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Living a New Era in Earthquake Engineering: targeting damage-resisting solutions to meet societal expectations

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ABSTRACT

Earthquake Engineering is facing an extraordinary challenging era, the ultimate target being set at increasingly higher levels by the demanding expectations of our modern society.
The Canterbury earthquakes sequence in 2010-2011 has confirmed a fundamental mismatch between societal expectations over the reality of seismic performance of modern buildings. By and large, with some unfortunate exceptions, modern multi-storey buildings performed as expected from a technical point of view, considering the intensity of the shaking they were subjected to.

In accordance to capacity design principles, plastic hinges developed in beams allowing for a ductile beam-sway mechanism to develop and the building to stand. Nevertheless, in many cases, these buildings were deemed too expensive to be repaired and were consequently demolished.

Targeting life-safety is clearly not enough for our modern society and a paradigm shift towards damage-control design philosophy and technologies is urgently required.
Is ductility-based design philosophy becoming obsolete and does it really imply irreparable damage?

This paper will discuss motivations, issues and, more importantly, cost-effective engineering solutions to design buildings capable of sustaining low-level of damage and thus limited business interruption after a design level earthquake. Focus will be given to the extensive research and developments in “jointed ductile” connections based on controlled rocking & dissipating systems for reinforced concrete, steel and more recently (laminated) timber structures.
An overview of recent on-site applications of low-damage or damage-control structural (and non-structural) systems, featuring some of the latest technical solutions developed in the laboratory and including proposals for the rebuild of Christchurch, will be provided as successful examples of practical implementation of performance-based seismic design theory and technology.

Keywords: performance-based design, low-damage structural systems, damage-control, post-tensioning, PRESSS, Pres-Lam, Canterbury Earthquake

1 INTRODUCTION: THE 22 FEB 2011 EARTHQUAKE EVENT

The Mw 6.3 Christchurch (Lyttelton) earthquake (afterschok) occurred at 12.51pm on Tuesday 22nd Feb 2011, approximately 5 months after the Mw 7.1 Darfield (Canterbury) main shock (Fig. 1). The epicentre of the February event was approximately 10km south-east of the Christchurch (Ōtautahi) Central Business District (CBD), near Lyttelton, at a depth of approximately 5 km. Due to the proximity of the epicenter to the CBD, its shallow depth and peculiar directionality effects (steep slope angle of the fault rupture), a significant shaking was experienced in the city centre, the eastern suburbs, Lyttleton-Sumner-Porter Hills areas resulting in 182 fatalities, the collapse of several unreinforced masonry buildings and of two RC buildings, extensive damage often beyond reparability levels to several reinforced concrete buildings, damage to tenths of thousands of timber houses and unprecedented liquefaction effects in whole parts of the city.

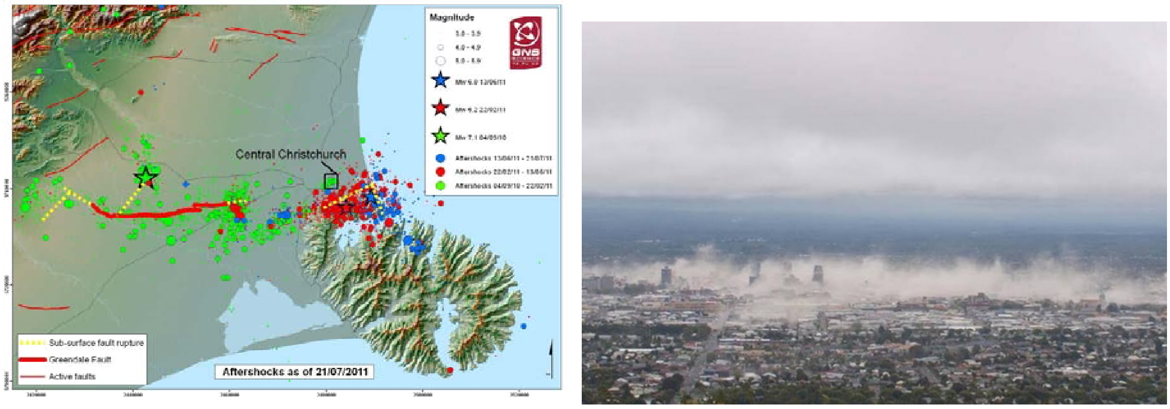


Figure 1. . Left: Fault rupture length and aftershock sequence for the 4 Sept 2011, 22nd Feb 2011 13th June 2011, 23 Dec 2011 earthquake events (source GNS) Science);
Right: 22 February event, impact on CBD

The combined effects of proximity, shallowness and directionality, led to a much greater shaking intensity of the 22 Feb aftershock, as recorded in the City of Christchurch, than that of the main shock on 4 Sept 2010. A wide range of medium-to- very high horizontal peak ground accelerations, PGA, were recorded by the GeoNet Network in the CBD area, with peaks exceeding 1.6g at Heathcove Valley and between 0.4g-0.7g in the CBD stations. This variation confirms in general strong dependence on the distance from the epicentre (as typical of attenuation relationships) but also on the site-specific soil characteristics and possibly basin amplification effects. Notably, the recorded values of vertical peak ground accelerations, in the range of 1.8-2.2g on the hills, were amongst the highest ever recorded worldwide. In the CBD the highest value of peak ground vertical accelerations recorded were in between 0.5g and 0.8g.

Figure 2 compares the elastic acceleration and displacement response spectra (5%-damped) after the 22 Feb 2011 event, from four ground motions recorded in the Christchurch CBD with the code-design level spectra (NZS1170:5, 2004 for 500-years and 2500-years return period, Soil Class D and Christchurch PGA=0.22g). As it can be noted, the level of shaking intensity, expressed in terms of spectral ordinates, that the buildings in the Centre Business District were subject to, was very high, well beyond the 1/500 years event code-level design when not (for a wide range of structural periods from 0.5s-1.75s) superior to the Maximum Credible Earthquake level (MCE, 1/2500 years event).

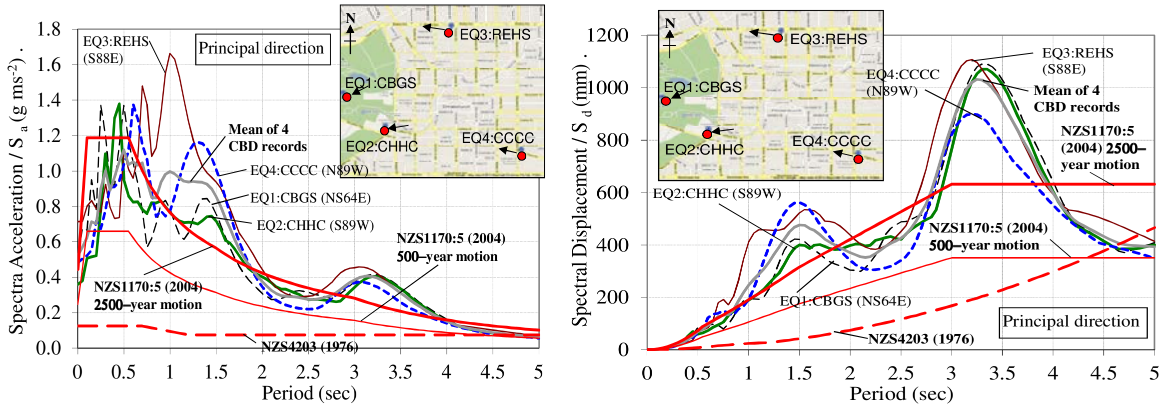


Figure 2. Acceleration and Displacement response spectra from 22 Feb 2011 Christchurch Earthquake records compared with code design spectra (NZS1170:5)

An overview on the level of shaking and overall structural performance of buildings in the 4 Sept 2010 and 22 Feb 2011 earthquakes events can be found in Kam et al. (2010) and Kam and Pampanin, (2011). For more comprehensive information on the overall earthquake impact, the reader is referred to the two Special Issues of the Bulletin of the New Zealand Society for Earthquake Engineering related to the 4 Sept 2010 and 22 Feb 2011 events (NZSEE, 2010, 2011).

