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Base Isolation and Damage-Resistant Technologies for Improved Seismic Performance of Buildings

A report written for the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes

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CONTENTS

1 SUMMARY

Modern methods of seismic design (since the 1970s) allow structural engineers to design new buildings with the aim of predictable and ductile behaviour in severe earthquakes, in order to prevent collapse and loss of life. However some controlled damage is expected, which may result in the building being damaged beyond economic repair after severe shaking.

Seismic protection of structures has seen significant advances in recent decades, due to the development of new technologies and advanced materials. It has only been recently recognised world-wide that it is possible to design economical structures which can resist severe earthquakes with limited or negligible structural damage.

There are two alternative ways of designing buildings to avoid permanent damage in severe earthquakes; base isolation and damage-resistant design. Base isolation requires the building to be separated from the ground by isolation devices which can dissipate energy. This is proven technology which may add a little to the initial cost of the building, but will prove to be less expensive in the long term.

Damage-resistant design is developing rapidly, in several different forms. These include rocking walls or rocking frames, with or without post-tensioning, and a variety of energy dissipating devices attached to the building in different ways. If not already the case, damage-resistant design will soon become no more expensive than conventional design for new buildings.

1.1 Scope

- This report is generally about structural damage to multi-storey buildings.
- Single family houses and other small residential buildings are beyond the scope of the report.
- Design to prevent damage to non-structural elements of buildings is also very important, but is not covered in this report.
- The emphasis is on design and construction of new buildings, not repair or reinstatement of damaged buildings, nor strengthening of existing buildings, although damage-resistant design can also be used for these purposes.
- This report does not address foundation engineering or geotechnical issues.

2 BACKGROUND

Many people are asking "*Why were so many modern buildings damaged beyond economic repair in the Christchurch earthquakes?*"The simple answer is that the current design methods rely on some damage to protect the buildings, and in addition, the ground shaking in Christchurch on 22 February was significantly more severe than the level of shaking used to design modern buildings. This report will focus on the causes of, and responses to, this damage caused by shaking. The other main reason for damage is the unprecedented soil liquefaction, lateral spreading, and foundation failure, which can only be managed in the future by careful site investigation and high quality geotechnical advice for the design of all buildings and foundations.

Considering the severity of the earthquake, the damage to buildings caused by ground shaking in Christchurch was somewhat less than expected by many structural engineers. Most of the old unreinforced masonry buildings were severely damaged, unless they had been systematically strengthened. Moderately aged reinforced concrete and reinforced masonry buildings generally suffered significant structural damage but no collapse, with two disastrous exceptions. Many well designed houses and industrial buildings did not have major problems which cannot be repaired.

The biggest concern of structural engineers is with those modern multi-storey buildings which have been damaged beyond economic repair. The seeds of this costly damage lie in the seismic design philosophy embedded in international building codes, based on the principle that a minor earthquake should cause no damage, a moderate earthquake may cause repairable damage, and a large earthquake, such as considered by modern design codes, can cause extensive damage but no collapse or loss of life.

As a very brief summary of the design process, when a structural engineer is designing a building for earthquake resistance, it is necessary to provide the building structure with the three key attributes of strength, stiffness, and ductility:

- Strength is necessary so that the building can resist lateral forces without failure of the whole structure, or failure of any critical parts. Increasing the strength of a structure costs money, but the required strength can be reduced if sufficient ductility is provided, as described below.
- Stiffness is essential to limit the lateral deflections of the building during the earthquake, to ensure that secondary structural elements such as stairs, facades and partitions are not damaged. The stiffness (or flexibility) of a building is a measure of how much lateral movement will occur when it is subjected to lateral loads. Modern building codes specify a maximum lateral deflection between two floors of about 75mm (2.5% of 3 metres) under the design level earthquake loading.
- Ductility is essential to avoid sudden failure after a building's strength limit is exceeded. Ductile materials like steel are often used locally in a building to increase the ductility of the whole building. Ductile buildings are subjected to much lower earthquake forces, making seismic design affordable, but they can be left with permanent structural damage. Ductility requires a building to undergo large displacements without losing overall strength in any of its critical elements.

A dilemma facing structural engineers is the trade-off between strength and ductility. Modern building codes provide for the design of safe but affordable buildings, by encouraging "capacity design" which allows for controlled damage in carefully selected ductile parts of the structure without exceeding the capacity of other components. In a severe earthquake, ductile buildings designed to minimum standards may have considerable damage in the ductile regions. Many Christchurch buildings have such damage, as expected, and some will need to be demolished because repair is not economically viable.

This dilemma raises another question *"Can structural engineers economically design new buildings for no structural damage?"* There are two recognised strategies for limiting damage in a major earthquake, to provide both life safety and property protection. These two are increased strength and stiffness, and energy dissipation to reduce damage:

1. The simplest and oldest method of limiting damage in a major earthquake is to overdesign the structure so that no damage occurs. This can be achieved by increasing the design level of strength and stiffness well above that required to resist the maximum expected earthquake. In this case the building will remain elastic in the design level earthquake, but ductility is still required to prevent collapse in a more severe earthquake. Overdesign may be an economical solution for houses and low-rise buildings such as factories and schools, but for multi-storey buildings this solution is very expensive, and usually unaffordable.

2. Base isolation will reduce damage in a major earthquake, by reducing the response of the building by partially isolating it from the shaking ground. This is done by placing the building on base-isolation units such as the lead-rubber bearings under Christchurch Women's Hospital, also used at Te Papa, and Parliament Buildings in Wellington. These devices allow an economical building to be built on an expensive foundation, with the total cost being only a little more than conventional design.

   Damage-resistant structures can also be designed to absorb energy in other parts of the structure, so that the building rocks back and forth in a major earthquake, returning to an undamaged position after the shaking. This combines ductility to reduce the design forces with little or no residual damage. New Zealand engineers are contributing to international developments in this field, including the recently completed reinforced concrete Endoscopy building at Southern Cross Hospital in Christchurch, TePuni Village steel building at Victoria University in Wellington, and the new NMIT timber building in Nelson. Experimental research at the University of Canterbury has supported these developments, which will allow new damage-resistant buildings at no more cost than conventional building designs.

The recent Christchurch earthquakes present a huge challenge and a huge opportunity to professional engineers. Now is the time to show how Kiwi structural engineers and geotechnical engineers can contribute to a sustainable cityscape for the new Christchurch, designing attractive and safe modern buildings which will not suffer the fate of today's older buildings in future earthquakes. The tools are available, with only a modest investment in building codes, education, and research necessary to make it happen.

3 PERFORMANCE-BASED DESIGN

This chapter describes the current international philosophy for seismic design, explaining why such large levels of structural damage occurred in the Christchurch earthquakes. Future standards for reducing the level of earthquake damage are also discussed.

3.1 Capacity Design

The seismic design philosophy embedded in international building codes is based on the principles that

- a minor earthquake should cause no damage,
- a moderate earthquake may cause repairable damage,
- a huge earthquake can cause extensive damage but no collapse or loss of life.

Recognising the economic disadvantages of designing buildings to withstand earthquakes elastically as well as the associated disastrous consequences following an event with an higher-than-expected earthquake intensity (i.e. as observed in Kobe 1995, and in Christchurch 2011), current seismic design philosophies favour the design of "ductile" structural systems. Ductile structures are able to withstand several cycles of severe loading, with materials stressed in the inelastic range, without losing structural integrity.

This design philosophy, referred to as "capacity design", was developed in the 1960s and 1970s by Professors Bob Park and Tom Paulay at the University of Canterbury. The basic steps in this design philosophy are to ensure that the "weakest link of the chain" within the structural system is located where the designer wants it, and that this weak link will behave as a ductile "fuse", protecting the structure from undesirable brittle failure. This will allow the structure to sway laterally in a severe earthquake without collapsing.

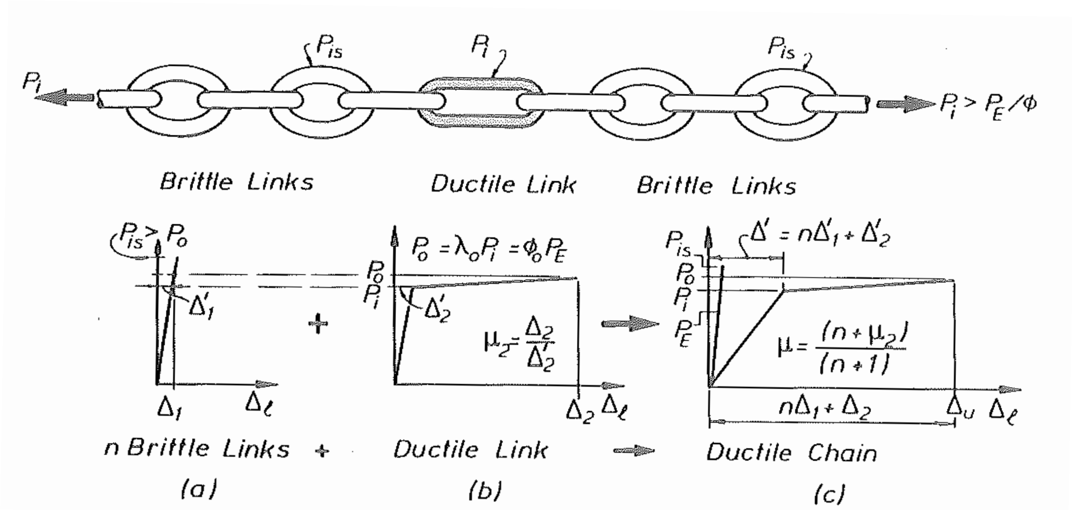
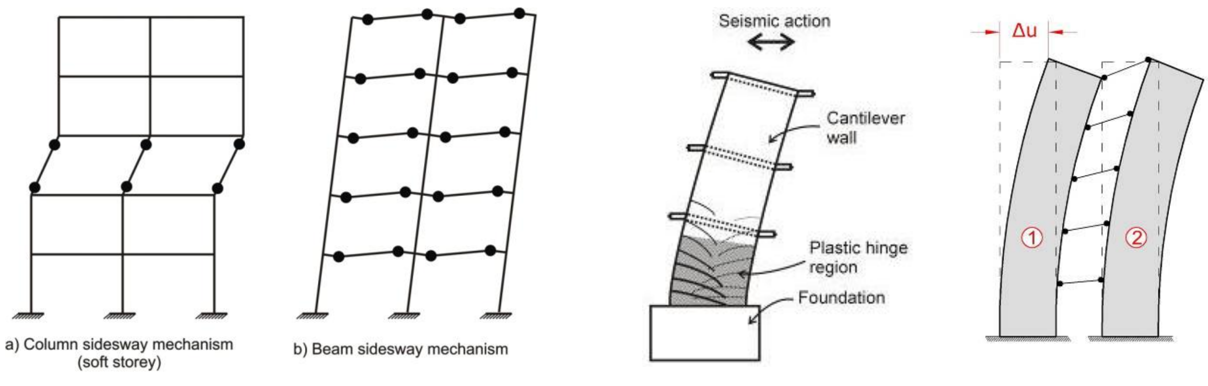


Figure3.1. Capacity design based on the weakest link of a chain (Paulay and Priestley, 1992).

