REALITY-CHECK AND RENEWED CHALLENGES IN EARTHQUAKE ENGINEERING: IMPLEMENTING LOW-DAMAGE SYSTEMS – FROM THEORY TO PRACTICE

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SUMMARY:

Earthquake Engineering is facing an extraordinarily challenging era, the ultimate target being set at increasingly higher levels by the demanding expectations of our modern society. The renewed challenge is to be able to provide low-cost, thus more widely affordable, high-seismic-performance structures capable of sustaining a design level earthquake with limited or negligible damage, minimum disruption of business (downtime) or, in more general terms, controllable socio-economical losses.

The Canterbury earthquakes sequence in 2010-2011 has represented a tough reality check, confirming the current mismatch between societal expectations over the reality of seismic performance of modern buildings. In general, albeit with some unfortunate exceptions, modern multi-storey buildings performed as expected from a technical point of view, in particular when considering the intensity of the shaking (higher than new code design) they were subjected to. As per capacity design principles, plastic hinges formed in discrete regions, allowing the buildings to sway and stand and people to evacuate. Nevertheless, in many cases, these buildings were deemed too expensive to be repaired and were consequently demolished.

Targeting life-safety is arguably not enough for our modern society, at least when dealing with new building construction. A paradigm shift towards damage-control design philosophy and technologies is urgently required.
This paper and the associated presentation 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 upon controlled rocking & dissipating mechanisms for either reinforced concrete and, more recently, laminated timber structures.

An overview of recent on-site applications of such 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 seismic design, damage-control, Canterbury earthquake

1. INTRODUCTION

1.1 Ductility and damage: is this an unavoidable equivalency?

Recognizing the economic disadvantages of designing buildings to withstand earthquakes elastically as well as the correlated disastrous socio-economic consequences after a design-level or higher-than designed level earthquake intensity (e.g. as for example observed in the Great Hanshin event, Kobe 1995 and, most recently in the 22 Feb 2011 Christchurch Earthquake), current seismic design philosophies promote the design of ductile structural systems able to undergo inelastic reverse cycles while sustaining their integrity.

The basic principle of this design philosophy, widely known and 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 that it will behave as a ductile "fuse", protecting the structure from more undesired brittle failure mechanisms (Fig. 1).

This approach would allow the building to sway laterally without collapsing in what in gergo is typically referred to as a "soft-storey" mechanism or, more simplistically a "pancake" collapse. Regardless of the structural material adopted (i.e. concrete, steel, timber) traditional ductile systems rely on the inelastic behaviour of the material. 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) connections has been assumed to inevitably lead to structural damage, thus implying that "ductility = damage", with associated repair costs and business downtime.

As discussed later in the paper, following the introduction of recently developed, cost-efficient and high-performance technologies, under the umbrella of an emerging damage-avoidance or damage-control design philosophy, the ductility-damage equivalency is not anymore a necessary compromise of a ductile design.

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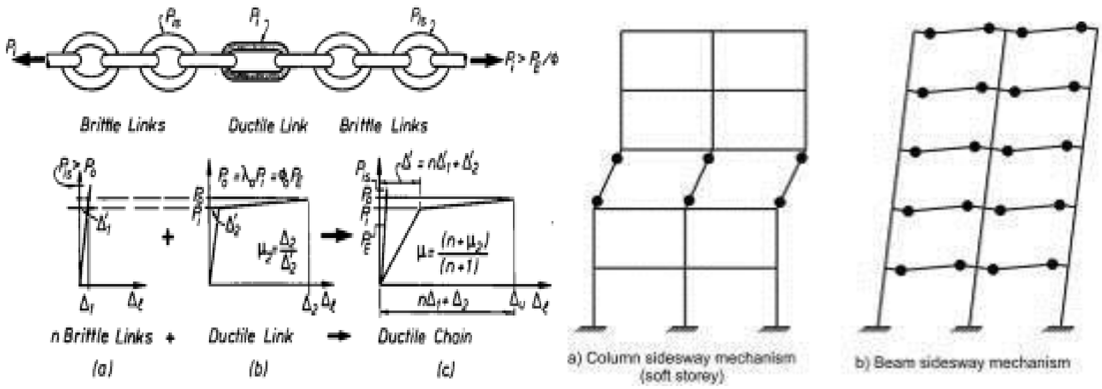


Figure 1: ***A tribute to the basic concept of capacity design: the "weakest link of the chain" concept (left) and its implementation in a frame system with the protection of a soft-storey (brittle) mechanism in favour of a beam side-sway (ductile) mechanism (Paulay and Priestley, 1992).***

1.2 What is an acceptable level of damage?

In response to a recognized urgent need to design, construct and maintain facilities with better damage control following an earthquake event, a special effort has been dedicated in the last two decades to the preparation of a platform for ad-hoc guidelines involving the whole building process, from the conceptual design to the detailing and construction aspects.

In the comprehensive document prepared by the SEAOC Vision 2000 Committee (1995), Performance Based Seismic Engineering (PBSE) was given a comprehensive definition, as consisting of "*a set of engineering procedures for design and construction of structures to achieve predictable levels of performance in response to specified levels of earthquake, within definable levels of reliability*" and interim recommendations have been provided to actuate it.

According to a performance-based design approach, 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.

Within this proposed framework, expected or desired performance levels are coupled with levels of seismic hazard by performance design objectives as illustrated by the Performance Design Objective Matrix shown in Figure 2.

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. As a further and fundamental step in the development of practical PBSE guidelines, the actual conditions of the building as a whole should be expressed not only through qualitative terms, intended to be meaningful to the general public, using general terminology and concepts describing the status of the facility (i.e. Fully Operational, Operational, Life Safety and Near Collapse) but also, more importantly, through appropriate technically-sound engineering terms and parameters, able to assess the extent of damage (varying from negligible to minor, moderate and severe) for the single structural or non-structural elements (ceiling, partitions, claddings/facades, content) as well as for the whole system.

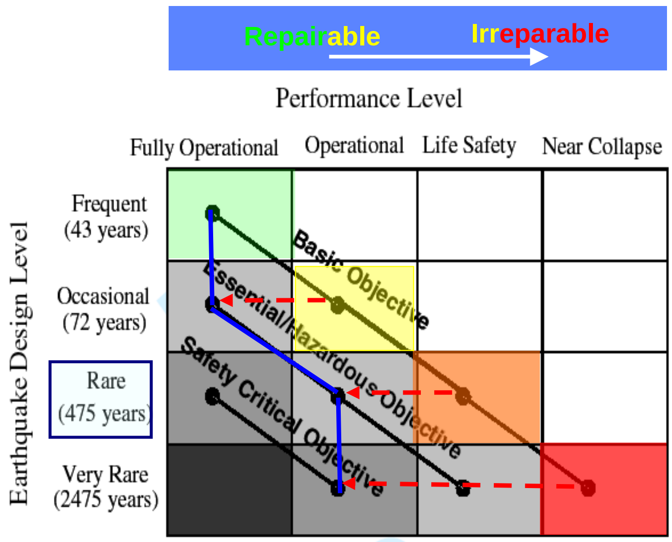


Figure 2: ***Seismic Performance Design Objective Matrix as defined by SEAOC Vision 2000 PBSE Guidelines, herein rearranged to match building tagging, and proposed/required modification of the Basic-Objective curve towards a damage-control approach (blue line, modified after Pampanin, 2010, Kam et al., 2011).***

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.

Such implications might be clear and obvious to the technical professionals, but not necessary to the general public. It would thus not come as a surprise if users, residents, clients, owners/stakeholders of the building/facilities as well as the territorial authorities had a remarkably different opinion, based on a clearly different understanding of the significance and expectation from the behaviour of an "earthquake-proof" building.
From the public perspective, not only life-safety and collapse prevention would be considered as "granted", but also only a minimum level of damage would be actually expected so to require minimum repairing costs and disruption of the daily activities.

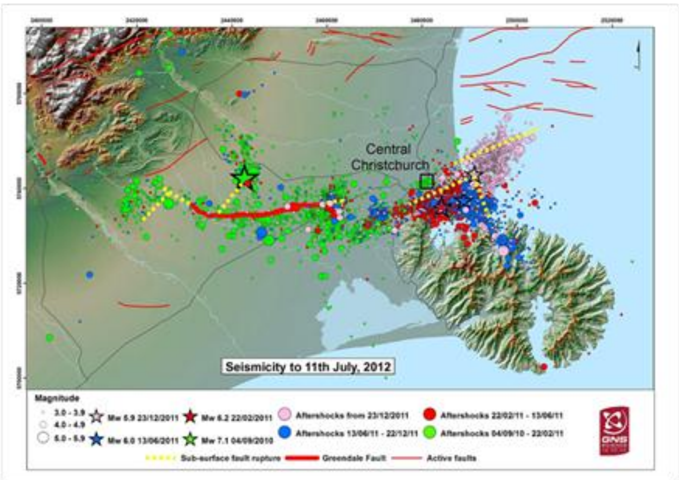
2. REALITY CHECK: THE CANTERBURY EARTHQUAKE SEQUENCE

The 22nd February 2011 Earthquake in Christchurch, New Zealand, has unfortunately been a tough reality check, further highlighting the severe mismatch between the expectations of building occupants and owners over the reality of the seismic performance of engineered buildings.

In the next paragraphs an overview of emerging lessons from the Canterbury Earthquake sequence will be provided, with focus on the on-going increased awareness of the concept of risk and performance by the general public and on the consequently market-driven re-writing of performance-based criteria and objectives, in lieu of the existence of cost-efficient technical solutions for low-damage seismic resisting systems able to accomplish and deliver to the higher public expectations.

2.1 The 22 February 2011 earthquake event and its overall impact

The Mw 6.3 Christchurch (Lyttelton) earthquake, itself officially referred to as an aftershock, occurred at 12.51pm on Tuesday 22nd February 2011, approximately 5 months after the Mw 7.1 Darfield (Canterbury) main shock (Fig. 3). The epicentre of the February event was approximately 10 km 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), significant shaking was experienced in the city centre, the eastern suburbs, Lyttleton-Sumner-Port 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 tens of thousands of timber houses and unprecedented liquefaction effects in whole parts of the city.