Observed building damage

Considering the high level of shaking, which led to high inelastic behaviour and severe displacement/deformation demands, the overall behaviour of modern reinforced concrete structures (dominant type of multi-storey building in the CBD) can be classified in general as quite satisfactory.

However, the extent of structural damage in the plastic hinge regions, intended to act as fuses as part of the ductile sway mechanism, highlighted the whole controversy of traditional design philosophies, mainly focused on collapse-prevention and life-safety and not yet embracing a damage-control objective. Figure 3 shows a examples of the extent of structural damage in frames and shear walls in reinforced concrete multi-storey buildings (typically precast with emulation of cast-in situ connections).

Figure 3. Damage to post-1980s RC moment-resisting frames and walls
Such post-earthquake damage situation and the following decision to demolish and rebuild instead of repairing and strengthen was the most common scenario for the vast majority of reinforced concrete multi-storey buildings in the CDB.

A summary and breakdown of the of the placard key statistics from the processed Building Safety Evaluation (Post-earthquake inspection) database according to the type of structural system and year of construction can be found in (Kam et al., 2011, Kam and Pampanin, 2011, Pampanin et al., 2012)

In general whilst when referring to pre-1970s buildings (most of which had not been seismically strengthened) their relatively poor performance did not come as a surprise (nearly 48% of pre-1970s buildings were assigned yellow or red tagged and the collapse of one 1960s RC building led to multiple fatalities, Kam et al, 2011), the high number of modern buildings (at least post-1976, or post-1980s, thus designed in accordance with the basic principles of capacity design) to be demolished represents a serious concern and a wake up for the international earthquake community. Approximately 30% of the RC buildings in this class were yellow or red tagged (Kam *et al.*, 2011). The collapse of one 1980s RC building, the Canterbury Television Building, or CTV, caused the highest number of fatalities (Canterbury Earthquake Royal Commission of Enquiry, CERC, 2012).

As a consequence of the excessive cost-of-repairing (a well as, to some extent, of the possibility to rely upon a significant insurance coverage for partial or full replacement of the building) many relatively modern buildings (mid-1980s and onwards) have been or are being demolished (Fig. 4 left, recent aerial view of the CBD). On a positive note, if any, this massive man-made demolition is providing the opportunity for a significant re-design of the urban plan of the city as part of the rebuild (Fig. 4 right).

Figure 4. Top Left: aerial view of the CBD at November 2012 and recently proposed urban plan for the city centre reconstruction (CERA Blueprint)

2 REALITY CHECK: IS CURRENT DESIGN PHILOSOPHY MEETING SOCIETAL EXPECTATIONS?

Ductility and damage: is this an unavoidable equivalency?

The 22nd Feb 2011 Earthquake in Christchurch, New Zealand, has indeed and once more highlighted the severe mismatch between the expectations of building occupants and owners over the reality of the seismic performance of engineered building

A question is being raised: is ductility-based design philosophy becoming obsolete and does it really imply irreparable damage?

As well known within the earthquake engineering community, but apparently not clear at all in the wider public arena, the basic principle of the current seismic design philosophy of

ductile structures, referred to as "capacity design" or hierarchy of strength, developed in the mid/late1960s by Professors Bob Park and Tom Paulay at the University of Canterbury in New Zealand, is to ensure that the "weakest link of the chain" within the structural system is located where the designer wants and will behave as a ductile "fuse", protecting the structure from more undesired brittle failure mechanisms (Fig. 5).
The inelastic action is intentionally concentrated within selected discrete "sacrificial" regions of the structure, typical referred to as plastic hinges.

Until recent years, the development of inelastic action in traditional monolithic (or emulative) ductile connections has been assumed to inevitably lead to structural damage, thus implying that "ductility= damage", with associated repair costs and business downtime.

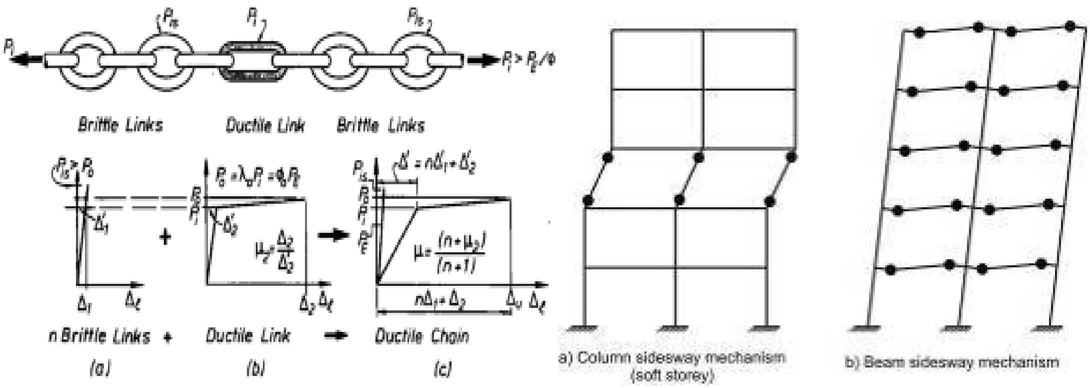


Figure 5: A tribute to the basic concept of capacity design: the "weakest link of the chain" concept (left) and its implementation in a frame system with the protection of a soft-storey (brittle) mechanism in favour of a beam side-sway (ductile) mechanism (Paulay and Priestley, 1992).

As later discussed in this paper, following the recent development of alternative cost-efficient solutions for high-performance low-damage structural systems, such ductility-damage equivalency is not anymore to be considered an unavoidable compromise of a ductile design.

What is the acceptable level of damage?

According to current performance-based design approach, visually summarized with the Performance Design Objective Matrix in Figure 6, different (and often not negligible) levels of structural damage and, consequently, repairing costs shall thus be expected and, depending on the seismic intensity, be typically accepted as unavoidable result of the inelastic behaviour. Performance levels are expression of the maximum acceptable extent of damage under a given level of seismic ground motion, thus representing losses and repair costs due to both structural and non-structural damage.

To give a practical example, according to the Basic Objective presented in this performance matrix, and associated to ordinary residential/commercial construction, a Life Safety damage level would be considered acceptable under a design level earthquake (traditionally taken as a 500 years return period event). This would imply that extensive damage, often beyond the reparability threshold (corresponding to a yellow/orange to red tag of the building), would be considered as an accepted/proposed target.

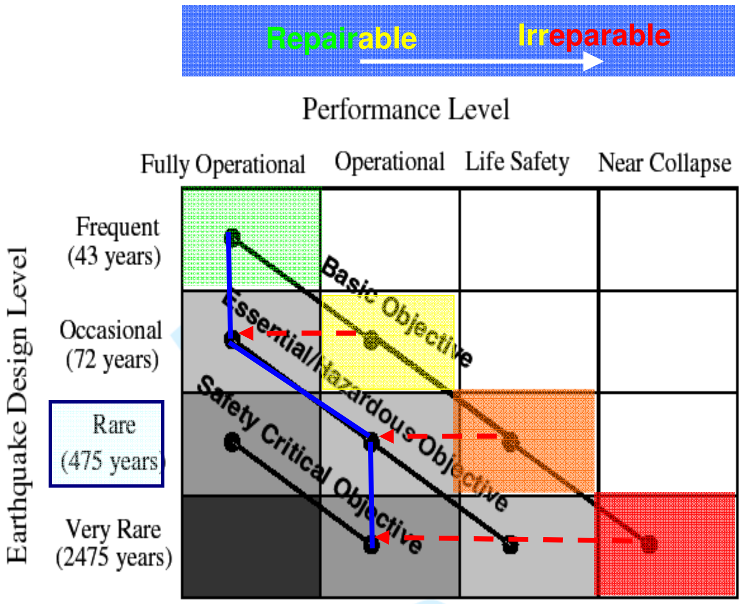


Figure 6. Seismic Performance Design Objective Matrix as defined by SEAOC Vision 2000 PBSE Guidelines, herein rearranged to match building tagging, and proposed/required modification of the Basic-Objective curve towards a damage-control approach (blue line, modified after Pampanin, 2010, Kam et al., 2011)

Such implications are clear and obvious to the technical professionals, but not necessary to the general public.

