Shake table testing of an integrated low-damage frame building.

H.C. Johnston, C.P. Watson, S. Pampanin & A. Palermo

Department of Civil and Natural Resources, University of Canterbury, Christchurch, New Zealand.

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ABSTRACT: Recent research has led to the development of multiple low-damage earthquake resistant structural and non-structural systems that are able to withstand high levels of drift or deflections with negligible damage. Dry jointed connections, articulated floor solutions, low damage drywall infills and low damage facade connections have all been developed separately and successfully tested with mainly pseudo-dynamic testing. Research is required to combine the individual technologies and gather better information of the entire system behaving dynamically.

The proposed paper will describe the design, fabrication, set-up and shake table testing of a 1/2 scale, two storey concrete frame building. The building consists of a post-tensioned rocking hybrid frame with a non-tearing dissipative floor, low damage drywall infills, and low damage façade connections.

1 INTRODUCTION

Damage to buildings that has been observed following the Canterbury Earthquake sequence has further emphasised the need for low damage solutions for the whole building system. Figure 1 shows the multiple forms of damage caused by earthquakes to both the structural and also more significantly the non-structural systems, when considering the cost of repair, typical to modern reinforced concrete (RC) frame buildings.

Beam hinges, similar to those in Figure 1 (a), are designed for ductility in RC buildings. While these hinges are effective at dissipating energy and preventing building collapse, the future level of shaking that frames with hinging can resist is unknown, often necessitating the demolition of these buildings after significant earthquake events. The hinging also causes beam elongation due to non-recoverable tensile strains in the reinforcing steel. If sufficient seating is not provided for in precast floor systems, beam elongation may lead to a catastrophic floor failure due to loss of seating. As well as beam elongation, further displacement incompatibilities between the floor diaphragm and seismic frames can cause serious damage to the floor in the form of cracks running parallel to the beam, as shown in Figure 1 (c). Permanent or residual strains in the reinforcing steel also cause an overall residual drift or deformation in the structure. Strategies in regards to repairing residual deformations that are significantly large are considered unpractical or cost more than the demolition and reconstruction of the building and as such most building are demolished.

Non-structural elements, such as infills and facades, are typically more vulnerable to seismic damage than structural elements due to them being more brittle and having a higher stiffness. The interaction between the non-structural elements and the structure itself can significantly alter the overall strength and stiffness of a building causing unexpected and potentially brittle failure mechanisms Baird (2012).