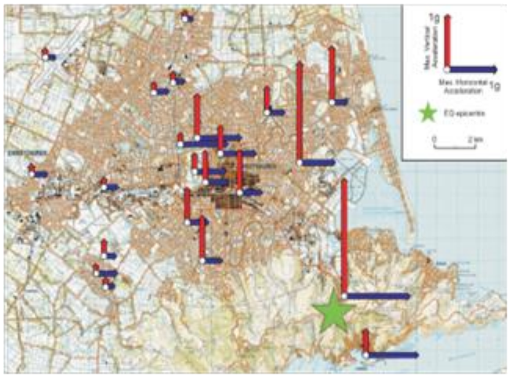


Figure 3: Left: Fault rupture length and aftershock sequence for the 4 Sept 2011, 22nd Feb 2011 13th June 2011, 23 Dec 2011 events; Right: peak ground accelerations during the 22 Feb 2011 aftershock (source GNS Science).

The combined effects of proximity, shallowness and directionality, led to a much greater shaking intensity of the 22 February aftershock, as recorded in the City of Christchurch, than that of the main shock on 4 September 2010. A wide range of medium-to-very high horizontal peak ground accelerations, PGA, were recorded by the GeoNet Network in the CBD area (Fig. 3, right), with peaks exceeding 1.6g at Heathcote Valley and between 0.4-0.7g 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.

2.2 "Spectrum compatibility" of the recorded ground motion

Figure 4 compares the elastic acceleration and displacement response spectra (5%-damped) after the 22 February 2011 event, from four ground motions recorded in the Christchurch CBD with the code-design level spectra (NZS1170:5, 2004 for 500-years and 2,500-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, 1/2,500 years event).

A more comprehensive 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).

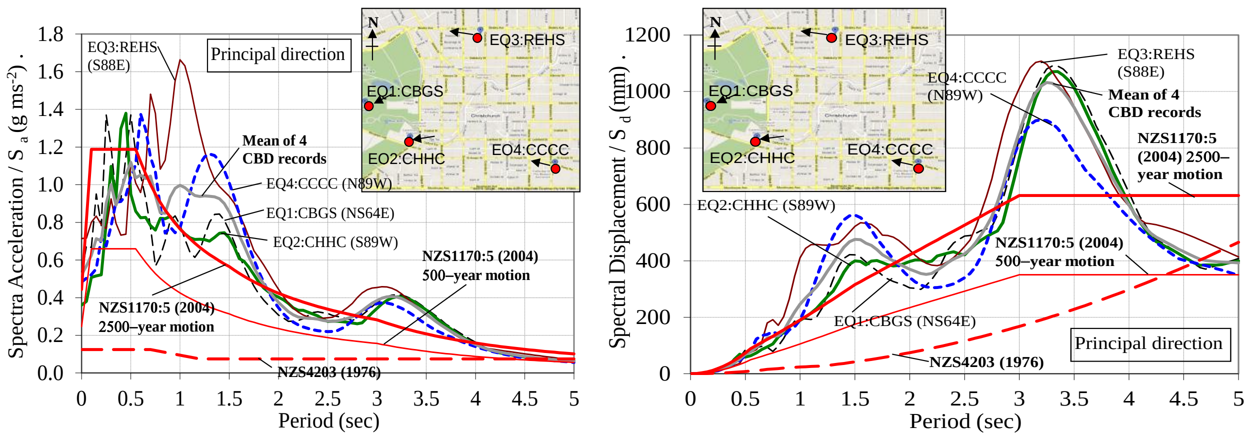


***Figure 4:** Acceleration and Displacement response spectra from 22 Feb 2011 Christchurch Earthquake records compared with code design spectra (NZS1170:5, Kam et al. 2011, Kam and Pampanin 2011).*

2.3 Observed building damage and overall statistics

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.

Many relatively modern buildings (mid-1980s and onwards) have already been or are being demolished as a consequence of the excessive cost-of-repairing (as well as, to some extent, to the possibility to relying upon a significant insurance coverage for partial or full replacement). Most of the buildings have suffered and will continue to suffer significant business interruption and downtime, also as a consequence of the closure of a widely affected area in the CBD.

Figure 5 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 5:** Damage to post-1980s RC moment-resisting frames and walls (Kam et al. 2011, Kam and Pampanin 2011).*

The Price Waterhouse Coopers (PWC) Building, a 22 storey reinforced concrete building designed and constructed in the mid-late 1980s (Restrepo, 1993; Park, 2002) represents somehow a "text-book" in terms of ductile seismic response according to a beam-sway mechanism. The building seismic resisting systems comprise perimeters moment-resisting frames in both directions, with flexible interior frames for gravity only/mainly. The precast concrete frames, constructed according to an emulation of cast-in-place concrete (as shown in Fig. 6), with a wet connection at mid-span of the beams and thus outside the plastic-hinge zone, behaved very well, with beam-hinging occurring at the beam-column interface at many floors up the elevation of the buildings, thus developing an exemplar beam-sway mechanism. A proper hierarchy of strength or capacity design protected the column from any inelastic mechanism. No noticeable cracking was evident even in the exterior-corner columns belonging to both direction frames and thus subject to a particularly high demand.

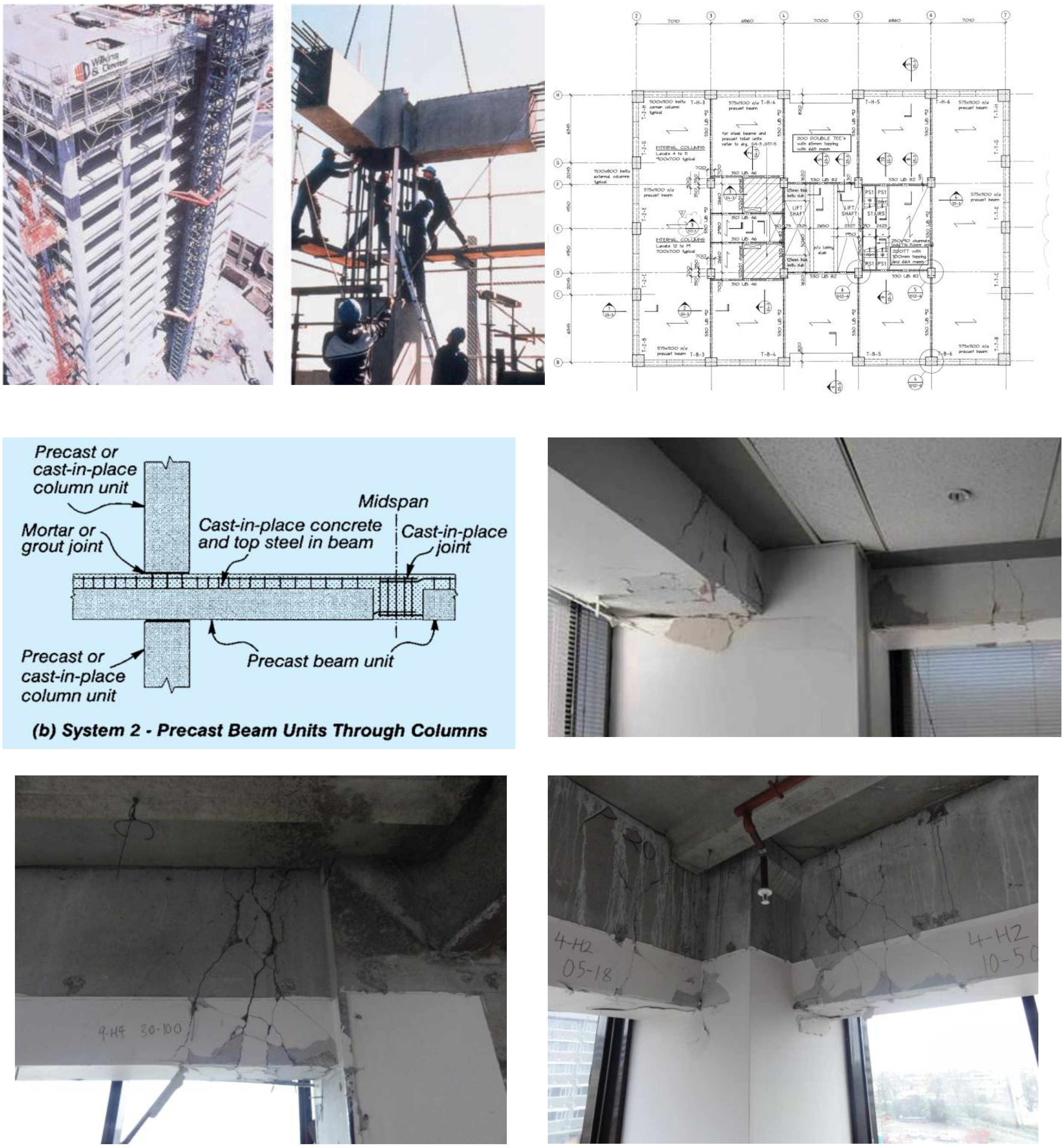


Figure 6: Beam plastic hinges in a 22-storey reinforced concrete building constructed in mid-end 1980s (currently under demolition). Top Left; photo of building under construction (courtesy of Restrepo).

Due to the inelastic mechanisms developed in the structural elements at most floors, the post-earthquake building state was characterized by low to moderate residual interstorey drifts. Furthermore, permanent deformations in the soil-foundation structures (consisting of shallow foundation) led to an overall leaning/tilting of the building. Repairing and strengthening options were considered, but found uneconomical when compared to the option of a controlled demolition and rebuild, partially or mostly covered by the insurance.

Such a post-earthquake damage situation and the following decision to demolish and rebuild instead of repairing and strengthening was the most common scenario for the vast majority of reinforced concrete multi-storey buildings in the CDB.

