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Lincoln University Waimarie building: An application of friction damping devices with recentring for low damage design

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ABSTRACT

The experience from Christchurch and Kaikōura seismic events demonstrated that the 'life safety' criterion, on its own, is not sufficient to protect the community from earthquake impact. Nowadays, the community expects more and expects measures taken to protect not only lives but also assets and maintain business continuity. The structural engineering community of New Zealand has moved accordingly and started to incorporate low-damage concepts in the seismic design of structures. There have been many examples of implementing one or a few aspects of low-damage design in buildings, but is this enough? The latest research shows that a structural system can only be resilient when aspects of low-damage design are considered and addressed (e.g. performance of the system as a whole). The Lincoln University project is a new flagship science building, named Waimarie (bountiful lakes). The project comprises a Teaching Building and Research Building. This paper presents this project as a case study where innovation made low-damage design possible. The paper presents the design concept, challenges, and outcomes. The lateral system for the Research Building included non-post tensioned rocking concrete shear walls (with innovative hold-downs) and resilient diagonal steel braces. The paper discusses the methods used for analysis and design, highlights the superior performance of the rocking concrete walls used and presents the innovative bracing system implemented. As a case study that has incorporated aspects of low damage design, this project will be of interest for those excited about resilient buildings and can lead the way towards having a seismic resilient built environment.

1 INTRODUCTION

The Canterbury Earthquakes resulted in significant damage across Canterbury. Many buildings were damaged, some requiring demolition or extensive and costly repairs. This held true for the buildings on Lincoln University's campus, which is located approximately 30 km from the Darfield fault. As a result of the Darfield earthquake, several buildings on the campus required demolition and varying degrees of damage occurred to other buildings. More than 12 years on from the Darfield earthquake, the campus is now in a significantly improved state, with several new and refurbished buildings. The centrepiece of Lincoln University's campus refurbishment programme is Waimarie, which will be Lincoln University's flagship science facility. This building will house modern, capable laboratories and equipment, enabling Lincoln University to maintain its position as one of the top agricultural universities globally. The facility will also allow close collaboration with AgResearch, a Crown Research Institute specialising in agricultural research, located immediately adjacent to the Waimarie site.

Beca was engaged as the structural engineers for Waimarie. Obviously, there was the possibly to provide a conventional structural system for the building. But the client driver was to find an alternative solution that provided improved performance and seismic resilience at a similar cost to that of a conventional structural system.

2 PROJECT DESCRIPTION

Waimarie comprises a Teaching Building and Research Building, located adjacent to one another but separated by a seismic gap, as well as a standalone Ancillary Building. Refer to Figure 1 for an overview of the structure for the buildings.

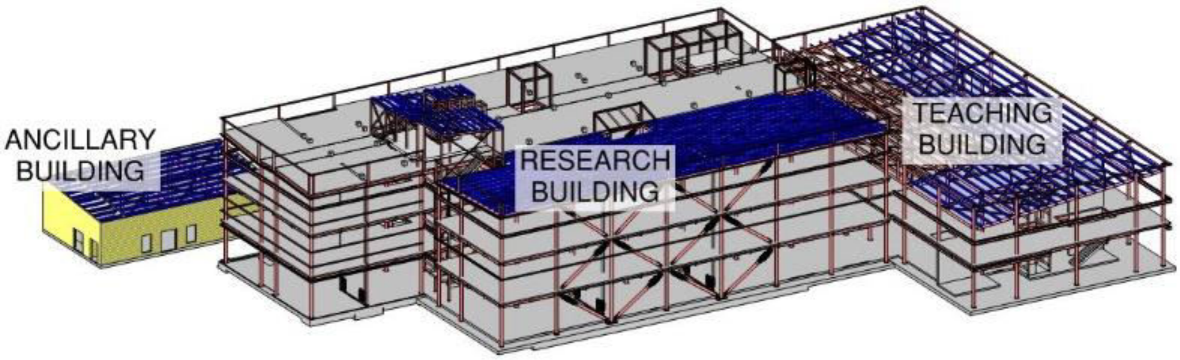

Figure 1: Waimarie building layout (note CBF bracing layout)

The Teaching building is two-storey structure with a conventional lateral load resisting system, namely, precast concrete shear walls. A unique aspect is the building's façade which includes curved brick veneer fins that cantilever from the main glazing line. These brick elements provide a materiality link to the adjacent historic Ivey building. Cantilevered precast shear walls provide a stiff lateral system, with the intention to minimise displacement demands on the façade.

