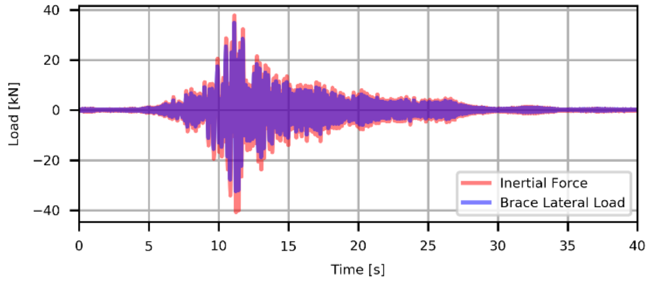
A pair of braces, with new strain gauges, was replaced at the beginning of each test series and the feasibility of the data acquisition system within the series was validated by its first test. This is achieved by comparing the inertial force at roof level with the lateral forces provided by the braces. If the two forces match well, the data acquisition system can be considered as reliable. The inertial force was computed according to the roof mass and the acceleration recorded from the accelerometer. As the braces behaved elastically and had no out-of-plane deformations in the first test, the compressive and tensile forces provided by the two braces can be computed from the strain values recorded at the mid-span. A Young's modulus value of 210000 MPa was assumed for the braces. Figure 10 shows the lateral forces comparison of three tests. It could be seen that the inertial force and the brace lateral load agrees well, which demonstrates good workability of the installed instrumentations. It should be noted that the forces caused by damping and frictions were ignored in the comparison as they were assumed that their contribution to the system under low GM intensity (PGA ≈ 0.1 g) was insignificant.

3.3 Fundamental Periods and Damping Ratios

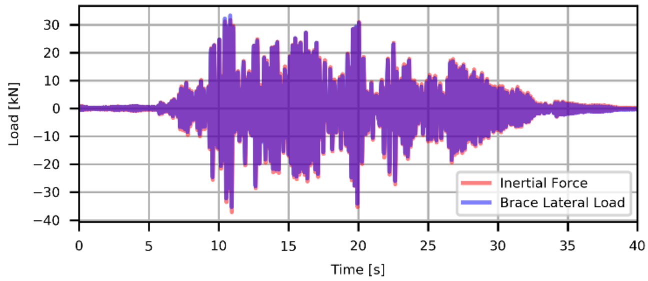
Table 3 summarises the structural fundamental periods and damping ratios of each testing series. With the increase of brace stiffness, the structural fundamental period decreases. By comparing the fundamental periods at the beginning and end of each series, an increase of the fundamental periods was found, which was caused by the damage of the braces. The structural damping ratios were computed based on the half-power bandwidth method. It could be noted that when the structure was not damaged, the damping ratios were generally less than 5%, a value commonly used in the design and numerical modelling of steel structures. When the brace failure occurred, there was an increase of damping ratios in all testing series except S2. It should be highlighted that when the braces were buckled, the damping ratio could increase to a value larger than 5% (S5 and S6).



(a) Test 3-1



(b) Test 4-1



(c) Test 5-1

Figure 10 Lateral loads comparisons of three tests estimated from accelerometers placed at roof level ('inertia force') and strain gauges applied to brace members ('brace lateral load')

Table 3 Natural frequencies and damping ratios of the SC-CBF before and after testing

	Brace	T [s]	ξ [%]	
S1	B20x20 (Pair 1)	0.23	4.28	Before testing
		0.25	4.83	After testing
S2	B20x20 (Pair 2)	0.21	3.49	Before testing
		0.22	3.39	After testing
S3	B25x25 (Pair 1)	0.20	3.10	Before testing
		0.23	3.79	After testing
S4	B25x25 (Pair 2)	0.20	2.56	Before testing
		0.24	3.51	After testing
S5	B30x30 (Pair 1)	0.19	2.57	Before testing
		0.22	7.02	After testing
S6	B30x30	0.19	3.29	Before testing

