NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING
2019 Pacific Conference on Earthquake Engineering
TURNING HAZARD AWARENESS INTO RISK MITIGATION
4 – 6 April | SkyCity, Auckland | New Zealand

Shaking table test of a near full scale low damage structural steel building: structural aspects

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ABSTRACT

Recent severe earthquakes worldwide have put emphasis on building resilience. To achieve this procedures for low damage seismic design have been developed to satisfy the life safety requirement and to minimize the undesirable economic effects of required building repair or replacement following a severe earthquake. The performance of these buildings is dependent on whole building system interactions, which are difficult to determine by numerical modelling. The purpose of this project is to experimentally test the seismic performance of a complete, low damage, full scale building system incorporating a number of friction energy dissipaters in forms of sliding hinge joint (SHJ), resilient slip friction joint (RSFJ), symmetric friction connection (SFC) and GripNGrab (GnG). This will also incorporate testing without and with non-structural elements (NSEs) to quantify their effect on the building response. Testing will be based on appropriately scaled actual earthquake records using two 70-ton shake tables at Tongji University, Shanghai, China. Both axis unidirectional and biaxial horizontal testing will be undertaken. The structure is expected to have at worst minor damage under a series of severe earthquakes. The design also aims to have economical methods for repairing and straightening such building systems after severe

seismic activities, if there is a need. This paper focuses on the design of the structural part in this project, presenting the preliminary design of the structure. By the time of publication, the dimension of the structure is revised, resulting in corresponding changes to the structural design, which is not included in the paper due to a limitation of time but will be updated in the slides and poster.

1 INTRODUCTION

Severe earthquakes occur infrequently but place very high demands on structures. To economically allow for this, the concept of designing for controlled damage in a severe earthquake has been well developed and implemented for several decades. However, experience from severe earthquakes has been that, while this approach is excellent for preserving life safety, the repair costs and downtime resulting from the controlled damage is very high. To reduce the damage and downtime, there is a need to develop a low damage structural system which can be occupied immediately following a design level ultimate limit state (ULS) earthquake and should be repairable with low cost in a short time due to more severe earthquake. The 2010-2011 Canterbury earthquake sequence showed the performance of controlled damage designed steel structures was very good, with these either not needing repair or able to be readily and rapidly repaired (MacRae and Clifton, 2015). However even here the advantage of a low damage design has been clearly shown. Development of such systems has been underway in New Zealand before Canterbury earthquakes and has continued (MacRae and Clifton 2013; MacRae et al. 2018). This project will incorporate many of these developments into the shaking table test of a complete structure under specified levels of earthquake loading.

The SHJ connection is a low damage beam to column asymmetric friction connection (AFC) of steel moment resisting frame (MRF) developed by Clifton (2002 and 2005a). It has already been used in many commercial and residential multi-storey buildings. The behaviour of AFC with Belleville springs (BeSs) has been investigated by Ramhormozian et al. (2014). Design procedures of BeS for the application in the AFC and Symmetrical Friction Connections (SFC) is proposed by Ramhormozian et al. (2017). Final steps towards developing an optimum low damage seismic-resisting steel MRF system are reported by Ramhormozian et al. (2018).

The SFC can be considered as an efficient means to dissipate energy because they are characterized by stable hysteretic behaviour, low strength degradation and assembling cost comparable to conventional construction (Chanchi et al. 2013). Given the stable hysteretic response and the low strength degradation exhibited by this type of brace, it can be considered as a low damage dissipater that can be incorporated in different structural systems for dissipating seismic energy (Chanchi et al. 2015).

Rocking structures are designed to uplift under severe lateral seismic accelerations, thus reducing the seismic demands on the superstructure. The first steel building designed to rock was built in Wellington in 2007. A tension limiting brace base level hinge, consisting Ringfeder friction springs and vertically orientated hinge joint plates, is used (Gledhill et al. 2008). With this system, the damage is limited and the deformation is controlled. A ratcheting, tension-only, Grip and Grab (GnG) device has been developed to offer resistance to loading in tension, while offering negligible resistance to compression (Cook et al. 2018).

A three-storey steel building with a number of replaceable low damage systems is going to be tested at the International joint research Laboratory of Earthquake Engineering (ILEE) facilities, Shanghai, China. The purpose is to develop damage avoidance design for steel structures. For a better understanding of the dynamic response of considered structural systems, the shake table test is needed to evaluate the seismic performance subjected to simulated ground motion. There are mainly three objectives of this study: 1) demonstrate the effectiveness of a variety of realistic scale structural friction technology solutions for resilient large-scale structures under dynamic load conditions, 2) develop and evaluate the performance of non-structural technology for use in resilient buildings and 3) test economical methods for inspecting, and repairing damage, and for straightening such building systems. Throughout the shake table testing, a comprehensive understanding of building's seismic performance will be gained for future facilitating the engineering application. This paper introduces the structural design considerations for steel structure using low damage design solutions. Overview of ILEE steel project, design of RSFJ and details of NSEs are described in separated papers.