The Research building is a three-storey structure which includes state-of-the-art research laboratories and workplaces. The lateral system for the Research Building initially included concrete shear walls and steel concentrically braced frames (CBFs) in the transverse and longitudinal directions respectively (refer Figure 2). The primary driver for introducing shear walls and CBFs was to provide a stiff lateral structure that limited interstorey drifts. CBFs were positioned on three bracing lines, the northern and southern elevations, and a central gridline in the middle of the building. CBFs are an efficient structural lateral system and were considered well integrated to the architecture on the perimeter of the building and at the interface between

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the laboratories and workplaces. Extending the CBFs over four bays allowed an increased leaver arm to resist overturning actions, which in turn reduced demands on the columns and foundation.

Six shear walls were positioned on four bracing lines. Four of these walls were integrated into the laboratories, located either side of a central corridor and one bay in from each end of the laboratory. The advantage of bringing the shear walls in one bay from the ends of the building allows for additional gravity loading to counteract uplift forces at the ends of the walls. Although the research laboratories were heavily serviced, the chosen shear wall arrangement kept the central corridor free of vertical structure. The two remaining walls were placed in the offices with even distribution to those in the laboratory. The walls are between 5.15 m to 5.85 m long, 300 mm thick, and three storeys high with no construction joints along their height. Refer to Figure 3 for an elevation of the shear walls.

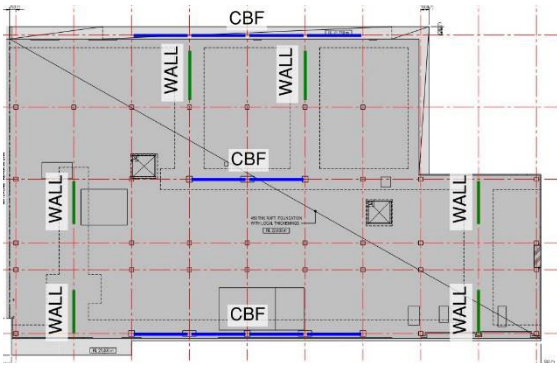

Figure 2 – Research Building lateral load resisting system

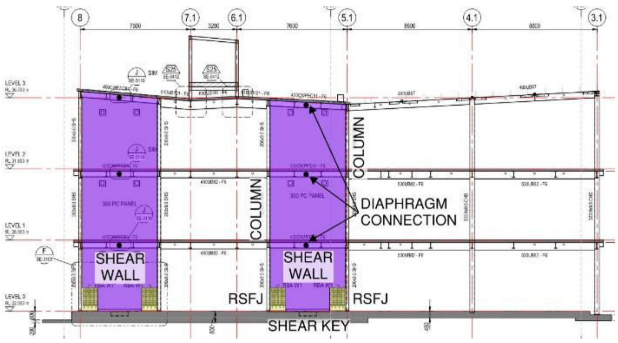

Figure 3 –Section across building showing rocking shear walls with RSFJ devices

During concept design consideration was given to implementing Resilient Slip Friction Joints (RSFJs) at the ends of shear walls and at one end of braces in the CBF direction. The driver for the introduction of these devices was to significantly improve the seismic performance of the structure at a similar cost to a conventionally designed structure. The introduction of RSFJs allowed to concentrate energy dissipation at the RSFJs and design the remaining building elements and connections for the corresponding building movements to avoid structural damage up to an Ultimate Limit State (ULS) event. The devices also exhibit recentring, meaning any residual building drifts should be avoided. The RSFJ devices could also be designed to limit device movements so that interstorey drifts could remain limited. An added advantage of shear walls with RSFJs is that the shear walls act as a stiff backbone and promote movement in the RSFJ devices. In the CBF direction, there is no deliberate spine element, however continuous columns assist with mobilising the devices up the height of the building.

3 RSFJ TECHNOLOGY AND PRIOR LITERATURE

3.1 The Resilient Slip Friction Joint (RSFJ)

The Resilient Slip Friction Joint (RSFJ) technology was introduced in 2015 (Zarnani and Quenneville 2015). This device is a friction device that can dissipate the seismic energy and provide a re-centering behaviour with a flag-shaped hysteresis up to a defined level of deformation. Figure 4 depicts the configuration and the load-deformation relationship of the RSFJ. Similar to conventional friction joints, the RSFJ dissipates energy via the sliding movement of the clamped plates. Figure 4(a) shows the assembly and different components of an RSFJ specimen. The hysteretic parameters (F_{slip}, $F_{ult,loading}$, $F_{ult,unloading}$, $F_{residual}$ and Δ_{max}) shown in the figure can be determined in accordance with the design requirements. The behaviour and performance of the RSFJ have previously been verified via joint component tests and large-scale experiments. Hashemi et al. experimentally tested full-scale rocking Cross Laminated Timber (CLT) (Hashemi et al. 2017) and

Laminated Veneer Lumber (LVL) (Hashemi et al. 2020) walls with RSFJ hold-own connections. Bagheri et al. (Bagheri et al. 2020) tested steel tension-only braces with RSFJs and Yousef-Beik et al. (Yousef-Beik et al. 2019) investigated the performance of tension-compression braces with RSFJs.