For a moment-resisting frame structures, capacity design will ensure a "strong column – weak beam" mechanism as shown in Figure 3.2(b), which will prevent the possibility of highly undesirable soft-storey mechanisms as shown in Figure 3.2(a), possibly leading to "pancake" collapses. For wall structures, a plastic hinge will occur at the base of the wall as shown in Figure 3.2(c), and in coupling beams between coupled walls as shown in Figure 3.2(d).



(a) (b) (c) (d)

Figure3.2. Plastic hinge locations in multi-storey buildings: (a) Frame with column side-sway mechanism. (b) Frame with beam side-sway mechanism. (c) Plastic hinge at base of multi storey shear wall. (d) Plastic hinges in beams of coupled shear wall.

Reinforced concrete building with masonry infill (Turkey, 1999)

Three-storey apartment building which collapsed to two storeys (Christchurch 2011)

Figure 3.3. Examples of soft-storey collapses in multi-storey buildings.

Regardless of the main structural material (i.e., concrete, steel, or timber), traditional ductile systems rely on the inelastic behaviour of the building. The structural damage is intentionally concentrated within selected discrete "sacrificial" regions of the structure, typical referred to as plastic hinges, most often at beam ends in moment-resisting frames or at the base of cantilevered structural walls. Soft storey collapses are not acceptable.

3.1.1 Ductility

Many of the observed problems in the Christchurch earthquakes result from the large level of *ductility* (inelastic deformation) being activated during the severe earthquakes. Ductile buildings do not have the sudden and catastrophic failures seen in unreinforced masonry buildings and older commercial buildings. The plastic hinge zones accommodate the large displacements during the earthquake, by absorbing energy through controlled damage in selected parts of the building. Design for ductility requires that buildings have the capacity for large displacements without significant loss of strength. Designers are strongly encouraged to provide ductile structures by their engineering education, modern building codes, building regulators, and the owners of the buildings who want to minimise construction costs.

With good design in all other respects, ductility is highly desirable because:

- Ductile components of buildings can absorb energy from earthquake shaking.
- Ductile buildings are required to resist lower seismic forces than buildings designed for elastic response, resulting in less expensive components. (For example, a typical multi storey building designed for a ductility factor of 4.0 will can be designed to resist lateral forces only about one quarter of those for a non-ductile building.)
- Ductile buildings will not suffer sudden collapse when the strength limit or displacement limit is exceeded, compared with more fragile or brittle buildings.
- Ductile buildings have built-in protection for an unpredictable earthquake much larger than the design-level earthquake.

However, if a very severe earthquake demands a high level of ductile deformation, as in the Christchurch earthquakes, ductile buildings can be left with permanent structural damage, which is very expensive to repair.

Ductility will always be a desirable attribute of modern building design, but this must be combined with new design methods which reduce the residual damage, even after the building has been subjected to large deformations. Ductile structures must be carefully designed and detailed to ensure that the required ductility can be provided as intended, especially if the design is for a high level of ductility.

Figure 3.4 (from Paulay and Priestley 1992) shows the strength-displacement relationship for different levels of ductility in a building. It can be seen that the strength required to resist seismic forces decreases as the designer-selected ductility increases from elastic response to fully ductile response. The total displacement of the building is similar for all cases, regardless of the level of ductility selected.

For more information on ductile design, standard references should be consulted, including Paulay and Priestley (1992), Charleson (2008), Dowrick (1988), Priestley et al (2007).

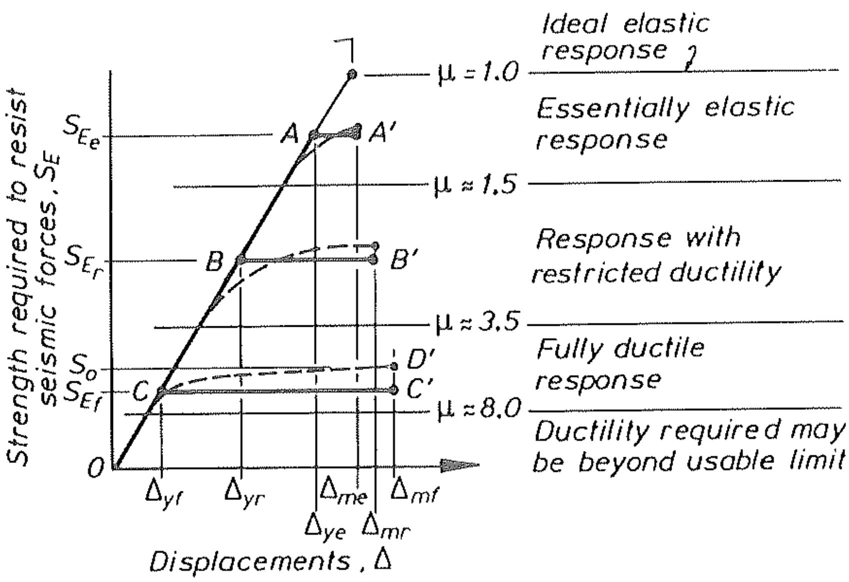


Figure 3.4. Relationship between strength and ductility (Paulay and Priestley 1991).

3.1.2 Code-based "acceptable" level of damage

In the last decade, in response to a recognised urgent need to design, construct and maintain facilities with better damage control following an earthquake, an unprecedented international effort has been dedicated to the preparation of a new philosophy for the design and construction of buildings, from the conceptual design to the detailing and final construction.

In the comprehensive document prepared by the SEAOC Vision 2000 Committee (1995), Performance Based Seismic Engineering (PBSE) has been given a comprehensive definition, 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"*

According to a performance-based seismic engineering approach, different levels of structural damage and, consequently, different levels of repair costs must be expected and, depending on the seismic intensity, be typically accepted as an 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 Objective Matrix shown in Figure 3.5, adapted from SEAOC (1995).

Performance levels are an 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 as shown in Figure 3.5) but also, more importantly, through appropriate technically-sound engineering terms and parameters, to assess the extent of damage (varying from negligible to minor, moderate and severe) for single structural components or non-structural elements (ceiling, partitions, claddings/facades, content) as well as of the whole system.

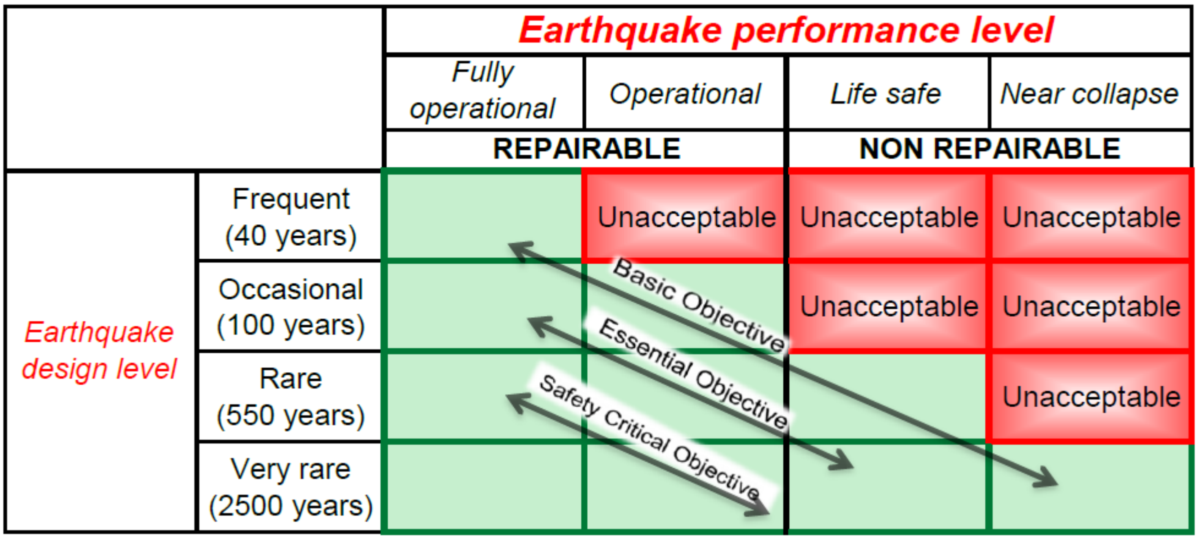


Figure 3.5. Current Performance Objective Matrix (modified from SEAOC, 1995).

