Kilmore Street Medical Centre: Application of an Advanced Flag-Shape Steel Rocking System

2013 NZSEE Conference

D.A. Latham & A.M. Reay

Alan Reay Consultants Ltd.

S. Pampanin

University of Canterbury, PRESSS Ltd.

ABSTRACT: The Kilmore Street Medical Centre is a new three storey building in the Christchurch CBD, utilising a post-tensioned steel rocking and dissipating (hybrid) "advanced flag-shape" system, the first application of this technology in New Zealand.

Coupled braced frames provide the lateral-load resistance, and are designed to rock in a significant seismic event, controlling damage. Significant features of the design include special detailing at the connections between the floor diaphragms and the braced frames in order to minimise displacement incompatibilities associated with the uplifting of "wall" systems during lateral sway (regardless of these walls relying upon a rocking mechanism or not), and the first application of University of Canterbury-developed High Force-to-Volume lead extrusion devices, which provide viscous damping in parallel with replaceable yielding mild steel fuse-bars. The combination of displacement-proportional and velocity-proportional energy dissipation mechanisms, in parallel to the re-centering contribution from the un-bonded post-tensioned bars, leads to what is referred to in literature as an "advanced flag-shape system", which has been shown to provide robust seismic performance under both far field and near field seismic events.

This paper provides details of the design process, the supporting non-linear static and dynamic analyses, as well as the experimental verification of the behaviour of the lead dampers via high speed cyclic testing.

1 INTRODUCTION

The Kilmore Street Medical Centre is a new building located in the Christchurch central business district. The building is three stories with over 5000m^2 of specialist medical facilities, including four operating theatres, patient bedrooms and urology, radiology, orthopaedics and women's health clinics. There is over 650m^2 of plant deck on the roof and a further plant room at ground level.

Figure 1: *Architectural Render of the Kilmore Street Medical Centre*

The 2010 and 2011 Canterbury earthquakes have highlighted the need for a wider implementation of a "Damage Control Design" philosophy (Pampanin, 2012). Several of the Centre's tenants experienced first-hand the impacts of having to relocate their patients and business into temporary facilities due to lack of access and damage caused to their former buildings.

A performance-based design methodology has been adopted for this new building, to achieve owner and tenant expectations that the building can remain operable following a major seismic event. A DCD system has been adopted for the structure, with the use of coupled post-tensioned steel braced frames that are designed to rock in a significant seismic event to control damage.

In addition to the structural design, the building includes features such as a generator, back-up water supply and emergency sewer tank so that the building can be self-sufficient and remain operable following a major event. Construction commenced in July 2012 and the building is due for completion later in 2013.

2 STRUCTURAL DESCRIPTION

The building floor plate is approximately 45m x 40m and consists of a predominantly steel structure. The suspended floors consist of steel-concrete composite slabs with eight sets of coupled steel post-tensioned braced frames located around the perimeter of the building to provide the lateral load resistance. Refer to Figure 2 for a typical floor plan.

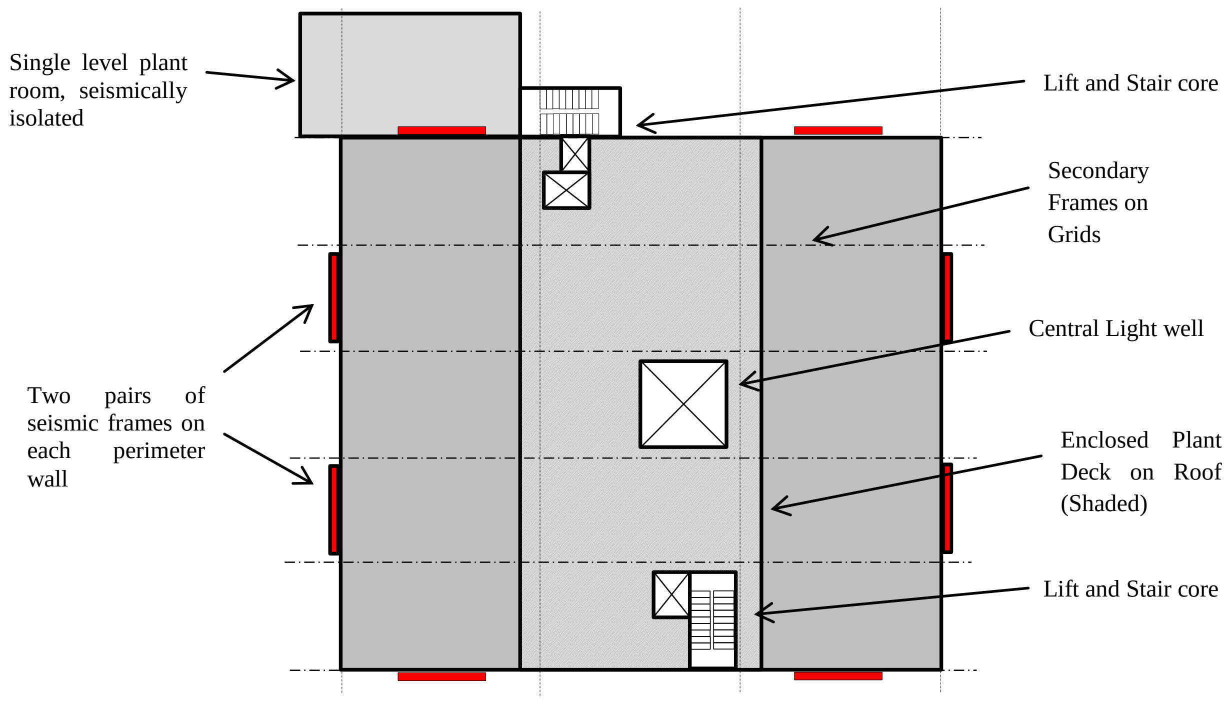

Figure 2: *Typical Floor Plan*

A large plant deck covering over one third of the floor plate is located on the roof for the intensive plant equipment required for a medical facility. A single level precast plant room structure is located to the north of the building, which is seismically isolated from the rest of the structure. The lift and stair cores are constructed from precast concrete walls.