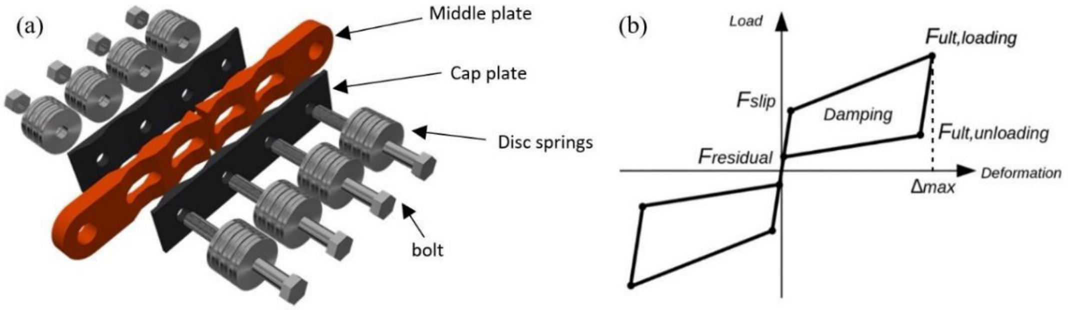

Figure 4: Resilient Slip Friction Joint (RSFJ): (a) configuration (b) hysteretic behaviour

This technology has made its way to practice two years after its introduction and has been implmented on a handful of projects in New Zealand and Canada. Some exmaples of New Zealand implementions are Nelson airport terminal, Catholic Cathedral College in Christchurch, Hutt Valley Health Hub in Wellington and 66 Oxford Terrace in Christchurch.

3.2 The over-strength mechanism of the RSFJ

The RSFJ is designed in a way that all components remain elastic up to the design load (F_{ult} of the device). However, with the aim of maintenance of an acceptable hierarchy of behaviour of the "brace" system if the applied loads are higher than the design earthquake loads, an over-strength mechanism in the body of the RSFJ is considered. When the load on the RSFJ increases beyond its maximum capacity (F_{ult}), the clamping bolts (or rods) start to yield. Inelastic elongation of the bolts provides additional travel distance for the joint allowing it to maintain a ductile behaviour whilst bolts travel across the slotted holes in the RSFJ middle plate. With this mechanism activated, the maximum force will be 1.25 times higher than the design F_{ult}. So, the over-strength factor applicable for the RSFJ is a minimum 1.25 to maintain the hierarchy of strengths following the capacity design principles. An over-strength factor of 1.35 is usually recommended for the attachments and the main structural members.

Figure 5 shows an example of a secondary fuse test performed on an RSFJ configuration related to the case study described in this paper. The specified characteristics of this configuration (ULS) were $F_{ult,loading}$=375 kN at Δ_{ult}=60 mm displacement. As can be seen, the red dashed rectangle shows the hysteretic performance at the specified design values, and the green dashed rectangle demonstrates the over-strength performance. It can be seen that the capacity of the device increased to about 450 kN, corresponding to 1.2 times of the F_{ult}. Also, the device continued displacing to 100 mm, which is about 1.7 times the design level displacement. Therefore, the available extra force and displacement provides additional capacity for events in excess of ULS. Structures with RSFJs can be analysed and investigated for scenarios in excess of ULS to confirm the structure, RSFJs and their connections meet the appropriate requirements for such scenarios.

4 DESIGN CRITERIA AND METHOD

The design method for the lateral system followed a displacement-based design approach (M.J.N. Priestley, G.M. Calvi 2007) with non-linear time history analysis to validate the final design solution. This section describes the adopted design process and key design considerations.

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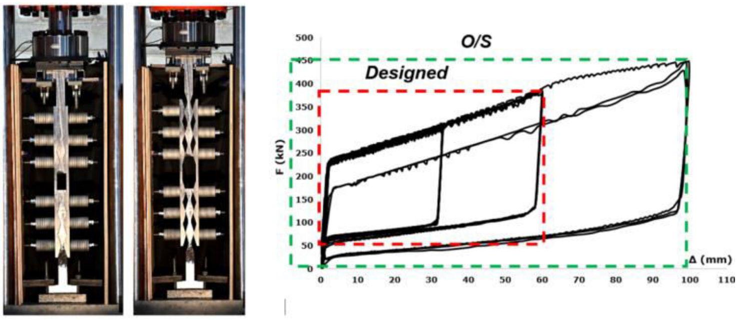
4.1 Design and Performance Criteria

Table 1 describes the earthquake design parameters were used in the design. These parameters are consistent with NZ 1170.5, except S_p was reduced following the recommendation of Marriott (2018) given a displacement- based design framework was adopted. Table 1 also includes the target drifts for the limit states.