2 PROPOSED STEEL STRUCTURE AND DESIGN CONSIDERATIONS

The general view of the proposed steel structure, comprising three storeys, two bays by one bay, is shown in Figure 1 (a). The total height of the structure is 12 m; 4 m for each storey. The structure mainly contains two parts, the main frame (circled in red as shown in Figure 1 (b)) and the transverse frame (circled in black as shown in Figure 1 (b)). These two frames are connected by a custom-made horizontal steel I section with a steel web plate to transfer the horizontal shear. The structure sits on a steel ringbeam (as shown in Figure 1 (a)), which is bolted to the shake tables, instead of to a concrete foundation. The elevation of X and Y directions is shown in Figure 2. The mass distribution of the structure is shown in Table 1.

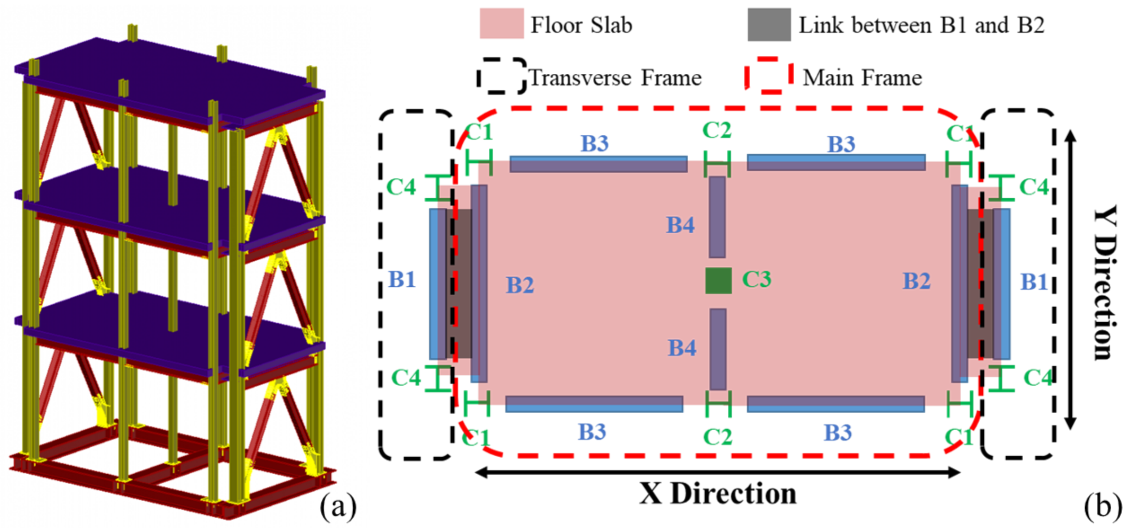

Figure 1: (a) General View of the Proposed Steel Structure and (b) Designed Plan Layout

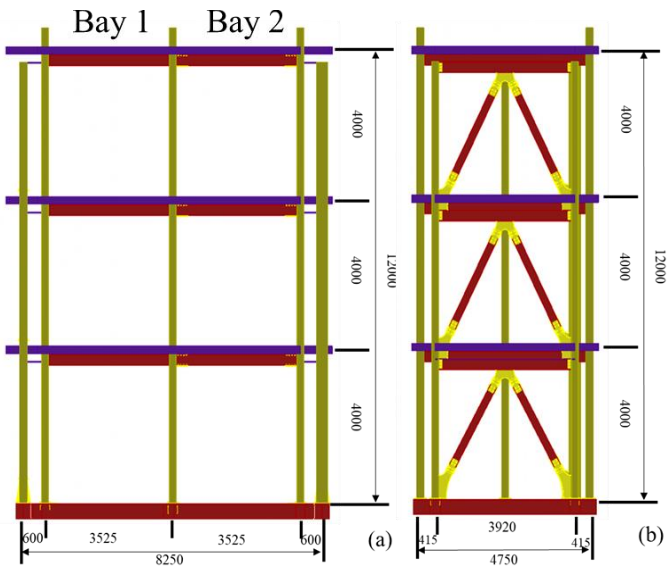

Figure 2: Elevation of Proposed Structure in (a) X Direction and (b) Y Direction

C3 is the gravity column located at the centre of the structure from bottom to the top continuously. B2 and B4 are gravity beams. The structure is designed in a way to be able to rapidly switch from system to system, as there are nine different systems or combinations of systems to be tested, as described in Table 3. The transverse frame is designed in a way that can be rocking or non-rocking as required to suit the different seismic-resisting systems. When it is a rocking frame, it means that the decking needs to be supported on a vertically stiff gravity supporting beam near the ends and then run on into the rocking transverse frame. The connection of the floor slabs into the transverse frame must be stiff horizontally to transfer diaphragm shear and ensure that the floor system acts as a rigid diaphragm, but needs to be vertically flexible to allow the rocking frame uplift to occur; this is the role of the flexible web link beam. The decking will terminate on the inside edge of the vertical supporting beam and an "I" section steel frame with a web thickness of 5 mm thick will be bolted into the two beams (B1 and B2). There will then be a stop edge to contain the concrete around the slab edge of the transverse frame with a soft lager rubber or ceramic fibre over the 5 mm plate and the slab reinforcement carried out to the normal edge detail over the transverse frame. As the transverse frame rocks, this flexible link beam will transfer horizontal diaphragm shear while allowing vertical uplift at the ends of the transverse frame to occur with reasonable ease.

