Chapter 13
Towards the "Ultimate Earthquake-Proof" Building: Development of an Integrated Low-Damage System

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Abstract The 2010–2011 Canterbury earthquake sequence has highlighted the severe mismatch between societal expectations over the reality of seismic performance of modern buildings. A paradigm shift in performance-based design criteria and objectives towards damage-control or low-damage design philosophy and technologies is urgently required. The increased awareness by the general public, tenants, building owners, territorial authorities as well as (re)insurers, of the severe socio-economic impacts of moderate-strong earthquakes in terms of damage/dollars/downtime, has indeed stimulated and facilitated the wider acceptance and implementation of cost-efficient damage-control (or low-damage) technologies.

The 'bar' has been raised significantly with the request to fast-track the development of what the wider general public would hope, and somehow expect, to live in, i.e. an "earthquake-proof" building system, capable of sustaining the shaking of a severe earthquake basically unscathed.

The paper provides an overview of recent advances through extensive research, carried out at the University of Canterbury in the past decade towards the development of a low-damage building system as a whole, within an integrated performance-based framework, including the skeleton of the superstructure, the non-structural components and the interaction with the soil/foundation system.

Examples of real on site-applications of such technology in New Zealand, using concrete, timber (engineered wood), steel or a combination of these materials, and featuring some of the latest innovative technical solutions developed in the laboratory are presented as examples of successful transfer of performance-based seismic design approach and advanced technology from theory to practice.

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13.1 Introduction

The Canterbury earthquakes sequence in 2010–2011 has represented a tough reality check for the international community of seismic engineering, highlighting the severe 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 they were subjected to. As per capacity design principles, plastic hinges formed in discrete predetermined regions, e.g. beam-to-column interface, column-to-foundation and wall-to foundation connections, 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 leading to the controlled demolition of large portion of the Central Building District of the second largest city in New Zealand and to an economic impact evaluated in the range of 40 Billion NZ$, corresponding to approximately 20 % of the GDP (Gross Domestic Product).

Targeting life-safety is arguably not enough for our modern society, at least when dealing with new building construction. A paradigm shift in performance-based design criteria and objective towards damage-control design philosophy and technologies is clearly and urgently required.

In general, the next steps in performance-based seismic design should more explicitly focus towards the development of an integrated approach, involving all aspects of design framework, design procedures and tools and technological solutions for engineers and stakeholders to control the performance/damage of the building system as a whole, thus including superstructure, non-structural elements and soil/foundation system.

In the aftermath of the Canterbury Earthquake sequence, the increased public awareness of seismic risk and better understanding on the concept of building performance, has resulted into a renewed appetite for cost-efficient technological solutions to meet the higher public expectations, i.e. sustaining low-level of damage and thus limited business interruption after a design level earthquake.

In additional to more "traditional" damage-control technology as base isolation and supplemental dissipative braces, which are experiencing a resurgence in New Zealand, particular interest is being received by alternative and more recently developed "low-damage" systems, based on post-tensioned rocking mechanisms, combining self-centering and dissipating capabilities, for either concrete, timber and steel.

In such a context, the first and core part of the paper will provide an overview of recent advances and on-going research carried out at the University of Canterbury in the past decade towards the development of a low-damage building system as a whole, within an integrated performance-based framework, including the skeleton of the superstructure, the non-structural components and the interaction with the soil/foundation system.

In the second and conclusive part, examples of real on site-applications of such technology in New Zealand, using concrete, timber (engineered wood), steel or a combination of these materials, and featuring some of the latest innovative technical solutions developed in the laboratory, are presented, as examples of successful transfer of performance-based seismic design approach and advanced technology from theory to practice.

13.2   The Canterbury Earthquake Sequence: A Reality Check for Current Performance-Based Earthquake Engineering

The Mw 6.3 Christchurch (Lyttelton) earthquake occurred at 12.51 pm on Tuesday 22nd Feb 2011, approximately 5 months after the Mw 7.1 Darfield (Canterbury) main shock. Due to the proximity of the epicenter to the Central Building District, CBD, (10 km south-east), its shallow depth (5 km) and peculiar directionality effects (steep slope angle of the fault rupture), significant shaking was experienced in the city centre (Fig. 13.1), the eastern suburbs, Lyttleton-Sumner-Porter Hills areas.

The aftermath counted 185 fatalities, the collapse of several unreinforced masonry buildings and of two reinforced concrete (RC) buildings, extensive damage deemed beyond reparability to several RC buildings, damage to tenths of thousands of (mostly timber) houses. Unprecedented liquefaction effects occurred in whole parts of the city, compromising housing and building foundations as well as causing severe damage and impact on the main infrastructures and lifelines systems of the city including road, water and wastewater networks, and the electricity transmission systems (though quickly restored within 2 weeks). The estimated total losses were in the range of NZ$ 40 Billion, corresponding to approximately 20 % of the GDP (Gross Domestic Product).

For a more comprehensive information on the overall earthquake impact, the reader is referred to Special Issues dedicated to the Canterbury Earthquake sequence (NZSEE 2010, 2011) and (EERI/NZSEE 2014).

Considering the high level of shaking, as indicated by the acceleration and displacement response spectra of the ground motions recorded in the CBD, shown in Fig. 13.2, the overall behaviour of modern reinforced concrete structures (dominant type of multi-storey building in the CBD) can be classified, in general terms and with some exceptions, as quite satisfactory.

However, the extent of structural damage (Fig. 13.3) was deemed in most cases beyond reparability level, for either technical and/or economical considerations, highlighting the whole controversy of traditional design philosophies, mainly focused on collapse-prevention and life-safety and not yet embracing a damage-control objective.

Fig. 13.1 Skyline of Christchurch CBD before (*Top*) and just after (*bottom*) the 22 Feb 2011 earthquake (Photo taken by Gilly Needham)
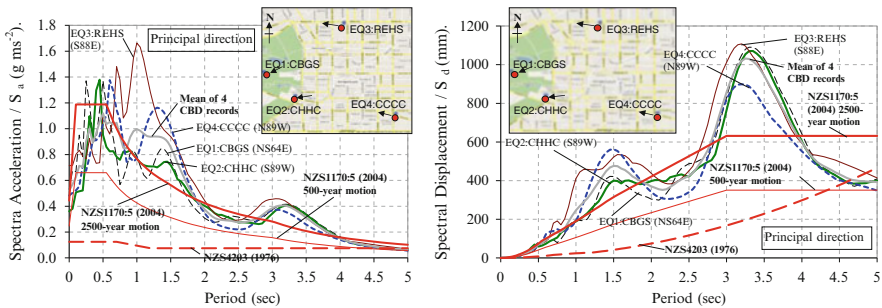

Fig. 13.2 Acceleration and Displacement response spectra from 4 records in the CBD of the 22nd Feb 2011 event, compared with the code design spectra (NZS1170:5 2004) *thick red line =* 1/2,500 years event (MCE); *red line =* 1/500 years event (DBE) (Kam and Pampanin 2011)

As a result, most of relatively modern buildings (mid-1980s and onwards) were demolished. The surprisingly high demolition rate (70 % in the CBD, Fig. 13.4) has been also arguably facilitated by the significant level of insurance coverage for partial or full replacement. In either cases, either demolition or repairing, the level of business interruption and downtime, was very severe and significantly beyond anticipations, also due to the long closure of a widely affected area in the CBD.

Fig. 13.3 Example of damage to RC frames and walls (all these buildings have been demolished) (From Kam et al. 2011; Pampanin 2012)

Fig. 13.4 *Left*: distribution of buildings tagging statistics in the CDB (updated to 12 June 2011, Kam et al. 2011); *Centre*: Aerial view of CBD with entire lots demolished and "cleaned up" (Photo courtesy of Kam Weng and Umut Akguzel); *Right*: CERA Blueprint

A Christchurch Central Recovery Plan (CCRP) has been developed by the Canterbury Earthquake Recovery Authority (CERA)'s Christchurch Central Development Unit (CCDU), outlining the future development of central Christchurch. The Plan incorporated a spatial Blueprint Plan (Fig. 13.4 right), developed by a professional consortium working with CERA/CCDU over a 100 days period and released to the public on 30 July 2012. The Blueprint provides a special framework for the development of the central city, including the locations of 'anchor' projects which are expected to stimulate further development.