Figures 7 and 8 summarise the key statistics and findings from the processed Building Safety Evaluation (Post-earthquake inspection) database. The breakdown of the placard statistics according to the type of structural system and year of construction is presented in Figure 7 (Kam *et al*., 2011, Kam and Pampanin, 2011, Pampanin *et al*., 2012)

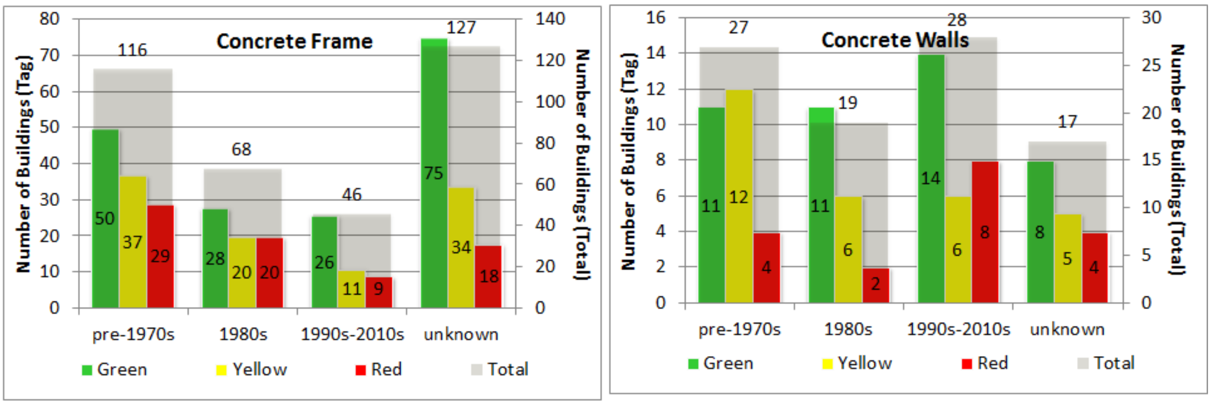


Figure 7: ***Distribution of Building Safety Evaluation placards of all buildings in the Christchurch CBD as per 12 June 2011 (source: CCC and research team inspections). The shaded bar on the secondary vertical axis shows the total number of buildings in each building construction age (Kam et al., 2011).***

Out of at least 3,000 buildings within the Christchurch CBD, as per 12 June 2011 (a day before the 13 June Mw 5.5 and 6.0 aftershocks), 53% of these were assessed as "Green – No restriction on use or occupancy", 23% as "Yellow - Restricted Use" and 24% as "Red – Unsafe". These tagging results were mostly based on a Level 1 (exterior only) and Level 2 (interior and exterior) assessment. Subsequently, a third evaluation phase consisting on a Detailed Engineering Evaluation (DEE) and relying upon structural drawings and calculations has been initiated (EAG, 2011).

Interestingly, the sum of the yellow and red tagged building (although based on L1 and L2 assessment only and prior to another damaging aftershock in June) represents approximately 1,300-1,400 buildings. According to a previous CERA (Canterbury Earthquake Recovery Authority) document, up to 1,300 buildings may be demolished.

Whilst when referring to pre-1970s buildings (most of which had not been seismically strengthened) their 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).

Figure 8: ***Top Left: distribution of buildings tagging statistics in Christchurch CBD. Building tagging is based on the CCC/Civil Defence Building Safety Evaluation procedure. (Statistics data is updated to 12 June 2011) (Kam et al., 2011). Top centre, right and bottom: example of multi-storey buildings under demolition and overview of CBD at August 2012 with entire lots "cleaned-up" as a result of the extensive demolitions.***

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

The excessive socio-economic impact of the Canterbury earthquakes sequence in 2010-2011 has clearly and critically highlighted the mismatch between the societal expectations over the reality of engineered buildings' seismic performance.

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 academia, practitioner engineers, territorial authorities, industry representatives and/or, generally speaking, end-users. Define, set, agree and disclose to the wider public the accepted/targeted performance levels built into a Building Act or in a design code, including the not-written considerations and compromises between socio-economical consequences and technical limitations and costs. It shall be clear that these are to be considered “minimum” not “maximum”, standards, with the possibility of achieving better performance if required/desired.

- 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. 2 right, dashed line).

Moreover, the focus of the next generation of performance base design frameworks 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 (Fig.8).

Figure 8: ***Example of extensive damage to non-structural elements (ceilings and partitions) and of tilting due to differential settlement of the soil-foundation system.***

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 in daily use by practitioners and contractors 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 repair (Fig. 8 left). 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 society expects (Baird *et al.,* 2011; Tasligedik, 2012).

Furthermore, the Canterbury earthquake has emphasised the actual impact of having combined damage in the superstructures and in the foundation-soil system (Fig. 8 right, 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. In the aftermath of the reconstruction of Christchurch, this issue is becoming more apparent, as the designers of new buildings are requested by the clients to be able to specify the targeted overall performance of the building, thus including the superstructure (skeleton and non-structural elements) and foundation-soil system.

In this specific contribution emphasis will be given to the possibility and opportunity to implement higher-performance structural systems and technology for superior seismic protections of the structural components of buildings.

3. NEXT GENERATION OF DAMAGE-RESISTING SYSTEMS

The increasing expectation of buildings capable of fulfilling the compelling requirements of cost-effectiveness and high seismic-performance have, in the recent past, led to a major effort towards the development of damage-control design approaches and technologies, in addition to, or better complementary and integrative of, the more common and renown (albeit not widely enough) base isolation and supplemental damping options.

In the next paragraphs an overview of the recent developments and on site implementations of emerging solutions for damage-control solutions, based on dry jointed ductile connections and referred to as PRESSS-technology (or Pres-Lam in its recent extension to timber) will be given.

3.1 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) programme 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. 9).

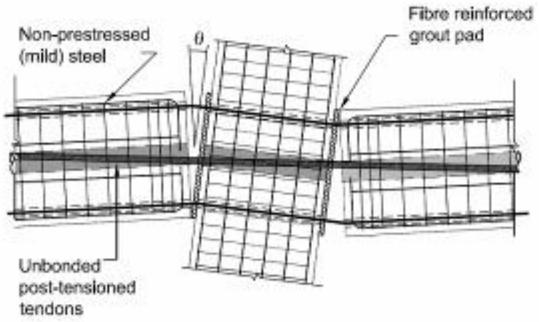
Figure 9: Five-Storey PRESSS Building tested at University of California, San Diego (Priestley et al., 1999).

The new construction system, based on dry jointed ductile connections, was conceived and developed for precast concrete buildings (frames and walls) in seismic regions with the intent to create a sound alternative to the traditional "wet" and/or "strong" connections typical of the emulation of the cast-in-place approach.

In PRESSS frame or wall systems, precast elements are jointed together through unbonded post-tensioning tendons/strands or bars creating moment-resisting connections.

A particularly efficient solution is given by the "hybrid" system (Priestley *et al*., 1996; Stanton *et al.* 1997, Fig. 10), which combines unbonded post-tensioned bars or tendons and 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 full 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, basically maintained in the elastic range. The basic structural skeleton of the building would thus remain undamaged after a major design level earthquake without any need for repairing intervention.



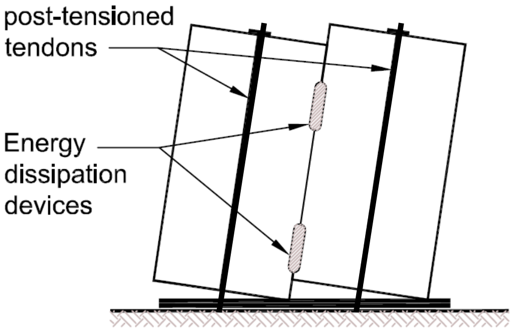


Figure 10: Jointed precast "hybrid" frame and wall systems (fib, 2003; NZS3101:2006).

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 plastic hinge, or sacrificial damage-mechanism, is thus substituted by a sort of "controlled rocking" (dissipative and re-centring) at the critical interface with no or negligible damage (Fig. 11).

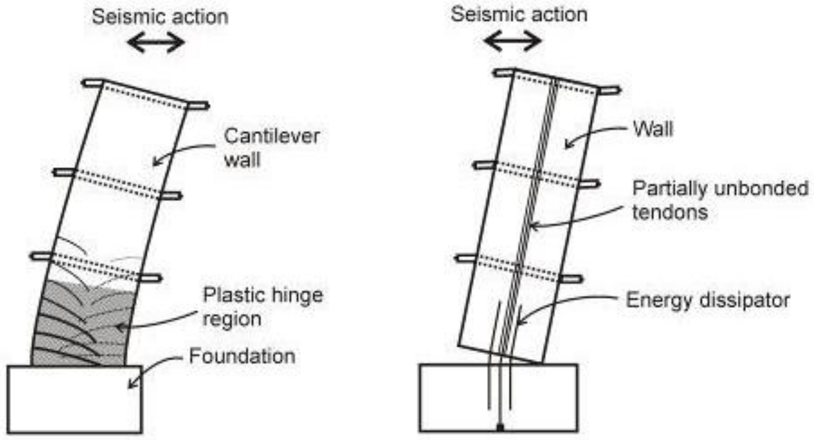


Figure 11: ***Comparative response of a traditional monolithic system (damage in the plastic hinge and residual deformations) and a jointed precast (hybrid) solution (rocking mechanism with negligible damage and negligible residual deformations, fib, 2003).***

Moreover, the tendons are unbonded so are able to elongate within the duct without yielding. They can thus act as re-centring "springs", guaranteeing that the structure comes back to its original at-rest position at the end of the shaking. As a result negligible residual or permanent deformations (offset or leaning of the building) would result, the repairing operations of which, as discussed can be more expensive and complicated than assumed in the design phase.

It is worth noting that residual deformations have been recently recognized as a fundamental and complementary damage indicator within a performance-based design or assessment procedure (Christopoulos *et al*., 2003, Pampanin *et al.* 2003, Garcia and Miranda 2006).

Post-tensioned rocking/dissipating wall system can take further advantage of coupling mechanisms between adjacent walls, using traditional coupling beams (either concrete or steel, possibly developing flexural-type yielding mechanisms instead of shear-type) and/or special dissipative elements/devices. e.g. U-shape Flexural Plates (Fig. 12, from the Five-Storey PRESSS Building) acting as dissipating rollers (Kelly *et al.,* 1982).