Table 1: Estimated Mass of the Structure (tonnes)

	Composite floor	Additional mass	Frame	NSE
Level 0	0	0	4.49 (ringbeam)	2.83
Level 1	11.38	12.41	4.25	0.42
Level 2	11.38	12.41	4.25	11.24
Level 3	11.38	16.54	4.56	11.46
Total mass		119		

The structure is designed using SAP 2000 by Computer and Structures Inc. of Berkeley California based on following parameters:

- Ductility Factor, μ=3
- Structural Performance Factor, S_p=0.7
- Returning Period Factor, R_u=1.0
- Wellington, Soil Class C, Shallow Soil Sites
- Important Level, IL 2
- Hazard Factor, Z = 0.4
- Near Fault Factor, N (T, D)=1.0
- Near Fault Distance, D = 5 km

The sections of the structural members are designed according to the abovementioned parameters. The sections of the column and braces are listed in Table 2. The transverse frame (B1) is designed not to carry any gravity load. As outlined in HERA DCB 68 section 3.1 (Clifton 2002), the beam depth is chosen for gravity strength and lateral stiffness without impacting on the column design. However due to the constructability of SHJ (the beam flange width has to be big enough), a minimum beam section size of 310UB40.4 is required. This is also important for limiting the lateral drift of the frame when operating as a moment frame only in the longitudinal direction.

Table 2: Section Properties

Member	Code	Section	Area (mm^2)
Column	C1	200UC46.2	5900
	C2	200UC46.2	5900
	C3	200X200X5 SHS	3810
	C4	200UC46.2	5900
Beam	B1	310UB40.4	5210
	B2	310UB40.4	5210
	B3	310UB40.4	5210
	B4	310UB40.4	5210

3 STRUCTURAL SYSTEMS CONSIDERED

The structural systems being used are shown in Table 3. There are mainly four types of structural systems considered, namely MRF, braced frame (BF), dual system and rocking frame (RKF). Periods and inter-storey drifts for each type are shown in Table 4. There are three different types of MRF. The first type is using SHJ as beam to column connection in one bay with another bay pinned (MRF-1). The second type is using RSFJ in one bay with another bay pinned (MRF-2). The third type is a combination of SHJ in one bay and RSFJ in another bay (MRF-3). There are four types of BF system considered. The first type is using SFC diagonal brace in one bay in X direction (the other bay is pinned, noted as CBF-D in Table 4). The second type is using RSFJ tension only braces in one bay in X direction (the other bay is pinned). The third type is using RSFJ tension and compression brace in one bay in X direction (the other bay is pinned). The fourth type is using SFC inverted V brace in Y direction on transverse frame (CBF-V). The dual system is a combination of MRF and CBF systems. There are two types of RKF. The first type is incorporating GnG and the second one is incorporating Asymmetric Flexible Plate (AFP). NSE will be installed on the structure after the testing of first three major types. The structure will then be tested with full NSE (type 4).

Table 3: Considered Structural System

Type	Subtype	Longitudinal (X)		Lateral (Y)
		Bay 1	Bay 2	
1	a	BRC CTB RSFJ	Pinned	CBF SFC STD Weak
	b	BRC TOB RSFJ	Pinned	CBF SFC STD Strong
	c	Pinned	MRF-RSFJ	CBF SFC BSW Weak
2	a	Pinned	SFC *** Strong	CBF SFC BSW Strong
	b	Pinned	Dual	CBF SFC BSW Strong
3	a	Pinned	MRF-SHJ	RKF AFP
	b	MRF-RSFJ	MRF-SHJ ***	RKF GnG
4	a	MRF-RSFJ	MRF-SHJ ***	CBF SFC *** Strong
	b	MRF-RSFJ	MRF-SHJ ***	RKF GnG

Notation:

AFP = Asymmetric Friction Plates	RSFJ = Resilient Slip Friction Joint
BRC = Brace	RKF = Rocking Frame
BSW = With Belleville Washers	SFC = Symmetric Friction Connection
CBF= Concentrically Braced Frame	STD = Standard with No Belleville Washers
CTB = Compression/Tension Brace	Strong = More Bolts Installed
Dual = SFC Braces + MRF SHJ ***	TOB = Tension Only Brace
GnG = GripNGrab	X = Longitudinal Direction (8.25 m)
Weak = Less Bolt Installed	Y = Transverse Direction (4.75 m)
Pinned = Pinned beam ends	*** is STD or BSW depending on performance in Type 1 tests