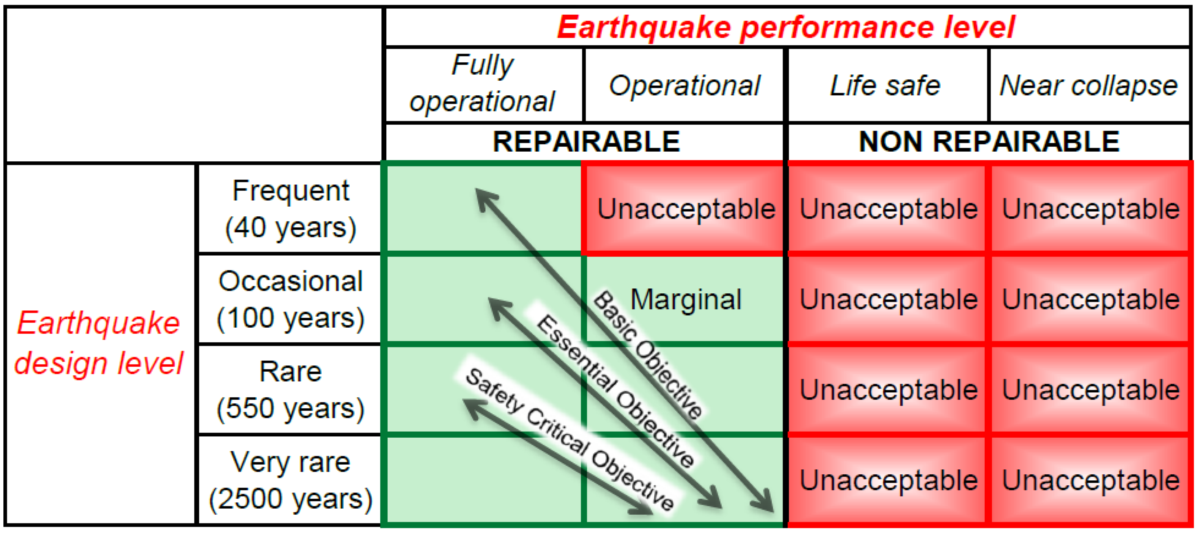


Figure 3.6. Proposed modification to Performance Objective Matrix.

3.1.3 Definition of Damage-Resistant Design

Before discussing the damage-resistant techniques, it is first necessary to define terms. It is actually not possible to design and build structures which are damage-resistant under all earthquakes, so the term "damage-resistant" should be used with care. In the context of this document, it simply means that there should be less damage than in existing construction during design level earthquake excitation. A structure which satisfies this criteria should also be available for occupation soon after the very large shaking associated with the Maximum Considered Earthquake (MCE) event.

3.1.4 Reality Check: is this enough?

It is clear from the cost of damage in Christchurch that the general public and their insurers had remarkably different expectations of the likely behaviour of an "earthquake-proof" building, compared with the building designers and the territorial authorities who consented the buildings in the knowledge that some damage was inevitable. All stakeholders clearly expected full life safety and collapse prevention, but the observed level of damage was certainly not expected by the building owners and occupiers and their insurers.

A broad consensus between the public, politicians and the engineering and scientific communities would agree that severe socio-economical losses due to earthquake events, as observed in Christchurch, are unacceptable, at least for "well-developed" modern countries like New Zealand. Higher standards are needed, which will result in much lower repair costs, and much less disruption of daily activities after major seismic events.

In order to resolve this major perception gap and dangerous misunderstanding, a twofold approach is required (Pampanin, 2009):

1. On one hand, it is necessary to clearly define, and disclose to the wider public, the targeted performance levels built into building codes (the New Zealand Building Code, and others) including any compromise between socio-economical consequences, on one hand, and technical limitations and costs, on the other. It must be clear that the targeted performance levels are considered "minimum standards", with the possibility of achieving better performance if desired.
2. On the other hand, it is also necessary to significantly "raise the bar" by modifying the New Zealand Building Code, to shift the targeted performance levels from the typically

accepted collapse prevention objective under a severe earthquake, to a fully operational objective. This is represented within the Performance Objective Matrix (Figure 3.5) by a tangible shift of the objective lines to the left, as shown in Figure 3.6. This will require a regulatory move towards higher performance levels (or lower acceptable damage levels).

In order to "raise the bar" two clear solutions are available:

- increase the level of seismic design loading (e.g., increase the Z factor),
- switch to higher-performance building technology.

A combination of these two could be used to guarantee more efficient results.

In this report, more emphasis is given on the latter option (e.g., implementation of higher-performance structural systems and technology for superior seismic protections of buildings).

These changes should apply not only to the structural skeleton, but also to the performance of the whole building system, including non-structural elements and all aspects of building operations.

In the following chapters, an overview of the development of emerging solutions for damage-resisting systems will be given. Some of these are based on base isolation, others on jointed ductile connections, or rocking structural systems, which could rely on the use of unbonded post-tensioned tendons to connect prefabricated elements. Recent examples of site-implementation will be shown for reinforced concrete, structural steel, and timber structures.

References

Charleson, A., (2008). Seismic Design for Architects. Elsevier.

Dowrick, D.J., (1988). Earthquake Resistant Design. John Wiley & Sons, Chichester, UK.

MacRae G. A., (2010a). "Some Steel Seismic Research Issues", in Proceedings of the Steel Structures Workshop 2010, Research Directions for Steel Structures, compiled by MacRae G. A. and Clifton G. C., University of Canterbury, 13-14 April.

Pampanin, S., (2009). "Alternative Performance-Based Retrofit Strategies and Solutions for Existing R.C. Buildings", Chapter 13 in "Seismic Risk Assessment and Retrofitting - with special emphasis on existing low rise structures"- (Editors: A. Ilki, F. Karadogan, S. Pala and E. Yuksel) Publisher Springer, pp. 267-295

Paulay, T. and Priestley, M.J.N., (1992). Seismic Design of Reinforced Concrete and Masonry Buildings. John Wiley & Sons, Chichester, UK.

Priestley, M.J.N., Calvi, G.M. and Kowalski, M.J., (2007). Displacement Based Seismic Design of Structures. IUSS Press, Italy.

SEAOC, (1995). Vision 2000 Committee, Performance Based Seismic Engineering, Structural Engineering Association of California, Sacramento, California.

Pampanin, S., (2009). "Alternative Performance-Based Retrofit Strategies andSolutions for Existing R.C. Buildings", Chapter 13 in "SeismicRisk Assessment and Retrofitting - With Special Emphasis on Existing Lowrise Structures"- (Editors: A. Ilki, F. Karadogan, S. Pala and E.Yuksel) Publisher Springer, pp. 267-295

4 THE NEED FOR DAMAGE-RESISTANT DESIGN

This chapter details the need for damage resistant design of new buildings, by giving a summary of serious damage observed in modern buildings in the 2010 and 2011 Christchurch earthquakes.

As well reported elsewhere, the February event had an extremely high level of shaking, significantly more than the design level earthquake, with vertical accelerations being among the highest ever recorded. This high level of shaking led to very high inelastic behaviour and severe displacement and deformation demands on a large number of buildings, and many of these will have to be demolished because of the excessive cost of repair. Many others have suffered significant business interruption and downtime costs. Given the high levels of recorded accelerations, the damage to buildings caused by ground shaking in Christchurch was largely as expected by structural engineers, because modern design standards encourage design for ductility, leading to controlled damage but avoidance of collapse.

Most of the modern buildings in central Christchurch are reinforced concrete, and a summary of damage to these buildings is given by Pampanin et al. (2011), highlighting a large amount of localised damage, especially in plastic hinge regions, exposing the limitations of traditional design philosophies not yet embracing a damage-control objective. A description of critical structural damage to non-residential buildings, and recommended assessment procedures, is given in a draft report by the Engineering Advisory Group (EAG, 2011).

The most important observed damage to structural components (excluding non-structural damage) includes:

- Major damage to plastic hinge zones of structural concrete walls including:
  - Loss of concrete and buckling of reinforcing bars.
  - Fractured reinforcing bars despite very little cracking of the surrounding concrete.
- Large inelastic deformations in moment-resisting frames, with plastic hinges and frame elongation, causing:
  - Serious cracking in concrete floor diaphragms, with fractured reinforcing bars.
  - Loss of seating of precast prestressed concrete floors.
- Fracture of welded steel in eccentrically braced structural steel frames.
- Excessive lateral displacements to parts of buildings, leading to:
  - Structural damage to frames which are not part of the lateral load resisting system.
  - Loss of support to stairs and ramps.

Most of this damage has required urgent repair, or demolition if the repairs are uneconomical. Even if the buildings are able to be re-used, there is often doubt about the residual ability to resist further major earthquakes. The solutions suggested below are for new buildings. Repair and reinstatement is not covered in this report, except in passing.

4.1 Serious Damage to Concrete Walls

4.1.1 Loss of concrete and buckling of reinforcing bars

Some buildings have suffered severe localised damage to structural walls that are holding the whole building up. This severe damage has often been in the lower stories where flexural and shear stresses are highest, as shown in Figure 4.1 and 4.2. Traditionally, the strength and performance of concrete in compression has been improved by confining the concrete in the critical regions with closely spaced hoops or stirrups of reinforcing bars. The observed damage shows that insufficient confinement was provided in many cases.

Thin walls have performed much worse than expected, largely due to insufficient confinement reinforcing bars. This may be partly because of oversight in the design, or just the practical difficulty of fitting high concentrations of reinforcement into areas that will be highly stressed.

(a) Photo from street. **(b) Severely damaged end of structural wall.**

Figure 4.1 Seven storey reinforced concrete office block.

(a) The system for joining precast concrete wall panels fails through insufficient confining reinforcement.

(b) Severe damage to reinforced concrete wall, with local buckling at the toe of the wall.

Figure 4.2 Damage to reinforced concrete walls .