13.3   Raising the Bar to Meet Societal Expectation: From Life-Safety to Damage Control and Holistic Approach

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

On one hand, a better communication between technical and non-technical communities could help clarifying and disclosing to the wider public what are the accepted/targeted performance levels built in a design code, itself to be considered a 'minimum' (not a maximum) standard. On the other hand, the earthquake engineering community is challenged with the complex task to "raise the bar", by shifting the targeted performance goals from the typically accepted Life-Safety level (for a design level earthquake or 1/500 years event for an ordinary structure), to a more appropriate and needed Damage-Control level (see performance matrix in Fig. 13.5), all this without increasing (too significantly) the cost of constructions. These increased expectations would require a significant paradigm shift in terms of performance-based design, which can be accomplished by the development and/or further refinement of design methodologies as well as of high seismic-performance, whilst cost-effective, technologies.

More importantly, the next steps in performance-based seismic design should more explicitly focus towards the development of an integrated approach,
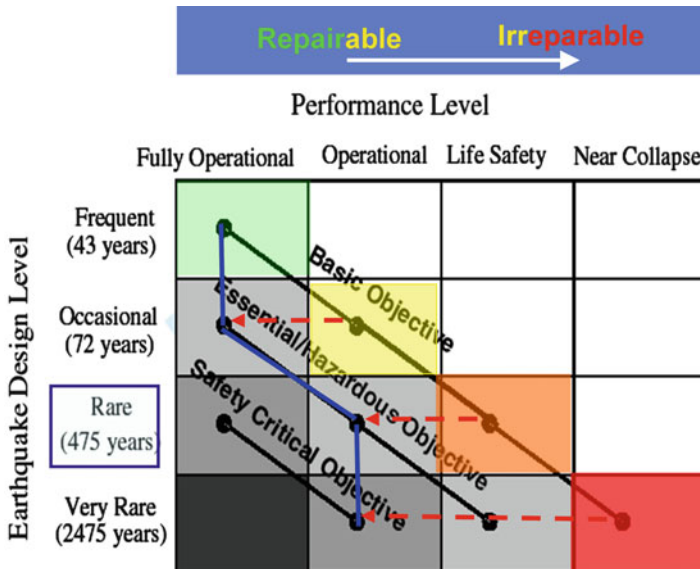

Fig. 13.5 Seismic Performance Design Objective Matrix as defined by SEAOC Vision 2000 PBSE Guidelines, herein rearranged to match building tagging, and proposed/required modification of the Basic-Objective curve towards a damage-control approach (*blue line*, Modified after Pampanin (2010), Kam et al. (2011))
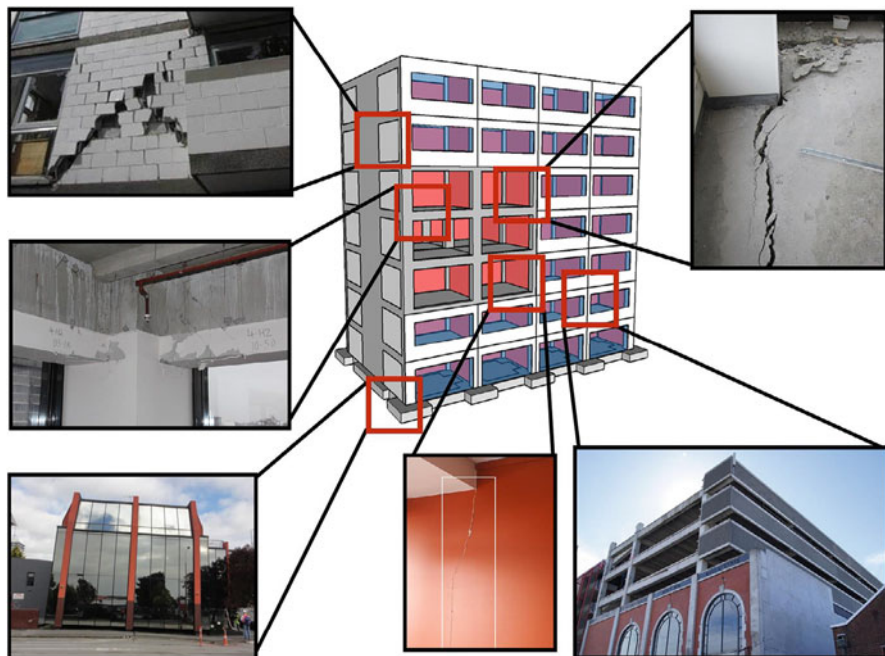
Fig. 13.6 Holistic representation of damage/performance to a modern building, including structural skeleton (frame system, floor diaphragm), non-structural components (lightweight partitions, heavy brick infills and precast concrete facades) and foundation system (significant settlements and residual tilting) (Modified after Johnston et al. (2014))

involving, in a holistic view, all aspects of the design framework, design procedures and tools and technological solutions for engineers and stakeholders to control the performance/damage of the building system as a whole, thus including superstructure, non-structural elements and soil/foundation system (Fig. 13.6).

13.4 The Next Generation of Low-Damage Seismic Resisting Systems

In addition to, or better complementary and integrative of, more "traditional" damage-control technology such as base isolation and dissipative braces, which are experiencing a resurgence in New Zealand after the Canterbury earthquake sequence, particular interest is being received by alternative and more recently developed "low-damage" systems, based on post-tensioned rocking & dissipative mechanisms for either concrete, timber and steel structures.

Such technology, also broadly referred to as PRESSS-technology from its original developments in the 1990s for precast concrete construction under the US PRESSS
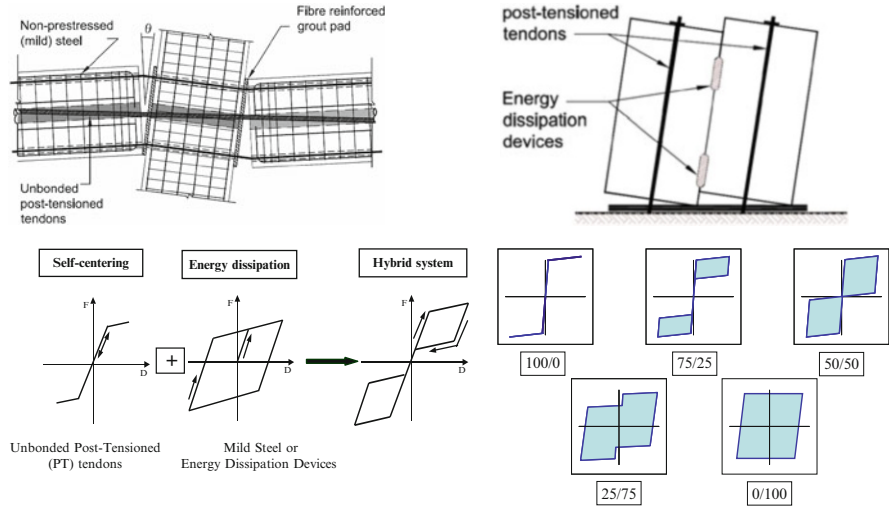
Fig. 13.7 *Top*: Jointed precast "hybrid" frame and wall connections developed in the US PRESSS-Program (*fib* 2003; NZS 3101:2006, NZCS 2010. *Bottom*: flag-shape hysteresis loop for a hybrid system (modified after *fib* (2003)) and effects of varying the ratio between re-centering vs. dissipative contribution (courtesy of Nakaki and Stanton)

Program (Priestley 1991; Priestley et al. 1999), relies upon the use of jointed ductile connections, where structural elements are jointed together through unbonded post-tensioning tendons/strands or bars creating moment-resisting connections. Additional damping and moment contribution can be provided by mild steel rebars either internally located (first generation) or by alternative dissipaters externally located and repleacable (recently developed). The combination of unbonded post-tensioning and additional dissipaters, lead to a so-called hybrid system (Priestley 1996; Stanton et al. 1997). The recentering and dissipative mechanism of a hybrid system, also referred to as controlled rocking, is described by a peculiar "flag-shape" hysteresis behaviour (Fig. 13.6, bottom), whose properties and shape can be modified by the designer by varying the ration between the re-centering and dissipative (moment) contributions, provided by the post-tensioned tendons/bars (and/or axial load) and mild steel/dissipaters, respectively (Fig. 13.7).

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.

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
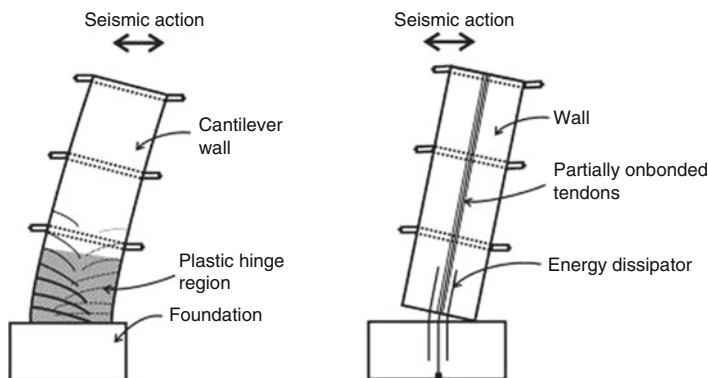
Fig. 13.8 Comparative response of a traditional monolithic system (damage in the plastic hinge and residual deformations) and a jointed precast (hybrid) solution (rocking mechanism with negligible damage and negligible residual deformations *fib* 2003)

accepted to occur in the plastic hinge regions, leading to substantial costs of repairing and business interruption.