(a) (b) (c)

Figure 5: Experimental testing on the RSFJ over-strength mechanism

Table 1: Summary of design and drift criteria.

Design Parameter	Serviceability Limit State (SLS)	Ultimate Limit State (ULS)	Collapse Avoidance Limit State (CALS)
Return Period	1/25	1/1000	1/2500
Return Period Factor, R	0.25	1.3	1.8
Structural Performance Factor	0.85	0.85	0.85
Target Drift	0.3%	1.75%	N/A

4.1.1 Serviceability Limit State

A key consideration for the design of the building was providing a stiff structure at the serviceability limit state to avoid the need for deflection head detailing, noting the complexity and costs associated with this detailing for containment laboratories, whilst avoiding damage at this limit state. A SLS drift limit of 0.3% was adopted to prevent damage to GIB partition walls at SLS even if these walls are translationally fixed at floor levels. The intention was for the RSFJ's to not experience joint slip at SLS. This was verified by ensuring each RSFJ link remains within the initial (nonslip) stiffness portion of the flag-shape curve. Noting that the RSFJ properties can be adjusted to meet the required force-displacement performance characteristics (varying bolts, springs etc). A second Serviceability Limit State (SLS2), with corresponding greater return period, was not adopted as this would introduce additional cost to the construction of the building. Noting the objective was achieve increased performance at minimal additional cost.

4.1.2 Ultimate Limit State

At ULS the design intent was for RSFJs to remain within the repeatable performance range. Namely, the range after the device slips (opens) and before permanent deformation occurs to the any parts of the device. This was done by determining the displacement demands on the devices against the ULS displacement capacity. The remaining structure and connections were designed to achieve an acceptable hierarchy of behaviour of the “brace” system if the applied loads are higher than the design earthquake loads including detailing to accommodate for the resulting building movements. The ULS drift of the building was calculated to be 1.75% with contributions of 1.5% and 0.25% from the superstructure and foundations respectively. The difference between the SLS drift and ULS drift can attributed to the behaviour of the devices and the performance criteria that were set. Namely for the building to remain stiff at SLS with no slip in the devices and within the repeatable bounds of the devices at ULS.

At ULS, validation of the design should include confirmation that there is sufficient structure present to force the devices to work over the height of the structure and just not predominantly at the level where slip first occurs.

4.1.3 Collapse Avoidance Limit State

There are no explicit requirements with the New Zealand Loadings Standards to check for an event greater than ULS. However, the commentary to NZS 1170.5 provides an example for a special study including determination of detailing capable of dependably sustaining the deformations resulting from the design level event and having sufficient reserve capacity for a rare earthquake event (quoting a 2500-year return period event). Accordingly, analysis was done to determine the displacement demands imposed on the devices at a 2500-year return period event and was a robustness check against device failure.

Similarly, as for ULS, validation of the design at CALS should also include confirmation of mobilisation of the devices up the height of the structure.

4.2 Displacement-based analysis

The lateral systems for the Research building were designed using a displacement-based design approach. This adopted approach can be summarised as:

1. Non-linear pushover analysis with the structural model including appropriate modelling for the dampers and intended structural detailing
2. Converting the pushover results to an equivalent single degree of freedom structure.
3. Determining the Acceleration-displacement response spectra (ADRS) for the site allowing for damping.
4. Determining the performance points at the considered limit states which is the intersection of the ADRS curve and response from the equivalent single degree of freedom structure.
5. Evaluating the structural performance and demands at the performance points.

The following sections describe a few key considerations that were made.

4.2.1 Damping

The effective damping for the building system can include contributions from inherent damping, hysteretic damping, and viscous damping. Although the NZS 1170.5 response spectra are based on 5% viscous damping, this level of damping would likely exceed the inherent damping provided by steel framed structure. ASCE-7 currently requires no more than 3% inherent damping for the design of structures with dampers.

4.2.2 Load vectors

Pushover modelling requires an input lateral load vector to push the structural model. This pushover analysis for this project adopted an inverted triangular distribution and uniform lateral load proportional to the storey mass. The Seismic Assessment of Existing Buildings technical guidelines provides useful commentary on lateral load vectors for pushover analysis. The demands in the superstructure may vary depending on the assumed lateral load vector.