The building is clad with curtain wall glazing and the braced frames sit in a frosted glass enclosure, so that they are incorporated as a feature of the building's architecture.

2.1 An Advanced Flag-Shape System

The concept of rocking and dissipative (hybrid) self-centering systems was originally developed and tested in the United States (Priestley 1991, 1996; Priestley et al. 1999) using precast concrete, commonly known as PRESSS (Precast or Prefabricated Structural Seismic Systems). It has since been applied in a number of structures around the world and in New Zealand (Pampanin, 2005; Cattanach and Pampanin, 2008; NZCS, 2010). Recently the concept has been applied using engineered timber, known as Pres-Lam system, with an increasing number of applications in New Zealand (Palermo et al. 2005; Devereux et al. 2011).

A major research programme was carried out by a collaboration of universities in the United States on steel self-centering braced frames, the steel equivalent of PRESSS post-tensioned rocking walls (Roke et al. 2009; Hajjar et al. 2010; Deierlein et al. 2010). The research included large scale dynamic testing at the E-Defence shake table in Japan (Figure 3), and confirmed the technology delivers a high seismic performance.

Figure 3: *Shake Table Testing of Steel Self-Centering Braced Frames at the E-Defence Shake Table in Japan (Deierlein et al. 2010)*

The concept of these systems is to allow controlled rocking of the structure to reduce damage to the primary structural elements themselves. Un-bonded post-tensioned tendons or bars provide a restoring and self-centering force. Energy dissipation occurs at the rocking interface, and can be provided in many different forms. Pairs of braced frame or wall elements can be coupled together with further energy dissipation devices to provide additional strength and stiffness. The braced frames and post-tensioned bars are designed to remain elastic, while the energy dissipation devices accommodate the inelastic demand and can be replaced if necessary.

This type of hybrid system has a recentering and dissipative hysteresis loop referred to as a "flag shape" hysteresis; refer to Figure 4. The hysteretic behaviour can be modified by adjusting the various contributions of the post-tensioning and the energy dissipation devices.

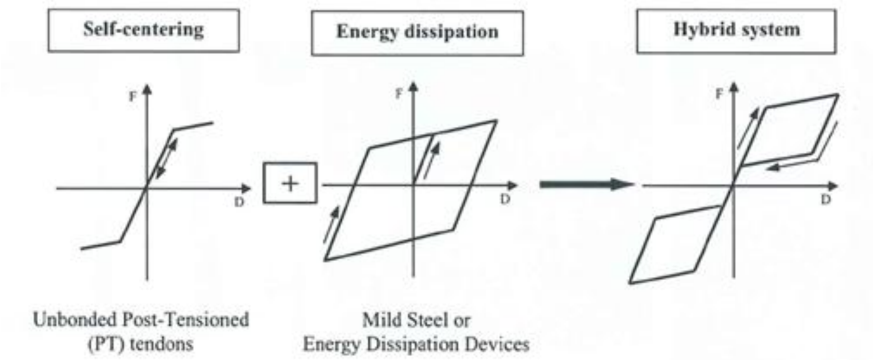

Figure 4: *Flag Shape Hysteresis Behaviour for Hybrid System (NZCS, 2010)*

A second generation of self-centering/dissipative high-performance systems, referred to as advanced flag-shape systems (AFS) has been recently proposed and tested at the University of Canterbury (Figure 5). AFS systems combine alternative forms of displacement-proportional and velocity-proportional energy dissipation in parallel with the main source of re-centering capacity.

As a result, it is possible to achieve an enhanced and very robust seismic performance, under either far field or near field events (high velocity pulse), as demonstrated by numerical investigations (Kam et al. 2010) as well as shake table testing (Marriot et al. 2008).