Table 4: Periods and Drift of each System

Structural Type	Period (s)	Inter-storey Drift		
		Level 1	Level 2	Level 3
MRF-1 / MRF-2	1.16	1.28%	1.48%	1.04%
MRF-3	1.02	1.18%	1.24%	0.82%
Dual System	0.38	0.36%	0.44%	0.41%
CBF-D	0.41	0.39%	0.50%	0.39%
CBF-V	0.27	0.17%	0.30%	0.30%

4 LOW DAMAGE CONNECTIONS

4.1 MRF incorporating SHJ

The SHJ is intended to be rigid under serviceability limit state (SLS) conditions and become semi-rigid to occur in ULS earthquake. Beam-to-column rotation can be observed during this stage. This rotation about the top flange, acting as pivot point, effectively isolates the floor slab limiting additional demands to other structural members. At the end of the earthquake, the joint seizes up and becomes rigid again. The design procedure of MRF incorporating SHJ has been first proposed by Charles (2002) and then reviewed for several times (Charles 2005a, 2005b, 2007a, 2007b and 2011). A designer summary of key lessons learnt based on observations from implementing several building projects with SHJ has been performed by Gledhill et al. (2013). The joint is designed based on abovementioned references. The SHJ layout is shown in Figure 3.

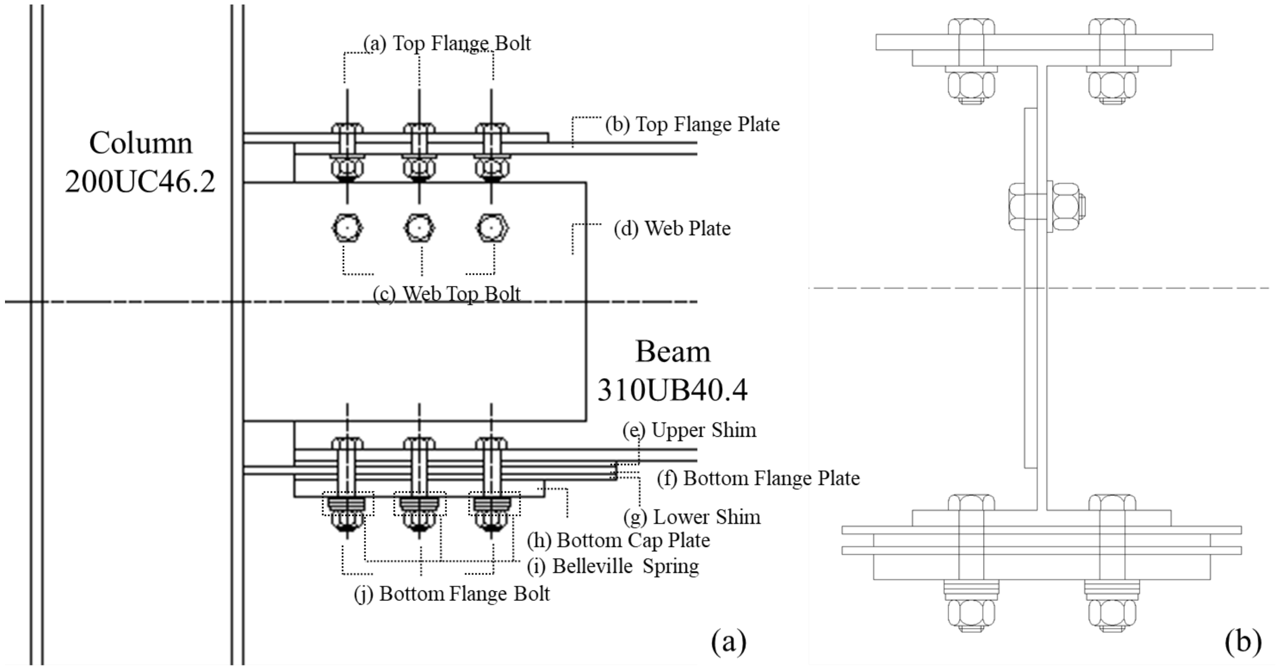

Figure 3: Sliding Hinge Joint Layout (a) Lateral View and (b) Beam Cross Sectional View

As can be seen, there are no bottom web bolts used in the design SHJ. The sliding will only take place at the bottom flange level where the AFCs are located. The reason is the joint would be too strong with the presence of bottom web bolts to be activated due to a design level earthquake. This is to reduce the effect of prying. Instead of using two row of bolts at bottom flange level, three rows of bolts are to be used. To accommodate the design moment (see Table 5) with three rows of bolts, M20, M16 and M12 structural bolts are used at level 1, 2 and 3.

Table 5: Design Actions

Level	Design Moment (kN*m)	Design Shear (kN)
1	71	72
2	54	61
3	23	34

BeSs are used at the bottom flange level to provide sufficient residual strength following an earthquake, significantly reducing the degradation of bolt tension (Ramhormozian et al, 2017). For repairing of the structure, only retighten of the bolts are expected following a ULS earthquake. Replacing of the bolts may be required beyond ULS earthquake. Behaviours of SHJ with BeS is under development using multi-plastic kinematic hysteresis loop in SAP 2000 for further analysis. A design summary using M20 bolt at bottom flange level is shown in Table 6 as an example.