4.2.3 Device variability

Device variability should be considered in the design of buildings with dampers. ASCE 7 provides equations and limits on the extent of variability to be considered. ASCE 7 considers the following factors when determining the device variability: aging and environmental effects, variability determined through testing, acceptance variability specified by the designer. Upper and lower bound damper properties should be considered in the design of damper structures. Lower bound properties will typically result in greater deflections, whilst upper bound properties will typically result in greater forces.

4.3 Non-linear time history analysis

The behaviour of structures with well distributed shear walls is largely dominated by a translational response in the first mode. However multi-storey structures with braced frames or moment frames have many more degrees of freedom than cantilevered shear wall structures. Multi-degree of freedom structures have the possibility of displacement demands concentrating across one or more stories, potentially resulting in soft storey mechanism forming. Because of this potential behaviour, non-linear time history analysis (NLTHA) was done to validate the design of the longitudinal lateral system with CBF frames incorporating RSFJs.

The following sections describe a few key considerations that were made.

4.3.1 Damping

For reasons described in Section 4.2.1 above, inherent damping of 3% was conservatively adopted for the NLTHA.

4.3.2 Ground Motion Scaling

RSFJ devices are less stiff beyond the point at which the devices slip. This means that structures with RSFJs can have high initial stiffness but operate at an effective period substantially greater than the period corresponding to the initial elastic stiffness of the structure. The ground motion scaling requirements within NZS 1170.5 specify a relatively narrow period range of interest and is a function of the elastic period, not the effective period. Scaling based on the narrow period range of NZS 1170.5 meant that the resulting spectra was significantly higher than the design spectra at the effective period. It is worth noting that ASCE 7-22 allows for a greater period range of interest by allowing for an upper bound period range of greater than or equal to twice the largest first-mode period in the principal horizontal directions. The NLTHA ground motion scaling adopted a period range of 0.2-2 seconds.

4.3.3 Spine Elements

Spine elements, articulating at their base, can mobilise lateral load resisting elements up the height of a building. The Research building has a relatively large floor plate with 57 continuous columns, free of splices. over the three stories from Ground to Roof Level. The six shear walls (approximately ~5m long and 300mm thick) provided additional continuous vertical structural elements. These elements limited the demands on the RSFJ devices as the continuous detailing meant that frame action could be developed. For example, Roof Level lateral loads could transfer to the lower floor by cantilever action of the columns and walls as well as via brace action of the diagonal braces with RSFJs.

5 RSFJ PRODUCTION AND TESTING

5.1 Prototype testing

Although many current building standards do not explicitly cover the implementation of energy dissipation devices, documents such as the ASCE-7 standards (American Society of Civil Engineers 2016) provides seismic design and testing requirements for structures with damping systems. Therefore, to address some of the performance aspects required by the standard, a series of prototype testing have been performed. The testing has been conducted at the Structures Lab of the Auckland University of Technology (AUT), at an ambient temperature of approximately 21°C. The maximum target displacement of the RSFJ was set at approximately 40 mm in tension and compression. The loading protocol was specified as per ASCE-7 providing a rigorous testing regime to verify the dynamic performance of the RSFJ. The number and amplitudes of the loading cycles were as follows: ten cycles at +/-15 mm, five cycles at +/-30 mm and three cycles at +/-40 mm. Note that the displacements were slightly lower than the design displacements specified for the project, due to limitations of the testing equipment. The testing program included two different tests with the following frequencies: Quasi-static at 0.1Hz (3 cycles at +/-40 mm) Test #2: Dynamic at 1.0Hz (as per the protocol mentioned above). Figure 6 provides an overview of test setup and the results of the quasi-static testing versus the dynamic testing. Figure 6 shows that the hysteresis curves are compatible demonstrating the repeatability of the RSFJ performance after severe events without stiffness and strength degradation. Furthermore, a comparison of the results of the two tests shows the load-deformation behaviour of the RSFJ is independent from the velocity, as the response is comparable from the two tests (orange and blue curves).