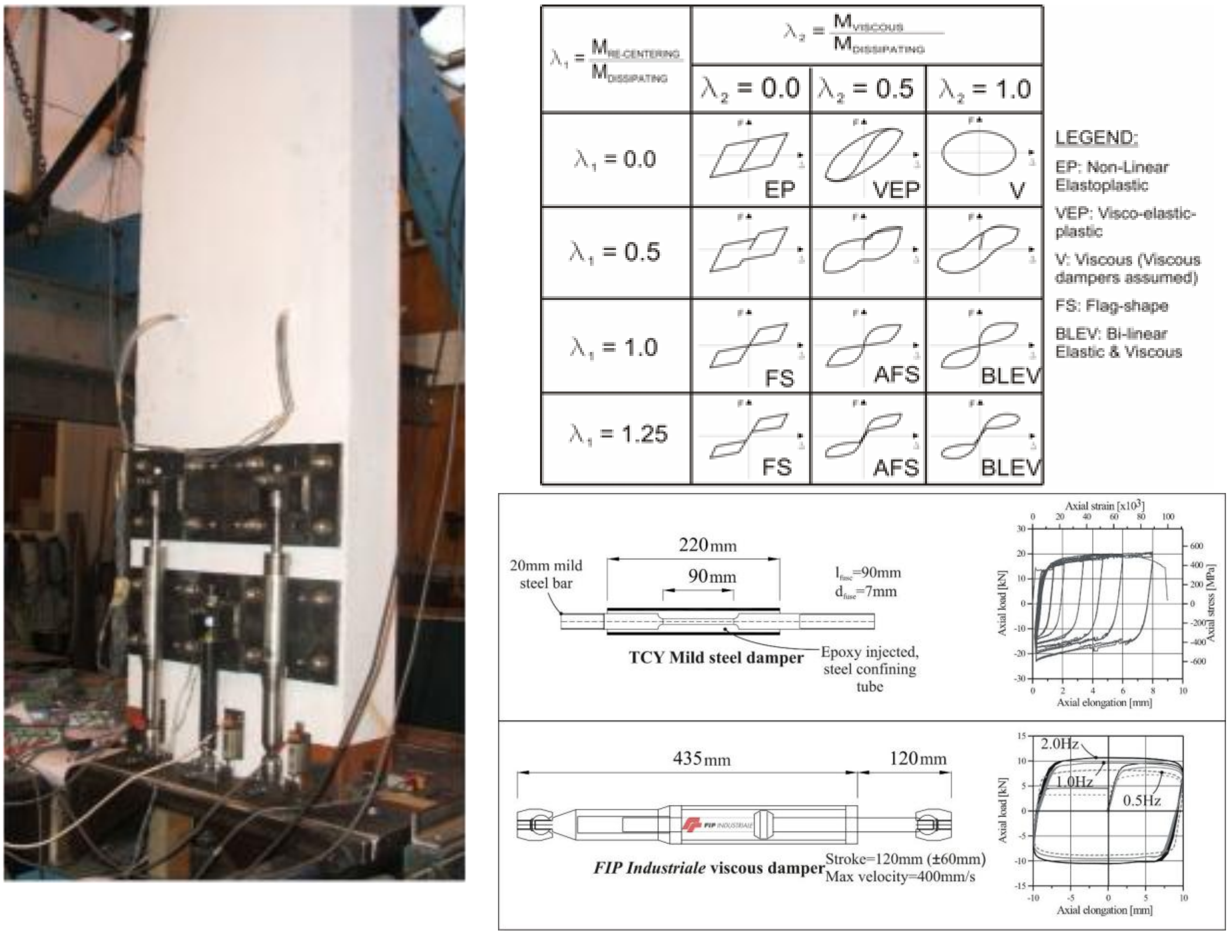

Figure 5: *Concept, implementation and experimental validation of the concept of an Advanced Flag-Shape system applied to a post-tensioned concrete wall (Kam et al., 2010; Marriot et al., 2008)*

2.2 Steel Braced Frames or "Walls"

The seismic resisting system at the Kilmore Street Medical Centre consists of pairs of concentrically braced frames (CBF's) coupled together, representing the equivalent of the concept of coupled post-tensioned and dissipative "walls" using structural steel.

Each CBF is a single fabricated element and is vertically post-tensioned to the foundation with two 75mm high strength Macalloy bars. The frame consists of a 400mm deep double-webbed I-section, which was used to enable the post-tensioned bars to pass through the middle of the frame and to reduce the unsupported length of the flanges. The frames sit in a base "shoe" which acts as a shear key under horizontal loading.

Figure 6: *Pair of steel braced frames assembled in the workshop with dampers*

Figure 7: *Pair of steel braced frames erected on-site.*

Axially yielding mild steel fuse rods provide hysteretic damping and are located at the base of the frames at the rocking interface and between the two frames to provide coupling between them. Similarly, University of Canterbury-developed High Force-to-Volume lead extrusion dampers (Rodgers et al. 2008; Rodgers 2009) provide viscous damping, and are also located at the base and between the frames. The combination of these dissipative devices, together with the post-tensioning (Macalloy bar), produces the Advanced Flag Shape (AFS) system.

A feature of the design includes special detailing at the connection between the floor diaphragms and the lateral load-resisting system in order to minimise the effect of the displacement incompatibilities associated with the uplifting of "wall" systems during the lateral sway (regardless of these walls relying upon a rocking mechanism or not). A steel tongue plate protrudes from the floor structure and fits through a slot in the seismic frame; refer to Figures 8 and 9. The tongue plate is able to slide up and down within the seismic frame to accommodate the vertical movements experienced by the frames as they rock.

Figure 8: *Tongue plate protruding from floor structure that slots through the seismic frame*

Figure 9: *Steel braced frame showing tongue plate (prior to installation of shims)*

Lateral loads are transferred via bearing of the steel plate onto the frames. Brass shims are used within the frame to reduce the friction whilst still providing the necessary bearing strength. Steel columns are provided within the floor structure at these connection points to resist the friction-induced vertical forces. Out-of-plane restraint is provided to the columns of the seismic frames at each floor level via a simple tension tie and compression bearing system. These are designed to provide buckling restraint to the seismic frame columns whilst providing out-of-plane support and satisfying the robustness requirements of NZS1170. The ties can accommodate the movements experienced by the frame as it rocks, and are isolated from the horizontal load transfer system.

2.3 Floors and Secondary Frames

The floor structure consists of a concrete-steel composite floor system, with relatively long spanning secondary beams to give large open spaces and to provide maximum flexibility for the use of the space. Deflection and vibration criteria generally governed the design of the floor.

Figure 10: *Concrete-steel composite floor structure under construction*

Figure 11: *Sliding Hinge Joint utilised for secondary frames*

The beam-column joints were designed as “top hinge” connections to minimise any displacement incompatibilities from lateral deflections and to limit damage to the concrete floor slab. The connections consisted of a double web cleat welded to the column, which bolt to the web of the beam in double shear. Slotted holes were provided to the lower bolts to allow the beam to pivot about the floor slab as the building displaces laterally.

The internal frames implement the “Sliding Hinge Joint” solution (MacRae et al., 2010) to provide a secondary moment resisting frame system (Fig. 11). This system can accommodate the lateral displacements required for displacement compatibility with the main seismic resisting system, while providing additional redundancy to the primary lateral system and stability to the building during erection and after a fire.

2.4 Stair and Lift Cores

The stair and lift cores consist of precast walls, which are detailed to pivot and rock about the centre of the wall to minimise any displacement incompatibilities with the primary steel braced frame structure. Precast walls provide the required fire rating as well as being stiff enough to limit distortions between floors that can damage non-structural elements and, importantly, affect the operation of the lifts.

2.5 Foundations

The site presents a significant liquefaction hazard. The foundations consist of 168 steel H piles driven to a typical depth of 24m. Steel H piles were the preferred solution due to the ability to drive through intermediate dense layers, reduced post-liquefaction negative skin friction and the inherent ductility capacity so that lateral movement can be accommodated by the piles.

Concrete foundation beams span between piles. The post-tensioned Macalloy bars are anchored near the bottom of the foundation under the seismic frames. The bars are encased in a steel box and baseplate assembly so that the bottom nut and washer is accessible from the side of the foundation. The assembly is grease filled to provide corrosion protection.

3 SEISMIC DESIGN AND ANALYSIS

3.1 General

A direct displacement based design (DDBD) procedure (Priestley et al., 2007) was adopted for the design of this structure, in conjunction with design guidelines available in New Zealand for similar systems (NZCS, 2010) and a procedure to modify the DDBD to allow for the contribution of the viscous dampers (Marriot, 2009).