	(Pair 2)	0.21	5.64	After testing

3.4 Rocking Mechanism and Inter-Storey Drift Ratio

In the tests, the rocking connections were observed to behave efficiently. This could be further verified by the scratches found on the column flange and the slotted connections (Figure 11). Figures 12, 13 and 14 plot the rocking connection gap-openings, recorded by the two LVDTs (Figure 6), of three selected tests. As the gaps closed at the end of each excitation, the SC-CBF was positioned back to its original position. It reveals that the forces offered by the post-tensioned strands could effectively close the rocking connection. Regarding the beam and column members, neither local buckling was observed around the rocking connections nor the yield strain exceeded, as determined from strain gauges. Therefore, it can be concluded that the rocking connections prevented the beams and columns from being damaged and all the imposed earthquake energy was dissipated by the braces. It should be noted that the upper strand forces had a decrease of 7 kN after test 4-4, but no damage was observed. The strand force decrease was mainly due to the sliding of the temporary strand anchors during strong excitation. Re-stressing was performed after test 4-4 to address this issue.

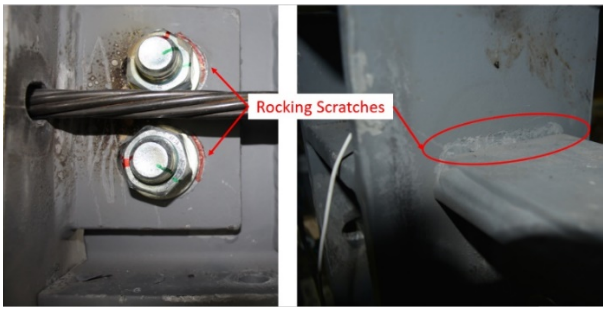
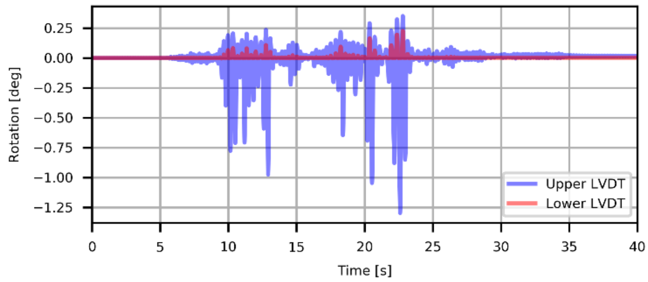
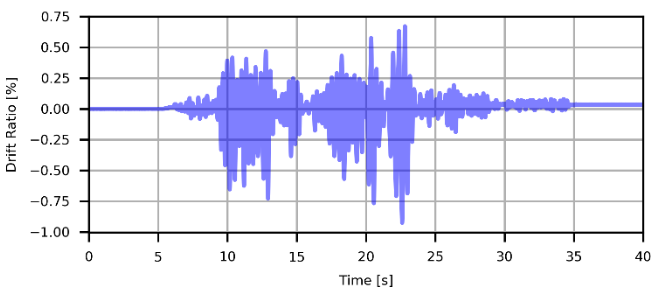


Figure 11 Scratches caused by the rocking mechanism

The maximum inter-storey drift ratios (DR_{max}) and residual drift ratios ($DR_{residual}$) of all the tests are summarised in Table 2. All the residual inter-storey drift ratios were less than 0.1%, which demonstrates the excellent self-centring behaviour of the SC-CBF. Figures 12, 13 and 14 show the inter-storey drift ratio plots of three selected tests. Even when the peak drift ratio reached 2.5% (test 4-4) and the braces were seriously damaged, the combination of rocking connections and post-tensioned strands could effectively minimise the residual displacement of the SC-CBF structure.