It should thus not come as a surprise if users, residents, clients, owners/stakeholders of the building/facilities as well as the territorial authorities had (not only in the occasion of Canterbury Earthquake, but also in similar situations in the past) a remarkably different opinion, based on a clearly different understanding of the significance of, and expectation from the behaviour of, an "earthquake-proof" building.

From the public perspective, not only life-safety and collapse prevention should be considered as "granted", but also only a minimum level of damage should be actually expected so to require minimum repairing costs and disruption of the daily activities.

The renewed challenged of earthquake engineering: raising the bar to meet societal expectation

In order to resolve this major perception gap and dangerous misunderstanding, a twofold approach is required:

- On one hand, increase the level of communication between scientists/researchers, practitioner engineers, territorial authorities, Industry representatives and/or, generally speaking, end-users. Define, set, agree and disclose to the wide public the accepted/targeted performance levels built in a Building Act or in a design code, including the not-written considerations and compromise between socio-economical consequences and technical limitations and costs. It shall be clear that these are to be considered "minimum", not "maximum" or target, standards, with the possibility, and somehow ethical duty, of achieving better performance if required/desired and when practically feasibile.

- On the other hand, significantly “raise the bar” by shifting the targeted performance goals from the typically accepted Collapse Prevention or Life-Safety level, to a more appropriate and needed Damage-Control level. This could be represented within the Performance Objective Matrix by a tangible shift of the Objective Curves to the left, i.e. towards higher performance levels or, equivalently, lower acceptable damage levels (Fig. 6, dashed line).

Moreover, the focus of the next generation of performance-based design framework should more explicitly directed towards the development of design tools and technical solutions for engineers and stakeholders to control the performance/damage of the building system as a whole, thus including superstructure, foundation systems and non-structural elements.

Valuable tentative recommendations/suggestions have been proposed in the past in terms of pair of limit states or performance requirements for both structural (the “skeleton”) and non-structural elements (the “dress”). Yet, practical cost-efficient solutions for low-damage resisting non-structural elements daily use of practitioners and contractor need to be specified and developed.
Not unexpectedly, the sequence of strong aftershocks that followed the main 4 September 2010 event, caused significant and repetitive damage to the non-structural components, requiring continuous and expensive repairing. Work is in progress in this space with the clear target to address this next fundamental step towards the development of an ultimate seismic resisting system as the society expects (Baird et al., 2011; Tasligedik, 2012).

Furthermore, the Canterbury earthquake has emphasised the actual impact of having a combined damage in the superstructures and in the foundation-soil system (Giorgini et al., 2011). The area of Soil-Foundation-Structure Interaction has received in the past decades a substantial attention reaching a significant maturity. Yet, there is strong need to convert the available information into practical guidelines for an integrated structure-soil-foundation performance based design.
This would require the definition and setting of specific and jointed limit states for the superstructure and the foundation and suggest the corresponding design parameters to achieve that “integrated” level of performance (Millen et al 2013). In the first phase of the Christchurch Rebuild, this issue is becoming more apparent, as the designers of new buildings are requested by the clients to be able to specify the targeted performance of the building property as a whole, thus including the superstructure (structural skeleton and non-structural elements) and foundation-soil system.

3 RAISING THE BAR: THE NEXT GENERATION OF DAMAGE-RESISTING SYSTEMS

The breakthrough of jointed ductile “articulated” systems: PRESSS-technology

A revolutionary alternative technological solution for precast concrete connections and system, capable of achieving high-performance (low-damage) at comparable costs has been introduced in the late 1990s as main outcome of the U.S. PRESSS (PREcast Seismic Structural System) program coordinated by the University of California, San Diego (Priestley, 1991, 1996; Priestley et al. 1999) and culminated with the pseudo-dynamic test of a large scale Five Storey Test Building (Fig. 7 left).

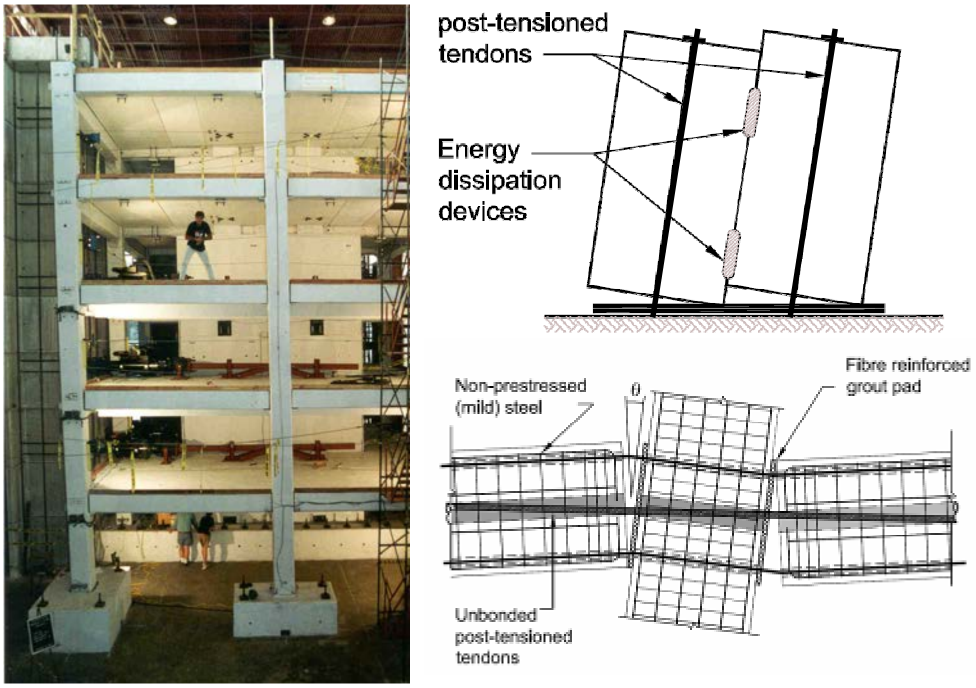


Figure 7. Left: Five-Storey PRESSS Building tested at University of California, San Diego (Priestley et al., 1999); Right: Jointed precast "hybrid" frame and wall systems (fib, 2003; NZS3101:2006)

In PRESSS frame or wall systems, moment-resisting dry jointed ductile connections are obtained by connecting precast elements through unbonded post-tensioning tendons/strands or bars. In the so-called hybrid system (Priestley et al., 1996; Stanton et al. 1997, Fig.7 right), unbonded post-tensioned bars or tendons are combined with non-prestressed mild steel (or similarly additional external dissipation devices as discussed in the next sections), inserted in corrugated metallic ducts and grouted to achieve fully bond conditions.