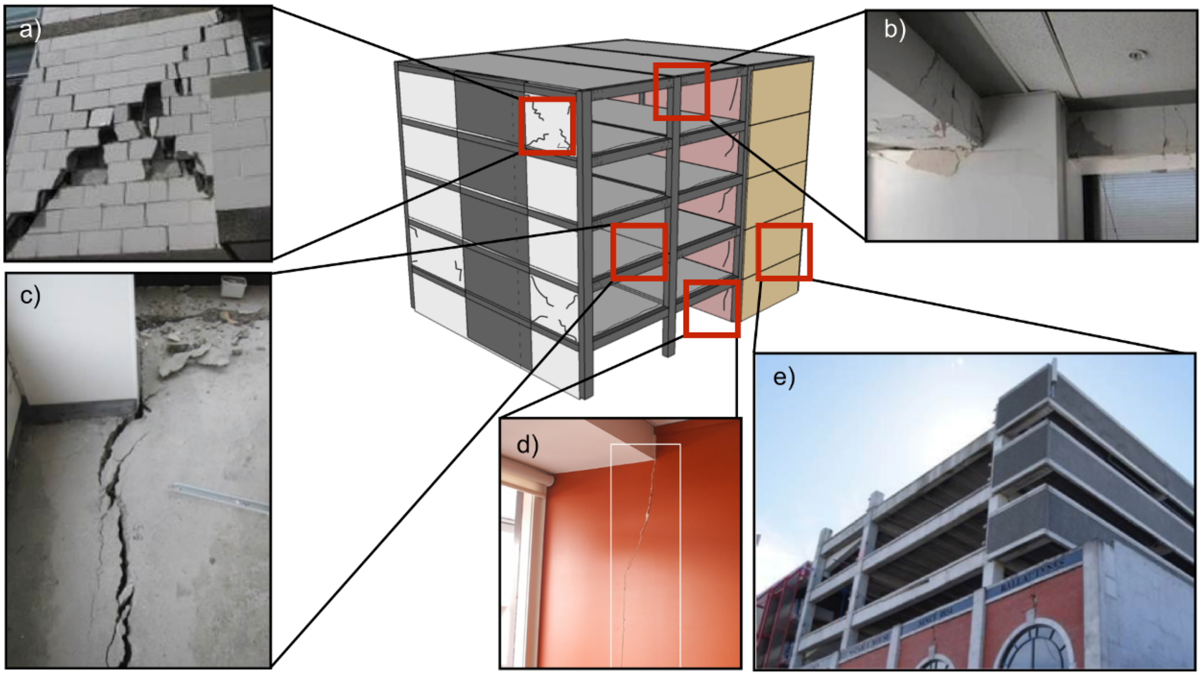


Figure 1. Typical damage to RC buildings from the Canterbury Earthquake sequence. a) Shear failure of infills, b) Beam end hinging, c) Floor cracking, d) Drywall infill cracking, e) Façade connection failure.

As part of a more general aim to increase the resilience of society against earthquake hazards, more emphasis in research has been given to damage-control design approaches (Priestley et al., 2007). Low-damage earthquake resistant structural and non-structural systems have emerged that are able to withstand high levels of shaking and large drifts or displacements without being damaged. These systems focus on concentrating earthquake energy in replaceable energy dissipators instead of plastic hinge zones.

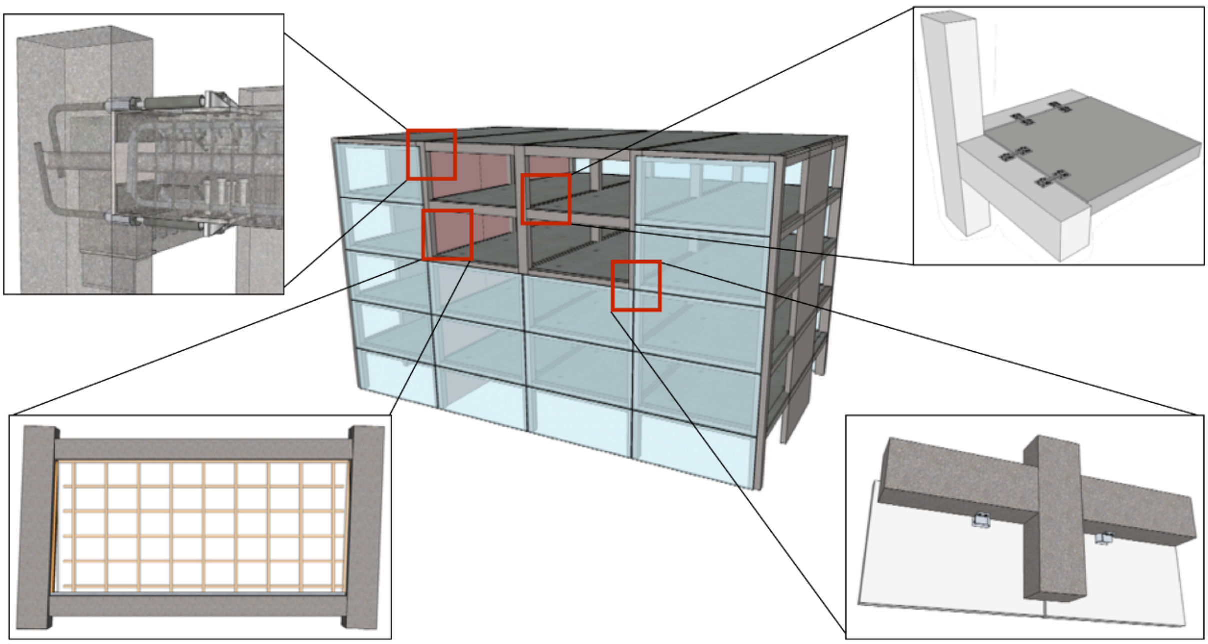


Figure 2. Five-storey PRESSS building with articulated floors and low damage non-structural element connections.