Due to the type of occupancy, this building was deemed an Importance Level 4 (IL4) structure, requiring design to the 1/2500 year earthquake at the Ultimate Limit State (ULS), along with Serviceability Limit State checks at the 1/45 year (SLS1) and 1/500 year (SLS2) earthquakes. In addition, specific checks were made at a level larger than the 1/2500 year earthquake to represent the Maximum Considered Event (MCE).

Target displacements or design drifts were selected for each limit state consistent with a performance-based design approach, refer Table 1. The targets reflected an objective to maintain the displacements in a range where the non-structural elements could be adequately detailed for the expected drifts, while providing sufficient flexibility within the structure to reduce demands on the structure and limit floor accelerations.

Table 1: *Target Design Displacement / Drift Criteria*

Limit State	Target Maximum Inter-storey Drift	
SLS1 (1/45 year)	0.4%	15 mm
SLS2 (1/500 year)	1.0%	40 mm
ULS (1/2500 year)	2.4%	100 mm

The DDBD procedure does not specifically consider floor accelerations as a direct input however target acceleration limits were set, refer Table 2, and checked with non-linear time history analysis. Non-structural damage threshold levels proposed by FEMA (HAZUS-MH MR3, 2003) consistent with the performance objectives were considered for each limit state.

Table 2: *Target Design Floor Acceleration Criteria*

Limit State	Target Maximum Floor Acceleration (HAZUS-MH MR3, 2003)
SLS1 (1/45 year)	0.25-0.30g (threshold of slight damage)
SLS2 (1/500 year)	0.50-0.60g (threshold of moderate damage)
ULS (1/2500 year)	1.00-1.20g (threshold of extensive damage)

A preliminary DDBD was used to determine the initial sizes of the steel frames, post-tensioning, energy dissipation and foundation requirements. A moment-rotation analysis at the rocking interface was carried out using principles of equilibrium.

3.2 Non-Linear Static Analysis

Non-linear 2D static analysis was carried out on the seismic frame system using an idealised model implemented into SAP2000, as shown in Figure 12.

Elastic frame elements were used to model the steel frame members. Non-linear elements with a bi-linear backbone were used to model the post-tensioned bars and hysteretic dampers. The viscous lead-extrusion dampers were modelled as discrete dashpots, although because of their velocity dependence their contribution was considered separately for the static analysis. The bases of the frames were modelled with a group of compression-only springs that allowed rocking behaviour. The spring stiffness was determined based on a calibration of the computer model with the calculated moment-rotation curve.

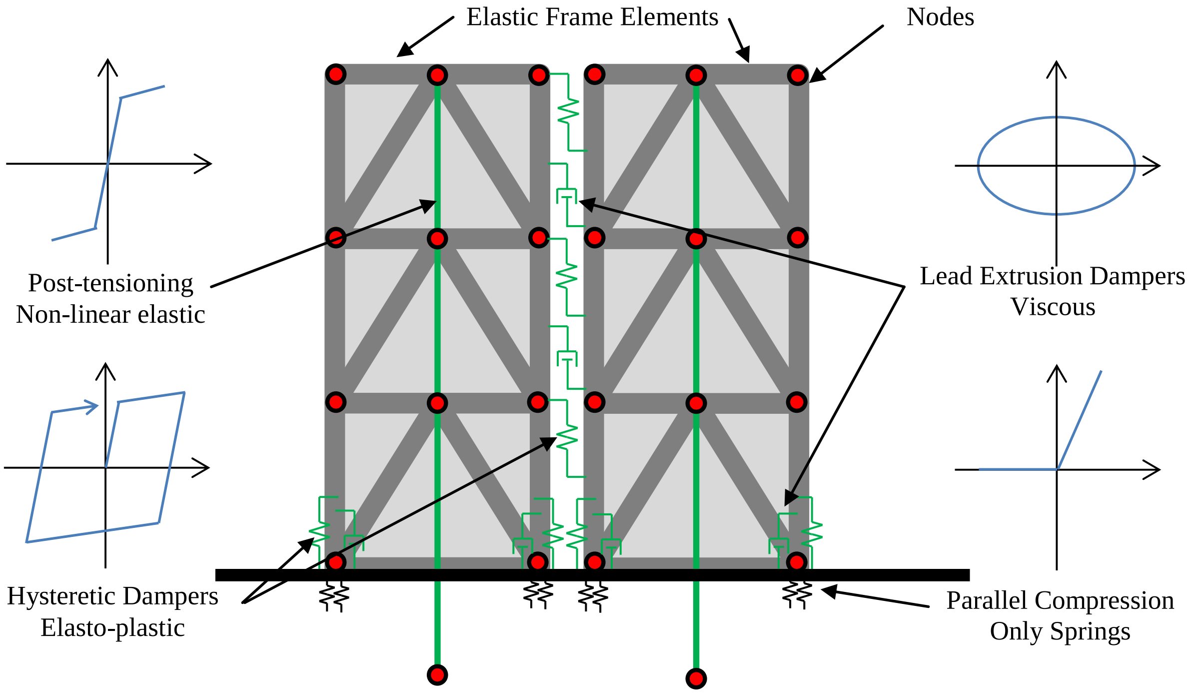

Figure 12: *Idealised Analysis Model of Coupled Frame System*

A cyclic pushover was carried out to determine the area-based equivalent viscous damping (EVD) at various drift levels. This demonstrated the expected "flag-shape" hysteresis behaviour, refer to Figure 13. The Acceleration-Displacement Response Spectrum (ADRS) was derived from NZS1170.5 at each limit state and scaled by a spectral reduction factor based on the level of EVD at that limit state. Both near-field and far-field relationships were considered for the spectral reduction factor for the ULS and MCE limit states. The ADRS could be directly compared to the pushover analysis, refer Figure 14. Allowing for accidental eccentricity of the centre of mass, this procedure was used to verify the preliminary design assumptions and target design drifts outlined in Table 1.