The plastic hinge, or sacrificial damage-mechanics, is thus substituted by this "controlled rocking" mechanism (dissipative and re-centering) at the critical interface with no or negligible damage (Figs. 13.8 and 13.18).

13.5 Reparability of the Weakest Link of the Chain: "Plug&Play" Replaceable Dissipaters

In the last decade, extensive research and developments have been carried out at the University of Canterbury in New Zealand on low-damage PRESSS-technology for both concrete and timber structures (buildings and bridges), resulting into the development of a wide range of improvements and new features.

As part of the overall scope, significant effort has been dedicated towards the development of cost-efficient external and replaceable dissipaters, which after an earthquake event could be easily accessed, inspected and, if needed, replaced (Pampanin 2005; Marriott et al. 2008, 2009; NZCS 2010; Sarti et al. 2013). These dissipaters, referred to as "Plug&Play" and consisting for example 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, have been developed and extensively tested within several subassemblies configurations, i.e. beam-column joint connections, wall systems, column (or bridge pier)-to-foundation connections (Fig. 13.9).

This option gives 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, with the additional feature of being
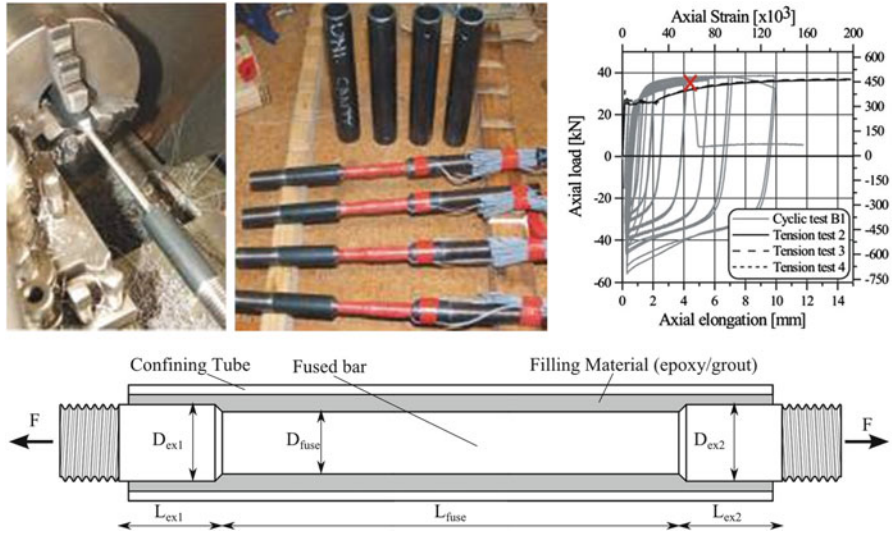
Fig. 13.9 Fused Type "Plug&Play" dissipaters: Manufacturing process and testing (Marriott et al. 2008, 2009); schematic of geometry and element composition (Sarti et al. 2013)

repairable. The traditional assumption "ductility equal to damage" (and consequent repair costs and business downtime) is thus not anymore a necessary compromise of a ductile design (Fig. 13.10).

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). Examples of application of friction and viscous devices in unbonded post-tensioned systems have been given in Kurama (2001) and Kurama and Shen (2004).

A second generation of self-centering/dissipative high-performance systems, referred to as advanced flag-shape systems (AFS) has been proposed, tested and implemented in real practice (Kam et al. 2006; Marriott et al. 2008; Latham et al. 2013). 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 (unbonded post-tensioned tendons, mechanical springs or Shape Memory Alloys (SMA) with super-elastic behaviour). As a result, an enhanced and very robust seismic performance, under either far field and near field events (high velocity pulse) can be achieved, as proven by numerical investigations (Kam et al. 2006) and shake table testing (Fig. 13.11) (Marriott et al. 2008).
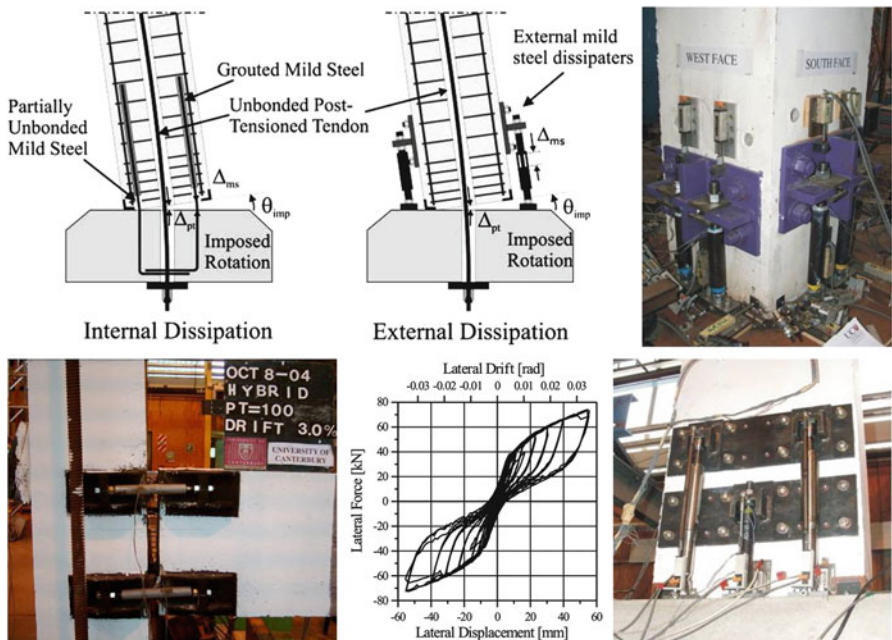
Fig. 13.10 *Top*: Internal vs. external replaceable dissipaters/fuses in a column/pier. *Bottom*: Alternative configuration of Plug&Play dissipaters for bc joints or walls (Marriott et al. 2008, 2009, 2010)

13.6 Low-Damage Solution for Multi-storey Timber Buildings: the Pres-Lam System

The concept of post-tensioned hybrid (recentering&dissipating) system has been in the past decade successfully extended from precast concrete to timber (engineered wood) frames and walls (Palermo et al. 2005; Pampanin et al. 2006b). Since 2004, a series of experimental tests, including quasi-static cyclic, pseudodynamic and shake-table, have been carried out on several subassemblies or larger scale structural systems at the University of Canterbury to develop different arrangements of connections for unbonded post-tensioned timber frame and walls (Fig. 13.12).

Due to its high homogeneity and good mechanical properties, laminated veneer lumber (LVL) was initially selected as the preferred engineered wood material for the first phase of the research and development. However, any other engineered wood product as Glulam or Cross-lam (X-lam) can be adopted as shown by recent experimental tests and numerical analyses on both materials (Smith et al. 2014; Dunbar et al. 2014).

The extensive experimental and numerical campaign has provided very satisfactory results and confirmation of the high potential of this new construction
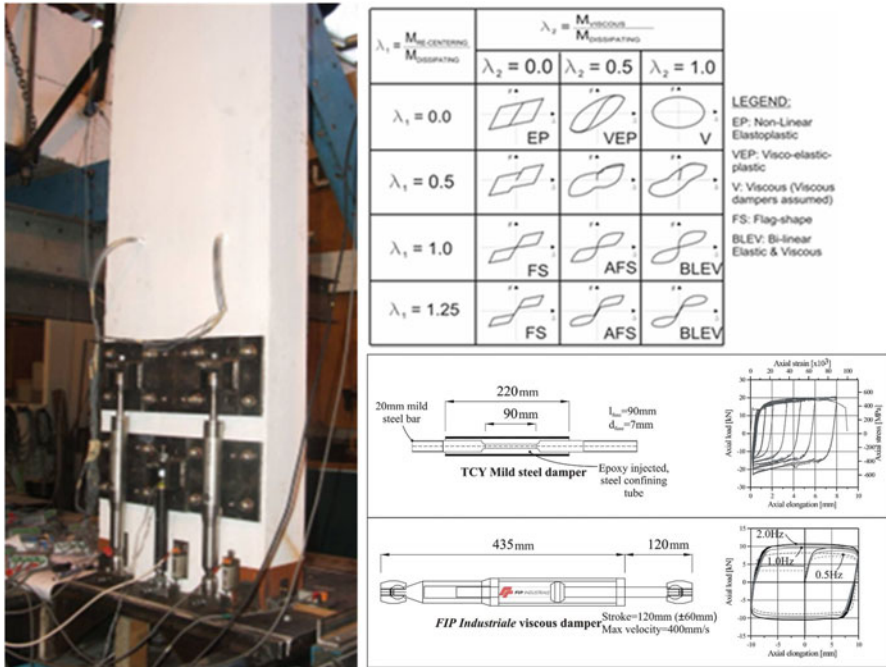
Fig. 13.11 Concept, implementation and experimental validation (*shake-table*) of the concept of Advanced Flag-Shape applied to a post-tensioned wall (Kam et al. 2010; Marriott et al. 2008, 2009). Combination in parallel of hysteretic and viscous dampers

system, referred to as a Pres-Lam system (acronym for Prestressed Laminated timber). The extension of low-damage systems to engineered wood solutions opens new opportunities for much greater use of timber and engineered wood products in multi-storey and 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. 2011).