Solutions:

- Use damage-resistant design. For example, design walls which will rock back and forth on the foundations under extreme lateral loading.
- Design buildings to avoid flexural plastic hinges in structural concrete walls.
- Provide more confinement in critical regions of structural concrete walls.
- Do not allow very thin structural concrete walls to be used.

4.1.2 Fracture of reinforcing bars in walls

Some semi-destructive investigation of structural walls in tall reinforced concrete buildings has identified a major problem of fractured reinforcing bars inside concrete elements that only show small cracks. An example is shown in Figure 4.3. This type of damage is due to the relatively small amounts of reinforcement in the walls, and a much higher concrete strength of the aged element than specified by the original designers. Extensive laboratory testing of reinforced concrete structures in New Zealand and around the world has shown that "plastic hinges" in beams and columns usually have a widespread pattern of cracks in the concrete, so that stresses in the internal reinforcing bars are distributed over a significant length of adjacent concrete.

(a) Small crack in base of a tall wall. Note the minor damage at far end of wall.

(b) Damaged end of wall after breaking out some concrete. The vertical bars have yielded then fractured.

Figure 4.3. Yielding and fracturing of wall reinforcing steel in a tall building.

If the concrete in a real building is much stronger than that tested in the laboratory, only one crack (rather than an array of cracks) occurs in the critical region, placing excessive strain demands on the reinforcing steel and sometimes leading to fracture of the bars. Many buildings have critical cracks which had clearly opened several centimetres during the earthquake, enough to fracture the bars, before closing up due to gravity loading after the shaking subsides. These fractured bars are hard to find, so there may be many more in damaged buildings that are undetected. Solutions for new buildings are not straightforward; simply placing more steel bars in the walls is not a solution because it will increase the strength of wall, in precisely the location where the intended "weak link" is supposed to be (this is a region of wall that is meant to "yield", or deform plastically; the "plastic hinge").

Solutions:

- Use damage-resistant design. For example, design buildings to avoid flexural plastic hinges in structural walls.
- Place upper and lower bounds on the strength of concrete in plastic hinge regions.

4.2 Large Deformations in Moment-Resisting Frames

Large inelastic deformations in moment-resisting frames result from plastic hinges occurring in the beams. Plastic hinge deformations often result in considerable lengthening of the beams, called "frame elongation". The effect of frame elongation is to cause the building to bulge or balloon out as described later in Chapter 6. This frame elongation effect causes several problems as outlined below (Peng, 2009).

4.2.1 Damage to floor diaphragms

Many buildings have suffered severe damage to reinforced concrete floor diaphragms. The main reason for this damage is frame elongation, with columns being forced apart by the formation of "plastic hinges" in the beams of moment-resisting frames (Figures 4.4and 4.5). This results in the whole building growing a bit bigger during the earthquake, causing major cracks in floor slabs, or in the topping concrete on precast concrete floor slabs. Reinforcing bars are often fractured in the cracked region. The initial concern about this cracking is the loss of the floor diaphragm action which holds the whole building together and transfers seismic forces to the lateral load resisting system.

Solutions:

- Use damage-resistant design. Design buildings without ductile moment-resisting frames.
- Find other ways of achieving ductility and hence dissipating seismic energy.
- Require larger amounts of reinforcing in topping concrete.
- Avoid the use of non-ductile welded wire reinforcing mesh.

Figure 4.4. Plastic hinges at ends of beams in reinforced concrete frames.

(a) Frame has elongated and moved away from the precast concrete floor

(b) Detail of the crack between the beam and the floor. Cold-drawn wire mesh has fractured.

Figure 4.5. Damage to floor slabs in a multi storey concrete building.

4.2.2 Seating of precast flooring systems

Most modern buildings in New Zealand have precast prestressed concrete floors with cast in-situ reinforced concrete toppings (50–75mm thick). These floors are usually simply supported one-way spanning systems, although flexural continuity is sometimes provided by placing additional reinforcing bars in the topping over the internal supports (the beams and walls).

Traditionally, the length of seating at the ends of precast prestressed concrete floors has been insufficient. Therefore when the building grows and the slab is damaged due to frame elongation as described above, the seating becomes marginal, as shown in Figure 4.6. It is extremely fortunate that no floor slabs actually collapsed in the earthquakes, although some of the observed stair collapses may have been from this cause.

Solutions:

- Design buildings without using ductile moment-resisting frames.
- Find other ways of achieving ductility.
- Require much larger seating for precast floor systems.
- Consider using two-way cast-in-place reinforced concrete floors.

Figure 4.6. Spalling of concrete ledge supporting the flange-hung Tee units.

4.2.3 Low cycle fatigue in reinforcing bars and structural steel

"Low cycle fatigue" refers to the fracture of steel due to a small number of strain reversals, well beyond the elastic strength of the steel. The best analogy is "a paperclip bent at right angles and bent back straight, a number of times, until the paperclip breaks". This is what happens to reinforcing bars that are stretched and compressed well beyond their yield strength during a seismic attack. A small number of big strains will cause the bar to fracture. This is called "low cycle fatigue".

This fracturing is hastened when the bars are bent sideways as the beams, columns or walls are damaged. Typically, visual inspection is not able to determine if steel bars or other steel members are close to fracture. Testing of samples is needed.

Steel members and steel bars that may have used up most of their plastic deformation capacity (and may be near fracture, in some cases) will be very difficult to find and repair. Repair will not be feasible in a lot of circumstances. This is elaborated upon in the next section.

Solutions:

- Design all structural members and their connections to avoid accumulative plastic strains.
- Design and install easily replaceable components that undertake the accumulative plastic strains needed to absorb energy from the earthquake.

4.3 Fracture of Welded Steel Members

Structural steel is normally considered to be a very ductile material. However, one serious brittle fracture was reported of an eccentrically braced frame (EBF) in a hospital car-parking building, very close to an eccentric welded connection (Figure 4.7). Brittle fractures are not normally expected in steel structures, but it is known that welding of structural steel can cause strain-age-embrittlement leading to brittle failures. Some new research in this area may be needed.

Solutions:

- Provide guidance on welding procedures for localised regions intended to be ductile.

Figure 4.7. Fracture in steel frame near welded connection.

4.4 Excessive Lateral Displacement of Buildings

Many buildings had large amounts of lateral displacement caused by earthquake shaking. In some of these buildings, the lateral load resisting system performed well, but the displacements caused structural damage to frames and other structural components, or to stairs and ramps or other secondary structure.

4.4.1 Structural damage to frames which are not part of the lateral load resisting system

In some buildings with large amounts of lateral displacement, the lateral load resisting system performed well, but the displacements caused extensive and expensive structural damage to frames and other structural components which are not part of the lateral load resisting system.

4.4.2 Stair failures

Some failures of stairs and ramps occurred because of the loss of stair and ramp supports, through underestimation of lateral displacements of parts of buildings.

Solutions:

- Use damage-resistant design. For example, provide base isolation to limit the inter-storey movements during earthquakes.
- Ensure that the recommended limits for lateral displacement are met, both at the serviceability limit state and at the ultimate limit state.
- Stairs and ramps which span from floor to floor must have sliding joints which are designed to accommodate sufficient floor-to-floor movement.

4.5 Summary

Much of the damage described in this chapter could have been prevented or minimised by the use of new high-performance solutions for damage-resistant design, as described in the following chapters.

References

EAG, (2011). Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-Residential Buildings in Canterbury. Part 2 - Evaluation Procedure (Revision 5). Engineering Advisory Group, Christchurch.

Pampanin, S., Kam, W.Y., Akguzel, U., Tasligedik, S., Quintana Gallo, P., (2011). " Seismic Performance of Reinforced Concrete Buildings in the Christchurch CBD after the 22 February 2011 Earthquake", Department of Civil and Natural Resources Engineering, University of Canterbury, report under preparation.

Peng, B., (2009). Seismic Performance Assessment of Precast Concrete Buildings with Precast Concrete Floor Systems. PhD Thesis, University of Canterbury.

5 BASE ISOLATION AND DAMPING DEVICES

5.1 Overview

Buildings respond to earthquake ground shaking in different ways. When the forces on a building or the displacement of the building exceeds certain limits, damage is incurred in different forms and to different extents. If a brittle building is designed to respond elastically with no ductility, it may fail when the ground motion induces a force that is more severe than the building strength. On the other hand, if the building is designed with ductility, it will be damaged but will still be able to weather severe ground shaking without failure.

As mentioned above, some alternatives to avoid significant damage in buildings in strong ground shaking are:

1. To provide the building with unreasonably high strength (which may not be economically justified).
2. To design the building to have a normal (economically justifiable) strength following damage resistant principles; in this case despite the seismic force being larger than the building strength damage will be minimal and restricted only to easily replaceable sacrificial components.
3. To alter the building's characteristics through external intervention such that even in strong ground shaking the demand is less than the design strength of the building and its components.

Following option 1, many structural engineers use the conventional approach to protect buildings from the destructive forces of earthquakes by increasing the strength of the buildings so that they do not collapse during such events. This approach is not entirely effective in terms of protection afforded to the contents and occupants because the maximum level of ground shaking is never known with certainty. Some level of ductility should always be provided for the case of extreme ground shaking, in which case there remains the risk of permanent damage to the building.

For option 2, research on damage resistant design has gained significant momentum in the last decade and design guidelines have been developed to design structures that incur little damage despite undergoing large deformation during strong ground shaking. The low damage solutions available for concrete, steel and timber buildings are explained in the later chapters of this report.

This section explains option 3(i.e., modifying the building externally to reduce its response/demand). Broadly speaking, this can be divided into two categories:

(1) Base isolating the building from the ground shaking; and/or

(2) Modifying the building's characteristics through the use of damping devices to reduce its response, and hence reduce the damage.

