The ROBUST Steel Building Response

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ABSTRACT

Shaking table testing of a full-scale three storey resilient and reparable composite steel framed building system is conducted as part of the RObust BUilding SysTem (ROBUST) collaborative China-New Zealand project. The steel frame building, with nine different interchangeable seismic resisting configurations and cold formed steel-concrete composite decks, is designed to dissipate energy by friction. Typical building non-skeletal elements (NSEs) are also included in one configuration. Testing is performed on the Jiading Campus shaking table at Tongji University, Shanghai, China. This paper provides a short overview of the test setups and preliminary results before full data reduction was conducted. It is shown that the systems with friction configurations, many of which were developed in New Zealand, behaved well during shaking intensities applied with peak ground accelerations up to 0.60g.

INTRODUCTION

The plan of the three-storey test structure, shown in Figure 1 is 7.25 m to the column centres in the longitudinal direction, and 4.75 m in the lateral direction (MacRae et al., 2019). The inter-storey height is 3 m. The structure is designed to dissipate energy by friction only. The building was designed for the design level of shaking, referred to as the ultimate limit state (ULS) shaking in New Zealand, for an office building in Wellington which has a zone factor of 0.4. An importance level of 1.0 is used which corresponds to an annual probability of exceedance, *APE*, of 1/500. The design ductility factor was $3/S_p$, which where the structural performance factor, S_p is 0.7. Devices used include the asymmetric friction connection (AFC), symmetric friction connection (SFC) (MacRae et al., 2010, 2015), and the resilient friction connection (RSFJ) (Zarnani et al., 2015, 2016). These are placed at beam ends, column bases, in braces, and in the tension-only “GripNGrab” device (Rangwani et al., 2020, 2023). For the AFC and SFC, axial compression is often applied by means of proofloaded bolts with traditional structural washers (STD), as well as with conical spring washers (CSWs) which are also known as Belleville Springs (BeSs) (Ramhormozian et al., 2017). When CSWs are used, a lower compressive force is applied to the sliding interfaces, but a larger range of interface compressive force adjustment is possible. An elevation view of the actual tested structure is shown in Figure 2.

Structural configurations include moment frames, braced frames, rocking frames, and rocking columns as shown in Figure 3 and Table 1. The NSEs include different configurations of fire sprinkler piping, suspended ceilings, glazed curtain wall (GCW), internal partition walls (IPWs) with access holes, precast concrete panels (PCP), and contents. These are subject to unidirectional and bidirectional horizontal shaking. The symbols have the following meanings: A - AFC, AB – AFC with Belleville springs (i.e. conical spring washers), BF - base fixed, BP - base pinned, BRC - brace, F - fixed, G - GripNGrab, R - RSFJ, RTC - tension-compression RSFJ, RC – rocking column, S - SFC, SB – SFC with Belleville springs, and U - uplift permitted. Table 1 describes the overall sequence of configurations tested. Further details about the test aims and details are given in MacRae et al. (2019).