Table 6: Joint Design Summary

BOLTS	Number	Size	
Top Flange	6	M20	
Web Top	3	M20	
Bottom Flange	6	M20	
PLATES	Thickness (mm)	Width/Depth (mm)	Length (mm)
Top Flange	8	210	295
Web Plate	12	230	295
Bottom Flange	12	210	300
Bottom Cap	16	210	235
PANEL ZONE			
90*16 stiffeners each side of web in line with top and bottom flange plates			
Doubler Plates Required (16PL)			
SHIMS	Thickness (mm)	Width/Depth (mm)	Length (mm)
Bottom Flange Shims (Upper)	5	250	245
Bottom Flange Shims (Lower)	5	250	235
SETUP			
Beam to Column Gap (mm)		55	
Slotted Hole Length (mm)		45	

4.2 Low damage connection at column base

The low damage AFC connections at the column base has two variations, column base strong axis-aligned asymmetric friction connection (SAFC base) and column base weak axis-aligned asymmetric friction connection (WAFC base). The cyclic performance of both column bases has been studied by Borzouie et al. (2015a and 2015b). Both of them (see Figure 4) have the ability to tolerate high levels of drift without significant strength degradation. They can be considered as low damage connections.

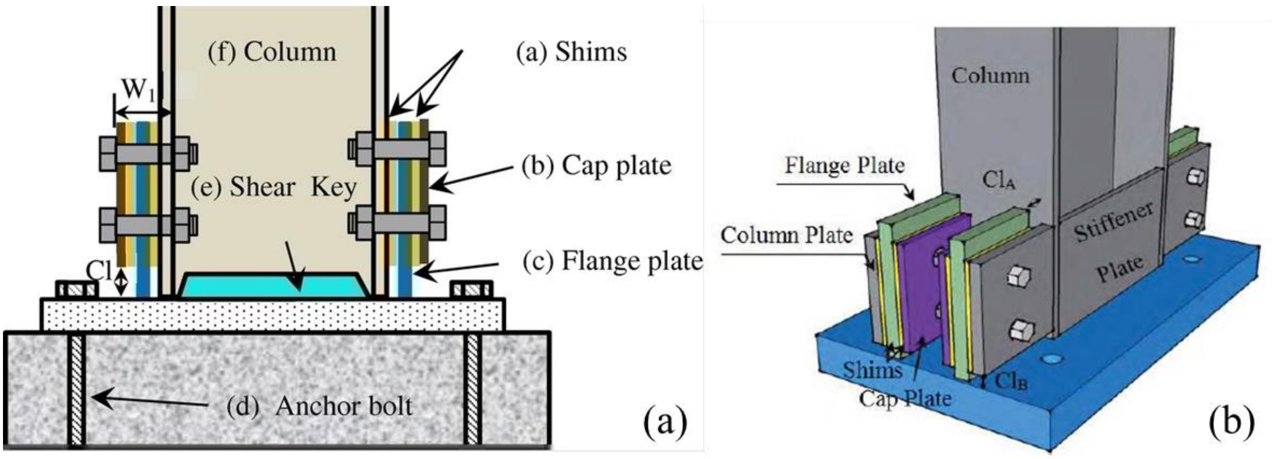

Figure 4: (a) SAFC Base (Borzouie et al. 2015a) and (b) WAFC Base (Borzouie et al. 2015b)

As for the application in this project, WAFC base requires additional plates outside the column flange and a stiffener plate (see Figure 4 (b)), which takes more space and adds complexity. A major concern is all the parts should be easy to install and uninstall as a number of structural systems are to be tested. The changeover time from system to system should be relatively short. SAFC base is design in a similar fashion as a sliding hinge joint. It requires additional plates (cap plate) parallel to the column flange welded to the baseplate. The shear key is bolted to the ringbeam through the baseplate. Along the edge of the shear key, an angle is designed to allow the column sitting back to original position after uplifting occurs. Other type of shear keys can be easily installed when there is another purpose. The constructability of SAFC base is better in this case. Therefore, SAFC base is selected in this project (see Figure 5). Any extreme rotation takes place in a sliding mode between two plates bolted together.