An alternative design approach to traditional precast reinforced concrete structures has been developed under the PRESSS (PREcast Seismic Structural System) program coordinated by the University of California, San Diego (Priestley 1991). Prefabricated elements are joined together through unbonded post-tensioning tendons, allowing the elements to rock against each other during seismic loading, and providing recentring capabilities to ensure negligible damage and residual deformations. Additional dissipation can be provided to the systems by means of internal or external mild steel bars (Pampanin, 2005, Priestley, 1999).

There is extensive research being performed on the development of low-damage non-structural elements at the University of Canterbury. Developments by Pampanin et al. (2006) have lead to innovative solutions to minimise the damage to the floor system while guaranteeing a reliable diaphragm action. An articulated “jointed” floor, shown in Figure 2, allows the floor to move relative to the beam without damaging either element. Taslegadik (2013) and Baird (2013) are developing infill systems and facade connections able to sustain their integrity under significant drifts and displacements. Furthermore, the elements are able to add additional damping to a structure and significantly improve the performance of the entire building.

2 EXPERIMENTAL RESEARCH

The primary objective of this research is to provide experimental evidence on the high seismic performance of integrated low-damage structural and non-structural systems whilst validating the constructability of a “fully dry” low-damage building .The experiment focuses on non-destructive shake-table testing of a half scaled two-storey single-bay concrete frame building with jointed ductile connections, non-tearing floor systems, low-damage drywall infills and low–damage façade connections. The dynamic testing will also investigate how each system contributes to the damping of the global structure.

There are three phases of testing:

Phase I is for benchmark testing of the bare frame shown in Figure 3a). The building has been designed with both Post-Tensioned only (PT only) and Hybrid connections. The non-tearing floor can be alternated between both fixed and dissipating UFP connections, allowing four different building configurations to be tested.

Phase II will investigate how non-structural elements influence the benchmark frame. Figure 3b shows how drywall timber infills will be installed into the frame, using both traditional methods and a new low-damage connection detail. Precast concrete facades, shown in Figure 3c), will be clipped on to the outside of the frame using dissipative UFP connections, which allow relative movement between the façades and structural elements.

Phase III involves combining the three low-damage non-structural systems within the low-damage frame and investigating the performance of the integrated systems.

For each phase, the test building will be exhibited to earthquake records across five different levels of shaking, ranging from 30% to 130% of a design level event. The shake table moves in one direction, so out-of plane behaviour will not be investigated.

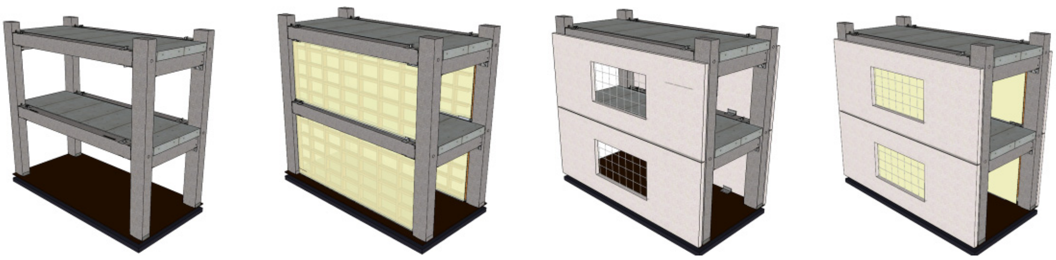


Figure 3. Left to right: a) Bare frame, b) frame with infills, c) frame with facades, d) integrated low-damage building