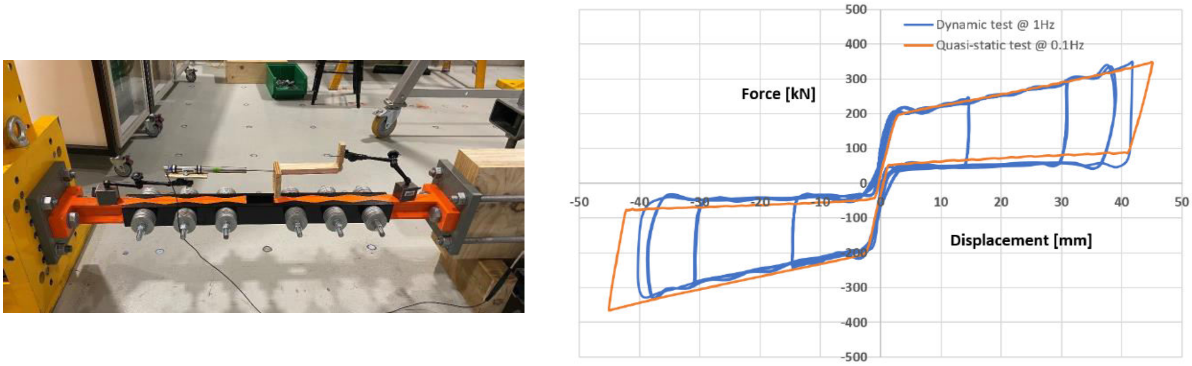


Figure 6: Prototype testing of the RSFJ

5.2 Production testing

The project required 108 RSFJ units with five performance categories from 325 kN to 1250 kN. All 108 units were tested (at a quasi-static frequency) to demonstrate that the specified load-deformation relationships were achieved within the specified tolerances. An acceptable tolerance of 5% was specified for the RSFJ devices on this project. Figure 7 provides the results for two representative device configurations and the upper and lower bounding.

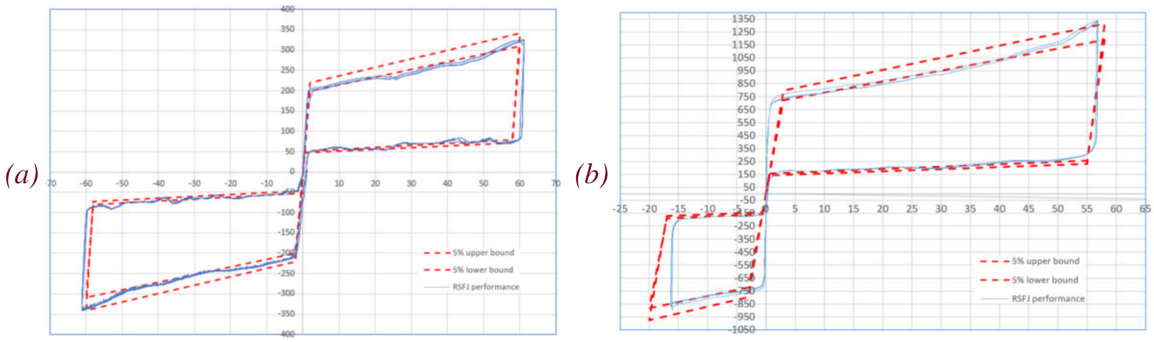


Figure 7: Production testing of the RSFJs (a) brace unit (b) shear wall hold-down unit

6 IMPLEMENTATION AND INSTALLATION

6.1 Implementation

This section describes the implementation of the RSFJs within the precast shear walls (transverse direction) and concentrically braced frames (longitudinal direction).

6.1.1 Shear walls with RSFJs

Tectonus RSFJs at the ends of the wall transfer overturning actions from the wall to the foundations. The base shear from the wall is transferred to the foundation via a steel shear key bolted to the base of the wall bearing against a steel armoured pocket cast into the foundation. The base of the wall bears on the top of the foundation and steel armouring is provided to both the wall and foundation at this location. Figure 8 shows an overview of the structural detailing at the base of the wall. The structural design determined that four Tectonus RSFJs were required at each end of each shear wall. With each device requiring a 1250kN tension capacity and 60 mm displacement capacity at ULS. The devices were attached to the precast wall with long threaded rods cast into the precast walls. A swivel bearing and pin was provided at the bottom end of the RSFJs which attached to a structural bracket bolted to the foundation. The inclusion of the swivel bearing was to allow for out-of-plane movement of the panel.

It was determined that the shear walls could be constructed as single precast panels without splices and remain transportable from a local precast yard to the project site. In situ and tilt-up construction were discounted due to programme and site space requirements. Differential movement can cause damage at the interface of suspended slabs and rocking walls, as indicated in Figure 9. To avoid damage at this location the slabs were detached from the walls. This approach required providing gravity columns at either end of the shear walls to support the floor at the walls. Vertical loading on the wall in the seismic load combinations provides additional overturning resistance. As the slabs were detached from the walls only the weight of the wall was considered in the lateral analysis as contributing to the restoring moment.