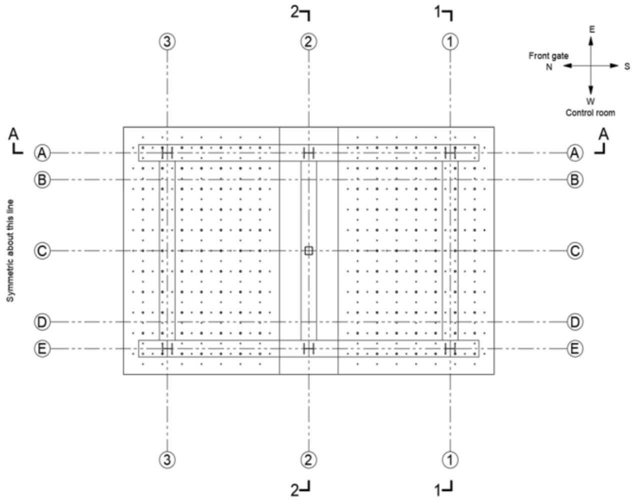


Figure 1. Plan of Test Frame

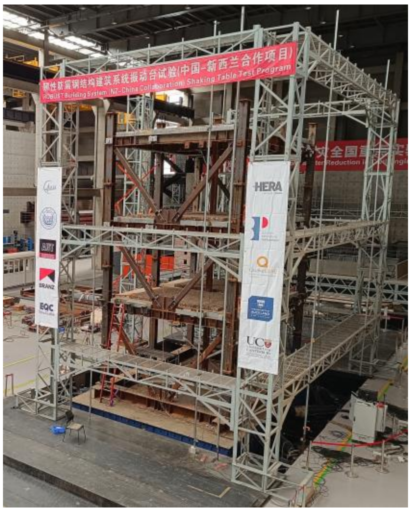


Figure 2. Actual Structure

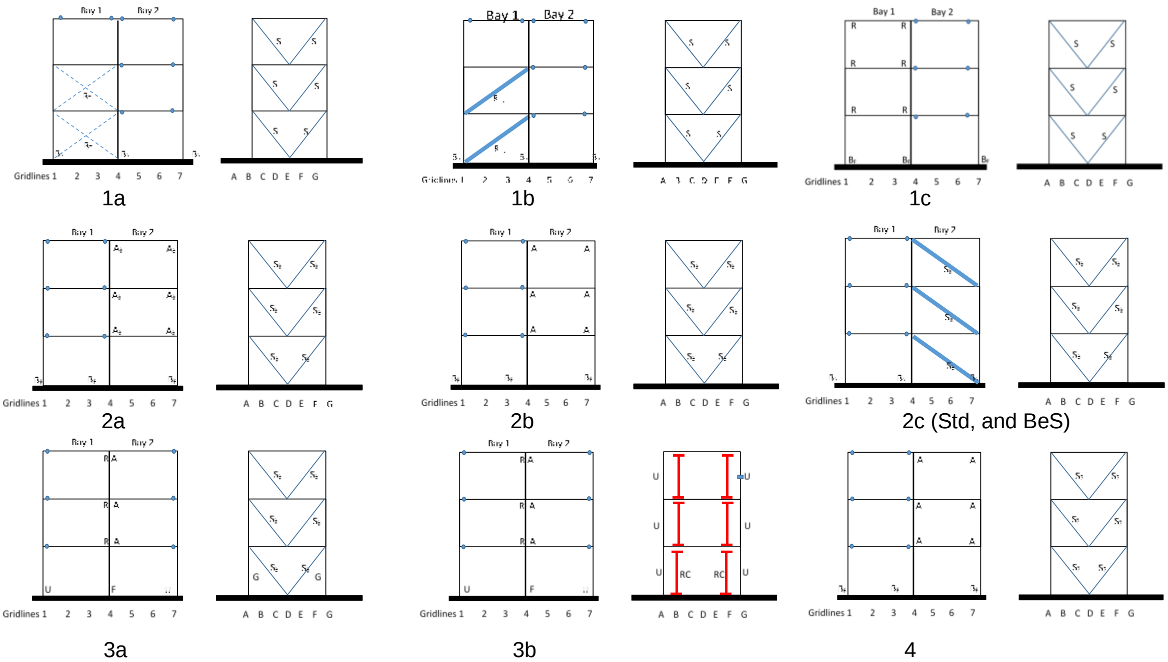


Figure 3. Elevations of the Different Test Configurations

Table 1. Shaking Table Test Configurations

CONFIGURATIONS			Longitudinal X Bay 1	Longitudinal X Bay 2	Transverse Y	Loading Type	Other
1	RSFJ	a	BRC TOB RSFJ, L1,2	Pinned	CBF V-braced System-SFC-BeS	X	**Pinned Base - up to 8 mm uplift allowed**
		b	BRC TCB RSFJ, L1,2	Pinned	Same as above	X&Bi	**Pinned Base**
		c	MRF RSFJ	Pinned	Same as above	X&Bi	**Fixed Base**
2	AFC&SFC	a	Pinned	MRF SHJAFC_BeS	CBF V-braced System-SFC-BeS	X&Y&Bi	**Pinned Base & Fixed Base**
		b	Pinned	MRF SHJAFC_STD	Same as above	X	**Pinned Base**
		c s	Pinned	CTB SFC-STD	Same as above	X	**Pinned Base**
		c b	Pinned	CTB SFC-BeS	Same as above	X	**Pinned Base**
3	Rocking System	a	MRF RSFJ at centre	MRF SHJAFC_BeS at centre	RKF GnG	Y	**Pinned Base - up to 175 mm uplift allowed**
		b	MRF RSFJ at centre	MRF SHJAFC_BeS at centre	TJ Rocking Column	Y	**Semi-rigid Base**
4	NSE	-	Pinned	MRF SHJAFC_BeS	CBF V-braced System-SFC-BeS	X&Y&Bi	**Pinned Base**

Many of the technical considerations involved with the design and construction have been described by MacRae et al. (2019, 2020a, 2020b), Yan et al. (2020), NSEs by Dhakal et al. (2020), and rocking columns by Jia et al. (2020), MacRae et al. (2023). It is noted that the central column was nominally pinned at the base. Connections into this column from the beams which exist only in the lateral direction, were also nominally pinned. However, the slab was composite and surrounding the column providing moment resistance there. Above the floor level, the upper storey exterior columns do not contact the slab to make changeover between the different configurations easy.

The term "non-skeletal elements (NSEs) are used only in the final configuration (Configuration 4). These in past literature these were often previously referred to as "non-structural elements". Here, the more technically correct term, with the same acronym, is used as many of the so-called non-structural elements often contribute to the structural resistance/action and are therefore structural (MacRae et al., 2020a)! These NSEs include interior partition walls (IPWs) at the ground level, cladding consisting of glazed curtain walls (GCW) at opposite corners, and precast concrete panels (PCP) at the other corners, ceilings, and piping in the top of the second and third storeys and building contents. Contents are placed on flat floors with either carpet or linoleum surfaces on the north side of the second storey. In Configuration 7, contents consist of short and taller stiff wooden boxes containing bricks placed to control the centre of the weight. These have wooden blocks of different height on top of them. The static and dynamic friction coefficients were determined both by (i) tension tests using a cable near the base, where the friction coefficient, μ, is given as F/N, and (ii) as $\mu = D/(2L)$, where L is the height of applied force to cause uplift rather than sliding as found by trial and error, and D is the base dimension in the direction of applied force. This last expression is found from moment equilibrium about the base at the point when the sliding force resistance, μW, is equal to the rocking resistance give $WD/(2L)$. In Configuration 8, a similar configuration is placed on the floor, but the boxes are supported by rolling coasters. In Configuration 9, typical building contents, including tables, chairs, and bookshelves are included.