Note that there can be significant overlap between these categories, because damping devices can be combined with base isolation, and can also be part of damage-resistant designs, as described later.

5.2 Base Isolation

Since the motion of earthquakes is vibrational in nature, the principle of vibration isolation can be utilised to protect a building (i.e., it is decoupled from the horizontal components of the earthquake ground motion by mounting rubber bearings between the building and its foundation). Such a system not only provides protection to the building but also to its contents and occupants.

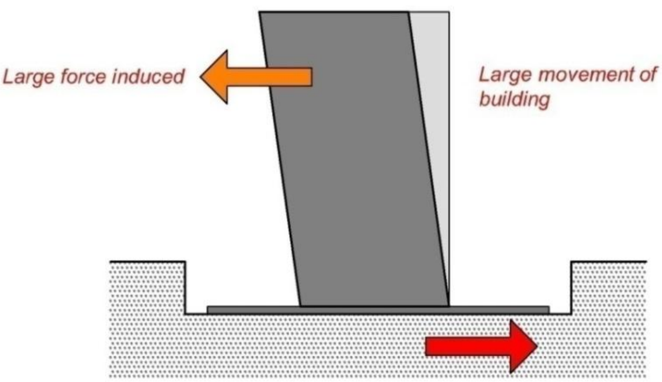
Base isolation is a passive structural control technique where a collection of structural elements is used to substantially decouple a building from its foundations resting on shaking ground, thus protecting the building's structural integrity. New Zealand is a leader in base isolation techniques, following pioneering work by Bill Robinson and Ivan Skinner (Skinner et al. 2000). Robinson Seismic Limited in Wellington is one of the leading base isolation suppliers and designers in the world. Base isolation enables a building or non-building structure (such as a bridge) to survive a potentially devastating seismic impact, following a proper initial design or subsequent modifications to the building. Contrary to popular belief base isolation does not make a building earthquake proof; it just enhances the earthquake resistance.

Base isolation can be used both for new structural design and seismic retrofit. Some prominent buildings in California (e.g., Pasadena City Hall, San Francisco City Hall, LA City Hall) have been seismically retrofitted using *Base Isolation Systems*. In New Zealand, Te Papa in Wellington and Christchurch Women Hospital are examples of base isolated new buildings, and Parliament buildings in Wellington have been seismically retrofitted. Christchurch Women's Hospital is the only base isolated building in the South Island and expectedly did not suffer any damage in the recent Canterbury earthquakes.

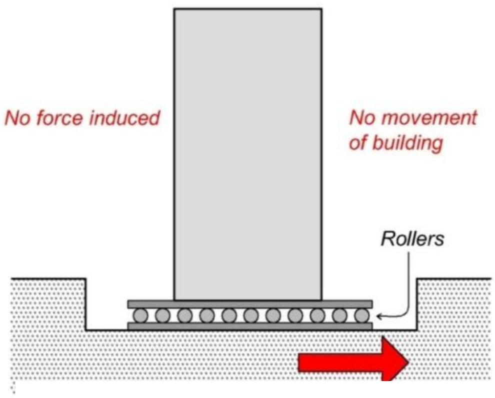
The concept of base isolation is explained through an example building resting on frictionless rollers; as shown in Figure 5.1(b). When the ground shakes, the rollers freely roll, but the building above does not move. Thus, no force is transferred to the building due to the horizontal shaking of the ground; simply, the building does not experience the earthquake. Now, if the same building is located on flexible pads that offer resistance against lateral movements (Figure 5.1(c)), then some effect of the ground shaking will be transferred to the building above. If the flexible pads are properly chosen, the forces induced by ground shaking can be much less than that experienced by a fixed base building built directly on the ground (Figure 5.1(a)). The flexible pads shown in Figure 5.1(c) are called base-isolators, whereas the structures protected by means of these devices are called base-isolated buildings.

The main feature of the base isolation technology is that it introduces flexibility into the connection between the structure and the foundation. In addition to allowing movement, the isolators are often designed to absorb energy and thus add damping to the system. This helps in further reducing the seismic response of the building. Many of the base isolators look like large rubber pads, although there are other types that are based on sliding of one part of the building relative to other. It should be noted that base isolation is not suitable for all buildings. Tall high-rise buildings or buildings on very soft soil are not suitable for base isolation. Base isolation is most effective for low to medium rise buildings which are located on hard soil.

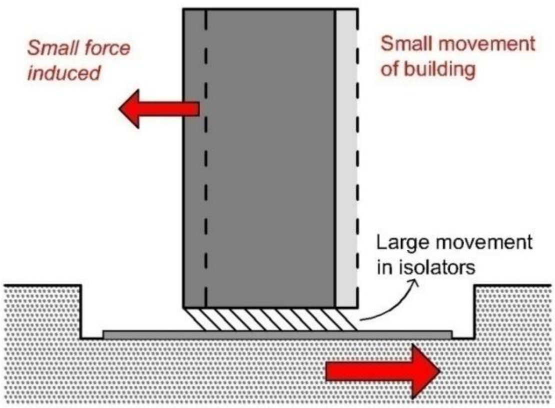
There are two basic types of base isolation systems; elastomeric bearings and sliding systems.



(a) Building resting directly on ground



(b) Building on rollers without any friction



(c) Building base isolated with lead-rubber bearing.

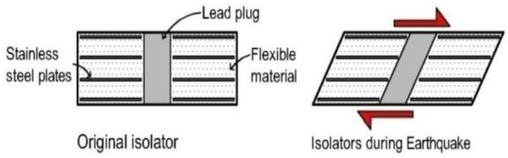
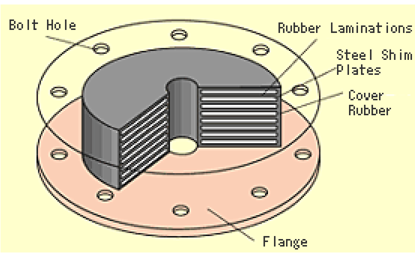


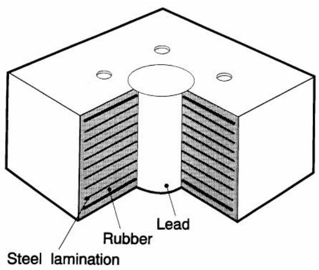
Figure 5.1.Principles of base isolation.

5.2.1 Elastomeric bearings

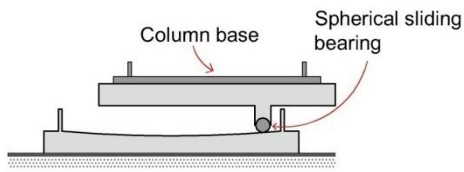
The base isolation system that has been adopted most widely in recent years is typified by the use of elastomeric bearings, where the elastomer is made of either natural rubber or neoprene. In this approach, the building or structure is decoupled from the horizontal components of the earthquake ground motion by interposing a layer with low horizontal stiffness between the structure and the foundation.



(c) Laminated rubber bearing



(b) Lead rubber bearing



(a) Spherical rubber bearing

Figure 5.2. Base isolation devices.

Rubber bearings are most commonly used for this purpose; a typical laminated rubber bearing (produced by Robinson Seismic Limited in Wellington) is shown in Figure 5.2(a). A rubber bearing typically consists of alternating laminations of thin rubber layers and steel plates (shims), bonded together to provide vertical rigidity and horizontal flexibility. These bearings are widely used for the support of bridges. On top and bottom, the bearing is fitted with steel plates which are used to attach the bearing to the building and foundation. The bearing is very stiff and strong in the vertical direction, but flexible in the horizontal direction. Vertical rigidity assures the isolator will support the weight of the structure, while horizontal flexibility converts destructive horizontal shaking into gentle movement. A slightly modified form with a solid lead "plug" in the middle to absorb energy and add damping is called a lead-rubber bearing which is very common in seismic isolation of buildings, as shown in figure 5.2(b).

The second basic type of base isolation system is typified by the sliding system. This works by limiting the transfer of shear across the isolation interface. Many sliding systems have been proposed and some have been used. One commonly used sliding system called "spherical sliding bearing" is shown in Figure 5.2(c). In this system, the building is supported by bearing pads that have a curved surface and low friction. During an earthquake the building is free to slide on the bearings. Since the bearings have a curved surface, the building slides both horizontally and vertically. The forces needed to move the building slightly upwards place a limit on the horizontal or lateral forces.

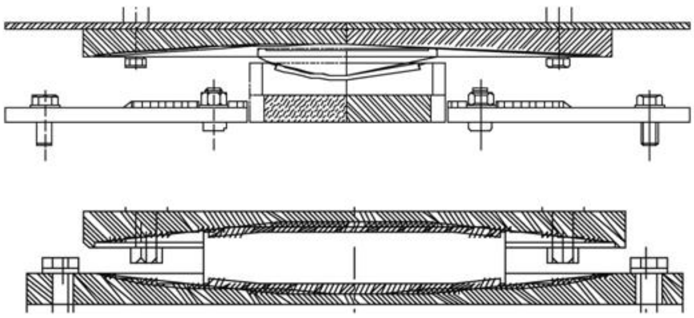
5.2.2 Friction pendulum bearing

A similar system is the Friction Pendulum Bearing (FPB), another name of Friction Pendulum System (FPS). It is based on three aspects: an articulated friction slider, a spherical concave sliding surface, and an enclosing cylinder for lateral displacement restraint (Zayas, 1990).

(a) Schematic cross section.

(b) Base isolators on steel columns

(c) Positioning a device



(d) Sections of single and double surface sliding devices

Figure 5.3. Three-storey residential construction on base-isolated ground-floor slab (Calvi, 2010)

Figure 5.3 shows an example of three-storey residential construction on base-isolated ground-floor slab, as part of the reconstruction after the 2009 L'Aquila earthquake in Italy (Calvi, 2010), using "friction pendulum" devices.