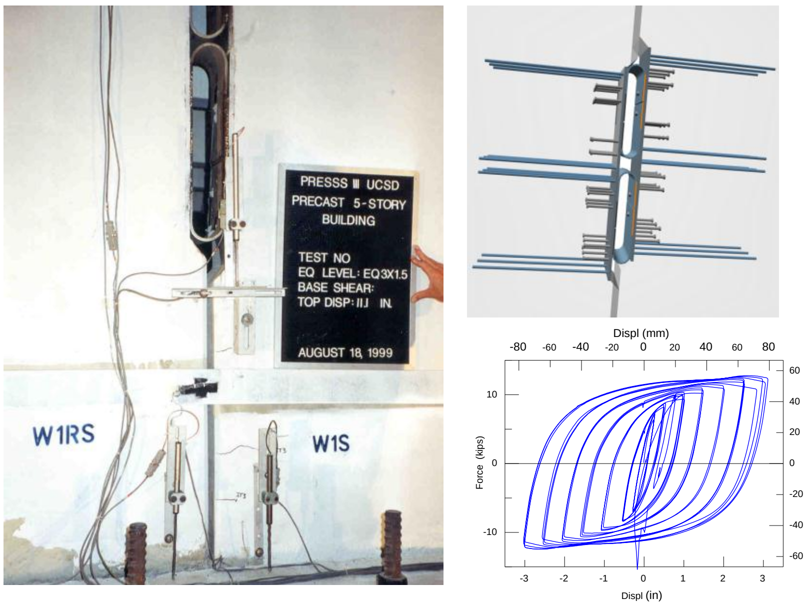


Figure 12: ***Behaviour of U-shape Flexural Plate Dissipaters in Post-tensioned coupled walls. (Priestley et al., 1999, UFP rendering courtesy of Nakaki and Stanton).***

The dissipative and re-centring mechanism of an hybrid systems 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-centring and the dissipation components (Fig. 13). A 50-50 flag shape (λ=1) would thus generate the maximum level of energy dissipation (typically in the order of ξ = 15-20% hysteretic damping) while maintaining fully re-centring capability.

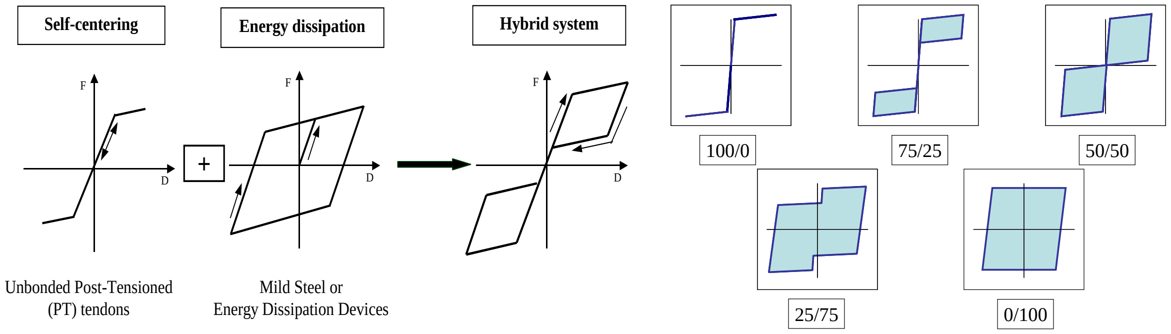


Figure 13: ***Flag-shape hysteresis loop for a hybrid system (modified after fib, 2003). Effects of varying the ratio between re-centring (nominator, post-tensioning and axial load) vs. dissipative (denominator, mild steel and dissipaters) contribution to the Flag-Shape Hysteresis loop (modified after Nakaki and Stanton 1999).***

3.2 Historical developments in earthquake engineering: understanding and implementing lessons from our ancient heritage

The conceptual innovation of "capacity design" introduced in the late 1960s- early 1970s is universally recognized as a major milestone in the development of earthquake engineering and of seismic design philosophies in particular. Similarly, the concept of jointed ductile connections able to accommodate high inelastic demand without suffering extensive material damage, developed in the 1990s, can be arguably anticipated to represent a critical milestone towards the development of the next generation of damage-resistant, high-performance systems, based on the use of conventional materials and techniques. Figure 14 provides an exemplification of the response of three beam-column joints representing different historical achievements: a) a pre-1970s or pre-capacity design era; b) current code (NZS3101:2006) with plastic hinge in the beam as per capacity design principles; c) a hybrid beam-column joint with rocking-dissipative mechanism (itself in accordance with the Appendix B of the NZS3101:2006). To confirm the simplicity of the technology both in terms of design and construction, it is worth noting that the shown specimen was entirely designed, constructed and tested by 3rd year engineering students at the University of Canterbury, as part of their first course in reinforced concrete.

Figure 14: ***Evolution of seismic resisting connections: performance of beam-column joints designed according to a) pre-1970 codes (shear damage in the joint or soft-storey mechanism); b) capacity design principles as per the NZS3101:1995 (beam plastic hinge) and c) hybrid jointed ductile connections as per Appendix B of NZS3101:2006 (controlled rocking).***

In a fascinating way, such a recent breakthrough represents a clear example of use of modern technology to further develop and refine very valuable solutions built on our ancient heritage. We could in fact clearly recognize the lessons and inspiration provided by the long-lasting earthquake resisting solutions used since the ancient Greek and Roman temples, consisting of segmental construction with marble blocks "rocking" on the top of each-other under the lateral sway. The weight of the blocks themselves and of the heavy roof-beams provided the required "clamping" and re-centring vertical force (Fig. 15). The shear in between elements was carried and transferred by shear keys, made of cast lead, preventing the occurrence of sliding but also probably acting as relocating pivot points.

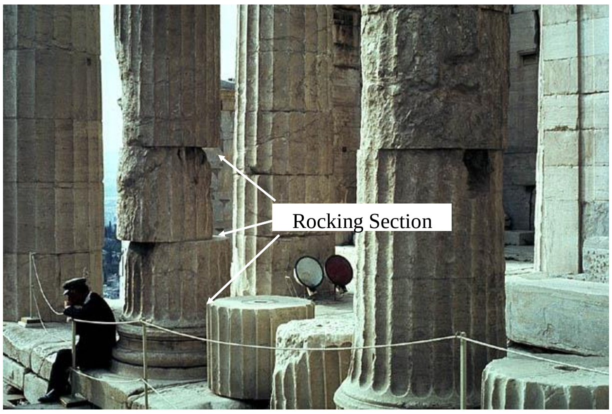


***Figure 15:** Examples of earlier implementation of rocking systems, self-centring and limited damage response under earthquake loading. Left: Dionysus temple in Athens, ancient agora, Right: Rocking segments of marble columns (Acropolis, Athens).*

3.3 Reparability of the weakest link of the chain: "Plug & Play" replaceable dissipaters

In principle, either internally (grouted) mild steel bars or, more recently developed, external & replaceable supplemental damping devices can be adopted (Figure 16). 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.

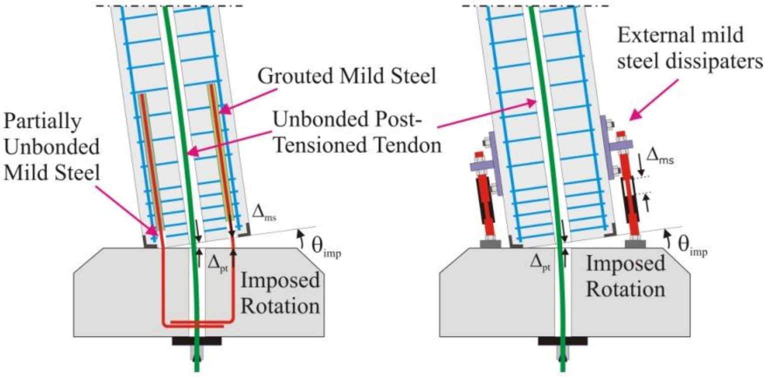


***Figure 16:** Internal versus external replaceable dissipaters/fuses at the base-column/pier connection (Marriott et al. 2008).*

A potential downside of such an approach is that following an earthquake the internal rebars would not be easily accessible nor replacable as per a typical monolithic solution (an insight of the Canterbury earthquake). Also the degradation of bond between concrete and steel during reversal cyclic loading causes some level of stiffness degradation, thus potentially causing a 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", 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 Plug & Play dissipater solutions, 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 (Figure 17).

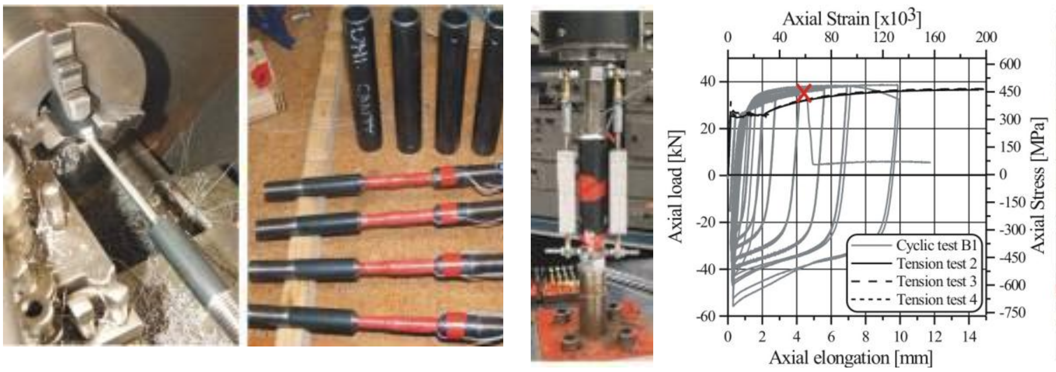


Figure 17: ***Manufacturing process and testing of the "Plug & Play" dissipaters (Marriott et al., 2008).***

The cyclic response of a typical dissipater is very stable and robust, allowing for many dissipative cycles prior to reaching failure, often due to low-cycle fatigue. It is worth noting that, as a further advantage of this type of external dissipater, very stable flag-shape hysteresis loops, with no stiffness degradation due to bond losses, can be obtained, when compared to internally grouted (bonded) mild steel bars.

A number of tests have been successfully carried out at the University of Canterbury in the past ten years on different subassembly configurations including beam-column joint connections, wall systems, column (or bridge pier)-to-foundation connections (Fig. 18) 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 with the direct repairing of the structural system and to the downtime (business interruption).