The structural periods were obtained using small magnitude (< 0.05g) "white noise" shaking in both directions for a period of 3 minutes. The periods were identified from the Fourier spectra obtained from the acceleration time-history response. These white noise events were applied after every major shake to evaluate any change in period. However, for all structures, a change was not observed indicating no major change in system stiffness which could be related to damage.

Each of these systems are subject to increasing intensities of earthquake shaking using the 1940 El Centro NS record, and bidirectional shaking (BI) is applied in some cases. This record was used because it gives strong shaking over a range of periods, and it does not require excessive table displacements, velocities, or forces. The response spectra for the Configuration 1a input record are given in Figure 4. The measured table accelerations from accelerometers on the table itself and on the top flange of the foundation beam are similar but they are not the same as the scaled El Centro record values. It is possible to calibrate the table so that it gives the same response as the target spectra over a range of periods. This can be updated when the configuration changes. In this preliminary run calibration was performed for the short period strictures. The same record with different scaling was used for all configurations.

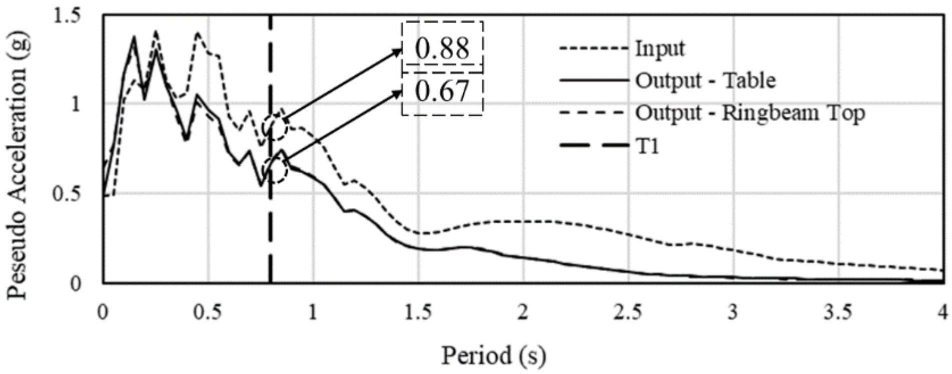
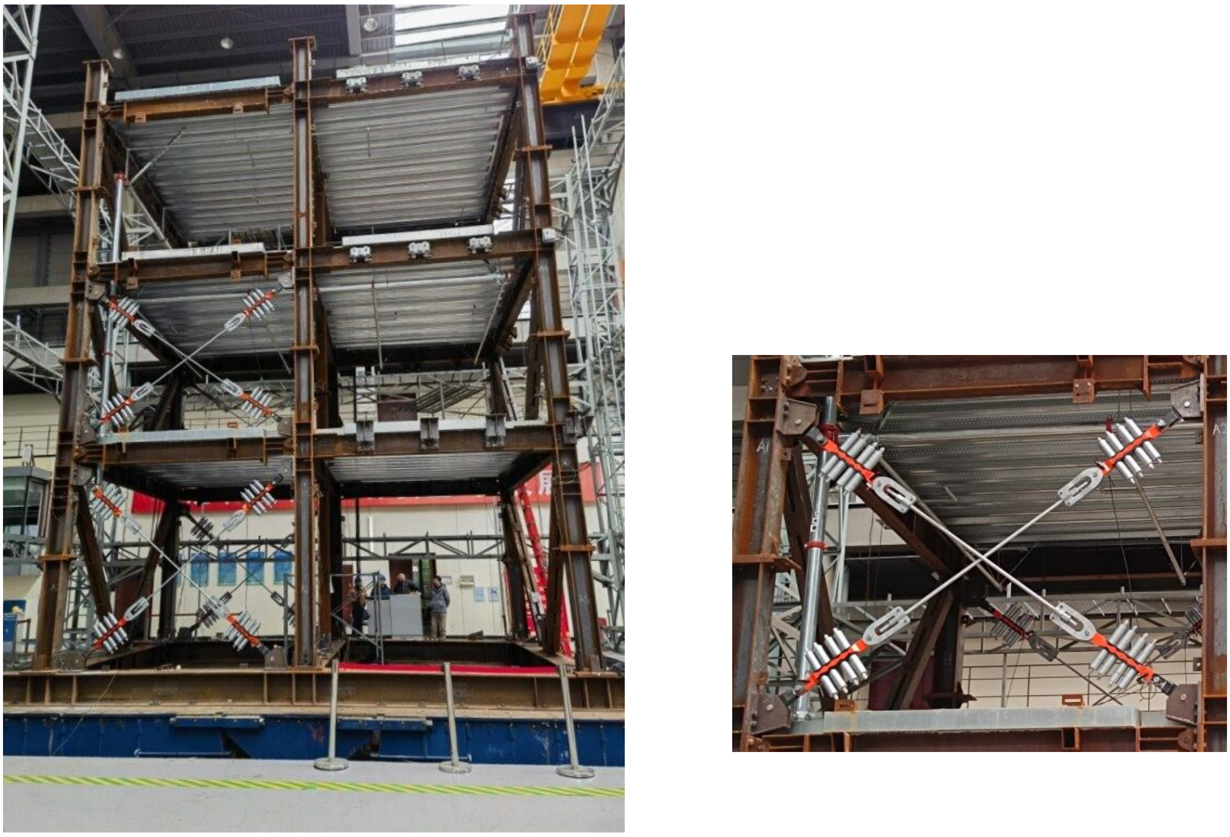