Examples of on-site applications of structural frames, walls, combination of them and hybrid material construction will be given in the later part of this paper.

13.7 Controlling and Reducing the Damage to the Floor-Diaphragm

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 ad control the damage to the diaphragms, as observed in recent earthquakes.
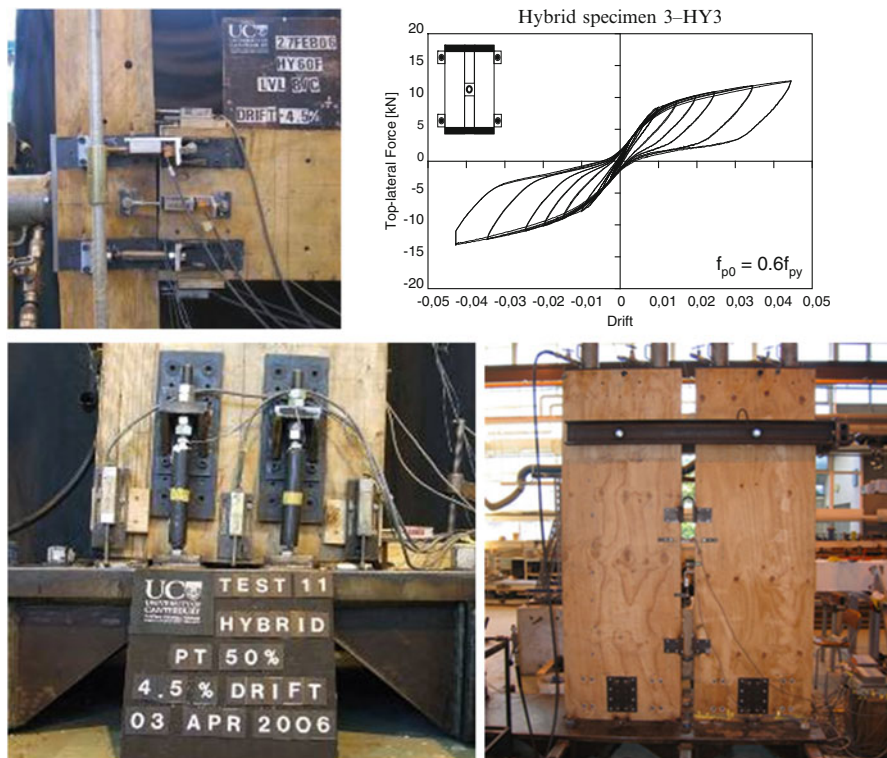
Fig. 13.12 Testing of hybrid post-tensioned timber (Pres-Lam) beam-column joints, column-to-foundation connections and coupled walls with replaceable dissipaters (Palermo et al. 2005, 2006)

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-diaphragm observed in recent earthquakes, including the Canterbury earthquake sequence (Fig. 13.14 right). Damage to the floor diaphragm can compromise the structural performance of the whole building when not leading to collapse of entire floors.

During the seismic response of a building, significant displacement incompatibilities issues can arise between the main lateral resisting systems (frames and walls) and the floor-diaphragm. In general terms they can be classified into vertical incompatibility (primarily associated to the wall response and uplifting, but also incurred into frames) and horizontal incompatibility (more typical of frame system subject to beam elongation effects, Fenwick and Megget 1993).

In the case of walls, regardless of them being based on a rocking mechanism or on a monolithic plastic hinge behaviour, the development of inelastic action at the base (in the form of a concentrated or distributed plastic hinge) result into a geometrical uplifting of the wall. If the axial load (or additional post-tensioning) acting on the wall is not sufficient to re-center the system, at each subsequent cycle
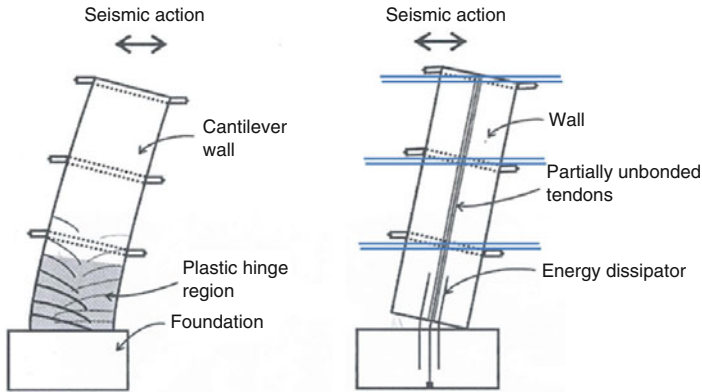
Fig. 13.13 Vertical displacement incompatibility between a ductile shear wall (uplifting) and the floor system. *Right*: slotted shear key solution to accommodate the relative movement (after *fib* 2003)

with larger ductility demand, the wall would tend to vertically elongate (beam elongation effects in the vertical directions). The resulting interaction with the floor-diaphragm can lead either to significant deformation and damage to the floor system itself (see Fig. 13.13b) and/or to a unexpected brittle mechanism in the walls due to the significant increased level of axial and shear forces acting in the wall (see Fig. 13.3). A conceptual solution to limit this effect is to develop connection details between wall and floors able to accommodate the relative vertical movement of the two systems while transferring the shear forces. An example of a practical solution to achieve this scope was proposed of in the PRESSS Five-Storey building tested at UCSD in 1999 at the culmination of the PRESSS Program and later adopted in the *fib* guidelines on seismic design of precast concrete construction (*fib* 2003, see Fig. 13.13 right): the shear connection between walls and floors should resemble the behavior of a shear key in the horizontal direction and be inserted into a vertical slot to accommodate the vertical displacement incompatibility.

Alternative solutions could include the use of a flexible (vertically, while stiff as needed horizontally) transfer/tie beams as well as cast-in-situ (timber infill) units adjacent to the wall, so to spread the localized relative deformation demand to a wider area.

When dealing with frame systems, both vertical and horizontal displacement compatibility issues between the lateral resisting systems and the floor-diaphragm can arise, as highlighted by a series of experimental tests on 3-dimensional performance of precast super-assemblages including frames and hollowcore units (Fig. 13.14) (Matthews et al. 2003; Muir et al. 2012)

Alternative innovative solutions have been recently developed and proposed in literature to minimize the damage to the floor system due to displacement
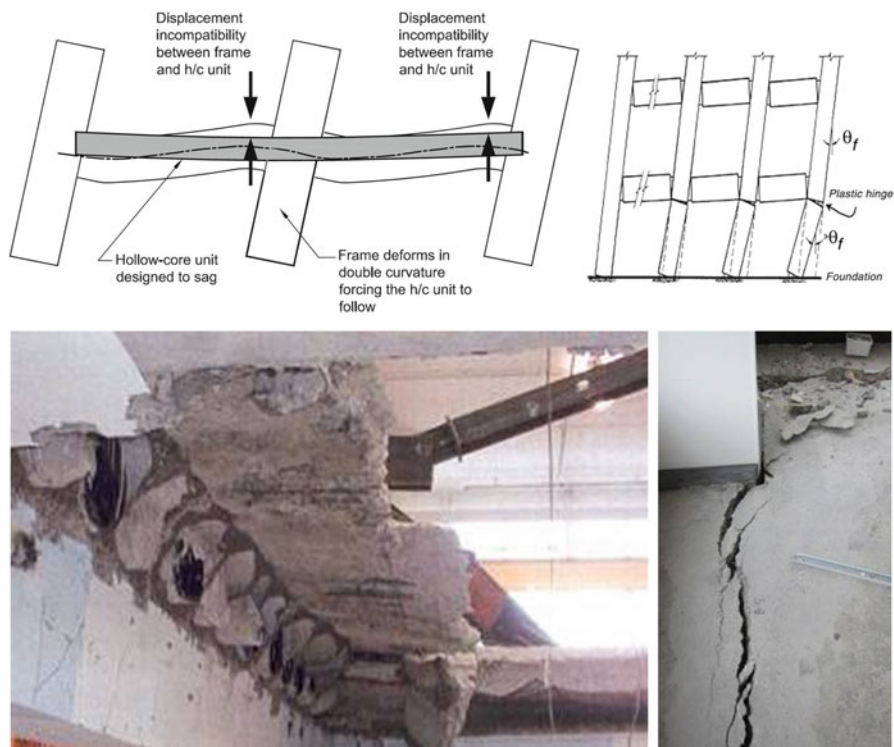
Fig. 13.14 *Top*: Example of vertical (*left*: after Matthews et al. 2003) and horizontal (due to beam elongation effects) displacement incompatibility (*right*: after *fib* 2003). *Bottom*: collapse of floor units in an 3-D experimental superassemblage test (*left*: after Matthews et al. 2003) and extensive damage to the diaphragm topping of precast concrete floors in a multi-storey building following the 22 Feb 2011 Canterbury Earthquake (*right*: after Kam et al. 2011)

incompatibilities with the response of the seismic resisting frame, while guaranteeing a reliable diaphragm action.