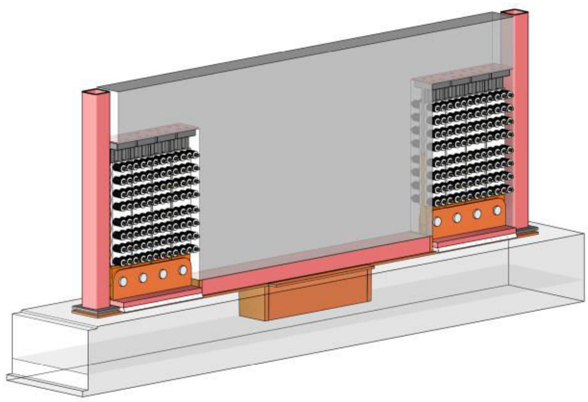


Figure 8 – Structural detailing at base of shear walls

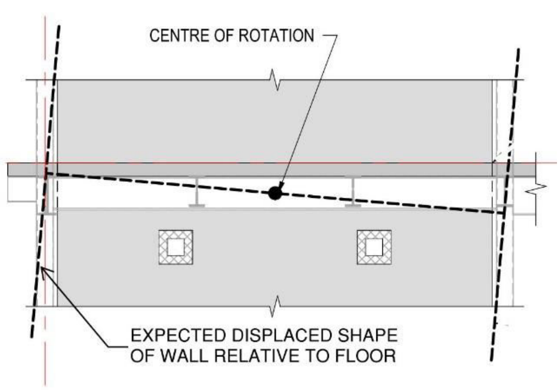


Figure 9 – Expected wall movement at suspended floors

As described previously, the suspended slabs were detailed so they weren't directly connected to the walls. This avoided the slab needing to accommodate differential movement at the ends of the walls which were expected to lift upwards by up to 60 mm. To transfer lateral loads to the shear walls a pair of custom welded parallel flange channels are provided either side of the shear walls. Shear studs connect the suspended slab to the wall and the CWPFC is connected to the wall with six M36 threaded rods in vertically slotted holes, refer Figure 10. The bolt spacing was minimised to reduce the distance to each bolt from the centroid of the bolt group. From the expected rotation at the bolt group, it was possible to predict the expected bolt positions. The movements were determined to be <3mm which was considered not to impose any excessive demands on the bolts given the slotted hole geometry.

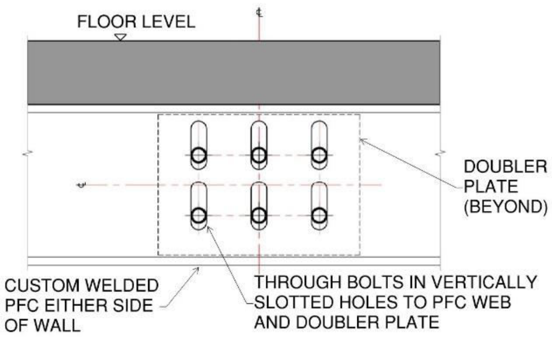


Figure 10 - Suspended floor to wall connection detail

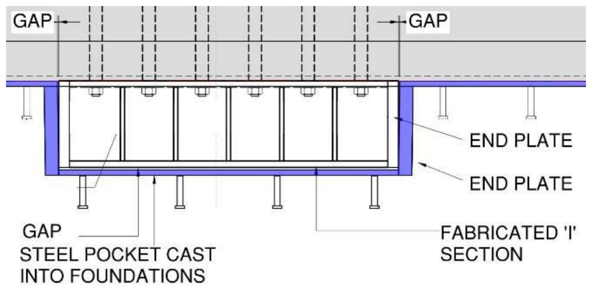


Figure 11 – Shear wall shear key detail

A shear key (refer Figure 11) was required at the base of each shear wall to transfer lateral loading at the base of the wall to the foundations, whilst also allowing for movement of the wall. It was not considered practical to have a reinforced concrete extension of the shear wall into a pocket, given tolerance and strength requirements. The design solution was for a steel shear key to be bolted to the base of the shear wall and located within a steel pocket cast into the foundations. Twelve M48 threaded rods were detailed to be cast into the wall. These rods were located with a fabricated steel channel also cast into the base of the wall, which provided correct set out of the rods and armoured the base of the wall. A fabricated steel section with thick end plates and stiffeners connect to the threaded rods. This assembly is in a pocket with thick end plates. A significant benefit of the adopted detailing is that the steel shear key can be constructed to relatively stringent tolerances and test fitted prior to installation to ensure that the shear wall can be installed without risk of shear key not fitting.

The endplates of the pocket were detailed with a slight taper to allow for the expected rotation of the wall. A geometrical check was done to validate that the shear key would not be expected to bind up at the design wall rotation. The design intent was for the endplates to bear uniformly against each other on the side transferring load, and not meeting each other on the opposite side.