During the earthquake shaking, the inelastic demand is accommodated within the connection itself (beam-column, column to foundation or wall-to-foundation critical interface), through the opening and closing of an existing gap (rocking motion). The mechanism acts as a fuse or "internal isolation system" with negligible or no damage accumulating in the structural elements, which are basically maintained in the elastic range. The structural skeleton of the building would thus remain practically undamaged after a major design level earthquake without any need for structural repairing intervention.

This is a major difference and improvement when compared to cast-in-situ solutions where, as mentioned, damage has to be expected and it is actually accepted to occur in the plastic hinge regions, leading to substantial costs of repairing and business interruption. The traditional plastic hinge, or sacrificial damage-mechanism, is thus substituted by a sort of "controlled rocking" (dissipative and re-centering) at the critical interface with no or negligible damage (Fig. 8 left).

Moreover, the tendons are unbonded so to elongate within the duct without yielding. They can thus act as re-centering "springs", guaranteeing that the structure come back to its original at-rest position at the end of the shaking This re-centering & dissipating mechanism is described by a peculiar "flag-shape" hysteresis behaviour, whose properties and shape can be modified by the designer by varying the (moment) contributions, between the re-centering and the dissipation components (Fig. 8 right).

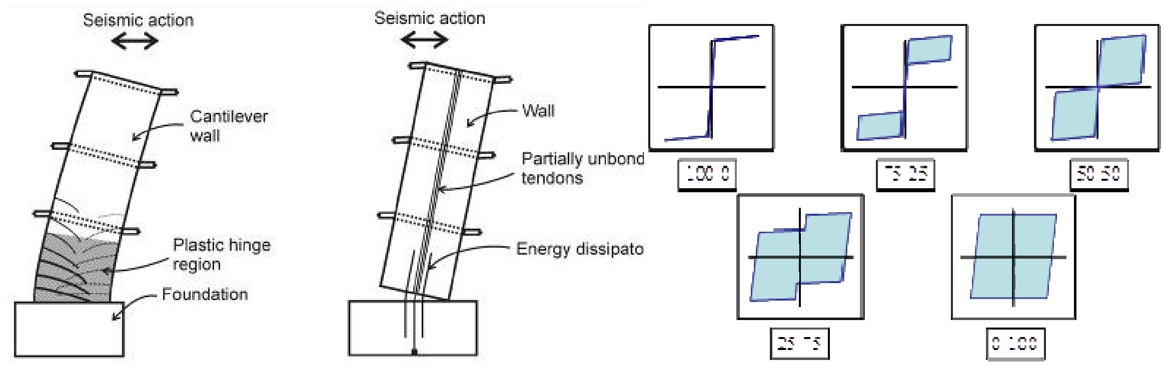


Figure 8. Left: Comparative response of a traditional monolithic system (damage in the plastic hinge and residual deformations) and a jointed precast (hybrid) solution (rocking mechanism with negligible damage and negligible residual deformations, fib, 2003); Right; Flag-shape hysteresis loop depending on the balance between re-centering and dissipative contribution.

One step further: reparability of the weakest link of the chain: "Plug&Play" replaceable dissipaters

In principle, either internal (grouted) mild steel bars or, more recently developed, external & replaceable supplemental damping devices can be adopted (Fig. 9). The original solution for hybrid connections proposed in the U.S.- PRESSS Program relied upon the use of grouted mild steel rebars, inserted in corrugated (metallic) ducts. A small unbonded length in the mild steel bars is typically adopted at the connection interface to limit the strain demand in the reinforcing bars and protect them from premature rupturing when the gap opens up to the design level of drift.

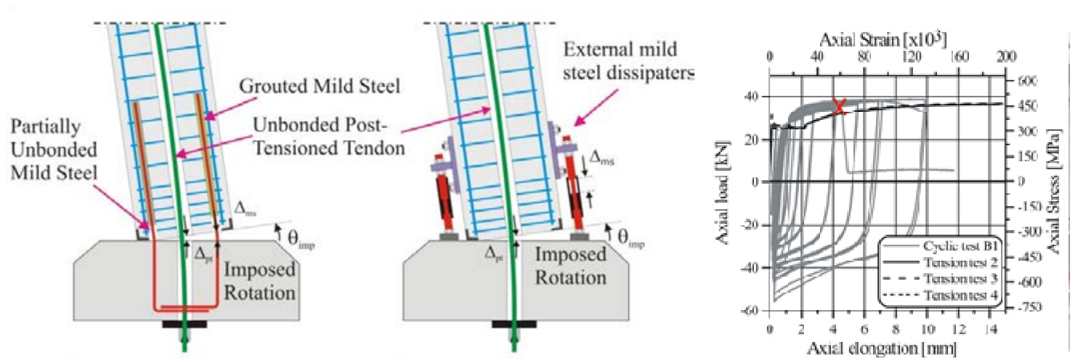


Figure 9. Left: Internal versus external replaceable dissipaters/fuses at the base-column/pier connection (Marriott et al. 2008); Right: stable hysteresis loop of a typical dissipater

A potential downside of such an approach is that, following an earthquake, the internal rebars would not be easily accessible nor replaceable as per a typical monolithic solution. Also the degradation of bond between concrete and steel during reversal cyclic loading causes some level of stiffness degradation thus potentially higher level of deformability of the structure.

More recently, following the declared target to achieve a low- (or no-) damage system, significant effort has been dedicated in the past few years towards the development of cost-efficient external dissipaters, referred to as "Plug&Play" (PnP), for their capability to be easily mounted and if required, demounted and replaced after an earthquake event (Pampanin, 2005). This option would give the possibility to conceive a modular system with replaceable sacrificial fuses at the rocking connection, acting as the "weakest link of the chain", according to capacity design principles.

One of the most efficient and practical PnP dissipater solution, developed and tested as part of several subassembly configurations, consist of axial, tension-compression yielding mild steel short-bar-elements, machined down to the desired "fuse" dimension and inserted and grouted (or epoxied) in a steel tube acting as anti-buckling restrainers.

A number of tests have been successfully carried out at the University of Canterbury in the past ten years on different subassemblies configuration including beam-column joint connections, wall systems, column (or bridge pier)-to-foundation connections (Fig. 10) with the aim to further simplify the constructability/assemblage and improve the reparability of the structure after an earthquake event, thus dramatically reducing the costs associated to the direct repairing of the structural system and to the downtime (business interruption).