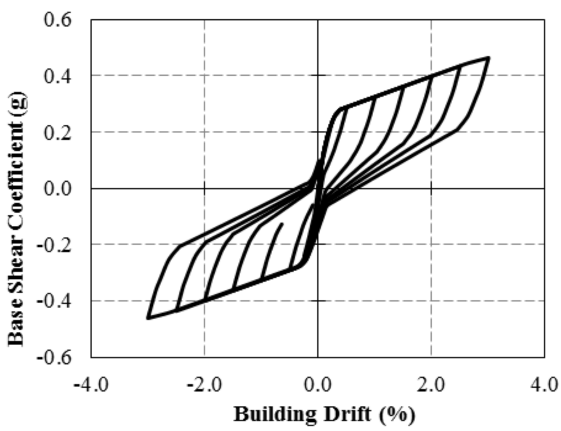

Figure 13: *Cyclic Pushover, used to determine the area-based Equivalent Viscous Damping. The contribution of the lead extrusion dampers is additional.*

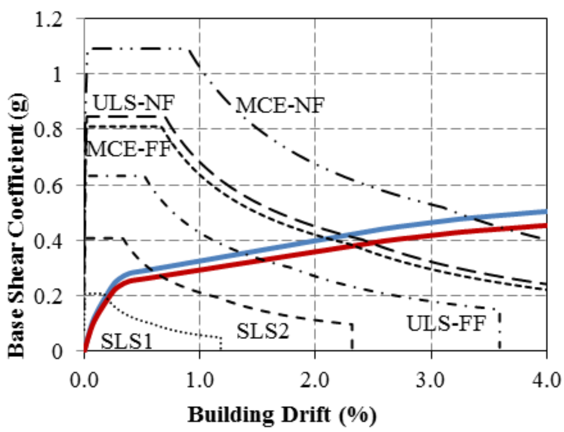

Figure 14: *Non-Linear Pushover (probable and dependable) and NZS1170.5 ADRS scaled for expected EVD at various limit states, used to verify the preliminary DDBD assumptions and drift demands.*

3.3 **Detailed Design**

Detailed design of the seismic frames, connections, floor diaphragms and foundations were based on the results of the non-linear static analysis. A capacity design procedure was implemented for individual elements and the system as a whole to ensure that that the system behaves as intended and undesirable mechanisms are avoided. Material strain limits were checked or used as the basis of the design as appropriate.

There was limited literature available on overstrength and dynamic magnification factors for this type of system. Overstrength actions were considered in accordance with the basic principle of ensuring that "protected" elements have an acceptably low probability of yielding. For this building, this was considered as the actions at a drift of 4.0%, in excess of the MCE level, combined with an increase in initial post-tensioning levels and material strengths in accordance with the relevant material codes (NZS3404).

A dynamic magnification factor was selected based on analysis work on similar steel rocking systems (Ma, 2010) and a review of typical factors provided in the current material codes for other types of structural systems (NZS3404). The magnification selected was verified using non-linear dynamic analysis. Further research into appropriate dynamic magnification levels would be beneficial for the design of these systems.

3.4 **Maximum Considered Event**

As part of a performance based design methodology, it is necessary to consider the performance of the structure as a whole in an event larger than the design level. For a normal building, the Maximum Considered Event (MCE) is typically assumed to correspond to the 1/2500 year earthquake (NZS1170.5 Commentary). However as this building was designed as IL4, the 1/2500 year earthquake represents the ULS design level. No guidance is given for what level of earthquake should be considered as the MCE for IL4 structures.

The commentary to the Structural Design Actions standard NZS1170.5 states that the maximum considered motions are not required to exceed that corresponding to ZRu = 0.7, representing the 84th percentile motions of an M8.1 Alpine Fault event. For a building in Christchurch with Z=0.30, this gives an effective return period factor of R=2.33 (compared to R=1.8 for the 1/2500 year earthquake). The return period factor of R=2.33 was adopted as the MCE level and the system performance and material strain limits were checked at this level.

3.5 **Non-Linear Time History Analysis**

Non-linear time history analysis (NTHA) was carried out on the seismic frame using the model in SAP2000 described previously. Due to the symmetry and regularity of the building and that all the seismic frames are located on the perimeter of the building providing a high resistance to torsion, it was considered that 2D analysis would be sufficient to represent the general dynamic behaviour.

Twelve earthquake ground motion records were selected, refer Table 3. These included the 04 September 2010 and 22 February 2011 events experienced in Canterbury, as well as a selection of overseas records, including some that exhibited near-fault characteristics.

Table 3: Earthquake Records Selected for Dynamic Analysis

Earthquake	Station	Year	Magnitude	Distance (m)	Site Class (USGS)	PGA (g) un-scaled
Christchurch	CCCC	2011	6.3	7	D	0.483
Christchurch	REHS	2011	6.3	8	D	0.714
Darfield	CHHC	2010	7.1	36	D	0.208
El Centro	Imperial Valley	1940	7.0	8	C	0.348
Kobe	JMA Observatory	1995	6.9	1	B	0.836
Loma Prieta	Santa Cruz	1989	6.9	19	C	0.630
Lytle Creek	Wrightwood	1970	5.3	13	D	0.200
Northridge	Sylmar	1994	6.7	6	C	0.843
Tabas	Boshrooyeh	1978	7.4	26	D	0.107
Taft	Kern County	1952	7.4	41	D	0.180
Taiwan	Smart45	1986	7.3	39	D	0.153
Tokachi-Oki	Hachinohe	1968	8.3	130	D	0.233

Each record was scaled to the different limit state levels including SLS1, SLS2 and ULS in accordance with the procedures outlined in NZS1170.5. The ULS scaled spectral acceleration plots for the twelve records considered are shown below in Figure 15.