Figure 4. Acceleration Response Spectra

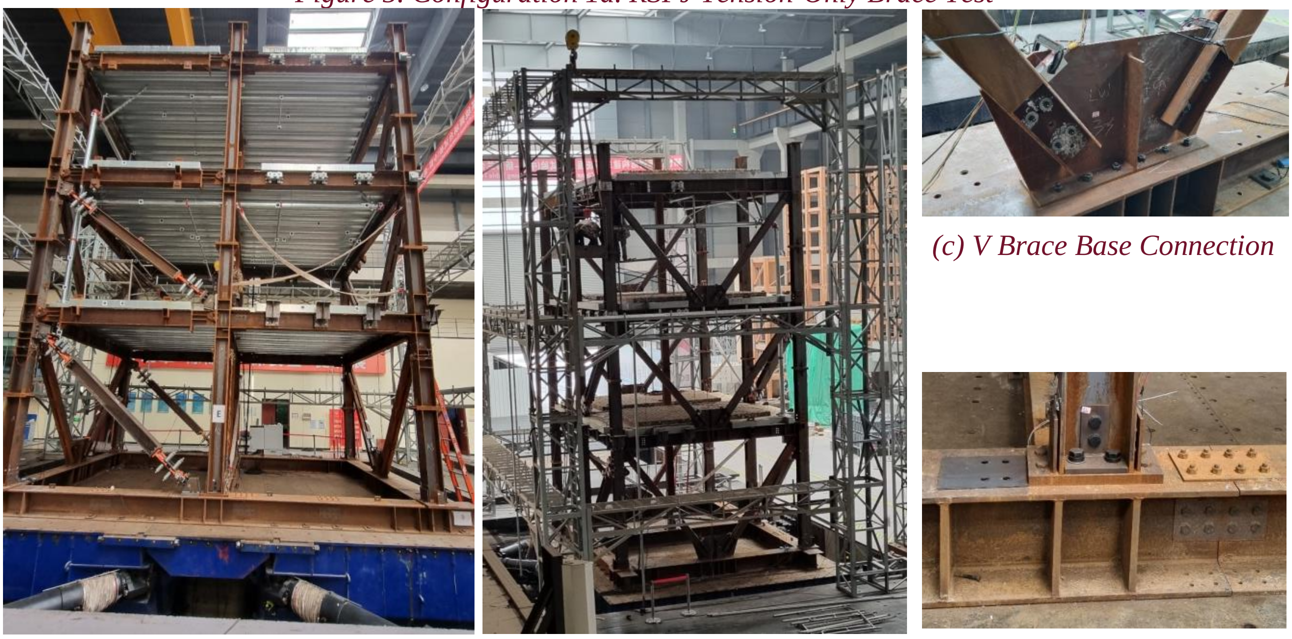
2. TEST CONFIGURATIONS

Photographs of the test configurations from Table 1 and Figure 3 are given below. In all configurations, some fire sprinkler pipes were present as shown. Figure 5 shows the tension RSFJ braces of **Configuration 1a** placed on the lower two levels. All columns had pinned bases. In Figure 6, the RSFJ tension compression braces of **Configuration 1b** are shown, again in the lower 2 stories. The V-braces connection for the transverse direction of loading is shown in Figures 6b and 6c. The end cleat of one of these braces fractured during the final shake as shown in Figure 7a. It was found that instead of the full strength butt weld specified shown in Figure 7b, a 5mm partial penetration weld was placed each side of the plate. This was due to fabrication error, not by the high quality main contractor, but by another fabricator used for a limited scope of work. There were similar issues with other items that contractor had performed, so checks were performed, and the issue was not critical in other cases. The third test with RSFJs was of moment connections as shown in Figure 8 associated with **Configuration 1c**. Friction flexural connections at the base of the columns were provided for this. Figure 9 shows **Configuration 2a and 2b** moment frame connections where sliding occurs in the beam bottom flange using asymmetric friction connections (AFCs). These were performed with conical spring washers (CSWs) and without CSWs. These are in the longitudinal frame on the north bays as shown in Figure 10. The crane cage was stabilised to enable the bolts to be tightened here. **Configuration 2c**, involving tension compression braces is shown in Figure 11. Sliding occurs at the lower end of the brace, and tests with and without conical spring washers (CSWs) were performed. Evidence of sliding on the shims beside the slotted gusset connection is shown in Figure 12. A number of visitors came to look at the tests. Figure 13 shows Charles Clifton describing the characteristics to the frame to Prof Zhao of Tongji university. GripNGrab components are shown in Figure 14a, and the complete device is placed in the frame in Figure 14b for **Configuration 3a**. Other elements of this configuration