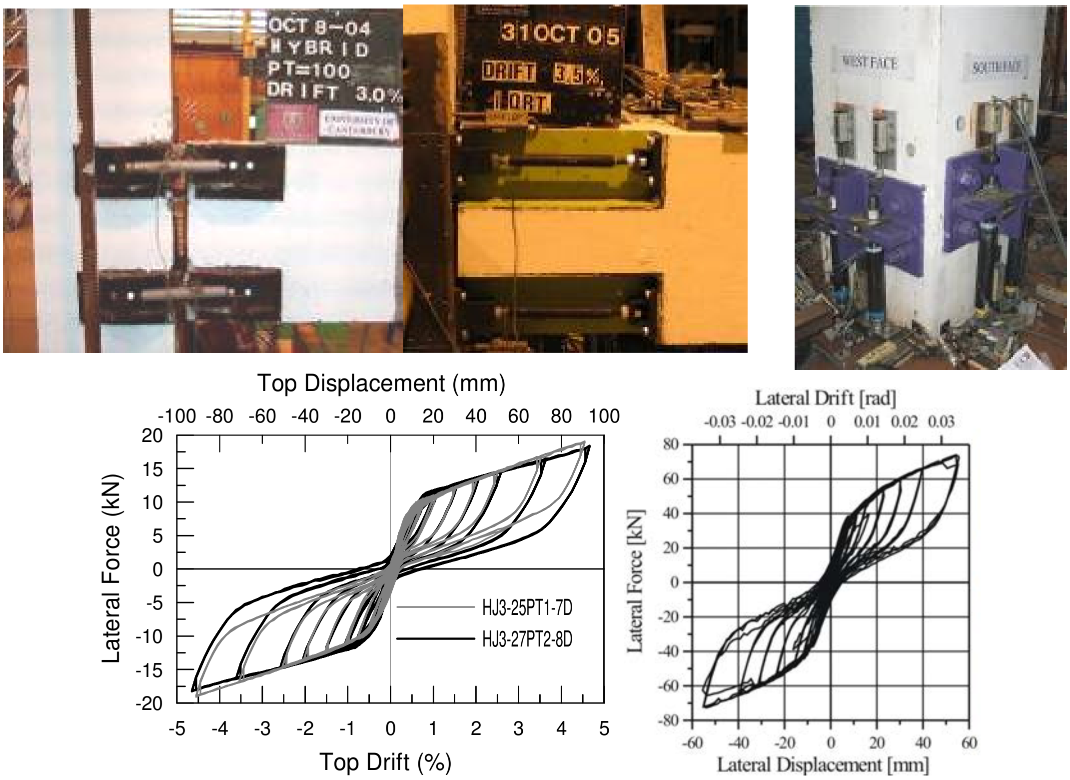
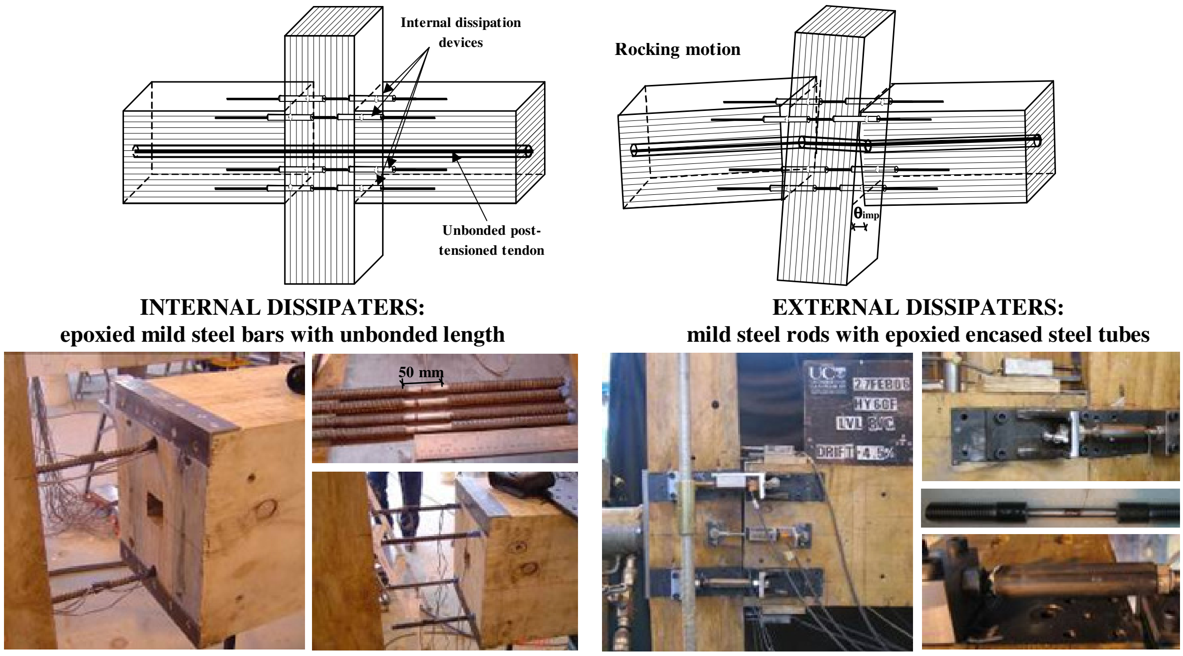


Figure 10. Alternative configurations of external replaceable (PnP) dissipaters for hybrid systems: Top left and centre: beam-column connections, with and without recess in the beam (from Pampanin et al. 2006); Top right: Column to foundation connections (from Marriott et al., 2009) Bottom: typical flag-shape hysteresis loops for a hybrid beam-column joint and a column-to-foundation connection with external dissipaters.

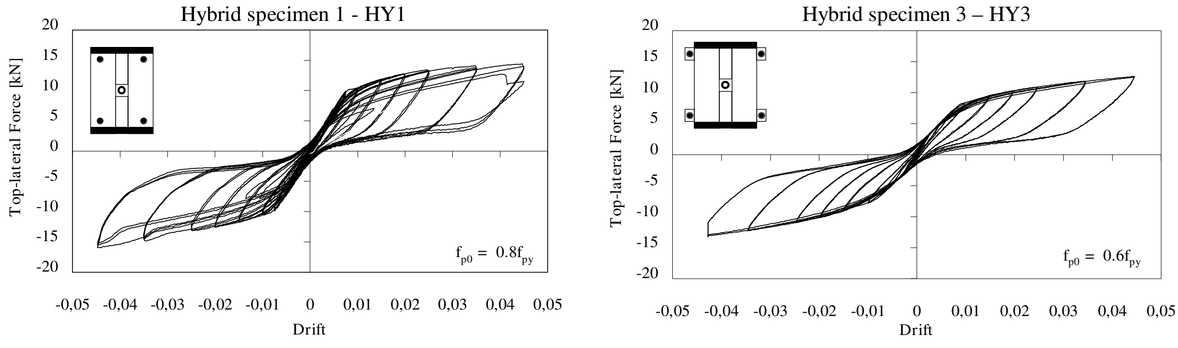
A second generation of self-centering/dissipative high-performance systems, referred to as Advanced Flag-Shape systems (AFS) has been recently proposed by Kam et al., 2010.
AFS systems combine alternative forms of displacement-proportional and velocity-proportional energy dissipation (i.e. yielding, friction or viscous damping) in series and/or in parallel with the main source of re-centering capacity (given by unbonded post-tensioned tendons, mechanical springs or Shape Memory Alloys, SMA, with super-elastic behaviour).
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 proven by numerical investigations (Kam et al., 2010) as well as shake table testing (Marriot et al., 2008).

4 EXTENSION TO MULTI-STOREY TIMBER BUILDINGS: THE PRES-LAM SYSTEM

The concept of post-tensioned hybrid (recentering/dissipating) systems has been recently and successfully extended from precast concrete to timber frames and walls (Palermo et al., 2005, 2006, Pampanin et al., 2006), in what is referred to as Pres-Lam (Prestressed Laminated timber) system. Since 2004, a series of experimental tests (comprising quasi-static cyclic, pseudodynamic and shake-table), have been carried out on several subassemblies or larger scale systems at the University of Canterbury to develop different arrangements of connections for unbonded post-tensioned timber frame and walls (Figs. 11-13).



(a) Internal and external dissipaters and construction details.



(b) Force-drift relationships for several different joints with internal and external dissipaters.