5.3 Supplemental Damping Devices

There are a number of supplemental damping devices which can absorb energy and add damping to buildings, in order to reduce seismic response. These devices can be can be combined with base isolation, or placed elsewhere up the height of the building, often in diagonal braces, or they can be used as part of damage-resistant designs, as described later.

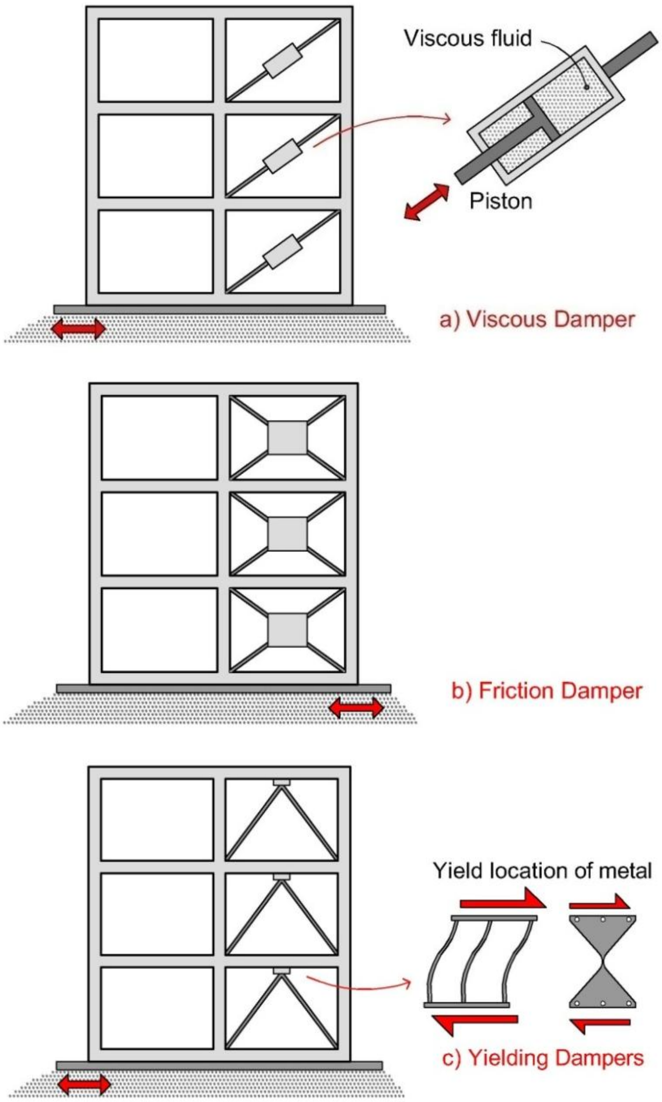


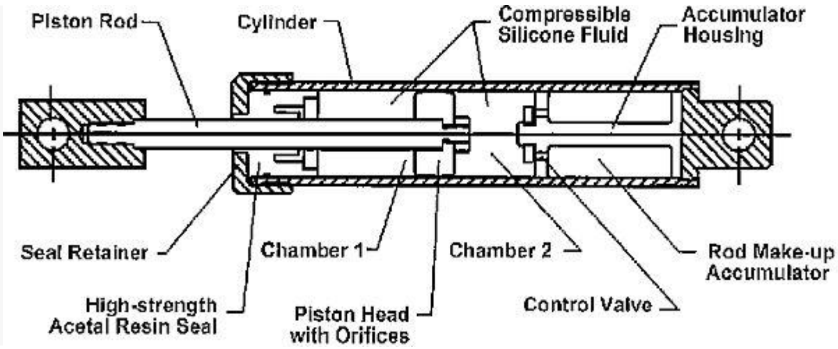
Figure 5.4. Dissipation devices.

Supplemental damping devices are especially suitable for tall buildings which cannot be effectively base-isolated. Being very flexible compared to low-rise buildings, their horizontal displacement needs to be controlled. This can be achieved by the use of damping devices, which absorb a good part of the energy making the displacement tolerable. Retrofitting existing buildings is often easier with dampers than with base isolators, especially if the application is external or does not interfere with the occupants. By equipping a building with additional devices which have high damping capacity, the seismic energy entering the building can be greatly reduced. In this concept, the dampers suppress the response of the building relative to its base.

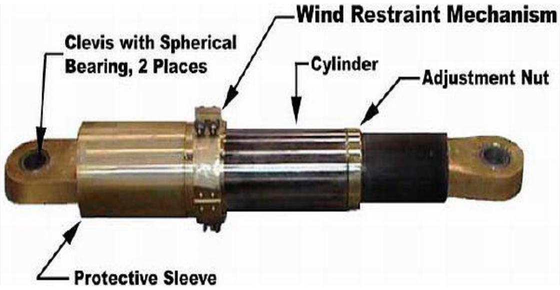
There are many different types of dampers used to mitigate seismic effects, as described below. Figure 5.4 shows typical applications of some of these dampers. More applications are shown in Chapter 7.

5.3.1 Fluid dampers

The construction of a fluid damper is shown in Figure 5.5. It consists of a stainless steel piston with bronze orifice head. It is filled with silicone oil. The piston head utilises specially shaped passages which alter the flow of the damper fluid and thus alter the resistance characteristics of the damper. Fluid dampers may be designed to behave as a pure energy dissipater or a spring or as a combination of the two. Shock-absorbers in cars are a type of fluid damper.



(a) Schematic



(b) Photograph

Figure 5.5. Typical fluid viscous damper.

(http://articles.architectjaved.com/earthquake_resistant_structures/energy-dissipation-devices-for-earthquake-resistant-building-design/

Fluid viscous dampers

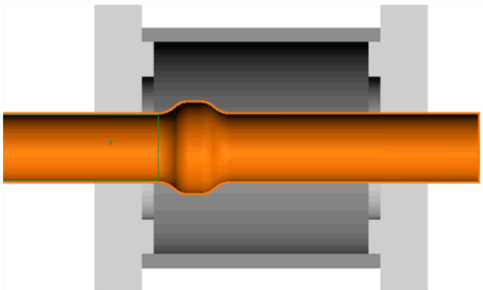
Figure 5.6. Application of fluid viscous damper.

http://www.wbdg.org/resources/seismic_design.php)

If the liquid is viscous, these dampers are called viscous dampers or fluid viscous dampers (Figures 5.5 and 5.6) in which energy is absorbed by a viscous fluid compressed by a piston in a cylinder. A fluid viscous damper resembles the common shock absorber such as those found in automobiles. The piston transmits energy entering the system to the fluid in the damper, causing it to move within the damper. The movement of the fluid within the damper fluid absorbs this kinetic energy by converting it into heat. In automobiles, this means that a shock received at the wheel is damped before it reaches the passengers compartment. Buildings protected by dampers as in Figure 5.3 will undergo considerably less horizontal movement and damage during an earthquake. Because the peak dissipater force occurs at the peak velocity, which is out of phase with the peak structural force/displacement, well designed dampers do not increases the forces on the structure. They may also be one of the only ways of minimising the effects of very large near-field pulse type accelerations. However, the cost of viscous dampers is generally considerable.

A variant of the viscous damper is the lead extrusion damper which uses solid lead as the viscous material. (Skinner et al 2000). Much has been written about lead extrusion dampers and how they allow structures to sustain large displacements without any damage. A high-force-to-volume (HF2V) lead extrusion damper has been developed at the University of Canterbury (Rodgers et al. (2010)) shown in Figure 5.7(a). It resists force as a bulge on the shaft pushes through lead as shown in Figure 5.7(b). The lead re-crystallises after the deformation thereby decreasing the likely permanent displacement.

(a) Size of the HF2V device.



(b) Shaft with bulge that passes through lead.

Figure 5.7. Lead extrusion damping device (Rodgers et al 2010).

5.3.2 Friction dampers

Friction dampers use metal or other surfaces in friction; and energy is absorbed by surfaces with friction between them rubbing against each other. Typically a friction damper device consists of several steel plates sliding against each other in opposite directions. The steel plates are separated by shims of friction pad material as shown in Figure 5.8. The damper dissipates energy by means of friction between the sliding surfaces. Friction dampers can be used in many applications including moment-frames and in diagonal braces, with several of these described in Chapter 7. This type of damper is also being developed for steel sliding hinge frames, as described in Chapter 7.

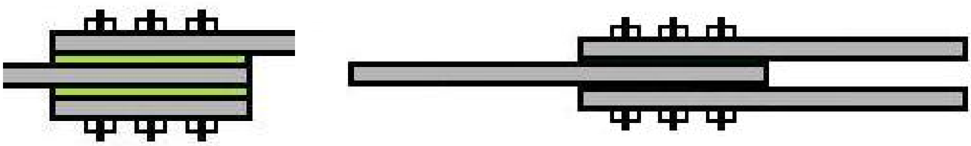


Figure 5.8. Possible arrangements of steel plates in friction dampers.

5.3.3 Visco-elastic dampers

Another type of damper is visco-elastic dampers which stretch an elastomer in combination with metal parts. In visco-elastic dampers, the energy is absorbed by utilising controlled shearing of solids. The latest friction-visco-elastic damper combines the advantages of pure frictional and visco-elastic mechanisms of energy dissipation. This new product consists of friction pads and visco-elastic polymer pads separated by steel plates. A pre-stressed bolt in combination with disk springs and hardened washers is used for maintaining the required clamping force on the interfaces as in original friction damping concept.

5.3.4 Hysteretic dampers

Hysteretic dampers (also called yielding dampers) are another type of dampers commonly used to dissipate energy in frame buildings. They typically are made of metal parts; in which energy is absorbed by yielding deformation of critical metallic components, usually made of steel. Hysteretic dampers can be designed to yield in bending, or in tension and compression.