shown in Figure 15 involve the corner column details which allow large uplift, the central base connection where shear is resisted during uplift, the central columns with the fixed base connections, and beside which the beams are provided with fixed connections to provide frame lateral resistance in the longitudinal direction during transverse shaking. Pinned connections were provided at the other ends of the beams, beside the exterior columns. These beams provided the transverse rocking frames with some resistance against uplift, together with gravity and GripNGrab forces. Figure 16 shows that 35mm of uplift occurred during the shaking. In addition, contents were provided on the north side of the second storey where the floor was partitioned into a linoleum and carpet sections as described earlier. Testing of the coefficient of friction is shown in Figure 17. Figure 18 shows the rocking column concept of Configuration 3b. This was not tested at the time of the paper writing. Also, the placement and testing of contents on wheels is shown in Figure 19. Figure 20 shows the precast concrete panel support details. There are two horizontally slotted connections at the bottom of the storey and two at the top. Similarly, studs from the PCP, can move vertically within the PCP. For assembly, the PCP are placed on two rectangular hollow section gravity supports at bottom of the storey, one of which is shown in Figure 20. These studs are placed through the slots. Then, plates are welded on the slots to restrain one panel support at the top and one at the bottom from moving laterally, thereby permitting the panels to rock as interstorey drift occurs. At the other top and bottom horizontal slots, less restraint is provided to prevent binding of the panel which can introduce more force into the system when rocking.



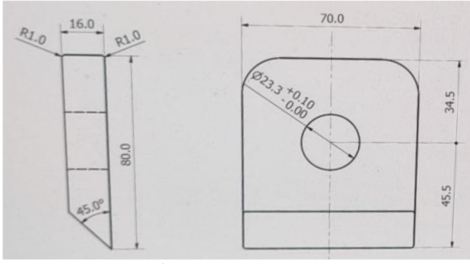
(a) Longitudinal View *(b) Connection View*

Figure 5. Configuration 1a. RSFJ Tension-Only Brace Test



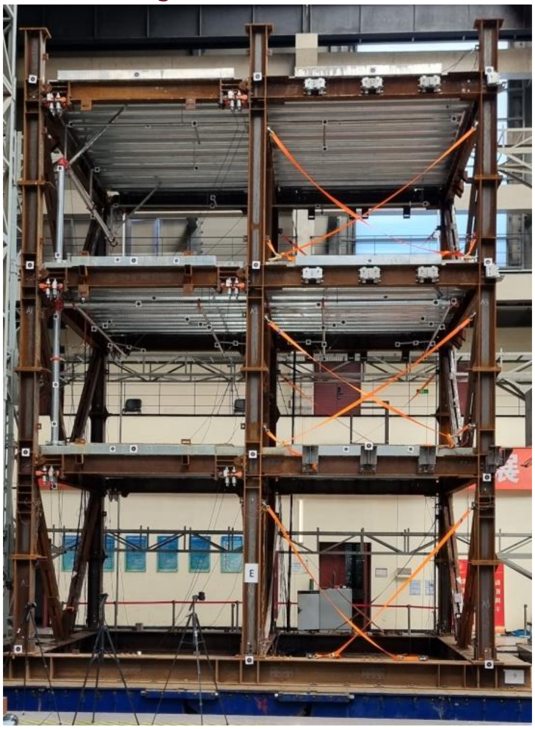
(a) Longitudinal View *(b) Lateral view* *(c) V Brace Base Connection* *(d) Col E Pinned Base Detail*

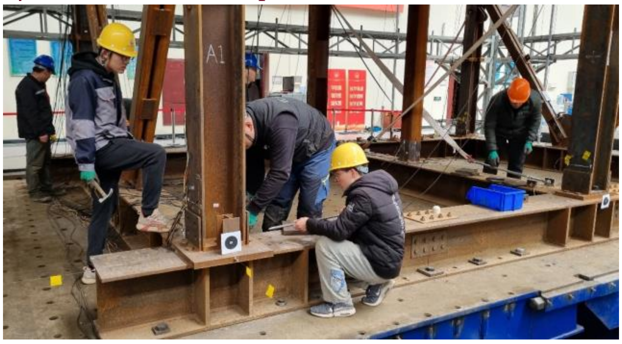
Figure 6. Configuration 1b. Tension Compression Brace Test Details



(a) Photo (see left cleat fracture) *(b) Detail specified in construction drawings*