Figure 11. Arrangements and testing results of Pres-Lam beam-column joints with internal or external reinforcement (Palermo et al., 2005, 2006)

Figure 12. Testing of hybrid post-tensioned column-to-foundation connections with replaceable dissipaters (observed performance at 4.5% drift) (Palermo et al., 2006)

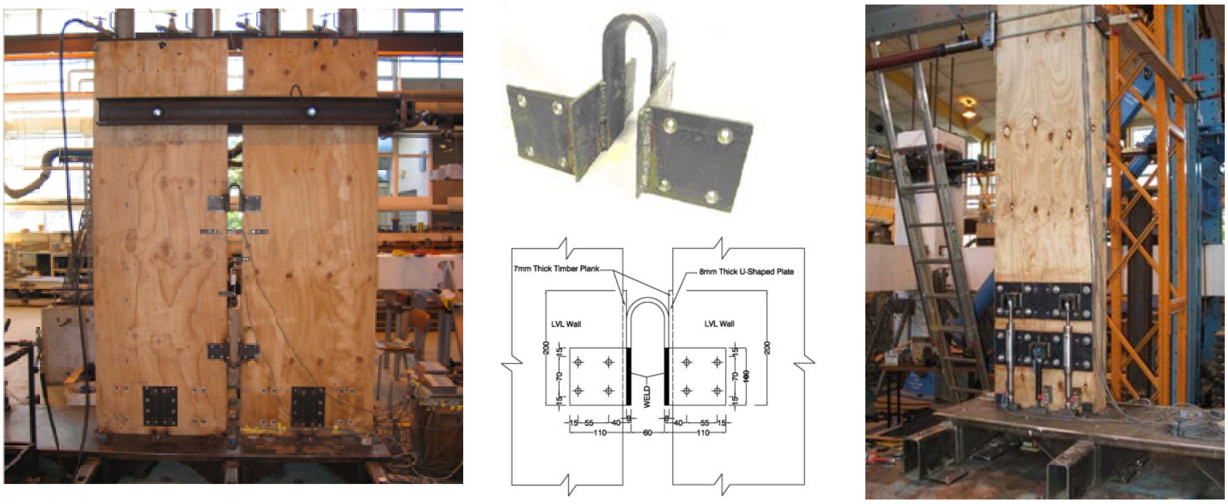


Figure 13. Left and Centre: Pres-Lam coupled walls with U-shape Flexural Plates dissipaters (Iqbal et al., 2007); Right: shake table test on Advanced-Flag-Shape Pres-Lam wall (viscous and hysteretic dampers in parallel), Marriott et al., 2008)

Due to its high homogeneity and good mechanical properties, Laminated Veneer Lumber (LVL) has been selected as the preferred engineered wood material for the first phase of the research and development. Any other engineered wood product as Glulam or Cross-lam (X-lam) can be adopted and in fact research is more recently on-going using both of them in addition to LVL.

The experimental results provided very satisfactory results and confirmation of the high potential of this new construction system, referred to as Pres-Lam, which gives opportunities for much greater use of timber and engineered wood products in large buildings, using innovative technologies for creating high quality buildings with large open spaces, excellent living and working environments, and resistance to hazards such as earthquakes, fires and extreme weather events (Buchanan et al., 2009).

A major multi-year R&D project has been ongoing from 2008-2013 under the umbrella of a NZ-Australia Research Consortium, STIC Ltd (Structural Timber Innovation Company).

5 ON-SITE IMPLEMENTATIONS OF PRESSS AND PRES-LAM TECHNOLOGY IN NEW ZEALAND

The continuous and rapid development of jointed ductile connections using PRESSS-technology for seismic resisting systems has resulted, within a bit more than one decade, in a wide range of alternative arrangements currently available to designers and contractors for practical applications, and to be selected on a case-by-case basis (following cost-benefit analysis). An overview of such developments, design criteria and examples of implementations have been given in Pampanin et al., (2005) and more recently in the PRESSS Design Handbook (2010)

Several on site applications of PRESSS-technology buildings have been implemented in different seismic-prone countries around the world, including, but not limited to, U.S., Central and South America, Europe and New Zealand. One of the first and most glamorous application of hybrid systems in high seismic regions was given by the Paramount Building in San Francisco (Fig. 14), consisting of a 39-storey apartment building and representing the highest precast concrete structure in a high seismic zone (Englerkirk, 2002). Perimeter seismic resisting frames were used in both directions. The dissipation was provided by internally grouted mild steel with a short unbonded length at the critical section interface to prevent premature fracture of the rebars.

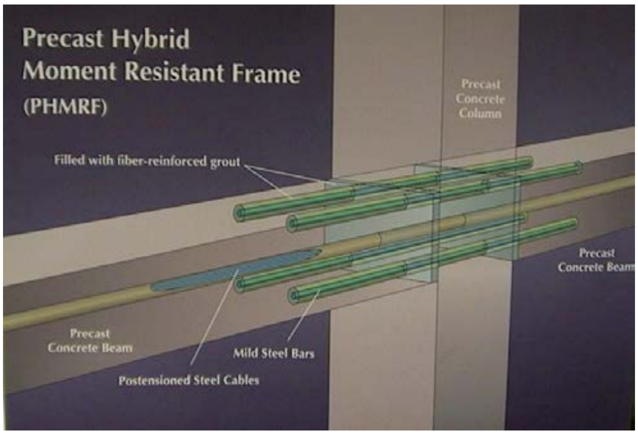


Figure 14. Paramount Building, 39-storey building, San Francisco (Englerkirk, 2002, photos courtesy of Pankow Builders, E. Miranda, Len McSaveney).

Given the evident structural efficiency and cost-effectiveness of these systems (e.g. high speed of erection) as well as flexibility in the architectural features (typical of precast concrete), several applications have quickly followed in Italy, through the implementation of the “Brooklyn System” (Fig. 15), developed by BS Italia, Bergamo, Italy, with draped tendons for longer spans and a hidden steel corbel (Pampanin et al., 2004). Several buildings, up to six storeys, have been designed and constructed in regions of low seismicity (gravity-load dominated frames). These buildings have different uses (commercial, exposition, industrial, hospital), plan configurations, and floor spans. A, overview on the system can be found in Pampanin et al. (2004).

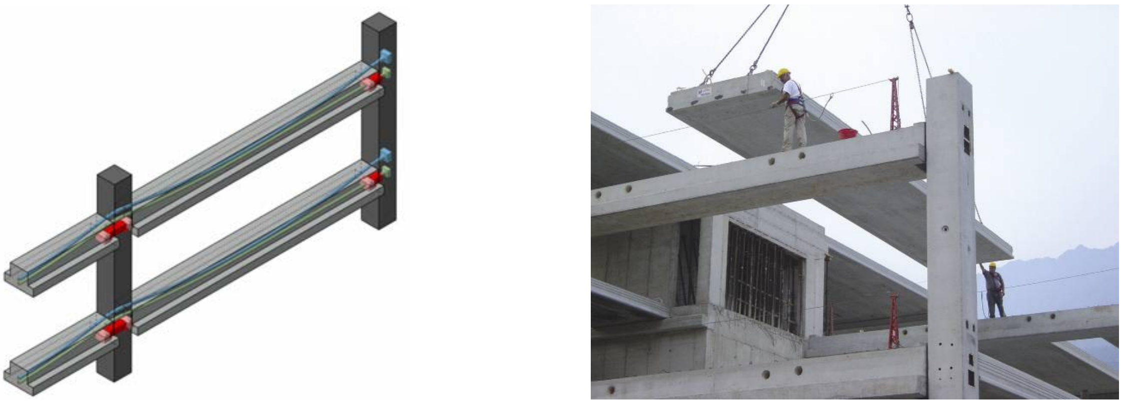


Figure 15. Application in Italy of the Brooklyn System, B.S. Italia, with draped tendons (Pampanin et al., 2004).

The first multi-storey PRESSS-building in New Zealand is the Alan MacDiarmid Building at Victoria University of Wellington (Fig. 16), designed by Dunning Thornton Consulting Ltd. The building has post-tensioned seismic frames in one direction and coupled post-tensioned walls in the other direction, with straight unbonded post-tensioned tendons.