Examples of bending devices include U-shaped flexural plates and triangular bending plates, both designed so that the yielding of the steel is spread over a significant length to avoid high strains and low-cycle fatigue. U-shaped flexural plates are used between closely spaced structural walls, as described later. Tension and compression devices are designed for axial yielding, so a high level of lateral restraint is necessary to prevent buckling in compression. The lateral restraint may be provided by steel tubes filled with concrete or epoxy, for example.

5.3.5 Buckling restrained braces (BRB)

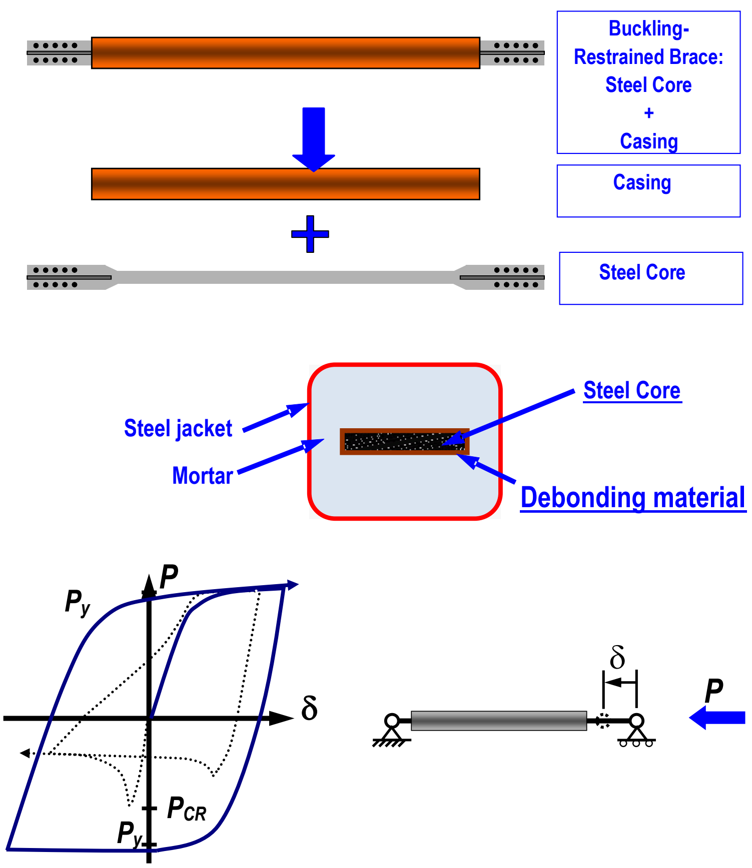


Figure 5.9. Buckling restrained brace and typical hysteresis loop.

Buckling restrained braces (BRB) are a special form of hysteretic damper, with energy dissipation built into a tension-compression brace, in such a way that the damper can yield in both axial tension and compression under reversed cyclic loading. The buckling restraint is needed to prevent the yielding steel component from buckling when loaded in compression. While the BRB sustains damage, the displacements are spread over a long length so that the strains are kept small enough to prevent low cycle fatigue failure.

5.4 Examples of Base Isolation

Some examples of real applications of base isolation and dampers follow.

Figure 5.10. Christchurch Women's Hospital, showing one of 40 lead-rubber bearings.

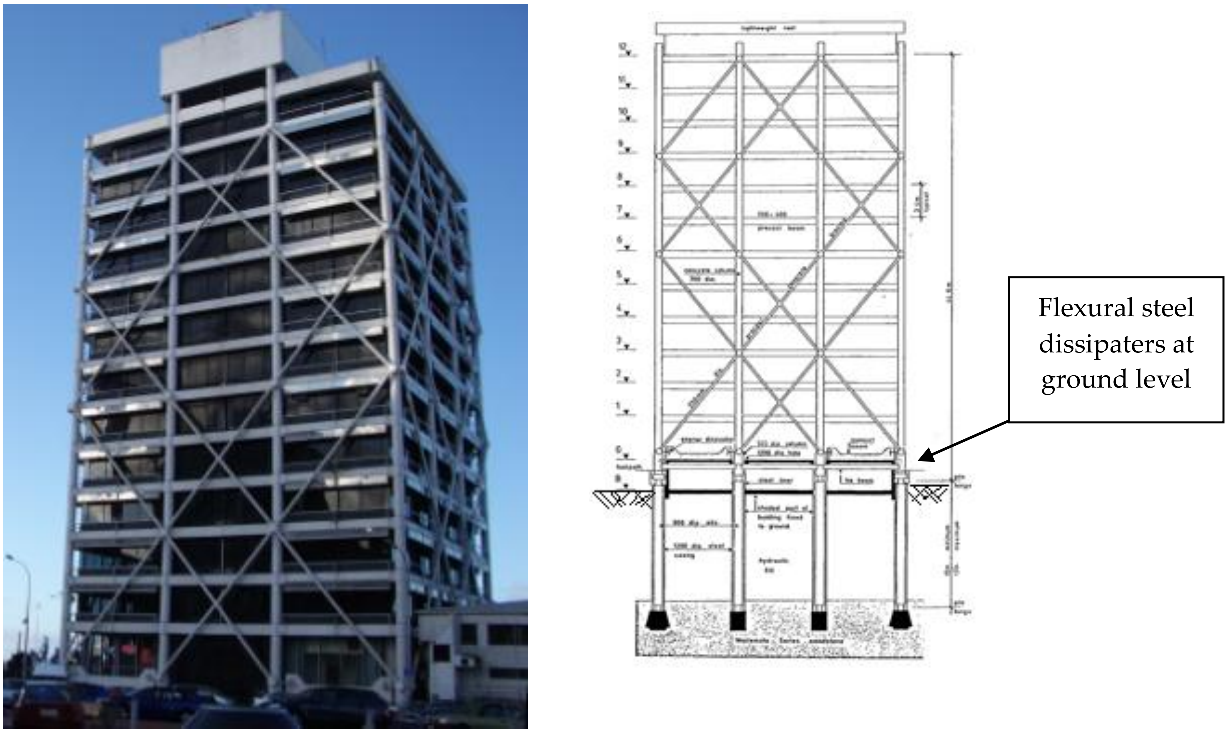
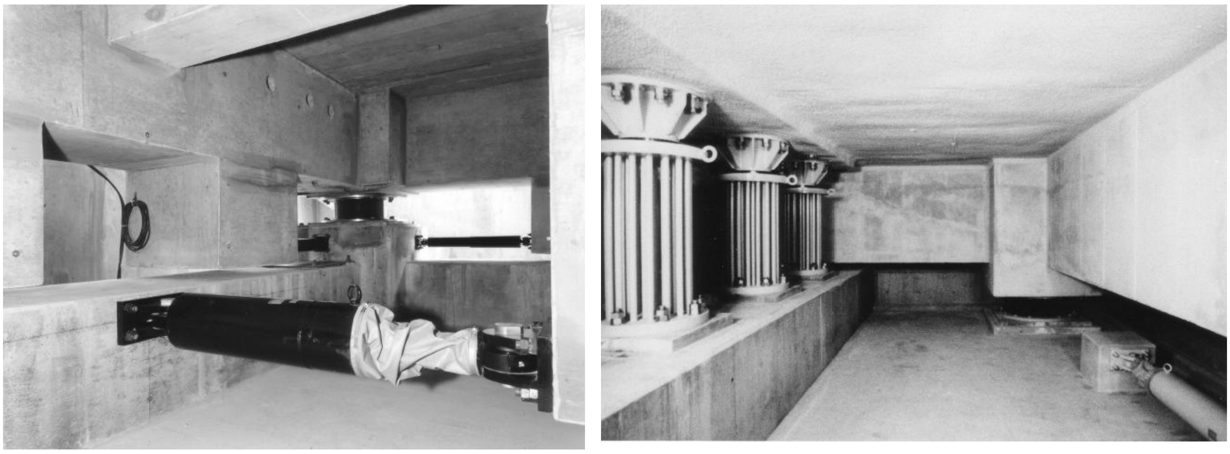


Figure 5.11. Union House, Auckland, base isolated using flexible piles and energy dissipaters.



Viscous dampers and laminated rubber bearings in Test Building at Tohoku University, Sendai, Japan.

High damping rubber bearing, steel dampers and oil damper in basement of Bridgestone Toranomon Building, Tokyo.

Figure 5.12. Base isolators in Japanese buildings (Skinner et al., 2000).

Figure 5.13. Te Papa Museum in Wellington has base isolation with lead rubber bearing (Skinner et al., 2000).

Many examples of steel buildings with damping devices are shown in chapter 8.

References

Calvi, G.M., (2010). L'Aquila Earthquake 2009: Reconstruction BetweenTemporary and Definitive. Proceedings, NZSEE 2010 Annual Conference. Wellington.

Christopoulos, C. and Filiatrault, A., (2006). Principles of Passive Supplemental Damping and Seismic Isolation. IUSS Press. First edition. Pavia, Italia.

Dowrick, D.J., (1988). Earthquake Resistant Design. John Wiley & Sons, Chichester, UK.

Rodgers G.W., Solberg, K.M., Mander J. B., Chase, J.G., Bradley, B.A., and Dhaka R.P., (2011). "High-Force-to-Volume Seismic Dissipaters Embedded in a Jointed Pre-Cast Concrete Frame". *ASCE Journal of Structural Engineering (JSE),* ISSN: 0733-9445, (In Press).

Rodgers, G.W., Solberg, K.M., Chase, J.G., Mander, J.B., Bradley, B.A., Dhakal, R.P. and Li, L., (2008). "Performance of a damage-protected beam-column subassembly utilising external

HF2V energy dissipation devices". *Earthquake Engineering and Structural Dynamics*, Vol. 37, No. 13, pp. 1549-1564.