3 TEST SPECIMEN

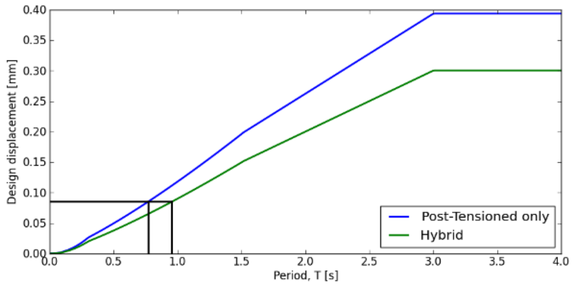
The specimen to be tested is a half-scale two-storey single-bay precast concrete frame building shown in Figure 4. In the long direction the building is composed of two post-tensioned frames with jointed ductile connections. In the transverse direction, the two frames are joined by short transverse beams bolted to a steel corbel cast-in to the column. The floor slabs span in the long direction and sit upon the transverse beams.

3.1 Building design

The building was designed as a full-scale prototype building following the Direct Displacement Based Design procedure. Two identical buildings, one with Post-tensioned only connections and the other with hybrid connections, were designed to reach 1.6% drift in a ULS level earthquake event based in Wellington, with Z=0.4 and solid type C. The building dampings used to modify the design displacement spectra for a concrete structure were calculated from Priestley et al. (2007) as 5.0% and 10.0% for the PT only and hybrid buildings respectively (Table 1).

Table 1. Design parameters of prototype frames

	PT only	Hybrid
M_{eff} [t]	5.33	5.33
H_{eff} [m]	23.8	23.8
D_{eff} [mm]	85	85
μ	2.70	2.70
Damping [%]	5.00	10.04
T_e [s]	0.77	0.95
K_e [kN/m]	1589.3	1044.1
V_b [kN]	135.6	89.1



The test specimen dimensions were scaled linearly from the prototype building by a scale factor of 0.5. Following similitude requirements, the Cauchy-Froude method was used to ensure constant acceleration and stress across the prototype and test buildings. Additional mass was required to be added to the test building to represent the increased density required by scaling. Table 2 and Figure 4 summarise the building dimensions.

Table 2. Properties of prototype and test frames

	Prototype	Test
Inter-storey height [m]	3.2	1.6
Bay length [m]	7.6	3.8
Building width [m]	3.6	1.8
Mass per floor [t]	26.6	6.6
Column dimensions [mm]	600x400	400x200
Beam dimension [mm]	500x400	250x200

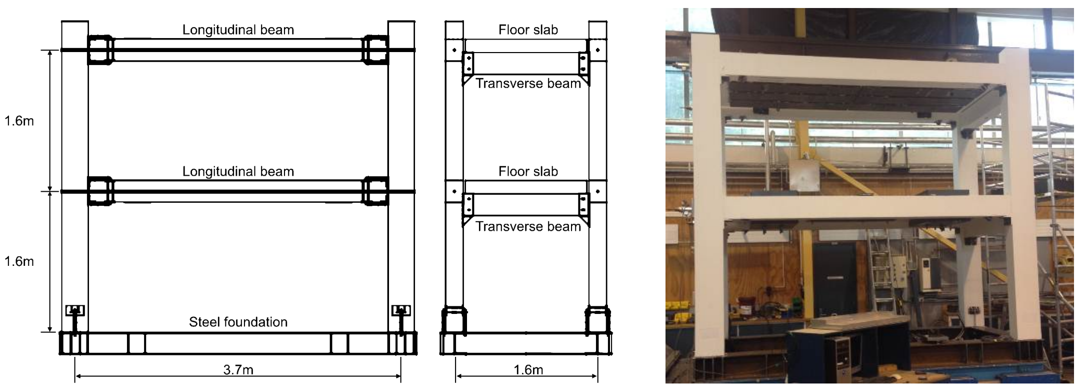


Figure 4. Test building dimensions

3.2 Building detailing

Connection design – The beam connections were designed following the procedure from the PRESSS design handbook (NZCS, 2010). The beam design moments were calculated following an equilibrium approach, with the assumption of pinned column foundation connections. The post-tension tendons available included 7mm wire (A=38.5mm^2), 12.7mm strand (A=100.1mm^2) and 15.2mm strand (A=143.3mm^2). The dissipators are fuse-type dissipators made from necked-down 16mm mild steel rod, shown in Figure 6. As the test building is very small, only one dissipator is needed top and bottom for each connection. The moment rotation curves for the connections of both buildings are included in Figure 7.