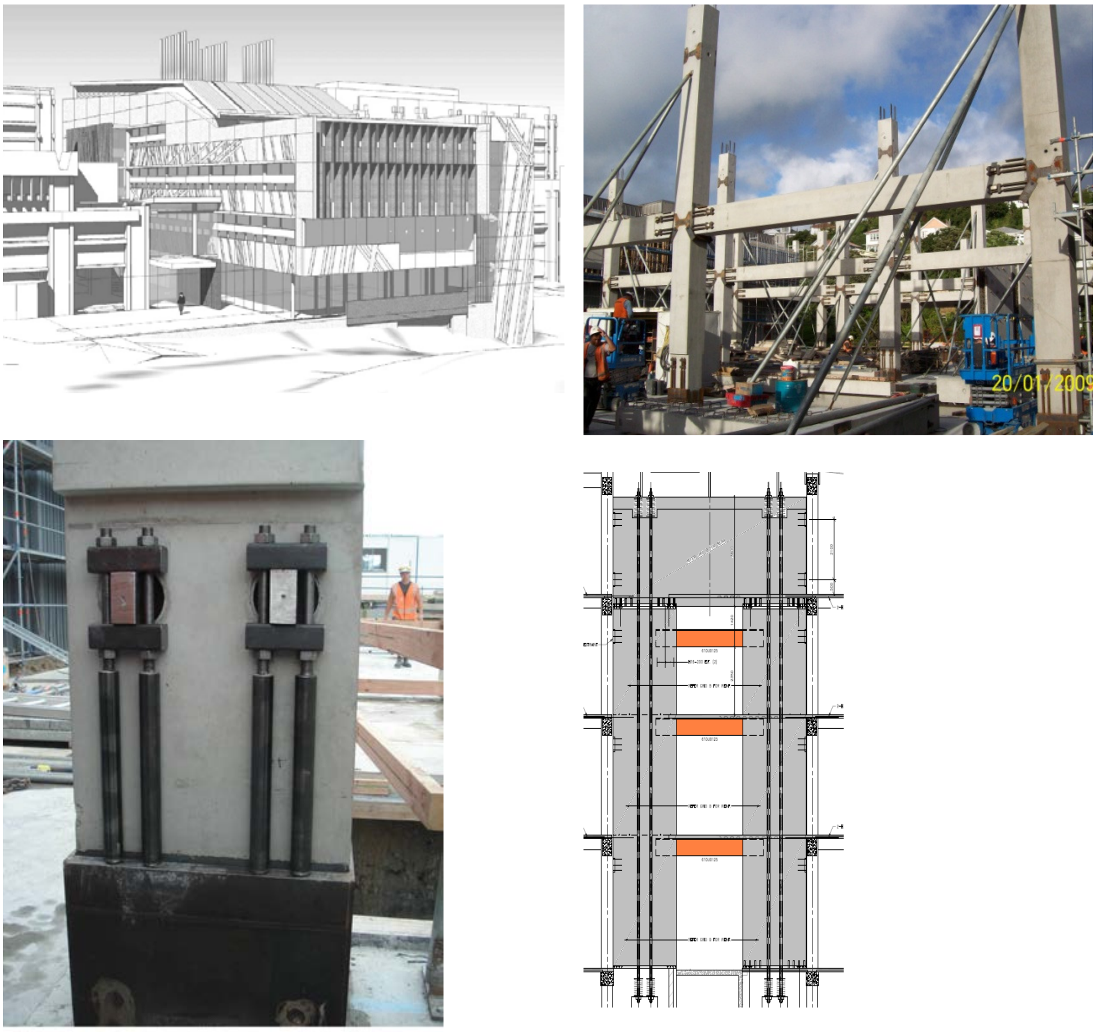


Figure 16. First multi-storey PRESSS-Building in New Zealand (Structural Engineers: Dunning Thornton Consultants; Cattanach and Pampanin, 2008).

This building features some of the latest technical solutions previously described, such as the external replaceable PnP dissipaters in the moment-resisting frame and unbonded post-tensioned sandwich walls coupled by slender coupling beams yielding in flexure. Additional novelty was the use of a deep cap-beam to guarantee rocking of the walls at both the base and the top sections (Cattanach and Pampanin, 2008). This building was awarded the NZ Concrete Society's Supreme Award in 2009 and several other innovation awards.

The design and construction of the second PRESSS-Building in NZ and first in South Island has followed at close distance and is represented by the Endoscopy Consultants' Building in Christchurch, designed for Southern Cross Hospitals Ltd by Structex Metro Ltd (Fig. 17). Also in this case both frames and coupled walls have been used in the two orthogonal directions. The post-tensioned frame system relies upon an asymmetric section reinforcement with internal mild steel located on the top of the beam only and casted on site along with the floor topping. The unbonded post- tensioned walls are coupled by using U-Shape Flexural Plates solutions.

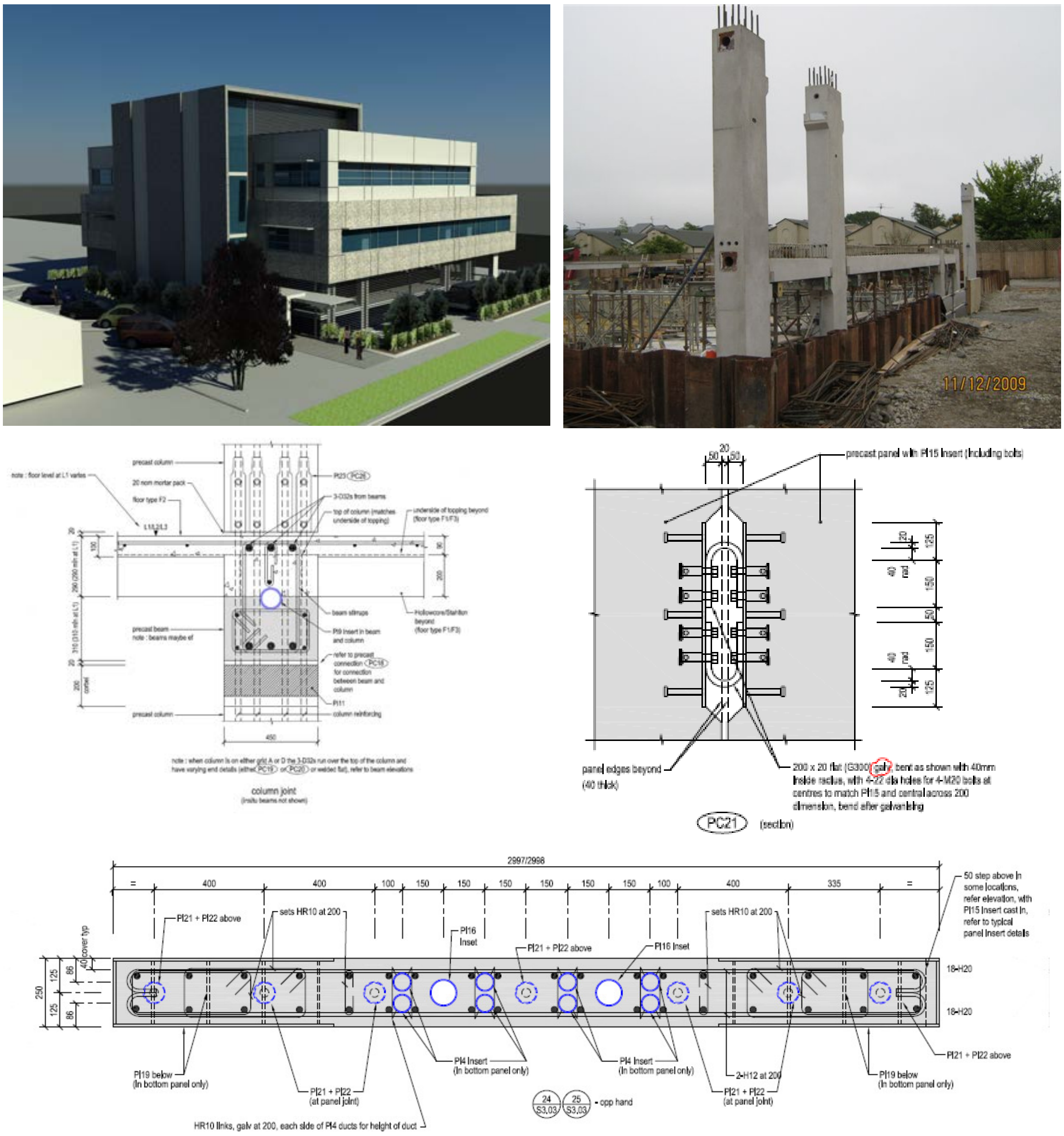


Figure 17. Southern Cross Hospital, Christchurch Rendering, construction of the frame, details of beams, walls and UFP dissipaters (Structural Engineers: Structex; Pampanin et al., 2011).