A jointed "articulated" floor system

The first approach is based on the concept of an articulated or "jointed" floor system to be combined with precast rocking/dissipative frames (Amaris et al. 2007, 2008). 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, Fig. 13.15), the floor (hollowcore in this case) units are connected to the beams by mechanical connectors, acting as shear keys when the floor moves orthogonal to the beam and as sliders when the floor moves parallel to the beam (Fig. 13.16).

As a result, the system is able to accommodate the displacement compatibilities 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 keys-connectors in the out-of-plane directions, torsion of the

Fig. 13.15 "X-connectors" between precast floor (pre-topped double-tee) units and frames as implemented in the PRESS Five Storey Building (Priestley et al. 1999)
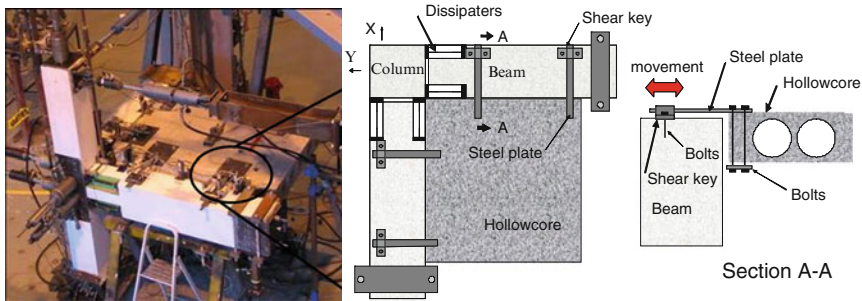

Fig. 13.16 "Articulated floor" system. Concept, connection details and response under uni-directional and bi-directional cyclic tests (Amaris et al. 2007, 2008)

beam elements, due to pull out of the floor or relative rotation of the floor and the edge support, can be limited.

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 directions 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 infill" slab in the critical regions adjacent to the beams, to enhance the capacity to accommodate relative deformations.

Top Hinge "Non-tearing floor" solution

An alternative method to prevent/control damage to the floor-diaphragm due to beam elongation effects 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 prevents direct contact between beams and columns, thus avoiding the beam elongation and the consequent tearing action on the floor. A debonded length is adopted in the bottom steel rebars to prevent premature buckling, as per a typical PRESSS jointed ductile connections.

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 solution would prevent the beam elongation effect but not the tearing action to 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 details refinement have led to the development at the University of Canterbury of what is referred to as a "non-tearing floor" beam-column connection which could be combined with any traditional floor system (Amaris et al. 2007, 2008; Au et al. 2010; Muir et al. 2012; Pampanin et al. 2006a). Based on a series of experimental testing on interior, 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 connection (Fig. 13.17).

Similar considerations on displacement compatibilities issues apply, in general, to low-damage (controlled rocking) timber connections.

A series of experimental testing have been carried out at University of Canterbury to investigate the extent of displacement incompatibilities and propose technical solutions to reduce or mitigate their effects (Moroder et al. 2013, 2014). In addition to proving the efficiency of a number of different connection detailing, the experimental results showed that the flexibility of the timber elements, combined with proper connection detailing, can provide some additional allowance to mitigate damage to the floor diaphragm at high level of interstorey drift demand.

13.8 Low-Damage Solutions for Non-structural Elements

A rapid and wide implementation of low-damage structural systems, capable of protecting the main "skeleton", including frames, walls and floor diaphragm from extensive damage at a design level earthquake would already be a major achievement. The next step towards the development of that "ultimate earthquake

Fig. 13.17 "Non tearing floor" or top-hinge solution: *Top left*: schematic (*left*, Muir et al. 2013) and comparison of damage to plastic hinges (*Top centre and right*) and to the floor (*bottom left and right*) from the testing of a 3-D superassemblage implementing a top-hinge solution (*top centre and bottom left*, Muir et al. 2012) vs. a traditional beam-column connection (*top right* from MacPherson 2005, *bottom right*, Lindsay 2004)

proof" building that the society expects would be to "dress" such structural skeleton with a compatible low-damage envelope and fit-outs, including all non-structural components (infills/partitions, facades, ceilings, services and contents).

Valuable tentative recommendations/suggestions have been proposed in the past in the form of pair of limit states or performance requirements for both structural and non-structural elements (e.g. FEMA 450 2003; FEMA E-74 2011). Yet, practical cost-efficient solutions for low-damage resisting non-structural elements for the daily use of practitioners and contractor need to be specified and developed.

Not unexpectedly, the sequence of strong aftershocks that followed the main event of the Canterbury earthquakes (4 September 2010 Darfield earthquake), caused significant and repetitive damage to the non-structural components requiring continuous and expensive repairing.

In parallel to the refinements of low-damage structural systems, a substantial effort has been dedicated at the University of Canterbury since 2009 (thus well before the main earthquake event) to the development of low-damage

non-structural components (Palermo et al. 2010), with focus on either vertical elements, e.g. infills/partitions (Tasligedik et al. 2012) and façades (Baird et al. 2011), or horizontal, e.g. ceilings (Dhakal et al. 2014).

In the case of infilled walls, either being lightweight partitions (drywalls) or "heavy" concrete or clay brick infills (more typical of the European Construction practice), the conceptual solution for a low-damage system is based once again on the possibility to create an articulated mechanism or jointed system, so to accommodate the interstorey drift demand through a sort of internal rocking mechanism of smaller panels with concentrated inelastic behaviour in few discrete locations, between adjacent panels and between panel and surrounding frame (Fig. 13.19). The low-damage infilled wall solutions were able to sustain 2–2.5 % interstorey drift, under quasi-static cyclic loading, corresponding to the maximum code-allowed demand under a design level earthquake, without evident cracking/damage, thus well beyond the expected performance of traditional infilled walls and in line with the ideal expectation of a more resilient building system.

Full details of the experimental campaign and suggested construction details can be found in Tasligedik (2014) and Tasligedik et al. (2014) (Fig. 13.18).

In the case of precast concrete facades/claddings, a number of connection solutions and detailing has been tested, ranging from traditional ones relying upon rods of different length, to slotted-bolted connections, to innovative solution with dissipative U-shape Flexural Plates (Kelly et al. 1972; Priestley et al. 1999), widely adopted in PRESSS or Pres-Lam structures as dissipative coupling systems for rocking walls. The target strategy could be either a full disconnection between the façade and the bare structures or a controlled disconnection with additional dissipation capability provided by ad-hoc designed elements (i.e., UFP). For detailed information the reader is referred to Baird et al. (2014) (Fig. 13.19).

## 13.9 First Prototype Test Building with Integrated Low-Damage Solutions

In the previous paragraph, an overview of the recently developed low-damage solutions for both structural and non-structural systems, capable to withstand high levels of drift with negligible damage has been presented, including dry jointed ductile connections for frames and walls, articulated floor solutions, low damage infilled walls (drywall/partitions) and low damage facade/cladding connections.

As inherent part of any research and development such solutions have been developed, refined and tested independently (mostly under quasi-static cyclic testing).