Table 3. Connection design

	Storey	Design moment (1.6% drift)	Post-Tensioning (Area , T_{pti})	Dissipation (Fuse diameter)
Post-Tension only	2	17.3 kNm	100.1 mm^2, 120kN	-
	1	24.9 kNm	143.3 mm^2, 200kN	-
Hybrid	2	11.4 kNm	38.5 mm^2, 55kN	9.2 mm
	1	16.4 kNm	100.1 mm^2, 70kN	7.7 mm

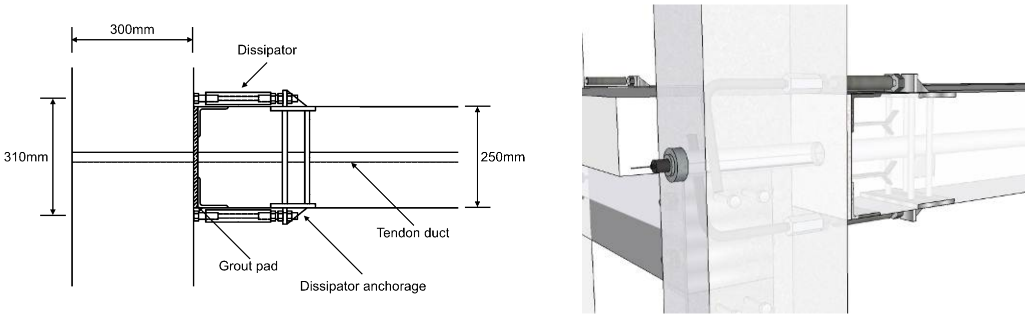


Figure 5. Schematic of beam-columns connections of test building

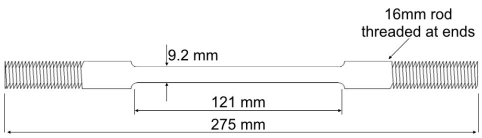


Figure 6. Fuse type mild steel dissipator

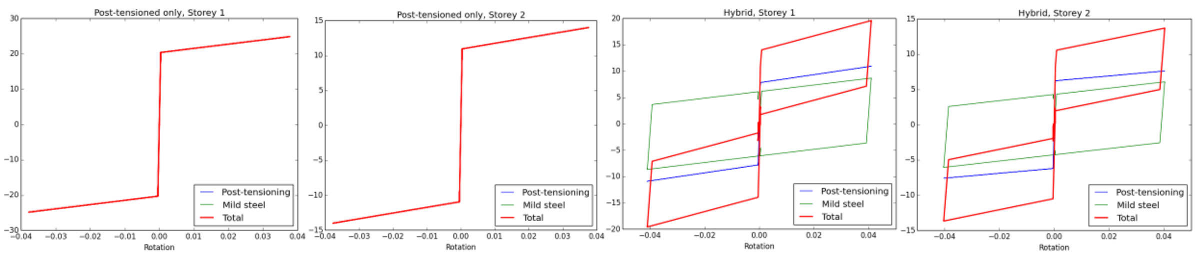
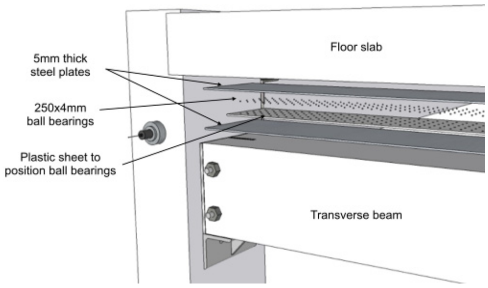


Figure 7. Moment rotation curves (from left to right): PT only Storey 1, PT only Storey 2, Hybrid Storey 1, Hybrid Storey 2

Non-tearing floor – The floor systems were designed as precast tensioned slabs, with a mechanical connection to the structural systems. The floor sits on top of the transverse beams, with a ball bearing plate (Figure 8a) to ensure no lateral friction between the two elements. The inertial forces from the slab are passed to the beam through either a fixed (angle) or dissipative (UFP) steel connection Figure 8b.