6.1.2 Concentrically braced frames with RSFJs

Each CBF brace includes one Tectonus RSFJ Brace Assembly (RBA). Each RBA includes a spigoted SHS assembly, two Tectonus RSFJs (one either side of the spigoted assembly) and an end plate at either end of the RBA. The spigoted SHS assembly provides lateral restraint to the RSFJs when the RBA is loaded in compression. RSFJ are provided either side of the spigot to evenly distribute loads to the RSFJs. A bolted connection with end plates allow attachment to the adjacent brace steelwork. At CHS section were adopted for the brace section a cruciform section was added to allow a bolted attachment of the RBA to the CHS brace.

The CBF braces were attached to the beam-column and column base connections with pin connections incorporating large gusset plates slotted through the columns. Additional bearing plates were added at the underside of the gusset to provide load transfer to the concrete infill within the column. The column baseplate connection detailing included four large diameter pins to transfer lateral load from the braces to the foundations. These pins extended through a tight-fitting hole in the baseplate and into pocket cast into the foundation which was then filled with grout.

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Figure 12 –RSFJ Brace Assembly (RBA)

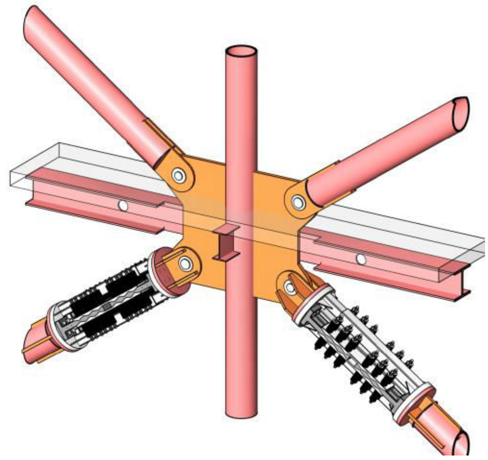

Figure 13 – Typical CBF detailing incorporating RBA's at the upper end of each brace

6.2 Installation

Beca worked closely with Leighs Construction, the design and build contractor for the project, to determine an appropriate construction methodology and preferred structural detailing.

6.2.1 Concentrically Braced Frames

After manufacture, the RSFJ Brace Assemblies (RBA's) were transported to the steel fabricators yard. Here the RBA's were mounted to the receiving CHS brace and pin cleat assemblies. The completed brace assemblies were then transported to site to allow installation within the braced frame. Finally, the structural pins were installed. The steel fabricator installed the braces with relative ease, despite the limited tolerance of the pinned connections, noting there was some flexibility in the steel frame before braces were installed and suspended slab pours had been made.

6.2.2 Shear walls

The RSFJ units for the shear walls were also transported to the steel fabricators yard. Here the RSFJs were mounted to steel cradles which were then brought to site and mounted to the precast panels orientated on-the-flat, with one cradle being installed at the base of either end of the panel. At this point, a PFC strongback was installed across the full width of the panel to support the devices whilst the panel was rotated from horizontal to vertical alignment. Finally, the precast panel was positioned in its final position and temporarily propped. Installation of the panel required locating the steel shear key attached at the base of the precast panels within an armoured steel pocket cast into the foundation. Test fitting was done in the steel fabricators yard to ensure the shear key fit within the steel pocket, noting the specified connection tolerance was 2mm.

One challenge with the adopted connection detailing was the limited construction tolerance available after the steel armoured pocket and steel armouring to the precast panel had been cast into the foundation and precast panel respectively. A gap was necessary between foundation and panel armouring to achieve plumbness of some panels. Steel shims were installed between any introduced gap, with shims welded to one side of the connection, which allowed for any future rocking movement at the base of the panel.

7 CONCLUSIONS

Incorporating damping devices in concentrically braced frames results in additional degrees of freedom. Analysis based on reducing the structure to an equivalent SDOF system may not provide sufficient verification of the behaviour of these types of structures. The design of such structures should include non-linear time history analysis to validate the design. Stiff spine elements can be incorporated to obtain more uniform mobilisation of all damping devices within the structure.

The construction of the building has shown that buildings with friction damping devices are readily constructible, with only simple temporary works and a few additional stages required during construction.

This case study has demonstrated that low damage performance can be achieved by introducing innovative friction damping system, at a similar cost to a building with conventional structural systems and achieve superior seismic performance.

8 ACKNOWLEDGEMENTS

The authors would like to thank Lincoln University for enabling the project team to adopt a low damage design solution. The authors would also like to thank Leighs Construction for facilitating a collaborative approach during the design and construction stages given the innovative structural system adopted.

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