It is worth noting that both these later structures have been designed and modelled, during the design and peer review process, following the theory and step-by-step procedures included in in the PRESSS Design Handbook (2010), published by the NZ Concrete Society, which provides a full design example of a five-storey building in accordance to the NZS3101:2006 concrete design code Appendix B.

Real earthquake testing: when reality meets expectations
The Southern Cross hospital endoscopy Building has very satisfactorily passed the very severe tests of the recent Christchurch earthquake series. In particular, the 22 February earthquake was very close to the hospital with a very high level of shaking. Figure 18 shows the minor/cosmetic level of damage sustained by the structural systems which comprise post-tensioned hybrid frames in one directions and post-tensioned hybrid walls coupled with U-shape Flexural Plate Dissipaters. Important to note, the medical theatres with very sophisticated and expensive machinery were basically operational the day after the earthquake. One of the main feature in the design of a rocking-dissipative solution is in fact the possibility to tune the level of floor accelerations (not only drift) to protect both structural and non-structural elements including content and acceleration-sensitive equipment.
More information on the design concept and performance criteria, modelling and analysis, construction and observed behaviour of the building can be found in Pampanin et al., (2011).

Figure 18. Negligible damage, to both structural and non-structural components, in the Southern Cross Hospital after the earthquake of 22 February.

Implementation of Pres-Lam Buildings
Following the aforementioned extensive research and development on post-tensioned timber buildings at the University of Canterbury, the first world-wide applications of the Pres-Lam technology are occurring in New Zealand. Several new buildings have been constructed incorporating Pres-lam technology.

The world’s first commercial building using this technology is the NMIT (Nelson, Marlborough Institute of Technology) building, constructed in Nelson. The building has vertically post-tensioned timber walls resisting all lateral loads as shown in Figure 19 (Devereux et al., 2011). Coupled walls in both direction are post-tensioned to the foundation through high strength bars with a cavity allocated for the bar couplers. Steel UFP devices link the pairs of structural walls together and provide dissipative capacity to the system. The building was opened in January 2011.

Figure 19. The World's first Pres-Lam building implementing unbonded post-tensioned rocking/dissipative timber walls. Nelson Marlborough Institute of Technology, (NMIT), New Zealand (Structural Engineers Aurecon, Devereux et al., 2011, Architects Irving-Smith-Jack)

The Carterton Events Centre (Fig. 20), located 100km north of Wellington, is the second building in the world to adopt the Pres-Lam concept. Post-tensioned rocking walls were designed as the lateral load resisting system (six walls in one direction and five in the other direction). The post-tensioning details are similar to the NMIT building, while internal epoxied internal bars are used for energy dissipation (Figure 20 right).

Figure 20 Carterton Events Centre. Single-storey building with LVL truss roof (Designed by Opus International: Dekker et al. 2012).

The University of Canterbury EXPAN building (Fig. 21) was originally a two-third scale prototype building tested in the laboratory under severe bi-directional loading conditions (Newcombe et al., 2010). After a successful testing programme, the building was dismantled and re-erected as the head office for the Research Consortium STIC (Structural Timber Innovation Company Ltd). Due to the low mass, the connections were redesigned from hybrid to purely post-tensioned without any dissipation devices. The light weight of the structure allowed the main timber frames of the building to be post-tensioned on the ground and lifted into places shown in Figure 21.

Figure 21. From laboratory specimen to office building: 3D Test Specimen tested in the lab (Newcombe et al, 2010), demounted and reconstructed (Smith et al., 2011) on UC campus as EXPAN/STIC office

The new College of Creative Arts (CoCa) building for Massey University's Wellington campus has been recently completed (Fig. 22). The building is the first to combine post-tensioned timber frame with innovative draped post-tensioning profiles to reduce deflections under vertical loading. Additional dissipation is added in the frame directions by using U-Shape Flexural plate devices, placed horizontally and activated by the relative movement between (some of) the first floor beams and elevated concrete walls/pedestal. This is a mixed material damage-resistant building which relies on rocking precast concrete PRESSS walls in one direction and Pres-Lam timber frames in the other direction.

Figure 22 College of Creating Arts (CoCa) Building, Massey University, Wellington, New Zealand (Structural Engineers: Dunning Thornton Consultants)

As part of the Christchurch Rebuild, a number of buildings under construction or design will implement the aforementioned damage-resisting technologies (Figs. 23-25), in some cases using mixed materials and/or a combination with base isolations and other supplemental damping devices.

Figure 23. Christchurch Rebuild: First Pres-Lam building in Christchurch, Merritt Building, Structural Engineers: Kirk and Roberts; Architects: Sheppard and Rout

Figure 24. Christchurch Rebuild. Left: St Elmo Courts a 1930 RC building demolished Right :rendering of the "new St. Elmo" using a combination of base-isolation and a post-tensioned timber-concrete two-way frame in the superstructures, Architect: Ricky Proko, Structural Engineers: Ruamoko Solutions;

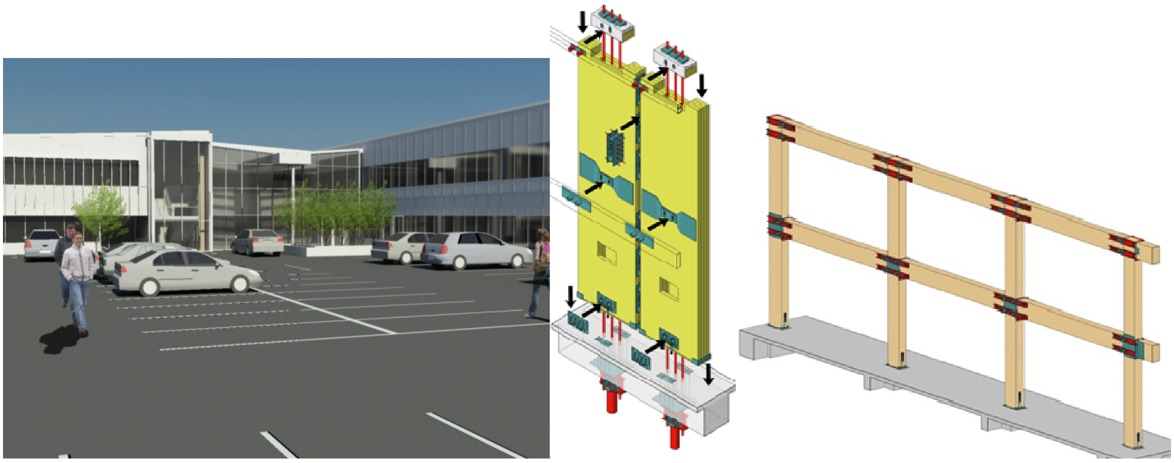


Figure 25. Christchurch Rebuild: Trimble Building, Architecture and Structures from Opus International (Brown et al.., 2012)

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