Interestingly and different from the traditional design approach for reinforced concrete structures, the new generation of reinforced concrete connections might thus have some critical connecting reinforcing bars placed outside the concrete elements, instead of inside.

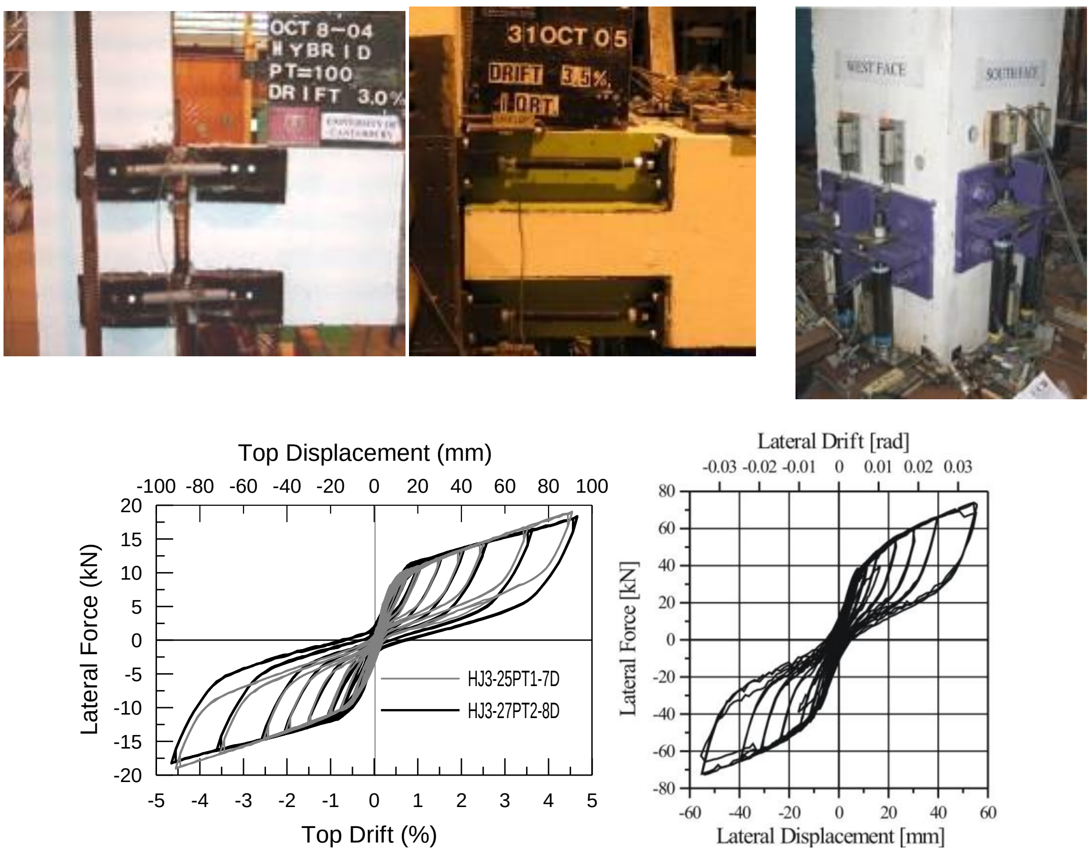


Figure 18: ***Alternative configurations of external replaceable dissipaters for hybrid systems: Top left and centre: beam-column connections, with and without recess in the beam (from Pampanin et al. 2006); Top right: Column to foundation connections (from Marriott et al., 2009); Bottom: typical flag-shape hysteresis loops for a hybrid beam-column joint and a column-to-foundation connection with external dissipaters.***

Hysteretic, Friction or Viscous Dampers?

In terms of material and type of dissipation, either metallic and/or other advanced materials (e.g. shape memory alloys, visco-elastic systems) can be used and implemented to provide alternative type of dissipation mechanisms (elasto-plastic due to axial or flexural yielding, friction, visco-elastic).

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 (e.g. Fig. 19) with the main source of re-centring capacity (given by unbonded post-tensioned tendons, mechanical springs or Shape Memory Alloys, SMA, with super-elastic behaviour).

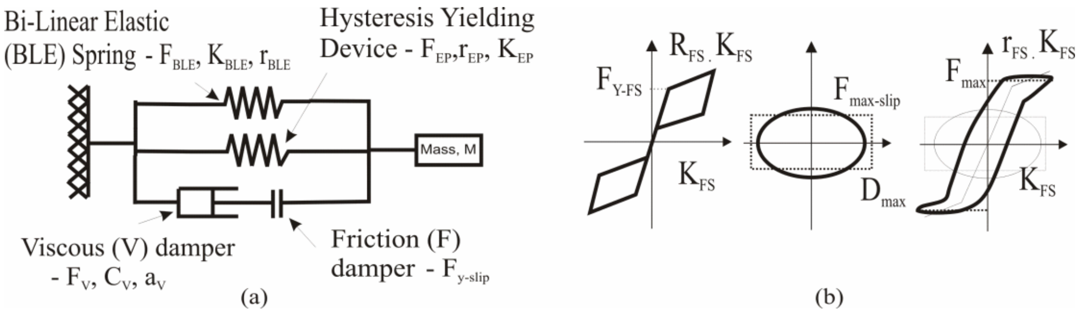


Figure 19: ***Example of Advanced-Flag-Shape System, combining the re-centring bi-linear elastic spring in parallel with "visco-elasto-plastic" dampers (viscous dampers in series with friction slip element) and hysteretic elasto-plastic spring. a) Schematic SDOF model; b) Idealized hysteretic model.***

In addition to the moment contribution ratio, λ, the designer can tune and control the damping contribution ratio, λ (i.e. ratio between the hysteretic moment and the viscous moment contribution, Fig. 20). 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, Fig. 20).

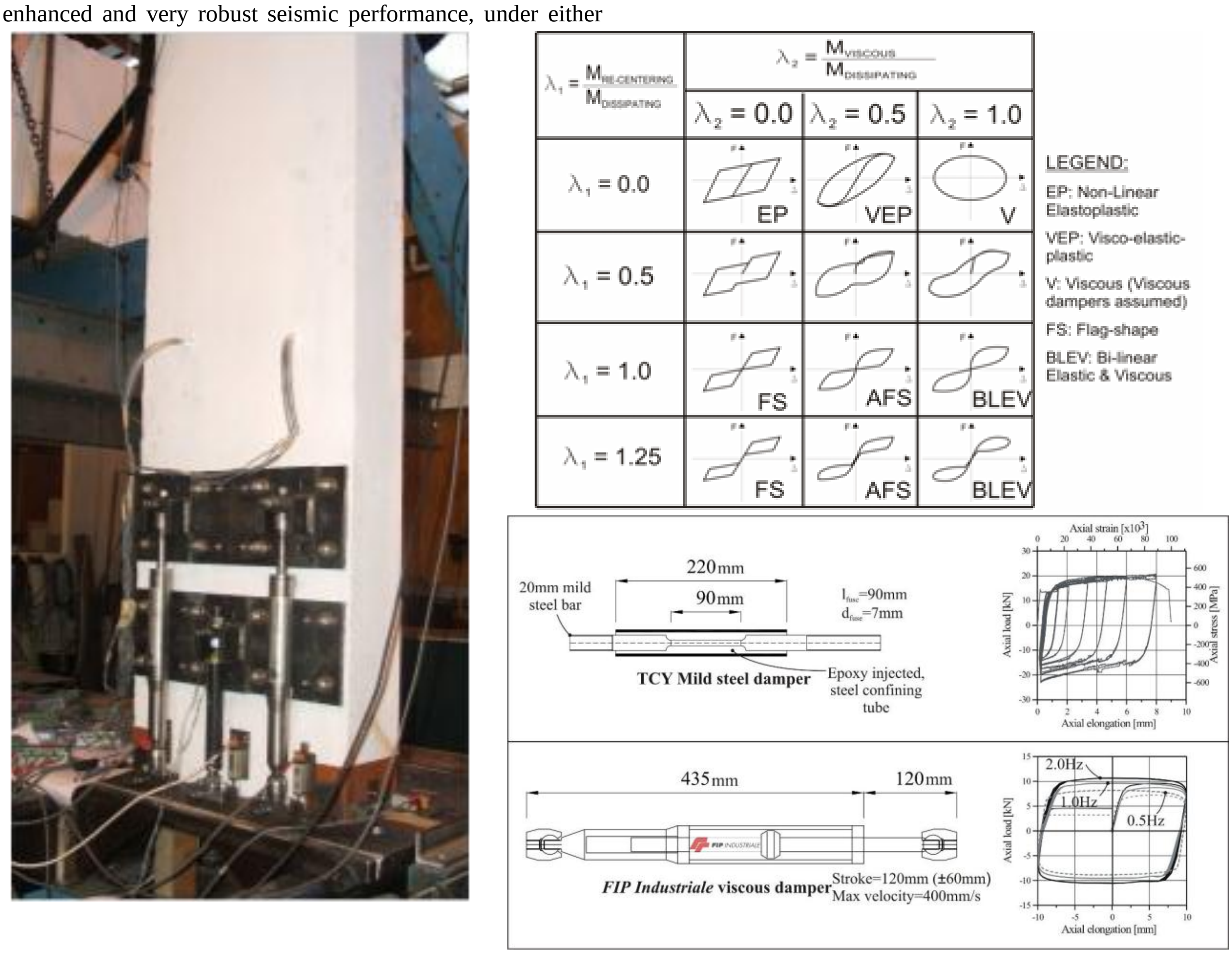


Figure 20: ***Concept, implementation and experimental validation (shake-table) of the concept of Advanced Flag-Shape applied to a post-tensioned wall (Kam et al., 2010; Marriott et al., 2008). Combination in parallel of hysteretic and viscous dampers.***

3.4 Controlling and reducing the damage to the floor

The peculiarity of a jointed ductile connection, consisting of an "articulated" assembly of precast elements can be further exploited and extended to the design of floor-to-lateral-load-resisting-system connections in order to minimize and control the damage to the diaphragms, as observed in recent earthquakes.