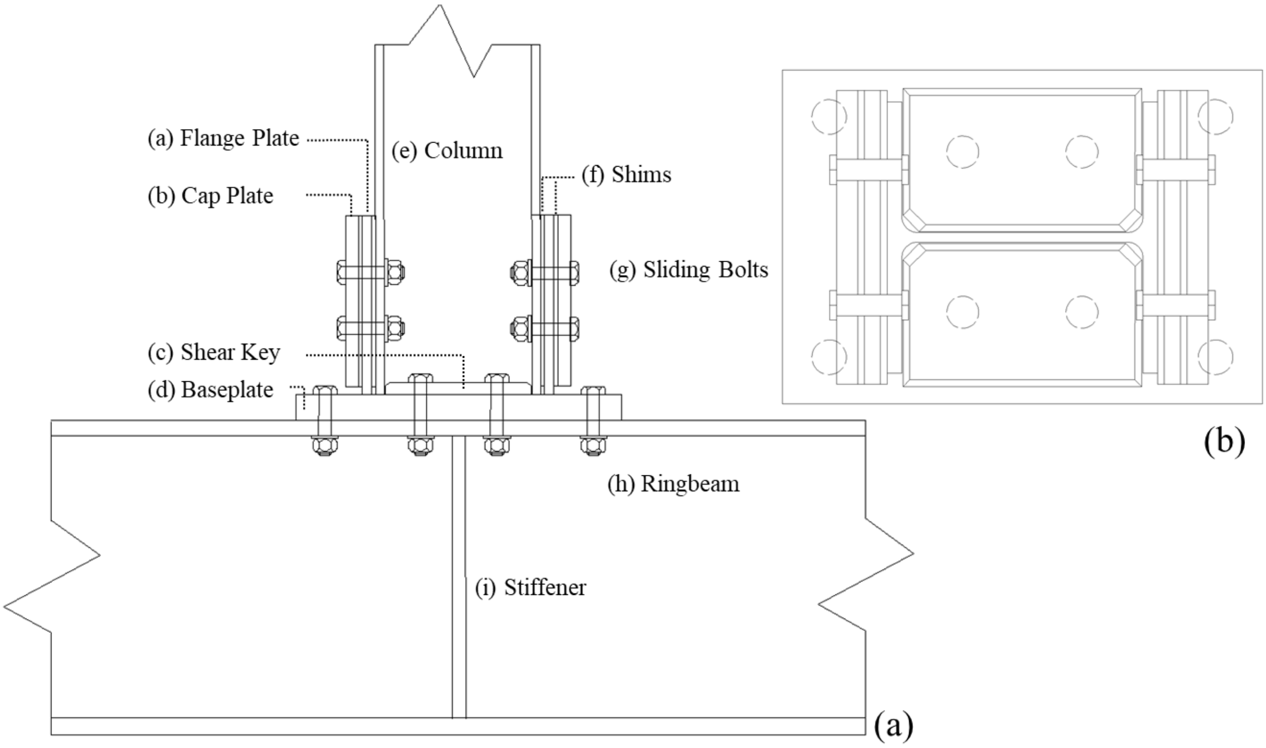

Figure 5: SAFC Base (a) Front View and (b) Plan View

The design actions is obtained from load combination 1.0 G + 1.0 Q and E_μ only with a return period of 500 years. The maximum required base moment, $M^*_{required}$, is given in Equation 1, where M_{Slide} is the moment resulting from sliding friction, M_{Prying} is the elastic-prying moment mainly from flange plate bending on the compression side of the column, and M_{Axial} is the moment the moment from axial force. In the equation, n_{Bolt} is the number of the bolts in each AFC. Fs is the sliding force for each bolt and d is the length of the lever arm. θ_{Base} is the base rotation from SAP 2000, H_{fp} is the distance from the top of then flange plate to the base plate, I_{fp} is the second moment of area about the weak axis of the flange plate and P is the vertical joint reaction force obtained from SAP 2000.

$$M^*_{required} = M_{Slide} + M_{Prying} + M_{Axial}$$

$$= n_{Bolt} * F_s * d + \frac{\theta_{base} * 3EI_{fp}}{H_{fp}} + \frac{d}{2} * P \tag{1}$$

4.3 Brace using SFC

The SFC is assembled by clamping the brace section, the slotted plate, and the shims if needed by means of high strength bolts (i.e. Grade 8.8 bolts) tensioned up to the proof load. To ensure a stable hysteretic behaviour of the brace, the slotted plate or the shims depending on the SFC detail configuration should be manufactured with a material with greater hardness than the hardness of the brace material. SFC has a stable hysteretic behaviour, low strength degradation and comparable assembling cost to conventional construction. The hysteresis loop of SFC braces is almost rectangular. Materials such as Bisalloy 400 or Bisalloy 500 can be considered as alternatives to steel for SFC as reliable and low damage dissipators (Chanchi et al. 2013).

The conceptual column base details of BF system using SFC is shown in Figure 6. A slender vertical plate with oversized holes is welded to top of the shear key and bolted to the column web, limiting the possible uplift occurring at the column base. The shear key is bolted to the foundation ringbeam through the baseplate A. The rest part of the connection is the same as the abovementioned SAFC base. The brace is located on baseplate B, which is bolted to the top flange of the foundation ringbeam, separated from baseplate A. This makes the changeover activity (switch from one system to another) fast and easy to do. The brace can be easily added or removed when required. The slotted holes are designed in the gusset plate to provide a perfect SFC rather than in the brace. Instead of channel sections, RHS (back to back) is selected to provide stable out-of-plane behaviour.