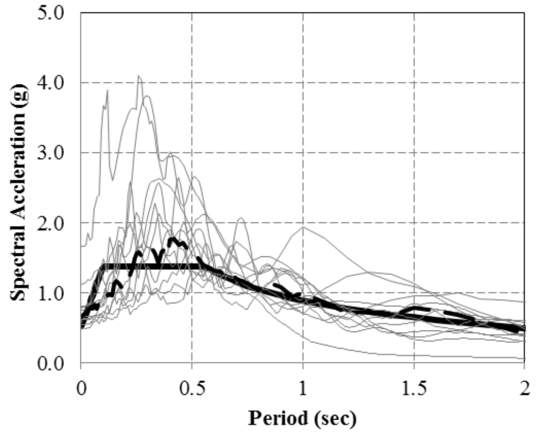

Figure 15: *Spectral Acceleration plots for the twelve records used in the time history analysis, scaled to the NZS1170.5 ultimate limit state (1/2500 year) design spectrum (solid) with median (dashed).*

The dynamic analysis was used as a means of checking the general performance of the system and the assumptions inherent in the design. The building drifts were checked for each record, as well as other key design parameters such as diaphragm shears, mild steel fuse bar material strains, post-tensioned bar material strains, viscous damper forces and velocities, frame element forces and floor accelerations. The analysis results were used to verify the overstrength and dynamic magnification factors assumed and applied to the static design.

Figure 16 shows the building drift time history record for the Christchurch CCCC record. The period of oscillation is approximately 1.3 seconds, which matches the design effective period obtained in the DDBD very closely. The structure exhibits a very similar period when subjected to other earthquake records scaled to ULS levels and once uplift of the frames occur and rocking behaviour is initiated.

Figure 17 shows the "flag-shape" hysteresis behaviour from the Christchurch CCCC record. The non-linear static cyclic pushover is also shown at an equivalent drift level. This demonstrates some of the higher mode and dynamic magnification effects observed from the time history analysis which increases the diaphragm shears and overall base shear over those obtained from a static analysis.

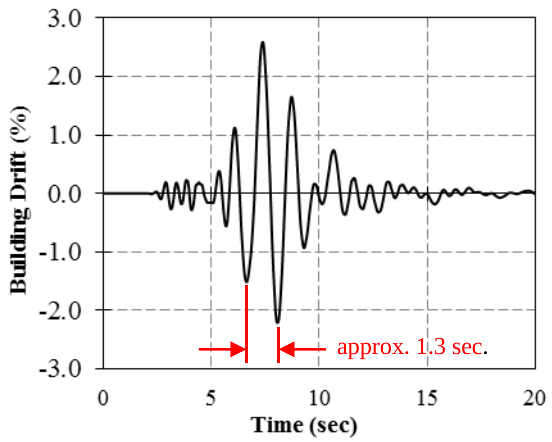

Figure 16: *NTHA Results of Building Drift of the CCCC record scaled up to ULS (1/2500 year).*

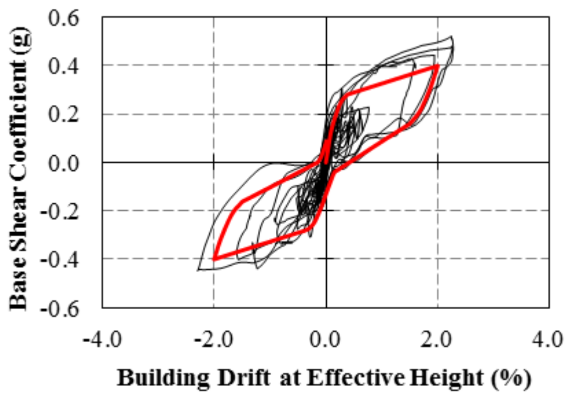

Figure 17: *Hysteresis behaviour from NHTA analysis of CCCC record scaled up to ULS (1/2500 year). Non-linear static cyclic pushover shown in red.*

Figure 18 shows the peak building drift recorded at each limit state for each record. A statistical analysis was carried out to determine the median, 5th and 95th percentile drift levels. Comparing to the target maximum displacements outlined in Table 1 (shown as red crosses on Figure 18), it shows the range of drift values obtained are consistent with the design assumptions and targets.

Figure 19 shows the peak floor acceleration at the top floor at each limit state for each record. A statistical analysis was carried out to determine the median, 5th and 95th percentile floor accelerations. Comparing to the target maximum floor accelerations outlined in Table 2 (shown as red crosses on Figure 19), it shows the range of floor acceleration values obtained are consistent with the design targets.

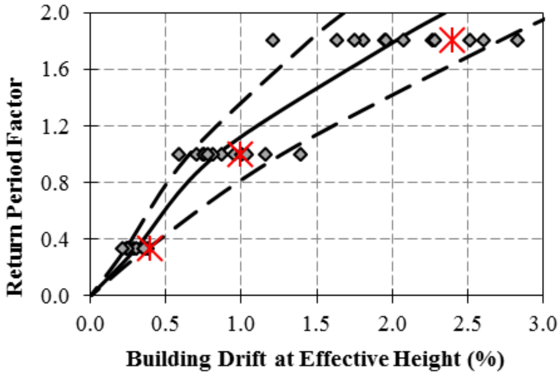

Figure 18: *NTHA Results of Peak Building Drift vs NZS1170.5 Return Period Factor, showing median, 5th percentile and 95th percentile. Red crosses show design target maximum drifts from Table 1.*

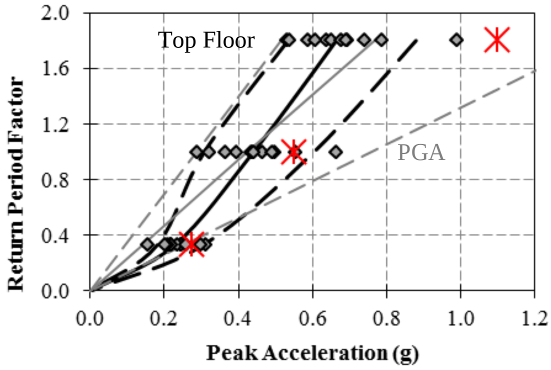

Figure 19: *NTHA Results of Peak Acceleration at Top Floor (black) and ground (grey) vs NZS1170.5 Return Period Factor, showing median, 5th percentile and 95th percentile. Red crosses show design target maximum floor accelerations from Table 2.*

Figure 19 also shows the median, 5th and 95th percentile scaled peak ground accelerations (PGA) for the twelve records. Comparing to the top floor accelerations, this demonstrates the amplification of accelerations from the ground to the top floor. For records scaled to low intensity levels, there is some amplification of the accelerations however the magnitude of acceleration remains low. At these levels, the response of the building is dominated by the linear-elastic response of the seismic frames therefore some amplification is expected. For records scaled to higher intensity earthquakes, the amplification decreases and a reduction in accelerations is observed at ULS (R=1.8) levels. At higher intensity levels, the response is dominated by the rocking behaviour along with larger displacements and velocities that can fully engage the dampers, leading to the reduction in acceleration at the top floor.