The next challenge towards the development of an integrated low-damage resisting building system would be to assess the feasibility and seismic performance
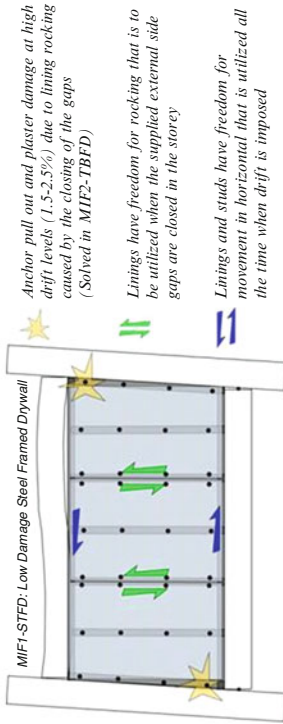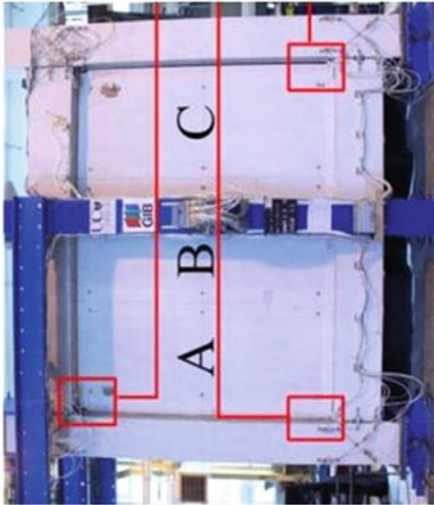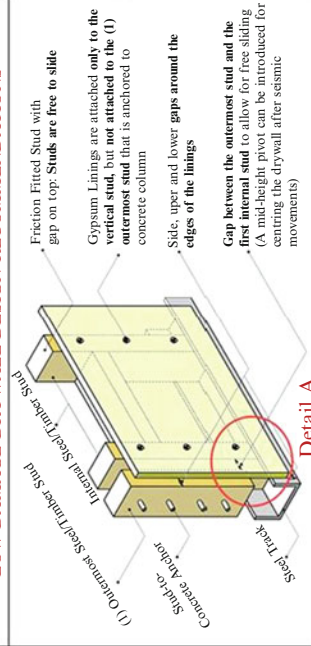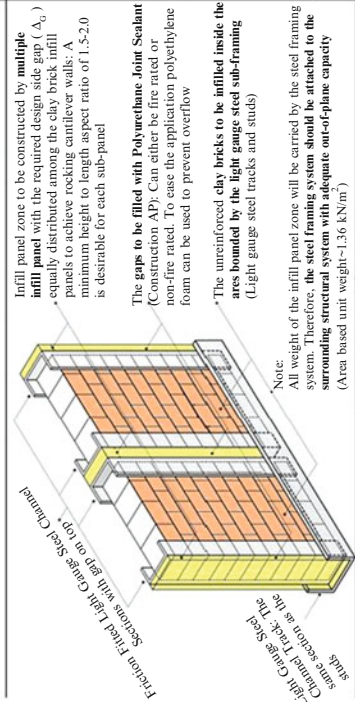
Fig. 13.18 Low-damage solution for infilled walls/partitions (After Tasligedik et al. 2014)
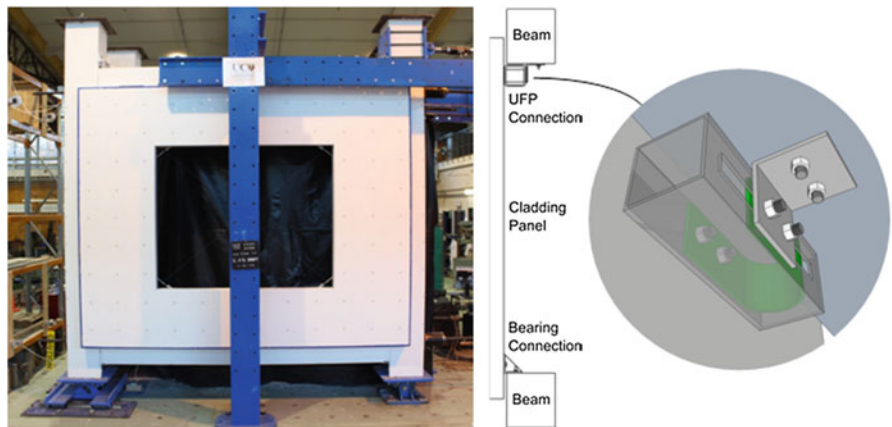
Fig. 13.19 Low-damage solution for precast concrete facades with UFP dissipative connectors (After Baird et al. 2014)
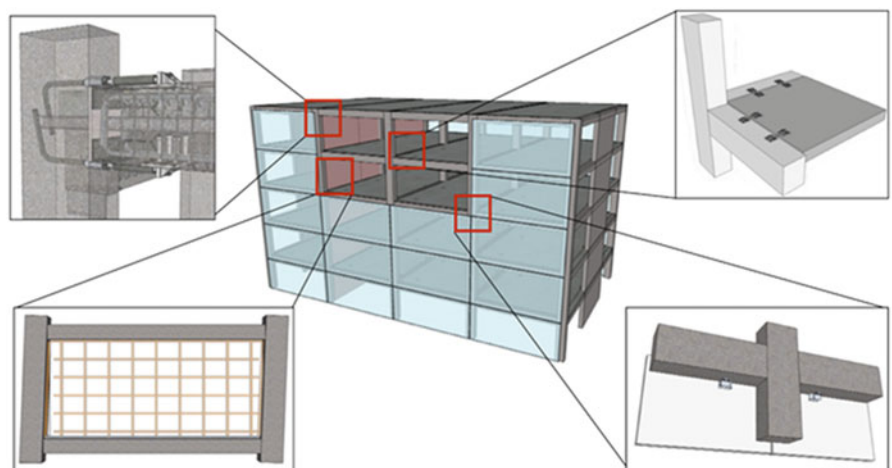

Fig. 13.20 Low-damage building system prototype (After Johnston et al. 2014)

of a building system prototype combining the aforementioned low-damage solutions for both skeleton and envelope (Fig. 13.20).

With this scope, shake table tests of a two storey, ½ scale, concrete frame building consisting of a post-tensioned rocking hybrid frame and incorporating an articulated floor solution (with U-shape Flexural Plates), low damage drywall infills and façades were carried out (Johnston et al. 2014). An overview on design, fabrication, set-up and preliminary shake table testing can be found in Johnston et al. (2014). The test building was tested under different configurations and subjected to over 400 earthquakes of different intensity levels, with no evident
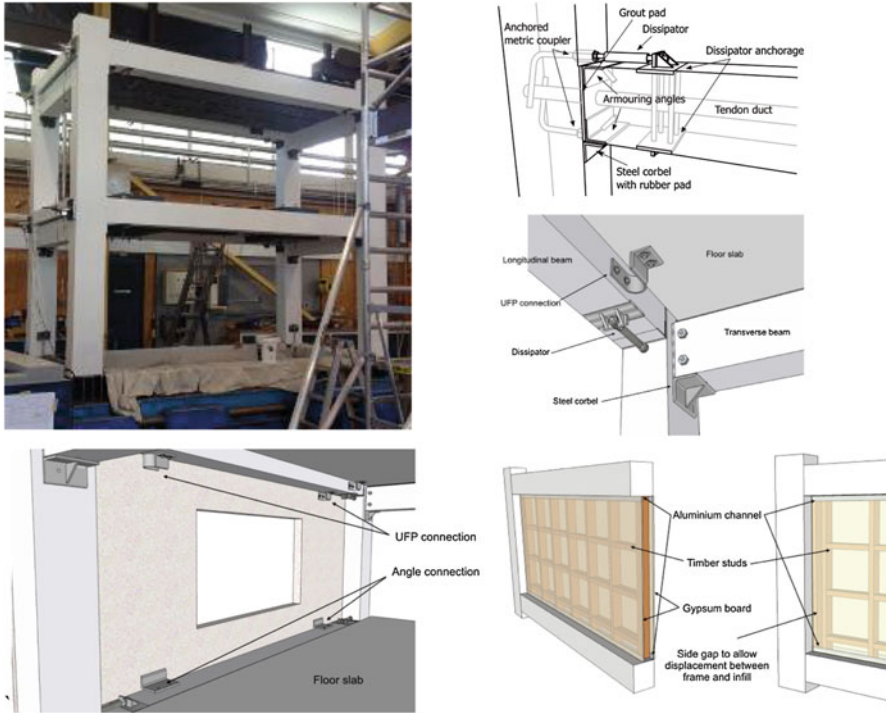
Fig. 13.21 Low-damage test-building (After Johnston et al. 2014)

level of structural and non-structural damage. More information can be found in Johnston et al. (2014) and in future publications under preparation (Fig. 13.21).