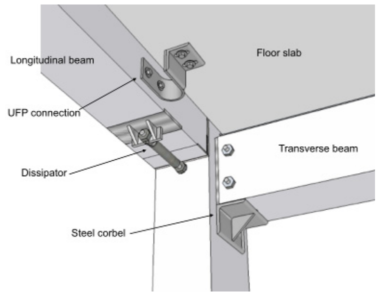


Figure 8. a) Exploded view of ball bearing system (left). b) UFP connection between underside of floor slab and beams (right).

The UFPs were designed following the procedure from the PRESSS design handbook (NZCS, 2010), where the desired displacement between the floor slab and frame was set at 20mm for a design level earthquake. As the floor connection to the lateral resisting frame determines the fundamental period of the building, the UFPs were designed by iterating on a time history analysis, where the size of the UFP was adjusted until the desired displacements were reached. Since the UFPs were constructed at Ballamy and East spring makers, the smallest thickness available was 5mm steel with 32mm width; the diameter of the spring was the only dimension able to be adjusted. The UFP dimensions are shown in Figure 9.

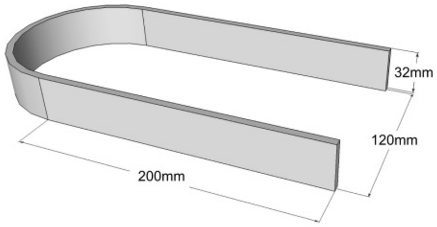
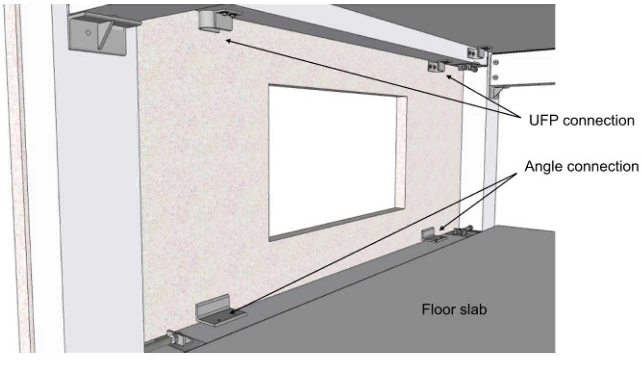


Figure 9. UFP dimensions for floor slab connections

Infill and façades – Both the façade and infill systems were designed to allow a building drift of 1.6% (which equates to 25mm per storey for the test building) in a design level earthquake without the high stiffness of the non-structural element affecting the overall behaviour. The facades were designed as full-scale elements and scaled down by 50% for testing. The thickness of the scaled facades was increased to 80mm to ensure sufficient mass for similitude requirements. The façade connections consist of rigid steel angles at the base to pass both the vertical and lateral loads into the longitudinal beams (Figure 10a). At the top, UFP connections prevent the façade from falling out of plane while allowing the relative in-plane displacement between the façade and frame.

The low damage infills are designed to allow little interaction with the surrounding structural frame. An aluminium channel, shown in Figure 10b, is attached to the beams and encases the timber studs. As the gypsum board is nailed on to the studs, the infill is free to move inside the channels without compromising the out-of-plane strength. A vertical stud is attached to the column but not the board to act as a shear key at the column.