The latter topic has been receiving a growing attention in the engineering community in the last decade, following the several examples of poor performance of floor-diaphragms observed in recent earthquakes. Damage to the floor diaphragm can compromise the structural performance of the whole building. Experimental tests on 3-dimensional performance of precast super-assemblages including frames and hollow-core units (Matthews *et al*., 2003) have further underlined issues related to the inherent displacement incompatibility between precast floor and lateral resisting systems, including beam elongation effects (Fenwick and Megget, 1993; fib 2003, Fig. 21 left).

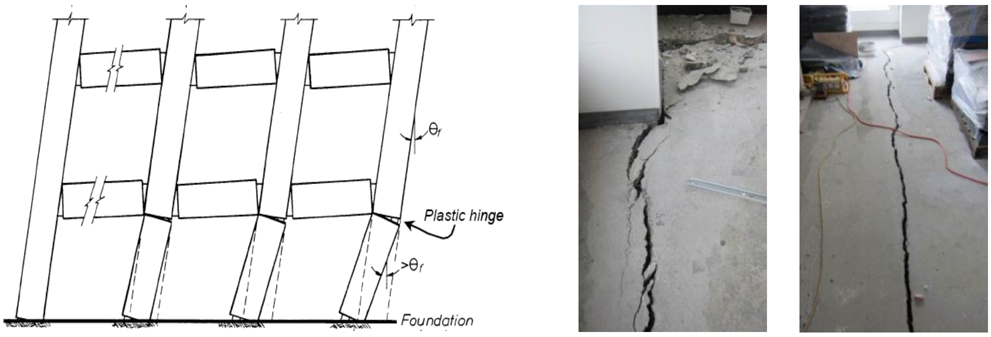


Figure 21. Beam elongation effects (after fib, 2003) and example of extensive cracking in the diaphragm topping of precast concrete floors within a multi-storey buildings following the 22 February 2011 Canterbury Earthquake.

Alternative innovative solutions have been recently developed and proposed in the literature to minimize the damage to the floor system, while guaranteeing a reliable diaphragm action, as described below.

3.4.1 Jointed "articulated" floor system

The first approach would consist of combining standard precast rocking/dissipative frame connections with an articulated or "jointed" floor system (Amaris *et al.*, 2007). According to this proposed solution, developed from the original concept of discrete X-plate mechanical connectors implemented in the Five-Storey PRESSS Building tested at UCSD (Priestley *et al.,* 1999), the floor (hollow-core in this case) unit is connected to the beams by mechanical connectors, acting as shear keys when the floor moves orthogonally to the beam and slides when the floor moves parallel to the beam (Fig. 22).

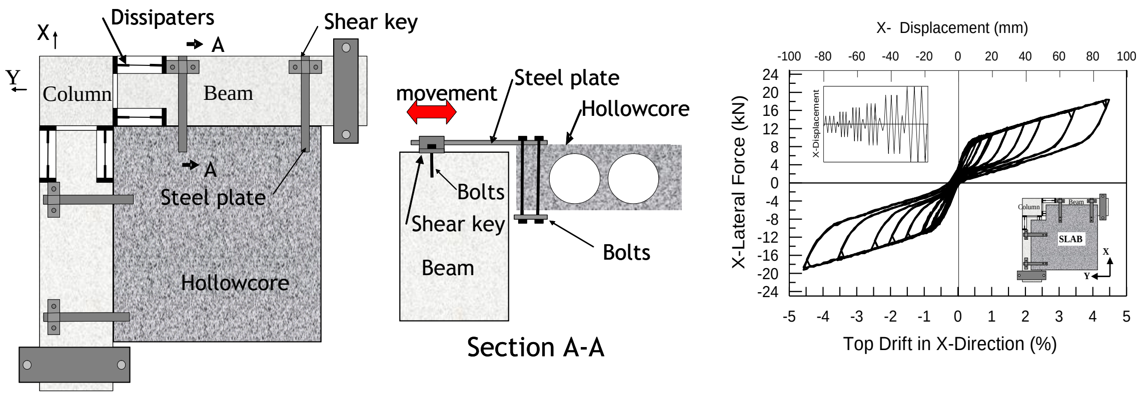


Figure 22: "Articulated floor" system. Concept, connection details and response under uni-directional and bi-directional cyclic tests (Amaris et al., 2007).

As a result, the system is able to accommodate the displacement compatibility demand between floor and frame by creating an articulated or jointed mechanism, which is effectively decoupled in the two directions. Also, due to the low flexural stiffness of the shear key-connectors in the out-of-plane directions, torsion of the beam elements due to pull out of the floor or relative rotation of the floor and the edge support, can be limited.

Note that a relatively simple design option which can reduce the extent of floor damage due to beam elongation is to use a combination of walls and frames to resist lateral loads, with walls in one direction and frames in the other. If the precast one-way floors run parallel to the walls and orthogonal to the frame, the elongation effects of the frame to the floor are reduced. This approach can be combined with partial de-bonding of the reinforcing bars (starters) in the concrete topping, and the use of a thin cast in-situ slab or timber infilled slab in the critical areas, to further increase the deformation compatibility.

3.4.2 Top Hinge "non-tearing floor" solution

An alternative method of preventing damage of floors due to beam elongation can rely upon a newly developed "top-hinge" or "top-hung" system in combination with a standard floor solution (i.e. topping and continuous starter bars). In its general concept, the top hinge allows the relative rotation between beams and column to occur and the bottom reinforcement to yield in tension and compression. The presence of a slot or gap on the bottom part of the beam will prevent direct contact to happen between the beams and columns, thus avoiding the elongation of the beam and the tearing of the floor. A debonded length is adopted in the bottom steel rebars to prevent premature buckling, as per a typical PRESSS jointed ductile connections.

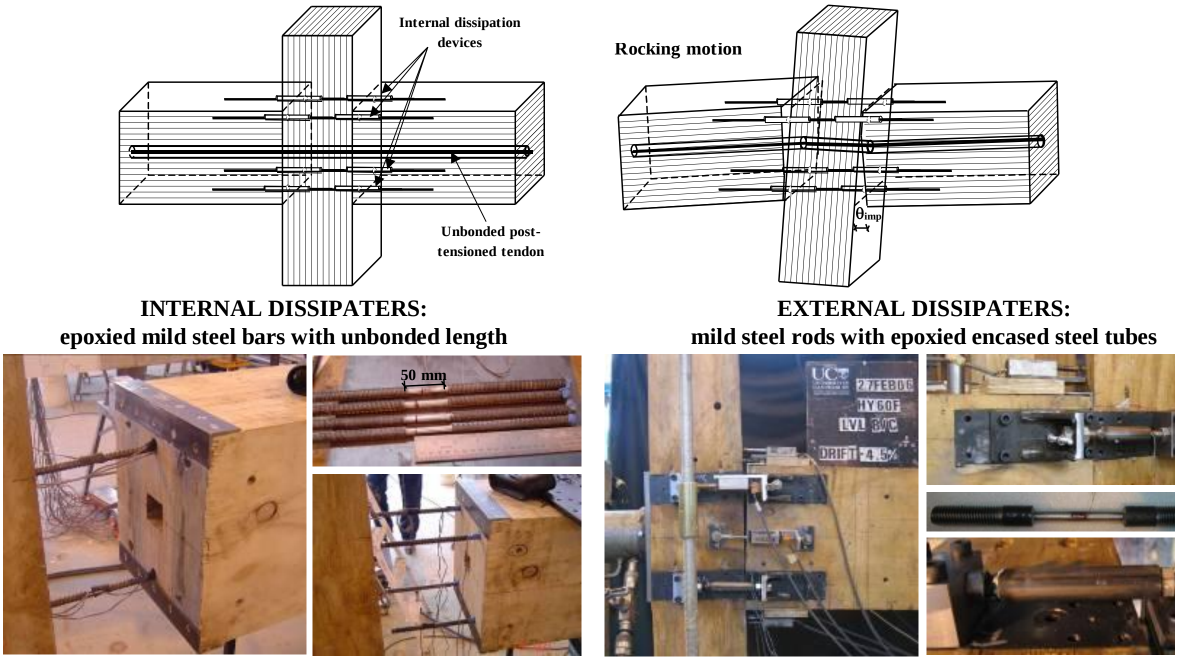
***Figure 23:** "Articulated floor" system. Concept, connection details and response under uni-directional and bi-directional cyclic tests (from Muir et al, 2012).*

The development of this concept originates from the evolution of the Tension-Compression Yield–Gap connection (TCY-Gap), developed during the PRESSS-Program, which used internally grouted mild-steel bars on the top, unbonded post-tensioned tendons at the bottom and a slot/gap at the interface between column and beam. Such a solution, would prevent beam elongation but not the tearing action in the floor due to the opening of the gap at the top of the beam. An intermediate improved version would consist of an "inverted" TCY-Gap solution based on a single top hinge with the gap and the grouted internal mild steel bars placed in the bottom part of the beam. This modification, as per the "slotted beam" connection proposed by Ohkubo and Hamamoto (2004), for cast in-situ frames (without post-tensioning), would succeed in preventing both elongation and tearing effects in the floor, but would not yet be capable of providing re-centring due to the location and straight profile of the tendons.

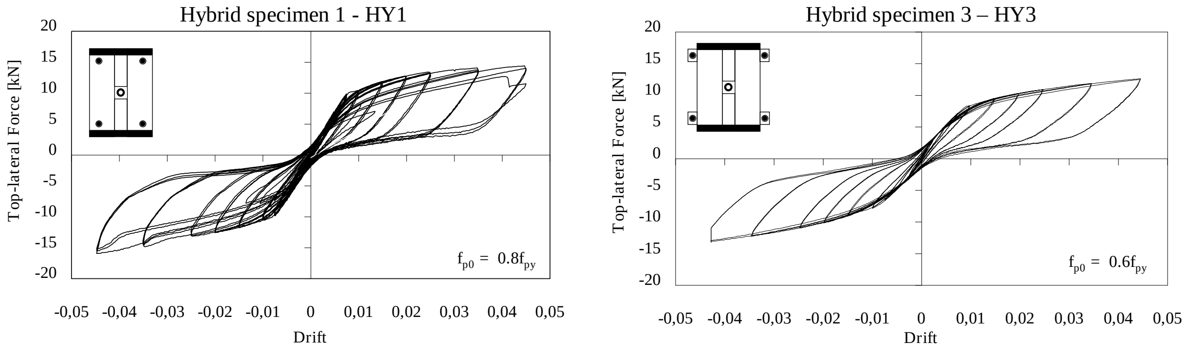
A further conceptual evolution and detail refinement have led to the development at the University of Canterbury of a "non-tearing floor" beam-column connection which could be combined with any traditional floor system (Amaris *et al.,* 2007, Eu *et al*., 2009, Muir *et al.,* 2012). Based on a series of experimental testing on interior and exterior beam column subassemblies, and on 2-D and 3D frame building specimens, a number of solutions have been developed, either with or without post-tensioning and ranging from partially to fully precast connections.