13.10   Towards an Integrated Structure-Foundation Performance-Based Design

The Canterbury earthquake has emphasised the actual impact (in terms of final outcome: demolition vs. repair) of combined damage to the superstructures and the foundation-soil system (Fig. 13.22, Giorgini et al. 2012, 2014). 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
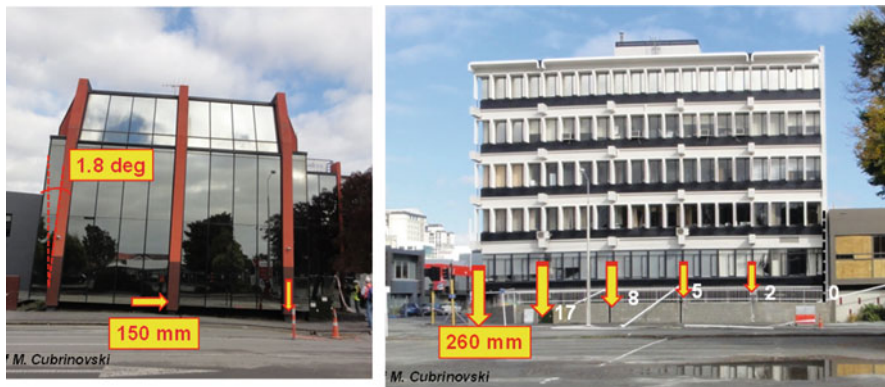
Fig. 13.22 Example of significant tilting and differential settlement in buildings in the CBD after the 22 Feb 2011 Canterbury Earthquake (From Giorgini et al. 2012, 2014)

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.

An attempt to develop a framework for an integrated structure-foundation performance-based design approach where limit stated and associated damage of superstructure and foundation can be combined into a performance matrix with defined objective and criteria is under-going at the University of Canterbury. More information on the overall integrated framework and on the more specific displacement based design approach can be found in Giorgini et al. (2014) and Millen et al. (2014), respectively (Fig. 13.23).

13.11 On Site Implementation of Low-Damage PRESSS and Pres-Lam Technology

The continuous and rapid developments of jointed ductile connections using PRESSS-technology for seismic resisting systems have resulted in a wide range of alternative arrangements currently available to designers and contractors for practical applications.

On site implementations of PRESSS-technology buildings have happened in different seismic-prone countries around the world, e.g. U.S., Central and South America, Europe and New Zealand. Overviews of research and developments, design criteria and examples of on-site implementations can be found for concrete structures in Pampanin (2005) and in the PRESSS Design Handbook (2010).

In the following sections, focus will be given to some implementations in New Zealand, highlighting the novel features resulting from the more recent experimental and numerical research and developments and presenting some
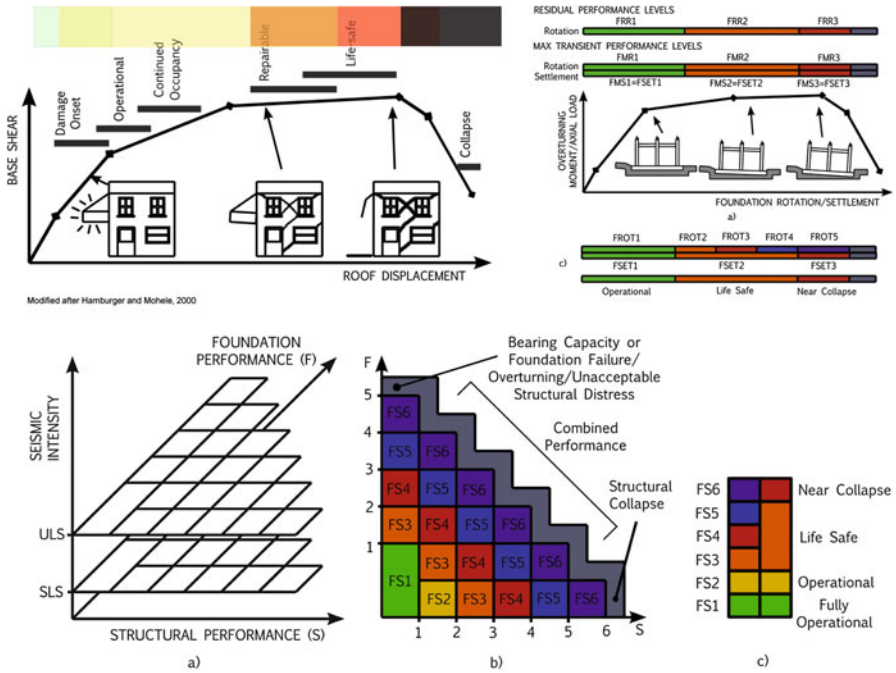
Fig. 13.23 Concept of a performance matrix (*bottom*) for integrated structure-foundation design combining limit states for structure (*top left*) and foundation (*top right*) (Giorgini et al. 2014)

more recent case studies designed and constructed following the Canterbury earthquake sequence in 2010–2011.

PRESSS (concrete) Buildings

The first multi-storey PRESSS-building in New Zealand is the Alan MacDiarmid Building at Victoria University of Wellington (Fig. 13.24), designed by Dunning Thornton Consulting Ltd. The building has post-tensioned seismic frames in one direction and coupled (by slender coupling beam yielding in flexure) post-tensioned walls (precast sandwich panels) in the other direction, with straight unbonded post-tensioned tendons. The seismic-resisting systems feature some of the latest technical solutions previously described, such as the external and replaceable dissipaters in the moment-resisting frame at both the beam-column connections and the base-column connections. Another 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 New Zealand and first in South Island followed at close distance and is represented by the Endoscopy Consultants' Building in Christchurch, designed for Southern Cross Hospitals (SCH) Ltd by Structex Metro Ltd (Fig. 13.25). Also in this case both
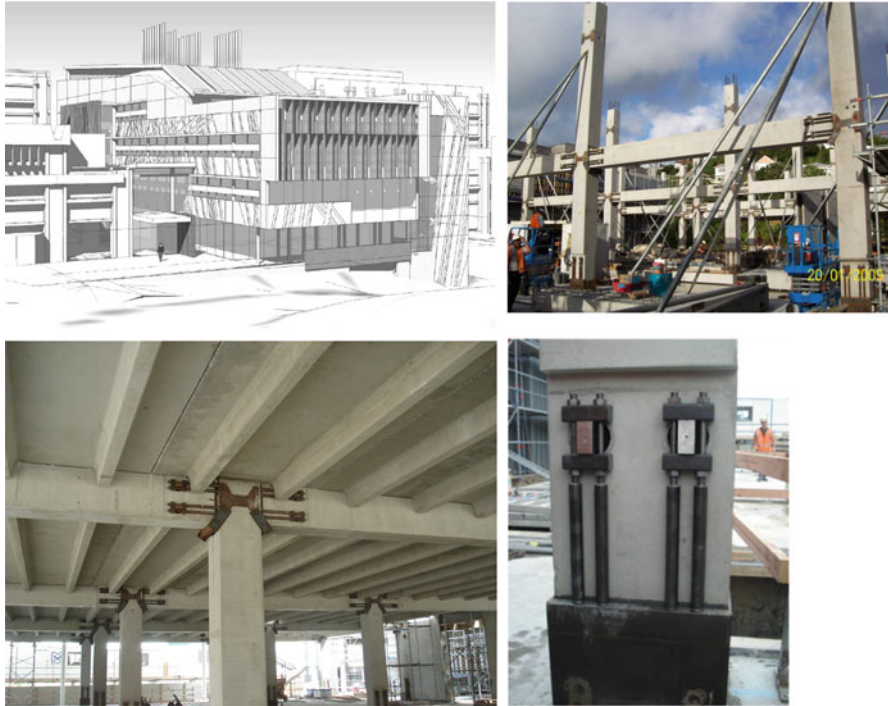
Fig. 13.24 First multi-storey PRESSS-Building in New Zealand (Structural Engineers: Dunning Thornton Consultants; Cattanach and Pampanin 2008)

frames and coupled walls were used in the two orthogonal directions. The post-tensioned frame system relies upon a 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 with UFPs.

The building passed with high performance the very severe tests of the Canterbury earthquake sequence in 2010–2011. The more devastating 22 February 2011 ground motion was very close to the hospital with a very high level of shaking. Only minor or cosmetic damage was sustained by the structural system. The medical theatres containing very sophisticated and expensive machineries 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 content and acceleration-sensitive equipment. More information on the design concept, performance criteria, modelling and analysis, construction and observed seismic behaviour can be found in Pampanin et al. (2011).