3.6 **Non-Structural Elements**

To achieve a performance-based system consistent with the owner and tenants expectations that the building will remain operable after a major seismic event, it is critical that the design and behaviour of non-structural elements such as glazing systems, internal partitions, ceilings, building services and plant are also consistent with the design objectives and damage control criteria.

Particular efforts were made to communicate key structural performance parameters such as building deflections and accelerations to the architects, mechanical and electrical engineers and others involved in the project. Specific sections on seismic design were included in the building fit-out specifications for each of these trades, outlining the minimum requirements for the performance and restraint of plant and services during a seismic event. To ensure that the requirements were adequately implemented, the contractor was required to engage a suitably qualified Chartered Professional Engineer to design the seismic restraint systems for each of the non-structural trades to achieve the performance criteria.

3.7 Review and Consent Process

A quasi-independent peer review process of the seismic design was carried out by Dr Stefano Pampanin, Associate Professor from the Civil Engineering Department of University of Canterbury and Director of PRESSS Ltd. This mechanism allowed for the review to be commenced at very early stages of the project, with significant benefits in terms of regular availability and critical feedback throughout the design.

This type of structural system and the use of DDBD is not covered by a verification method to the New Zealand Building Code (NZBC). Instead the design required an alternative solution, which involves specific justification and evidence that the performance requirements of the NZBC are achieved. The Christchurch City Council (CCC) was familiar with the design of rocking and dissipative systems, a DDBD procedure and the quasi-independent review process adopted in this project. There were no specific issues raised by CCC related to the structural design and the alternative solution.

4 LEAD EXTRUSION DAMPERS

The lead extrusion High Force-to-Volume (HF2V) dampers have been developed at the University of Canterbury over the last 10 years (Rodgers et al. 2008; Rodgers 2009). This will be the first application of the devices into a structure. The device consists of a high strength bulged steel shaft encased in lead which is prestressed. As the shaft is displaced, the bulge induces a plastic flow of lead around the bulge providing a resistive force and dissipating energy, as shown in Figure 20.

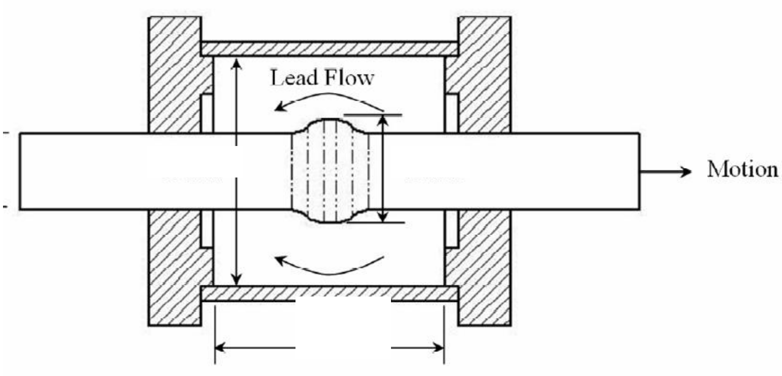

Figure 20: *Schematic of HF2V Lead Extrusion Damper (Rodgers, 2008)*

4.1 High Speed Cyclic Testing of Prototypes

As the devices are not explicitly covered by a verification method of the NZBC and that it is the first application of the devices in a real building, it was considered necessary to carry out testing specifically for this application. The devices have undergone extensive quasi-static testing over their development, however they have had limited high speed cyclic testing. A specific testing campaign was developed for this project, which involved testing at velocity levels comparable to that expected during an event in a range of magnitudes.

The testing was carried out by the University of Canterbury (Department of Mechanical Engineering) at the Quest Lab in Wellington. The Quest Lab was one of the only laboratories available that could provide the required force levels at velocities consistent with the design demands. A lever system was developed for the testing to further increase the velocity through the device, refer Figure 21.

Three different designs with slightly different shaft parameters were tested, with three identical devices for each design, making a total of nine devices. Each device was tested at 2.5, 10, 25, 50, 100, 150 and 200 mm/s. The laboratory setup restricted the ability to test multiple cycles at the highest speeds, and only one cycle could be achieved at 200mm/s. However this is considered to be representative of a real earthquake motion with a near-field velocity pulse. Figure 22 shows a typical hysteresis plot for two of the devices tested.

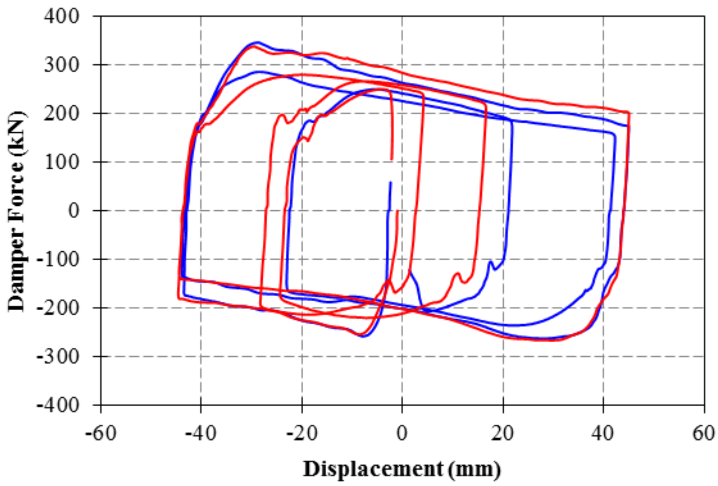

Figure 21: *High Speed Testing of Devices at Quest Laboratory, showing lead damper and lever system*

Figure 22: *Experimental test results from two identical devices at 50mm/s velocities*

4.2 Quasi-Static Verification of Actual Devices

An advantage of the lead extrusion dampers is that they can undergo a large number of cycles without degradation. This enabled a proportion of the actual constructed devices to be tested prior to installation. The testing was carried out in a quasi-static manner so that the testing could be carried out using available laboratory equipment at the University of Canterbury. The quasi-static test results were compared to the devices that underwent the high speed testing to ensure consistency across all the devices.