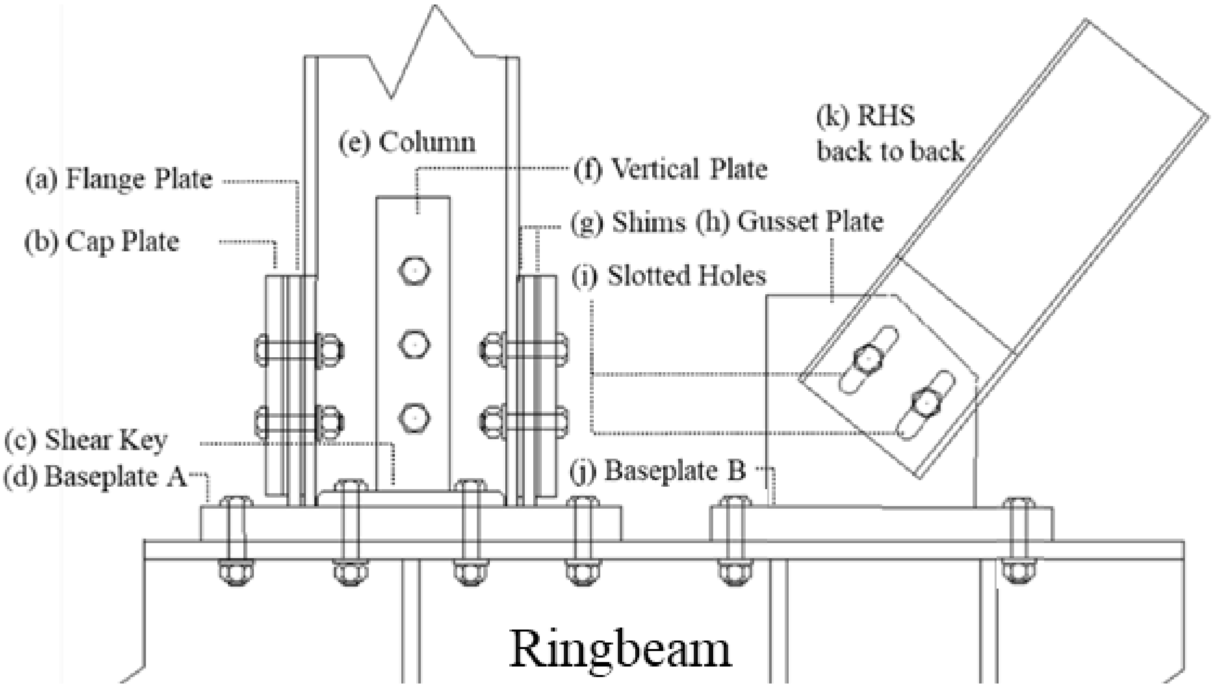

Figure 6: Column Base Connection of BF system

4.4 Rocking frame incorporating GnG

The rocking frame is designed at the transverse frame, which is a concentrically braced frame (CBF) with inverted V brace. Instead of using SFC braces, conventional braces (no friction) are used. The conceptual drawings of the GnG device and the column base connection are shown in Figure 7 (a).

Comparing to the column base connection of CBF system (see Figure 6) which does not allow rocking, the vertical plate is removed and the shear key is replaced with a steel tube (filled with concrete inside). GnG (see Figure 7 (b)) is a tension-only dissipater device developed to offer resistance to loading in tension, while offering negligible resistance to compressive motion (Cook et al. 2016). GnG device can be installed on the outside of the column flange or along the column web. For the convenience of constructions and changeover activities, GnG device is attached to the outside of the column flange.

The dissipative element would need to be replaced, and the device reset, after every major event. It is worth to mention that the GnG device is not supposed to be post-tensioned. It is designed to work together with the frame to resist forces of 100% of the base moment caused by the design level elastic force (DLEF). During the analyses, a lower force, 67% of DLEF, is used.