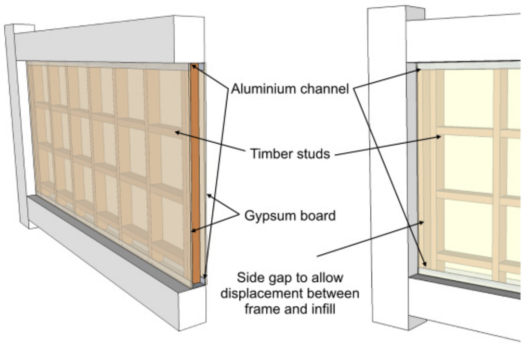


Figure 10. a) View from inside of building of facade connections (left). b) Low-damage infill design (right)

3.3 Construction

The test building elements were precast at Stahlton Engineered Concrete to ensure the construction methods used replicated those used in industry (Figure 11). The steel corbels and dissipator anchors were fabricated by Autobend and cast into the beams and columns.

Figure 11. Fabrication of the test building

4 PREDICTIONS

Analytical predictions of the test building behaviour were performed using the non-linear analysis software Ruaumoko2D (Carr, 2008). A lumped-mass lumped-plasticity model was created using rotational springs to model the beam-column connections and pinned connections for the column-foundation connections. The analysis was performed on the building for the first phase of testing; PT only building with fixed floor connections, Hybrid building with fixed floor connections, PT only building with UFP floor connections and Hybrid building with UFP floor connections. The UFP connections were modelled as springs with a Ramsberg-Osgood hysteresis.

4.1 Pushover with Acceleration Displacement Response Spectra

A force-based non-linear pushover was performed on the analytical models to determine the equivalent single degree of freedom performance point of the building. Figure 12 shows the pushover plots of the buildings against the acceleration displacement design spectra. It is evident that due to the increased damping associated to the hybrid connections, the drifts of the hybrid buildings will be lower than the post-tensioned only equivalents.

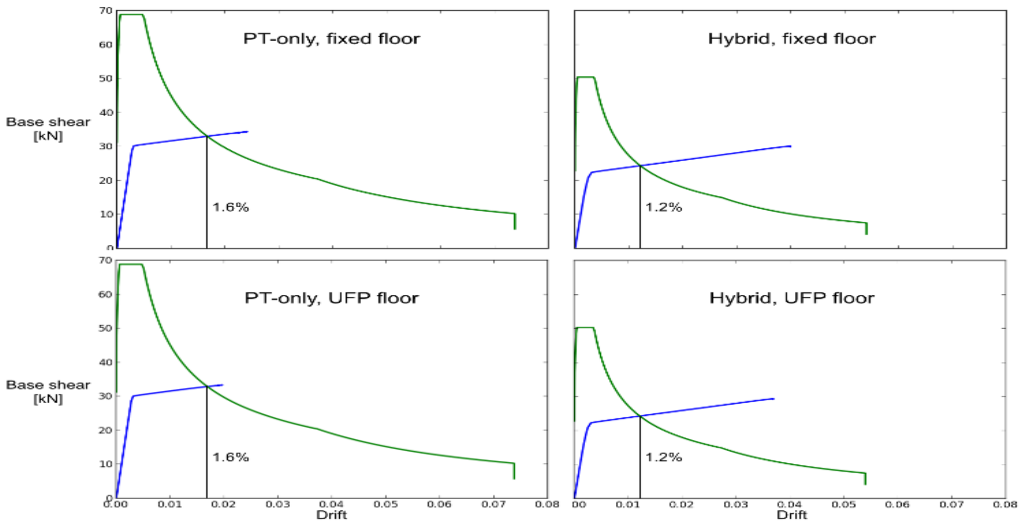


Figure 12. Design level ADRS model for the Phase I buildings

As the pushover is compared to the reduced acceleration-displacement response spectrum, the results are heavily dependent upon the assumptions used in the analysis. At this stage it is not fully known what levels of damping the UFP will be able to provide and how this contributes to the overall damping of the system.

Analysis from testing will be able to confirm the dynamic performance of these dissipators, as previous testing sequences have focused on Quasi-Static loadings, and assist in the ongoing development of design guidelines for the design of UFP's as seismic dissipation devices.