3.5 Extension to multi-storey timber buildings: the Pres-Lam system

The concept of post-tensioned hybrid (re-centring/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 (Fig. 24-26).



(a) Internal and external dissipaters and construction details.



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

Figure 24: ***Arrangements and testing results of Pres-Lam beam-column joints with internal or external reinforcement (Palermo et al., 2005, 2006).***

Figure 25: ***Testing of an hybrid post-tensioned column-to-foundation connections with replaceable dissipaters (observed performance at 4.5% drift) (Palermo et al., 2006).***

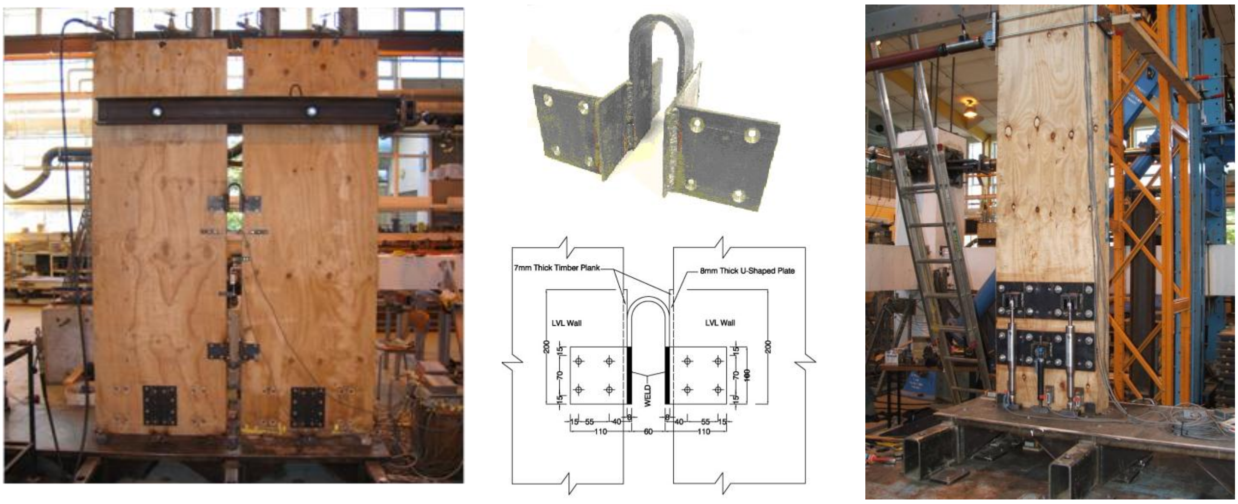


Figure 26: ***Left: Pres-Lam coupled walls with U-shape Flexural Plates dissipaters (Centre); Right: shake table test on Advanced-Flag-Shape Pres-Lam wall (viscous and hysteretic dampers in parallel) (Iqbal et al., 2007, Marriott et al., 2008).***

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 these, in addition to LVL.

The experimental testing provided very satisfactory results and confirmation of the high potential of this new construction system, 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).

4. ON-SITE IMPLEMENTATIONS OF PRESSS AND PRES-LAM TECHNOLOGY

The continuous and rapid development of jointed ductile connections using the PRESSS-technology for seismic resisting systems has resulted, in a little 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 the 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. 27), 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.

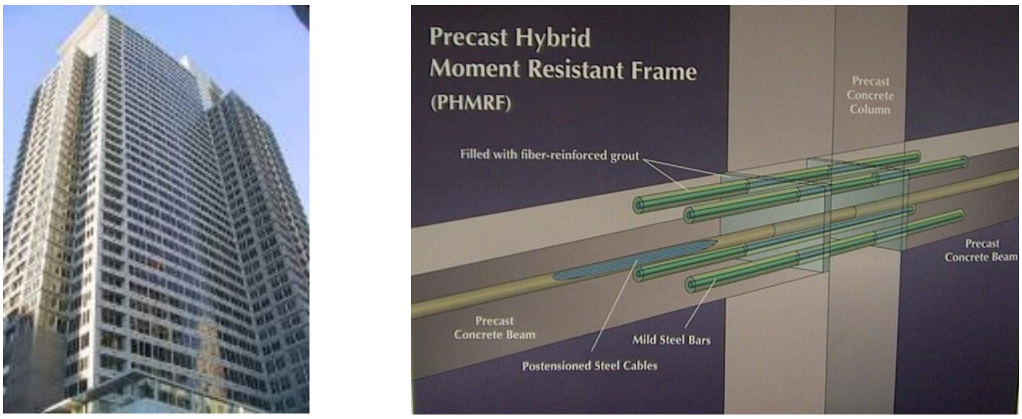


Figure 27: ***Paramount Building, 39-storey building, San Francisco (Englerkirk, 2002, photos courtesy of Pankow Builders, E. Miranda, Len McSaveney).***

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. 28), 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, a hospital), plan configurations, and floor spans. A brief description has been given in Pampanin *et al.* (2004).

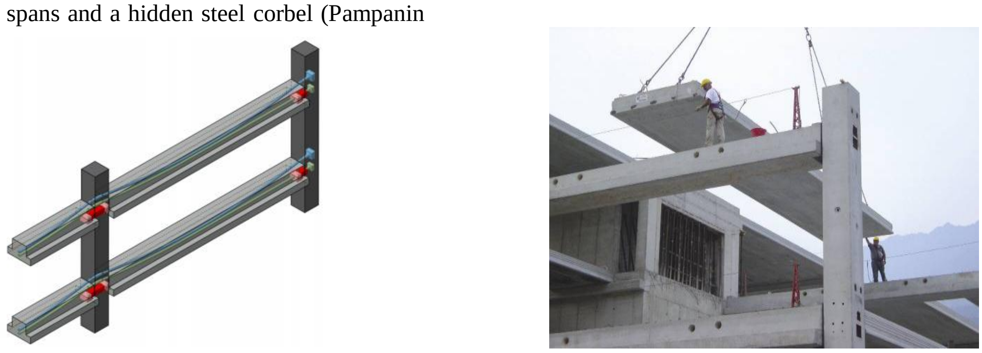


Figure 28: Application in Italy of the Brooklyn System, B.S. Italia, with draped tendons (Pampanin et al., 2004).

The first multi-storey PRESSS-building in New Zealand is the Alan MacDiarmid Building at Victoria University of Wellington (Fig. 29), 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. This building features some of the latest technical solutions previously described, such as the external replaceable 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.

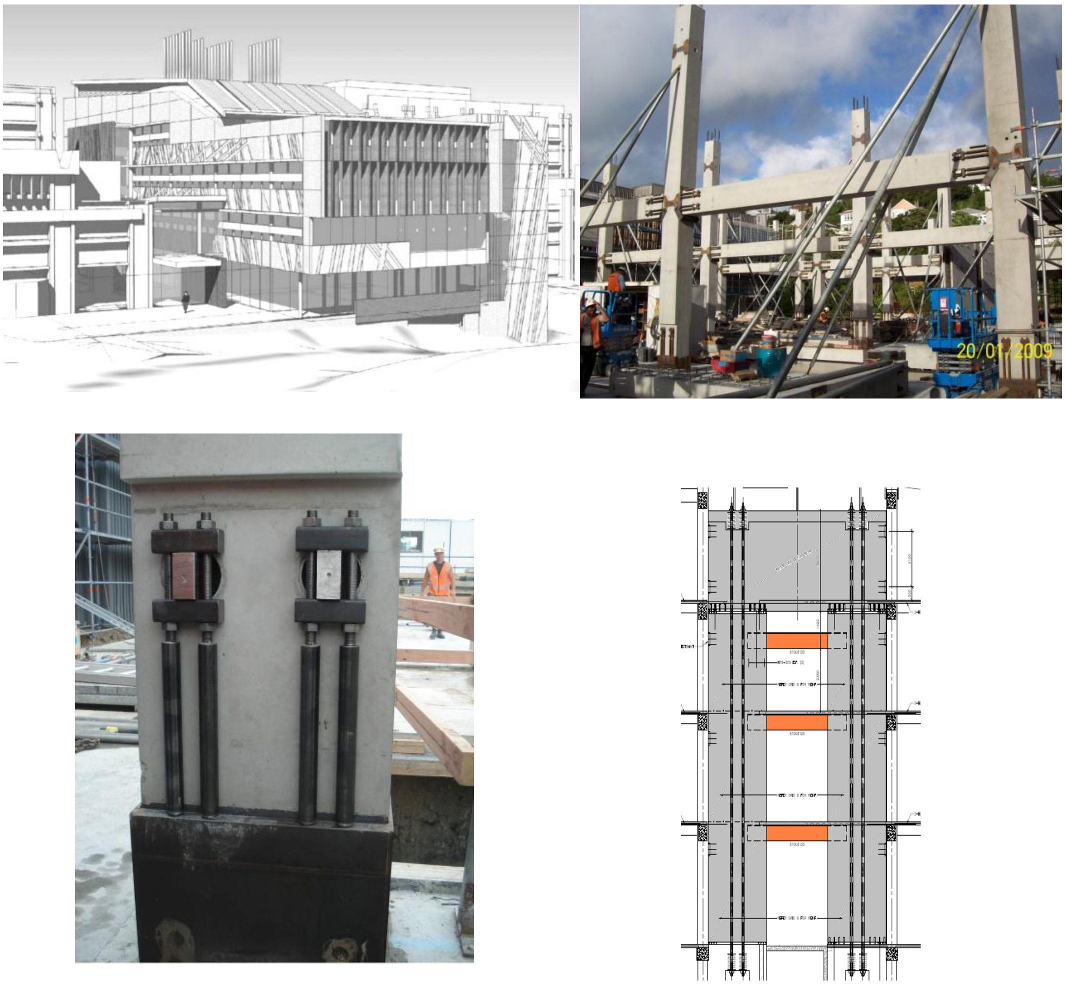


Figure 29: First multi-storey PRESSS-Building in New Zealand (Structural Engineers: Dunning Thornton Consultants; Cattanach and Pampanin, 2008).