5 CONCLUSION

The Kilmore Street Medical Centre utilises post-tensioned steel rocking and dissipating braced frames, with alternative forms of displacement-proportional and velocity-proportional energy dissipation in parallel with the main source of re-centering capacity to deliver an "Advanced Flag-Shape" system.

This type of system delivers high seismic performance, consistent with a "Damage Control Design" philosophy. The project is under construction and is generally within budget and on schedule. It has shown that this type of system is viable for future developments.

The client, design team and contractor have supported the use of this innovative steel structural system and shown enthusiasm about the project and the use of the technology. Together with the University of Canterbury, the lead extrusion HF2V devices have been implemented into a building for the first time.

Further development and research, in conjunction with designers, contractors and academia, would be beneficial for this type of system, with an aim to further optimise the design and construction of these types of buildings.

6 REFERENCES

Cattanach A., and Pampanin, S, 2008. 21st Century Precast: the Detailing and Manufacture of NZ's First Multi-Storey PRESSS-Building, NZ Concrete Industry Conference, Rotorua

Deierlein, G.G., Ma, X., Hajjar, J.F., Eatherton, M., Krawinkler, H., Takeuchi, T. Midorikawa, M., Hikino, T., and Kasai, K., 2010. Seismic Resilience of Self-Centering Steel Braced Frames with Replaceable Energy-Dissipating Fuses – Part II: E-Defence Shake Table Test, 7th International Conference on Urban Earthquake Engineering & 5th International Conference on Earthquake Engineering Joint Conference, March 3-5, 2010, Tokyo

Devereux, C.P., Holden, T.J., Buchanan, A.H., Pampanin, S., 2011 “NMIT Arts & Media Building - Damage Mitigation Using Post-tensioned Timber Walls”, Proceedings of Proceedings of the Ninth Pacific Conference on Earthquake Engineering, “Building an Earthquake-Resilient Society”, 14-16 April, Auckland, New Zealand, paper 90

Hajjar, J.F., Eatherton, M., Ma, X., Deierlein, G.G. and Krawinkler, H., 2010. Seismic Resilience of Self-Centering Steel Braced Frames with Replaceable Energy-Dissipating Fuses – Part I: Large-Scale Cyclic Testing, 7th International Conference on Urban Earthquake Engineering & 5th International Conference on Earthquake Engineering Joint Conference, March 3-5, 2010, Tokyo

HAZUS-MH MR3, 2003. “Technical Manual for HAZUS-MH MR3” FEMA, Washington D.C.

Kam, W.Y., Pampanin, S., Palermo, A. and Carr, A., 2010. Self-centering Structural Systems with Combination of Hysteretic and Viscous Energy Dissipations. Earthquake Engineering and Structural Dynamics Aug 2010, 39 (10), 1083-1108

Marriott, D., Pampanin, S., Bull, D. and Palermo A., 2008. Dynamic Testing of Precast, Post-Tensioned Rocking Wall Systems with Alternative Dissipating Solutions, Bulletin of NZ Society for Earthq. Engineering, Vol. 41(2), 90-103

Marriott, D., 2009. The Development of High-Performance Post-Tensioned Rocking Systems for the Seismic Design of Structures, Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand.

MacRae, G.A., Clifton, G.C., Mackinven, H., Mago, N., Butterworth, J. and Pampanin, S., 2010. The Sliding Hinge Joint Moment Connection. Bulletin of the New Zealand Society for Earthquake Engineering, 43 (3) 202-212

New Zealand Standard (NZS 3101:2006), Concrete Structures Standard, Standards New Zealand, Wellington

New Zealand Standard (NZS 3404:1997), Steel Structures Standard, Standards New Zealand, Wellington

New Zealand Standard (NZS 1170.5:2004), Structural Design Actions, Part 5: Earthquake Actions – New Zealand, Standards New Zealand, Wellington

New Zealand Concrete Society, 2010. PRESSS Design Handbook (Editor: S. Pampanin), NZCS, Wellington, New Zealand

Pampanin, S., 2012. Reality-check and Renewed challenges in Earthquake Engineering: Implementing low-damage structural Systems – from theory to practice, 15th World Conference on Earthquake Engineering, 24-28 September, Lisbon, Keynote Address

Priestley, M.J.N., 1991. Overview of the PRESSS Research Programme, PCI Journal, 36 (4) 50-57

Priestley, M.J.N., 1996. The PRESSS Program Current Status and Proposed Plans for Phase III, PCI Journal, 41 (2) 22-40

Priestley, M.J.N., Sritharan, S., Conley, J.R. and Pampanin, S., 1999. Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building, PCI Journal, 44 (6) 42-67

Priestley, M.J.N., Calvi, G.M., Kowalsky, M.J., 2007 Displacement-based Seismic Design of Structures. IUSS Press, Pavia, Italy

Rodgers, G.W., Dhakal, R.P., Chase, J.G., Mander, J.B., MacRae, G.A., Mander, T.J. and Solberg, K.M. 2008. Investigation of Rocking Connections Designed for Damage Avoidance with High Force-to-Volume Energy Dissipation, The 14th World Conference on Earthquake Engineering, October 12-17, 2008, Beijing, China

Rodgers, G.W., 2009. Next Generation Structural Technologies: Implementing High Force-To-Volume Energy Absorbers, Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand.

Roke, D., Sause, R., Ricles, J.M., and Gonner, N., 2009. Damage-free seismic-resistant self-centering steel concentrically-braced frames, STESSA, Philadelphia, Taylor & Francis Group, London

Sause, R., Ricles, J.M., Roke, D., Seo, C.Y. and Lee, K.S., 2006. Design of Self-Centering Steel Concentrically-Braced Frames, 4th International Conference on Earthquake Engineering, October 12-13, 2006, Taipei, Taiwan