Figure 7. RSFJ Fractured Cleat at end of RSFJ Tension Compression Brace

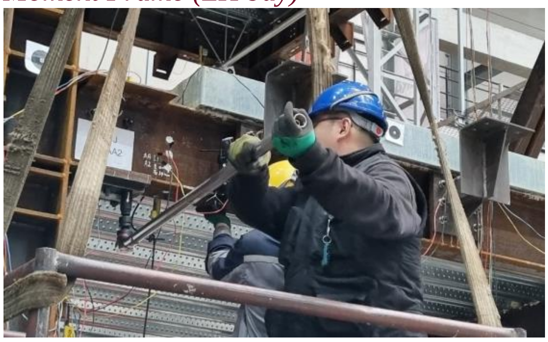




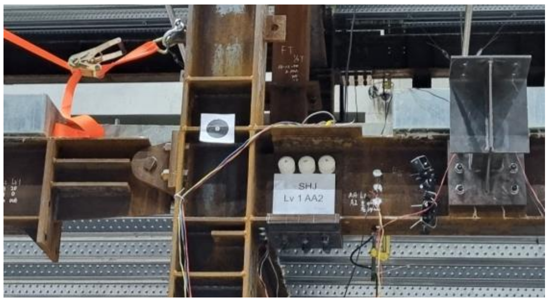
(b) Fixing Column Base Connection (1)

(a) Longitudinal View *(c) Fixing Column Base Connection (2)*

Figure 8. Configuration 1c. RSFJ Moment Frame (LH bay)

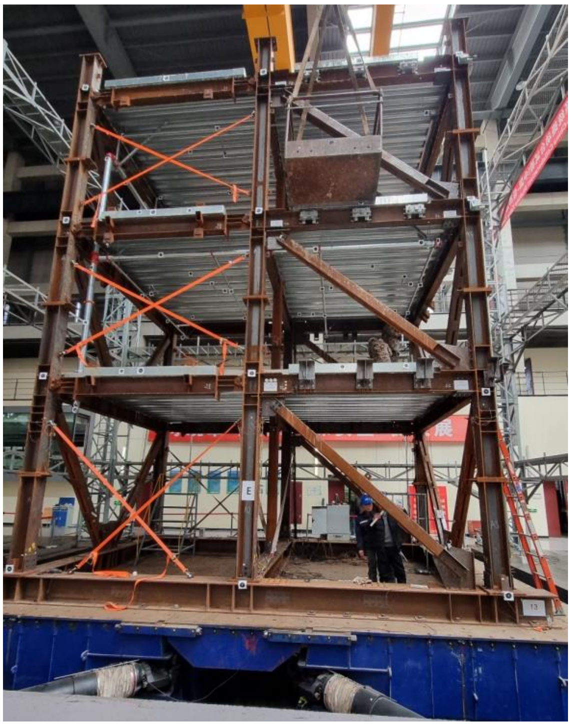


(b) Fixing Tightening Bolts for Moment Connections



(a) Longitudinal View *(c) Sliding Hinge Joint (RH side)*

Figure 9. Configuration 2a. and 2b. Moment Frame View (RH bay) with and without CSWs



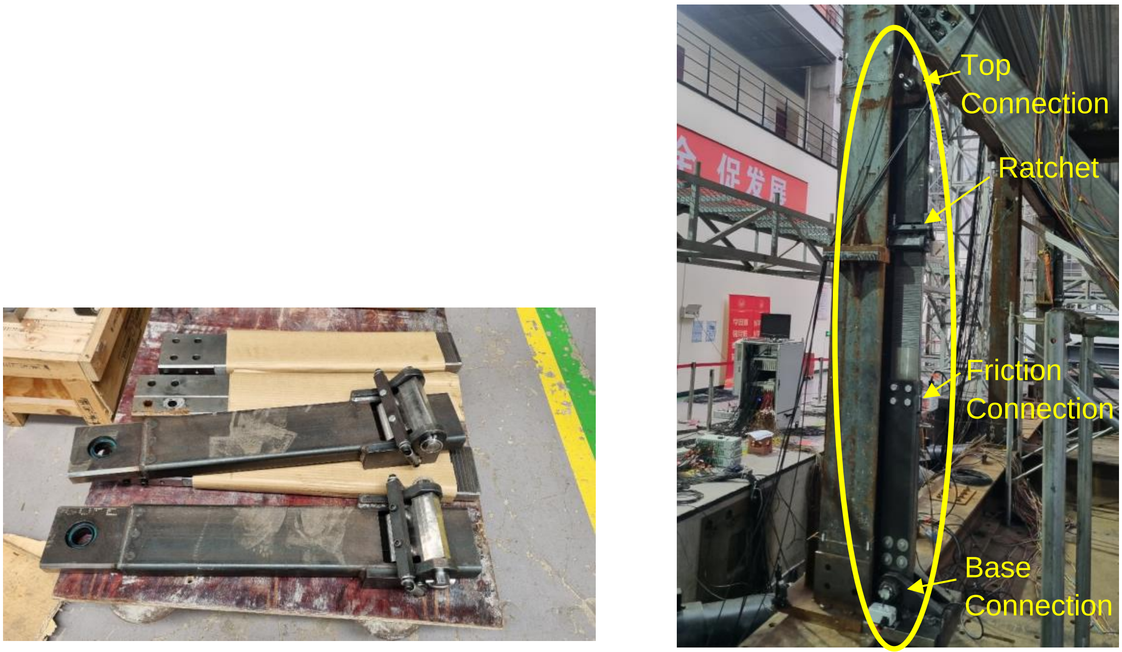
(b) Fixed Connection at Brace Top

(a) Longitudinal View *(c) Sliding Connection at Brace Bottom Before SHJ bolts removed*