The design and construction of the second PRESSS-Building in NZ and first in South Island has followed at close duration and is represented by the Endoscopy Consultants' Building in Christchurch, designed for Southern Cross Hospitals Ltd by Structex Metro Ltd (Fig. 30). Also in this case both frames and coupled walls have been used in the two orthogonal directions. The post-tensioned frame system relies upon non-symmetric 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.

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 now presented in this PRESSS Design Handbook (2010), in accordance to the NZS3101:2006 concrete design code Appendix B, including a) a Direct Displacement Based Design Methodology, b) the section analysis approach based on the Monolithic Beam Analogy procedure, c) a lumped plasticity model based on rotational springs in parallel and implemented in the time-history analyses software Ruaumoko (Carr, 2008).

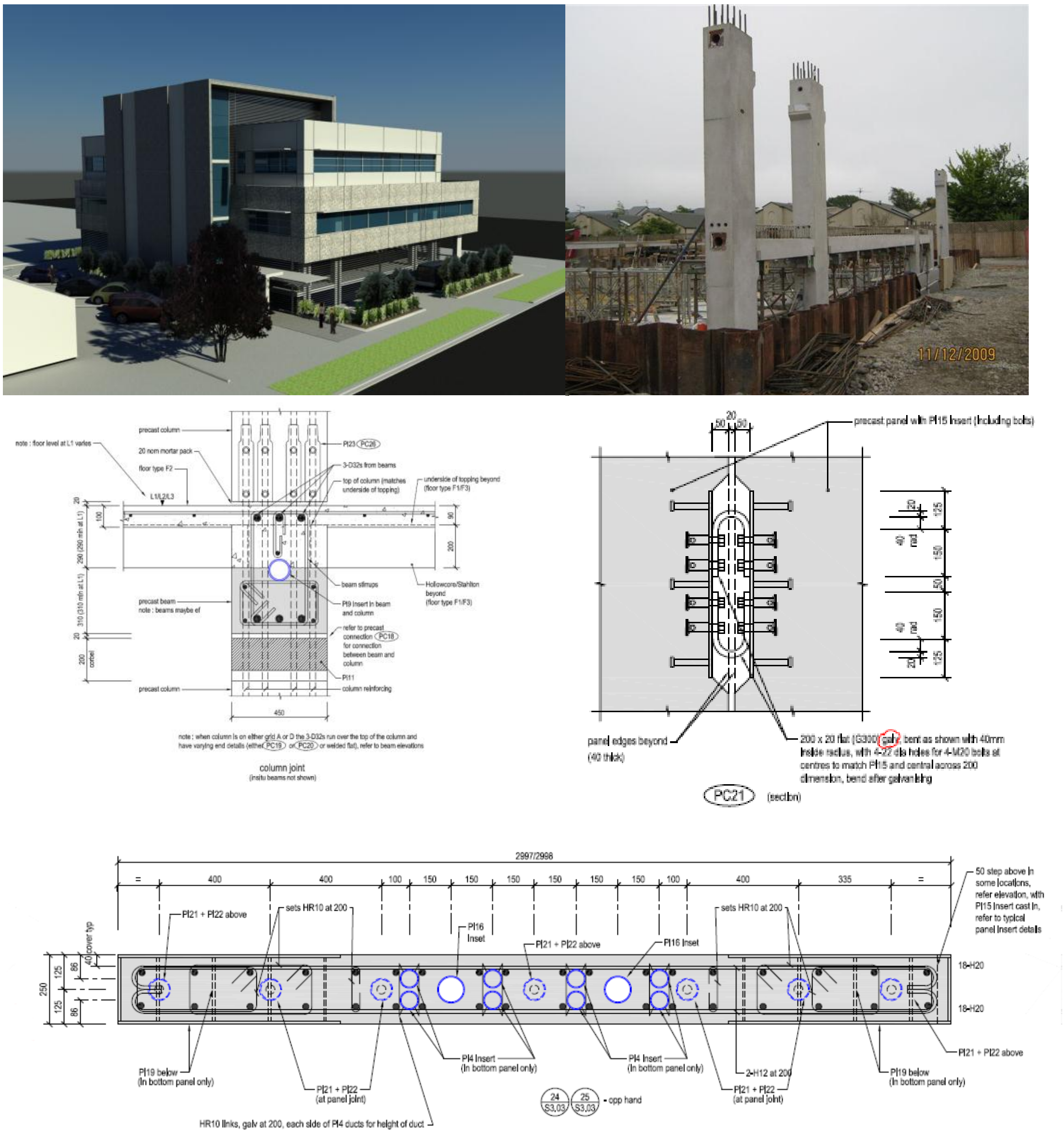


Figure 30: ***Southern Cross Hospital, Christchurch Rendering, construction of the frame, details of beams, walls and UFP dissipaters (Structural Engineers: Structex; Pampanin et al., 2011).***

4.1 Real earthquake testing: when reality meets expectations

The Southern Cross Hospital Endoscopy Building has very satisfactorily passed the very severe tests of the two recent Christchurch earthquakes. The 22 February earthquake was very close to the hospital with a very high level of shaking. Figure 31 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 features 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 contents 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 31: ***Negligible damage, to both structural and non-structural components, in the Southern Cross Hospital after the earthquake of 22 February.***

4.2 Implementation of Pres-Lam Buildings

Following the research described on post-tensioned timber (Pres-Lam) buildings at the University of Canterbury, the first world-wide applications of the technology are occurring in New Zealand. Several new post-tensioned timber buildings have been constructed incorporating Pres-lam technology.
The world's first commercial building using this technology is the NMIT building, constructed in Nelson. This building has vertically post-tensioned timber walls resisting all lateral loads as shown in Figure 32 (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 32: ***The world 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, located 100 km north of Wellington, is the second building in the world to adopt the Pres-Lam concept, as shown in Figure 33. 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 33).

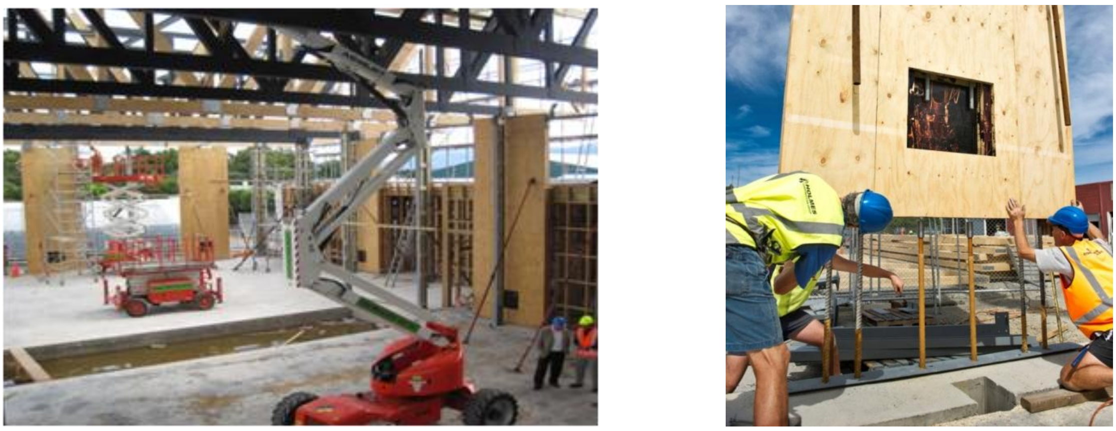


Figure 33: ***Carterton Events Centre. Single-storey building with LVL truss roof. (Designed by Opus International: Dekker et al. 2012).***

The University of Canterbury EXPAN building (Fig. 34) 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 are 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 34.


Figure 34: ***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. 35). 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 walls (PRESSS) in one direction and Pres-Lam timber frames in the other direction.



Figure 35: ***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 (Fig. 36), in some cases using mixed materials and/or a combination with base isolation and other supplemental damping devices.

Figure 36: ***Christchurch Rebuild: several Pres-Lam buildings in the final stage of their design or under current construction. Top left: Merritt Building, Structural Engineers: Kirk and Roberts; Architects: Sheppard and RoutTop; Right: Trimble, Architecture and Structures from Opus International (Brown et al., 2012); Bottom Left: St Elmo Courts a 1930 RC building demolished; Bottom-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.***

The increased awareness by the general public/tenants, building owners, territorial authorities as well as insurers/reinsurers, of the severe economical impacts in terms of damage/dollars/downtime of moderate-strong earthquakes is indeed facilitating the wider acceptance and implementation of cost-efficient damage-control technologies in New Zealand.

From an earthquake engineering community prospective, the challenge is still significant:

- on one hand, maintaining and supporting this (locally and temporary) renewed appetite for seismic protection for both new buildings and existing ones (retrofit);
- on the other hand, pushing towards a wider international dissemination and acceptance of damage-resisting technologies according to current best know-how and practice.

In a way, the target goal has not changed but the expectations (the bar) are higher with a shorter time frame: to develop, at comparable cost, an ultimate earthquake resisting building system (including both the structural skeleton and non-structural components/contents) capable of sustaining the shaking of a severe earthquake basically unscathed.

ACKNOWLEDGEMENTS

The research, development and implementation of damage-control solutions described in this paper are the results of the exceptional support and collaborative effort between a number of individuals and organizations from academia, the wider industry, governmental and funding agencies, at national and international level, a list of whom would be practically impossible to prepare. The author wishes to acknowledge and sincerely thank all those involved in this extended "research team".

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