The Police Station in Rotorua (North Island, New Zealand) is a three storey building designed as a critical facility (or importance level IL4) with post-tensioned

Fig. 13.25 Southern Cross Hospital Endoscopy Building, Christchurch Rendering, construction of the frame, details of beams, walls and U-shape Flexural Plate dissipaters (Structural Engineers: Structex Metro, Pampanin et al. 2011)

rocking/dissipative concrete (PRESSS) walls in both directions, implementing external and replaceable (Plug & Play) dissipaters (Fig. 13.26)

Pres-Lam (timber) Buildings

Following the research described on post-tensioned timber (Pres-Lam) buildings at the University of Canterbury, several new post-tensioned timber buildings have been constructed in New Zealand incorporating this technology. The world's first commercial building using a Pres-Lam system is the Nelson Marlborough Institute of Technology (NMIT) building, constructed in Nelson. This building has vertically post-tensioned timber walls resisting all lateral loads as shown in Fig. 13.27 (Devereux et al. 2011). Coupled walls in both direction are post-tensioned to the

Fig. 13.26 Police Station in Rotorua. Post-tensioned concrete (PRESSS) walls with external & replaceable dissipaters in both directions (Structural Engineers: Spiire)

Fig. 13.27 Nelson Marlborough Institute of Technology, (NMIT), Nelson, New Zealand. Post-tensioned timber (Pres-Lam) walls coupled with UFPs (Structural Engineers Aurecon; Architects Irving-Smith-Jack, Devereux et al. 2011)

Fig. 13.28 Carterton Events Centre, New Zealand. Single-storey building with LVL truss roof (Designed by Opus International: Dekker et al. 2012)

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.

The Carterton Events Centre, located 100 km north of Wellington, is the second building in the World to adopt the Pres-Lam system (Fig. 13.28). 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.

The University of Canterbury EXPAN building (Fig. 13.29) was originally a two-third scaled prototype building tested in the laboratory under severe bi-directional loading conditions (Newcombe et al. 2010) After a successful testing programme, the building was demounted and re-erected as the head office for the Research Consortium STIC (Structural Timber Innovation Company Ltd). Due to the low mass, the connections of the remounted building ended up being post-tensioned only without 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.

The new College of Creative Arts (CoCa) building for Massey University's Wellington campus (Fig. 13.30) is the first to combine post-tensioned timber (Pres-Lam) frames with innovative draped post-tensioning profiles to reduce deflections under vertical loading. Additional dissipation is added in the frame directions by using UFP devices, placed horizontally and activated by the relative movement between (some of) the first floor beams and the elevated concrete walls/pedestal. This is a mixed material damage-resistant building which relies on post-tensioned rocking precast concrete walls (PRESSS) in one direction and Pres-Lam timber frames in the other direction.

Fig. 13.29 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

Fig. 13.30 College of Creating Arts – MacDiarmid Building, Massey University, Wellington, New Zealand. Post-tensioned timber (Pres-Lam) frames in the transverse directions with horizontal U-Shape flexural plate dissipaters on the first floor and Post-tensioned concrete (PRESSS) walls in the longitudinal direction (Structural Engineers: Dunning Thornton Consultants)

Fig. 13.31 Merritt Building, Victoria Street, Christchurch. Three Storey commercial Building consisting of Post-tensioned timber (Pres-Lam) frames in the transverse direction and cast-in-situ reinforced concrete wall in the longitudinal direction (Structural Engineers: Kirk and Roberts; Architects: Sheppard and Rout)

As part of the Christchurch Rebuild, a number of buildings implementing the aforementioned damage-resisting technologies have been already completed and more are under construction or design (e.g. Figs. 13.31, 13.32, and 13.33). In some cases the structural systems use mixed materials (timber/concrete/steel) and/or a combination of rocking systems with base isolations and other supplemental damping devices. As notable from the pictures shown in the following paragraphs, in most cases the low-damage seismic resisting systems, and in particular the details of the external and replaceable dissipaters, have been partly or fully exposed to the view of the public/tenants as architectural features.

PRESSS-Steel Buildings

The Forté Health Medical Centre in Kilmore Street, Christchurch is the first PRESSS-Steel building in New Zealand and possibly the first in the World using this technology in steel. The three storey building includes over 5,000 m^2 of specialist medical facilities, including four operating theatres, patient bedrooms and urology, radiology, orthopaedics and fertility clinics. The lateral load resistance, with the high performance requirements of a critical facility (IL4 design level), is provided by post-tensioned steel rocking coupled 'walls' (or braced-frames) in both directions, combining hysteretic and viscous dampers (High

Fig. 13.32 Trimble Building , Christchurch. Two storey office building (more than 5,000 m^2) consisting of post-tensioned timber (Pres-Lam) frames with external replaceable dissipaters at the beam-column connections and at the column-to-foundation connection and Pres-Lam coupled (with UFP, U-shape Flexural Plates) walls with external dissipaters at the base-connections (Design-build project with Architecture and Structures by Opus International and construction by Mainzeal/City Care, Brown et al. 2012)

Force-to-Volume Lead extrusion devices, developed at University of Canterbury Mechanical Engineering Department, Rodgers 2009) in parallel, for what is referred to as Advanced-Flag Shape System, AFS (Kam et al. 2010).

The internal frames implement another low-damage system, widely adopted in New Zealand as part of the Christchurch Rebuild and referred to as "sliding hinge joint" solution (MacRae et al. 2010) acting as second moment-resisting frame, in order to provide a additional redundancy to the primary lateral resisting systems and stability to the building during erection and after a fire (Fig. 13.34). The beam-column connections, consisting of a "top flange hinge" and a slotted- bolted connection at the bottom flange, are designed to accommodate the lateral displacements required for displacement compatibility with the main seismic resisting system with minimum stresses/strain demand on the floor/slab plate.

Fig. 13.33 Former 'St Elmo Courts' Building, Christchurch. Five storey building, combining base-isolation and two-ways post-tensioned frames in the superstructure with timber beams and concrete columns (Architect: Ricky Proko, Structural Engineers: Ruamoko Solutions)

13.12 Conclusions

The increased awareness by the general public/tenants, building owners, territorial authorities as well as insurers/reinsurers, of the severe economic impacts in terms of damage/dollars/downtime of moderate-strong earthquakes has indeed stimulated and facilitated the wider acceptance and implementation of cost-efficient damage-control, also referred to as low-damage, technologies in New Zealand, based on concrete, timber, steel or combination of the above material.

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

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

Somehow the target goal has not changed but the societal expectations (the 'bar') are higher and the allowed time frame shorter: to develop, at comparable
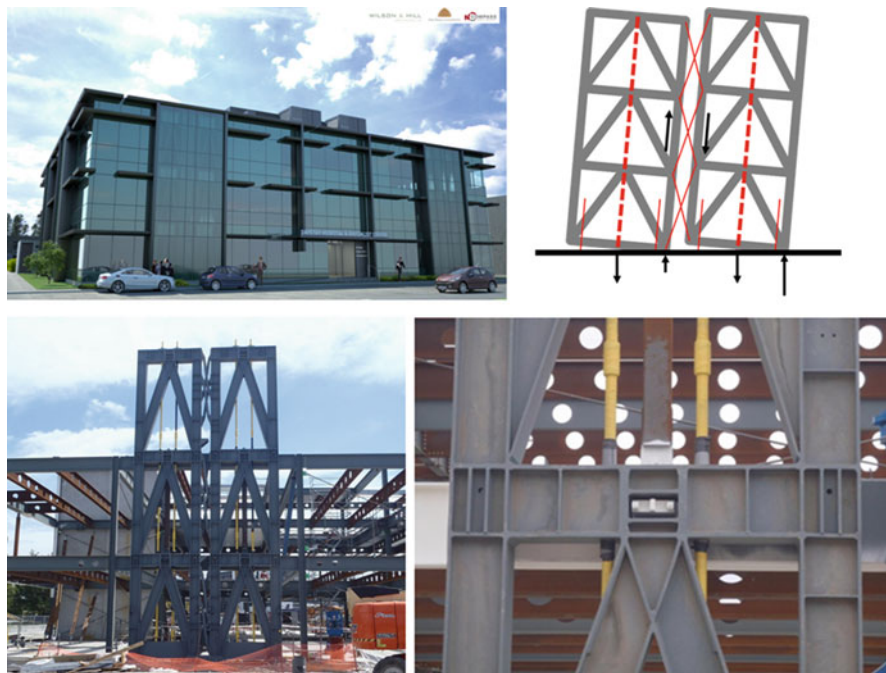
Fig. 13.34 Forté Health Medical Centre, three storey building with over 5,000 m^2 of specialist medical facilities. Post-tensioned steel rocking coupled 'walls' (or braced-frames) in both directions, combining hysteretic and viscous dampers in parallel for "an advanced flag-shape" system. *Bottom right*: secondary interior moment-resisting frames implementing a sliding hinge joint beam-column connection solution

costs, what the general public would referred to as an "ultimate earthquake-proof" building system (including skeleton, non-structural components/contents and foundation systems) capable as a whole of sustaining the shaking of a severe earthquake basically unscathed, thus including structural skeleton, non-structural components/contents and the soil-foundation system.

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