Figure 10. Configuration 2c. SFC Braced Frame View (RH bay)

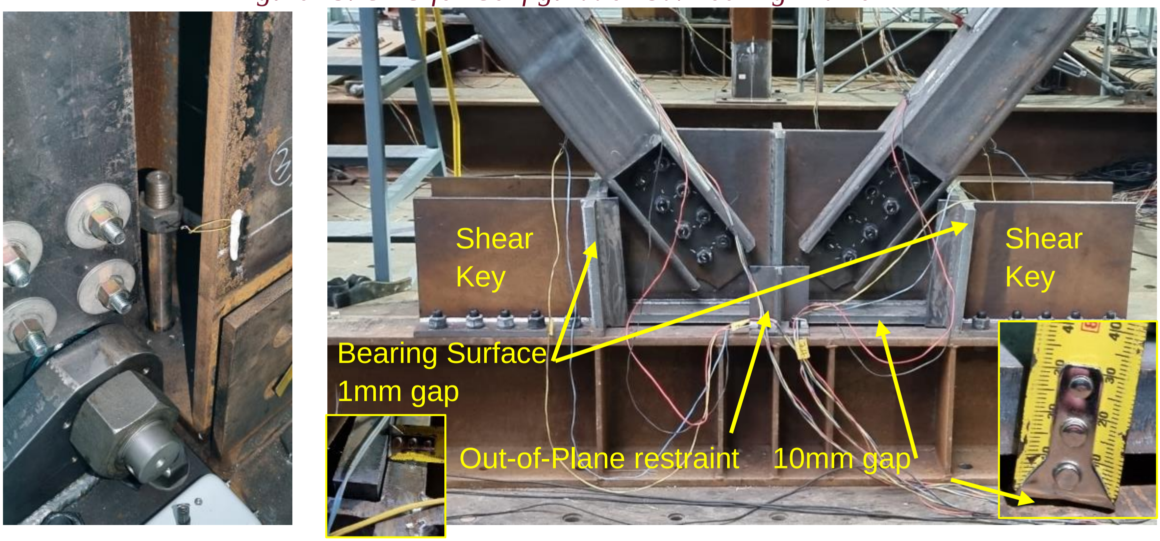
Figure 11. Zhenduo Yan Brace Shim Wear from Test 2c

Figure 12. Charles Clifton discussing testing with Prof. Zhao, Vice President of Tongji University and Yuao



(a) GNG Components developed by Rodgers *(b) GNG System Installed*

Figure 13. GNG for Configuration 3a. Rocking Frame



(a) Uplift Limit through Column Baseplate *(b) GNG System Installed*

(c) Fixed base central column (d) Beam connections beside central column (others pinned), 3a and 3b

Figure 14. Configuration 3a. Rocking Frame Details

Figure 15. Configuration 3a. Rocking Frame Column Uplift (up to 35mm)