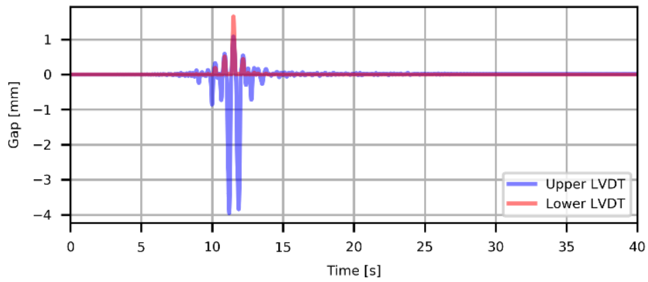


(a) Gap-Opening

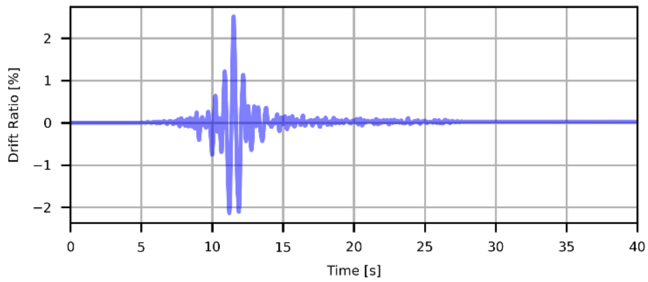


(b) Inter-Storey Drift

Figure 12 Rocking connection gap-opening and Inter-storey drift ratio time histories of test 2-4

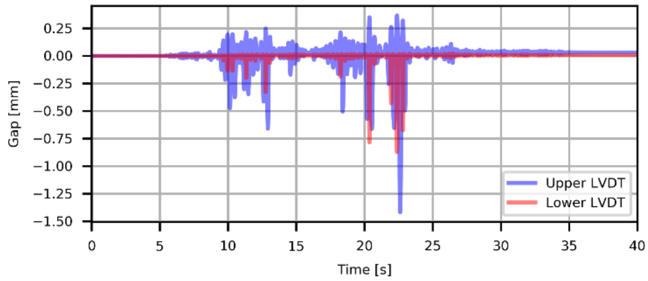


(a) Gap-Opening

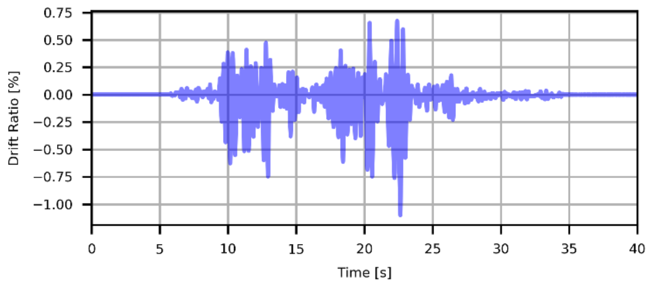


(b) Inter-Storey Drift

Figure 13 Rocking connection gap-opening and Inter-storey drift ratio time histories of test 4-4



(a) Gap-Opening



(b) Inter-Storey Drift

Figure 14 Rocking connection gap-opening and Inter-storey drift ratio time histories of test 5-3

4 Conclusion

Building on the research outputs of O'Reilly *et al.* [10, 11, 12], an extensive study has been carried out to characterise the seismic behaviour of SC-CBFs through shake table testing. The innovative frame designed by O'Reilly *et al.* [10, 11, 12] was modified and extended to a SC-CBF structure. SHS structural steel braces with different cross-section dimensions were selected as the energy-dissipating members of the SC-CBF. A series of shake table tests were conducted under two ground motions with different intensities. Based on the testing results, the following conclusions can be made:

– The damping ratios of the structure, with four types of SHS braces installed, were generally less than 5%.

- Benefiting from the proposed rocking connection, the beams and columns of the SC-CBF were prevented from yielding under earthquake excitation. Therefore, the braces dissipated all the imposed energy.
- The combination of post-tensioned strands and rocking connections provided the SC-CBF with excellent self-centring behaviour under earthquake excitation.
- Easy brace member replacement between experiments was achieved due to the excellent self-centring behaviour of the SC-CBF. Hence, downtime of a structure comprising a SC-CBF can be reduced after the occurrence of earthquake events.

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