4.2 Non-linear time history analysis

A non-linear time history analysis was performed to produce blind predictions of the test building response to ground motion simulations. Ten earthquakes with average spectra equivalent to the design spectra were used for the analysis. The ground motion used in Figure 13 is a record of the 22 February 2011 Christchurch Earthquake taken from Christchurch Cathedral College (CCCC). When comparing the four buildings directly, the analysis showed improved performance in terms reduced drifts and accelerations within the building through added dissipation both with the hybrid connections and more significantly with the added UFP floor connections.

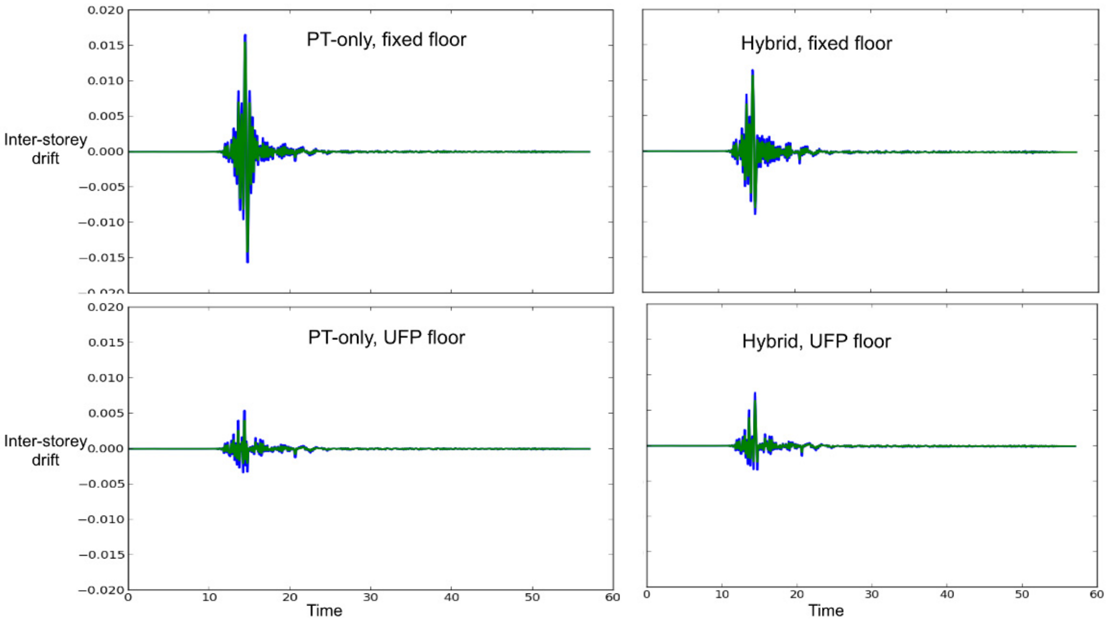


Figure 13. Inter-storey drift response history of test buildings in 2011 Christchurch Earthquake

Dynamic shake table testing will be carried out to confirm whether the response predicted by time history analysis can be validated.

5 CONCLUSION

A half-scale two-storey recast concrete building has been designed and constructed for shake-table testing. The testing will investigate the constructability and performance of a building with integrated low-damage structural and non-structural systems. Construction of the test specimen is complete and testing is ongoing, however preliminary results have been omitted from this paper.

Systems investigated as part of testing include the dynamic testing of a post-tensioned rocking hybrid frame, non-tearing floor solutions and the response of infill and façade structures with and without low-damage connections. The combined response of a hybrid structure with non-tearing floors and façade and infill structures with low damage connections will also be investigated.

Non-linear pushover and time-history analysis of the test specimen has been carried out. The numerical analysis indicates that the introduction of dissipation devices reduces the overall drift od the frame. Analysis has also shown a positive effect of providing dissipative floor connection devices.

Dynamic shake table testing is ongoing to provide experimental evidence of the concept of integrated low damage solutions.

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