Figure 16. Configuration 3a. Storey 2 Contents without Wheels

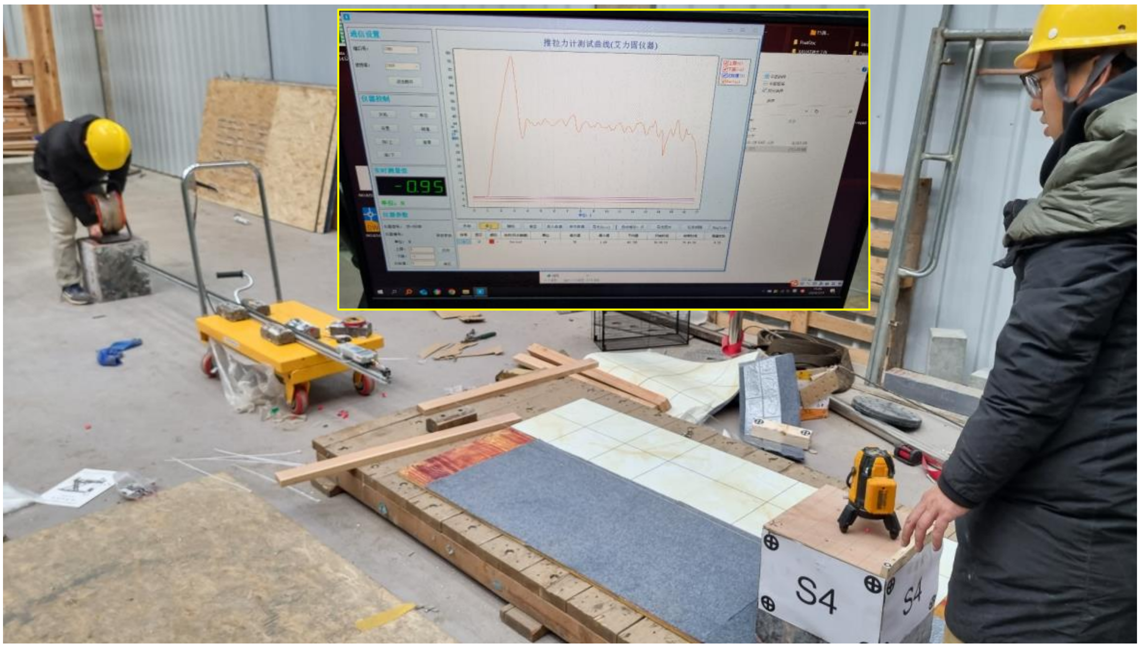
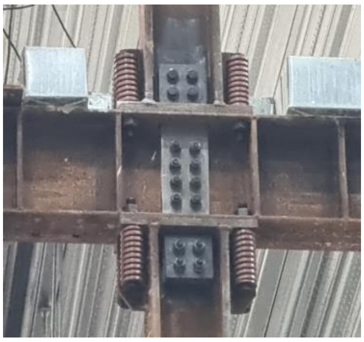


Figure 17. Contents without wheels Friction Tests on Carpet and Linoleum

(b) Separated External Columns at Each Level



(a) Transverse View *(c) Rocking Column Detail*

Figure 18. Configuration 3b. Rocking Column Configuration (Not yet tested)

(a) Configuration 3b. Storey 2 (not yet tested) *(b) Element tests*

Figure 19. Contents with Wheels (photos courtesy of Dr Zheng LUO)

(a) PCP support details *(b) PCPs with slotted holes*

Figure 20. Precast Concrete Panel (PCP) support details for upcoming NSE test, Config. 4.

3. BEHAVIOUR

At the paper submission deadline testing is underway and preliminary test information is only available for the first 6 test configurations as listed in Table 2. The test configurations performed well with the expected modes of deformation to more than 1.2 times the ULS shaking.

Table 2. Shaking Table Tests

Structural system	RSFJ - TOB	RSFJ - TCB	RSFJ - MRF	MRSF-OSHJ	CBF-V-SFCBeSs	MRSF-SHJ	CBF-D-SFC	CBF-D-SFCBeSs
Configuration	1.a	1.b	1.c	2.a	2.a	2.b	2.c s	2.c b
Direction	X	X	X	X	Y	X	X	X
Translational Period (s)	0.4	0.8	0.8	0.8	0.24	0.8	0.27	0.27
Maximum Input (g)	0.49	0.49	0.49	0.60	0.49	0.49	0.60	0.60
Table Peak Acceleration (g)	0.54	0.49	0.48	0.60	0.50	0.47	0.59	0.55
Base shear (kN)	305	294	271	289	343	330	316	278
Peak Roof Displacement (mm)	59.2	87.9	113	122	18.2	112	88	94
Peak Roof Drift (%)	0.66%	0.98%	1.26%	1.36%	0.20%	1.24%	0.63%	0.77%
Maximum Inter-storey Drift (%)	0.87%	1.11%	1.64%	1.72%	0.32%	1.56%	0.90%	0.97%

Other points are:

a) The slip between the foundation ringbeam and shake table surface in both directions was less than 0.01 mm in both directions so their displacements were considered to be the same.
b) Some noises were heard, and acceleration spikes were observed in the hysteresis loops for some configurations with large shaking intensity. Reasons for these, which may be due to impact from rocking or sliding mechanisms are being investigated. Nevertheless, in no case did this cause any poor behaviour of the frame.
c) The fundamental periods of the frames, as measured from white noise excitations, did not change for any of the configurations.
d) Residual displacements were small with the largest being less 1mm after the strongest shaking.
e) Testing was not conducted to with an input table acceleration of more than 0.6g, due to concerns of the laboratory staff in terms of preventing damage to the shaking table.
f) The central gravity column (nominally pinned) baseplate indicated no sign of yielding.
g) For the moment frames, there was no indication of sliding at the top flange level.

Further details will be available as the remaining 2 configurations are tested and data is analysed.

CONCLUSIONS

This paper describes some aspects of the ROBUST Project collaboration. In particular:

1) The ROBUST test involves the shaking table testing of a full scale 3 storey building with a steel frame and cold formed steel-concrete composite deck on the Jiading Campus shaking table at Tongji University, Shanghai, China. Energy is dissipated by means of different types of friction connections in frames with different structural configurations.

2) A photographic summary of the test configurations is given. These were constructed and seven of the nine major configurations in the sequence were tested before the paper submission deadline.

3) The first 6 structural configurations studied all behaved in the expected modes during, and for many beyond, the design shaking level. Further analysis of the test results will provide more information about the behaviour.

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