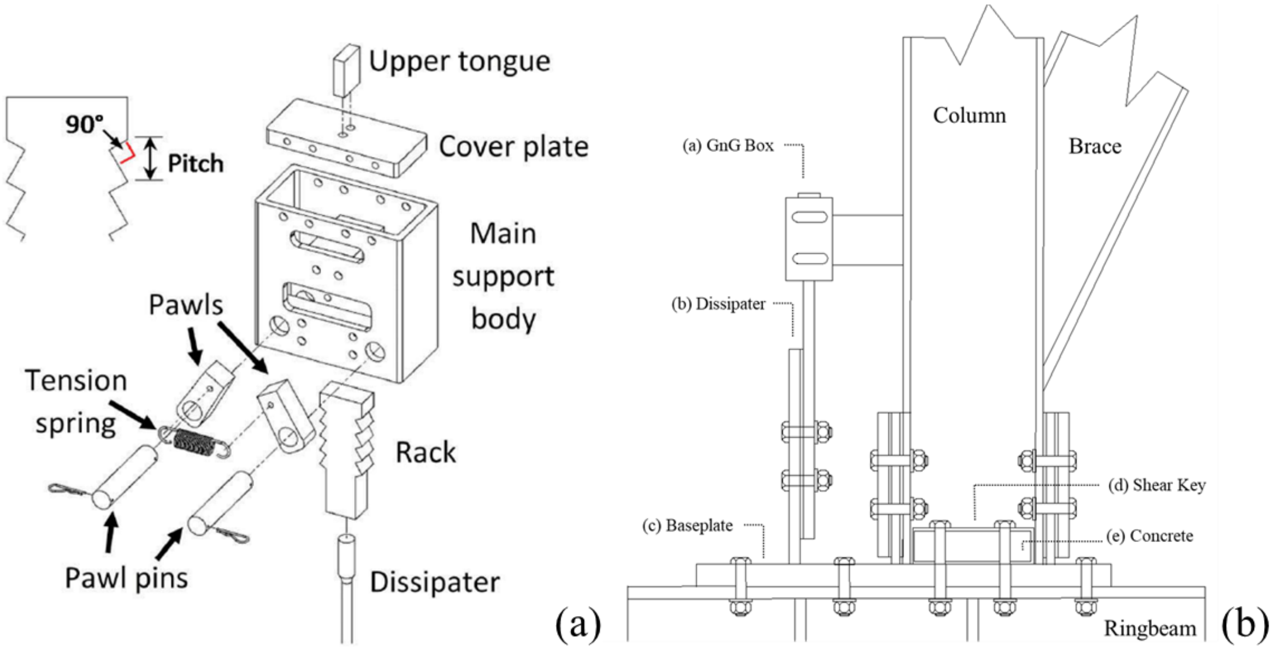

Figure 7: (a) GnG Device (Cook et al. 2018) and (b) Column Base Connection

5 GROUND MOTION AND LOADING PROTOCOL

The ground motion selection follows the regulation of New Zealand Loading Standard. The ground motion are scaled based on the design considerations. The strong earthquake record is EI Centro record from the 1940 California Imperial Valley earthquake (EI Centro). When the structure is loaded bi-directionally, one direction is set as the primary direction and the peak ground acceleration (PGA) in another direction is proportioned by a reduction factor of 0.85.

The test is composed of 8 phases as listed in Table 7. Under SLS and ULS stages, the structure is expected to stand minor damage. During the last two phases, the tested structure is subjected to severe earthquakes and the main frame may yield.

Table 7: Test Phases

Phases	Name	Hazard Level	
1	SLS	Service Limit State	10%
2			50%
3			100%
4	ULS	Ultimate Limit State	50%
5			100%
6			150%
7	MCE	Maximum Considered Earthquake	-
8	GM-MCE	Ground Motions Exceeding MCE	-

The draft loading protocol is shown in Table 8. The earthquake record is scaled to represent these hazard levels. To identify the contribution of NSEs, no permanent or unrepairable damage is allowed before testing the structure with full NSE on. For the testing of Type 1 to 3 (without NSE), the structure may not be tested up to MCE level. Impulse followed by free vibration (I/FV) scanning is conducted to obtain the natural period of the structure at different stages.

Table 8: Draft Loading Protocol

Sequence Number	Phase	Input	PGA (g) X	PGA (g) Y
1		I/FV		
2		EI Centro	0.01	0.01
3	SLS	EI Centro	0.05	0.05
4		EI Centro	0.10	0.10
5		I/FV		
6		EI Centro	0.20	0.20
7	ULS	EI Centro	0.40	0.40
8		EI Centro	0.60	0.60
9		I/FV		
10	MCE	EI Centro	0.72	0.72
11		I/FV		
12	E-MCE	EI Centro	0.90	0.90
13		I/FV		

6 CONCLUSIONS

This paper presents the preliminary plans and drawings of the proposed structure, reports on design progress that have taken place to date, and describes about the testing phase. The low damage structural systems being considered of the structure are discussed. Design example of SHJ with BeS is given. The use of SAFC base, low damage brace using SFC and rocking frame with GnG are discussed, respectively. The testing will be conducted at ILEE facilities, Shanghai, China. This test is expected to provide an exemplar of how economic resilient technology can protect the whole building.

7 ACKNOWLEDGEMENTS

This work described is part of a joint NZ-China research programme with the International Laboratories on Earthquake Engineering (ILEE), Tongji University, Shanghai, China. Direct NZ funding is kindly provided by the Building Research Association of NZ (BRANZ) under the Building Research Levy, the Earthquake Commission (EQC), QuakeCentre, the Tertiary Education Commission funded QuakeCoRE (the NZ ILEE partner through whom the NZ funding is also coordinated), and the University of Auckland (UA). Donation of materials is kindly provided through ComFlor, the NZ Heavy Engineering Research Association (HERA), and Steltech. Expertise has been generously provided by staff at Auckland University of Technology (AUT), Tongji University, the University of Auckland, the University of Canterbury (UC), as well as by a number of NZ industry representatives. The authors gratefully acknowledge this support. Opinions expressed are those of the authors alone. The QuakeCoRE paper number is 0374.

The first author would like to thank the University of Auckland for providing the Doctoral Scholarship to perform his Ph.D. research.

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