Skinner, R.I., Kelly, T.E., Robinson, W.H., (2000). "Seismic Isolation for Designers and Structural Engineers". Robinson Seismic Ltd. and Holmes Consulting Group, Wellington.

Zayas, V.A. et al., (1990). *A Simple Pendulum Technique for Achieving Seismic Isolation.* Earthquake Spectra.pp. 317, Vol.6, No.2.

6 NEW FORMS OF DAMAGE-RESISTANT STRUCTURE

Damage-resistant design is the newest way of limiting damage in a major earthquake, whereby damage-resistant structures can be designed to absorb energy in a major earthquake, rocking back to an undamaged position after the shaking. This combines ductility to reduce the design forces with little or no residual damage. New Zealand engineers are contributing to international developments in this field, as described in the following chapters. Experimental research at the University of Canterbury has supported these developments, which will allow new damage-resistant buildings to be built at no more cost than conventional designs.

6.1 Rocking controlled dissipative rocking or hybrid concept

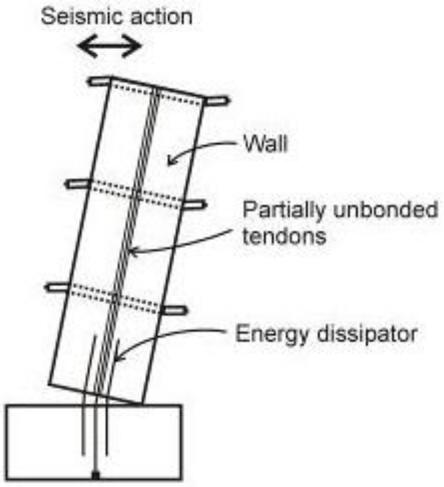
The main type of damage-resistant design is to use one or more of many new rocking structural systems being developed, in concrete, steel, timber, or mixed materials. The introduction of jointed ductile systems, assembled by unbonded post-tensioning and able to undergo severe seismic events with minor structural damage, represents a major development in seismic engineering.

The conceptual innovation of "capacity design" introduced by professors Park and Paulay in the 1960s and 1970s is universally recognised as a major milestone in the development of earthquake engineering, and of seismic design philosophies in particular. Similarly, the concept of ductile connections able to accommodate high inelastic demand without suffering extensive material damage, developed in the 1990s, is the next development in high-performance damage-resistant structural systems.

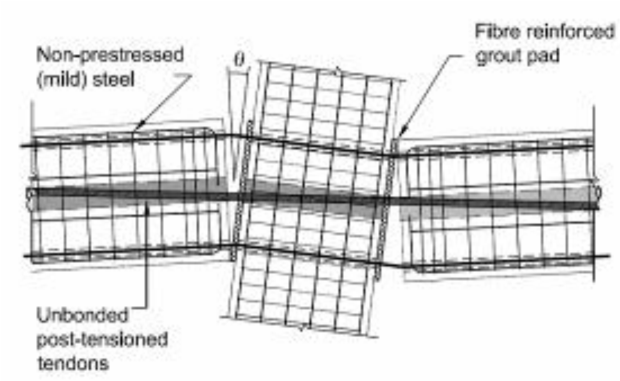
This revolutionary technological solution and the associated conceptual design philosophy was developed in the 1990s as an outcome of the U.S. PRESSS Program (PREcast Seismic Structural System) coordinated by the University of California, San Diego (Priestley et al. 1999) . The main goal of the project was to create innovative damage-resistant solutions for precast concrete buildings, as an alternative to the traditional connections based on cast-in-situ concrete. High-performance, low-damage structural systems for both frames and walls were developed through the use of dry jointed ductile connections, where prefabricated elements are joined together by means of unbonded post-tensioned tendons. A wall system is shown in Figure 6.1(a) and part of a frame system in Figure 6.1(b).

During the seismic response, the articulated or segmented elements are subjected to a controlled rocking mechanism. After the earthquake shaking, due to the elastic clamping action of the unbonded tendons the structure returns back to the original position, with negligible damage and negligible residual deformation. Additional energy dissipation capability can be provided by means of grouted mild steel bars or other supplemental damping devices to create a "hybrid" system (Stanton et al., 1997) which combines re-centering capability with energy absorption, resulting in particular "flag-shaped" hysteresis behaviour, to be described later, as shown in Figure 6.2.

This type of damage-resistant structural system has been further developed for concrete, steel, and timber structures as described in the following chapters.



(a) Rocking wall system.



(b) Beam-column joint in a rocking frame system.

Figure 6.1. Rocking hybrid frame or wall system (after fib, 2003).

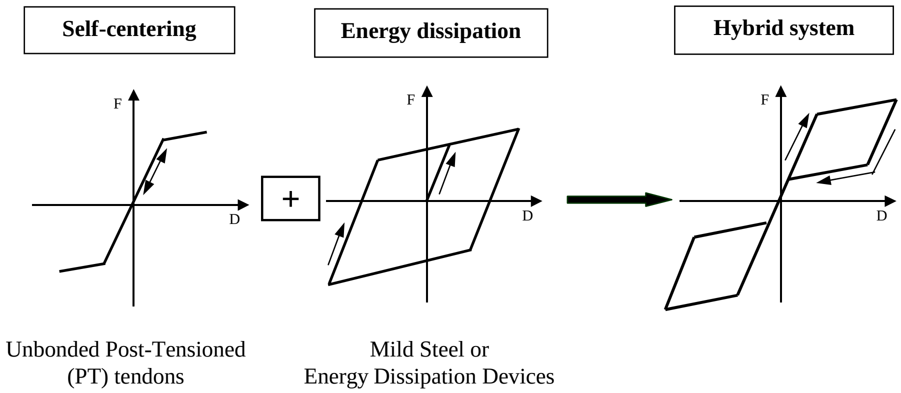


Figure 6.2. "Flag-shape" hysteresis loops for a hybrid frame or wall system (after fib, 2003).

6.1.1 Ancient Technology

In a fascinating way, buildings with rocking walls represent a clear example of modern technology based on our ancient heritage. We could in fact clearly recognise the lessons and inspiration provided by the long-lasting earthquake resisting solutions in the ancient Greek and Roman temples consisting of segmental construction with marble blocks "rocking" on the top of each other under the lateral sway.

Figure 6.3. Earlier implementation of a self-centering limited-damage rocking system, for earthquake loading (Dionysus temple in Athens).

The weight of the blocks themselves and the heavy roof-beams provided the required "clamping" and re-centering vertical force (Figure 7.17). The shear force between the elements was transferred by shear keys, made of cast lead, preventing the occurrence of sliding and also probably acting as relocating pivot points.
Exemples of concrete and timber buildings wall system are presented in Chapters 7 and 9.

6.2 Rocking Wall and Rocking Frame Systems

The most simple form of rocking damage-resistant structural elements are rocking walls. Rocking precast concrete walls will be described in Chapter 7 and rocking timber walls in Chapter 9. Multiple walls can be used with damping devices inserted between the walls, as described in Chapters 7 and 9. A similar system can be used with rocking braced frames in steel structures, as described in Chapter 8.

Rocking frame systems have similar overall performance to rocking wall systems, with the same flag shape loops shown in Figure 6.1, but there are many more points of gap opening throughout the building, and potential for so careful detailing is required to make sure that the whole structural system performs as intended with no significant damage, especially to floors, as described below. Several techniques for managing this issue for steel structures are described in Chapter 8.

Some buildings may have mixed wall and frame systems, with walls resisting lateral loads in one direction and frames in the orthogonal direction.

6.3 Avoiding Damage to Floors

In addition to protecting the structural skeleton from damage, one of the largest problems to be overcome in earthquake resistant structural systems is potential damage to floors .Most of the standards and codes around the world allow the use of design forces that are generally smaller than those required for elastic response, providing that the critical regions of the structure have adequate ductility and energy dissipation capacity. Such approaches are fundamentally based on a casualty-prevention principle, where structural damage is accepted providing that collapse is avoided. Designers must select a proper mechanism of plastic deformation and use capacity design principles to ensure that the chosen mechanism can be developed.

Both conventional frames and rocking beam-column systems can cause gaps to form between to floor and the neighbouring beams. The mechanisms that cause the gapping are described below. This gapping disrupts the flow of forces across the floors to the supporting frames and, in extreme cases, can reduce the support of the floor to where the floor drops off the supporting beams.

The most likely cause of floor damage is frame elongation, described below. New techniques are being developed for the design of non-tearing floors which can remain undamaged after a rocking structure deforms and returns to its original position after a major earthquake, as described later. This is also a problem for conventional cast-in-place reinforced concrete buildings where plastic hinges cause frame elongation.

6.3.1 Floor diaphragms

In many multi storey buildings, the bulk of the weight of the building is in the concrete floor slabs, so the largest inertial forces arising during an earthquake are forces in the floors. To resist these forces, it is essential that the floor diaphragms remain undamaged, even during major earthquakes.

In-plane diaphragm action of floors is exceedingly important for the lateral load resistance of multi storey buildings. The floor diaphragm is a critical structural element which holds the building together, and transfers lateral loads from the floor or elsewhere in the building into the lateral load resisting system of frames, walls or braces. Damage to a floor diaphragm can compromise the structural performance of the whole building. Floor diaphragms must always be carefully designed for the high in-plane forces which are obtained from analysis of the whole structure. The diaphragm forces are much larger in floors with openings, or in irregular buildings, especially if there is a change in the lateral load resisting system with storey height, or if the building has a number of different lateral load resisting systems.

The development of forces that cross the floors (in diaphragm action) during earthquakes is due to firstly, the mass of the floor and contents on that floor being accelerated by the earthquake motions and secondly, the various vertical structures (frames and walls) that resist the horizontal displacements of the earthquake. If the vertical structures acted in isolation from each other, each would form an individual displaced shape under the lateral forces, but the floor diaphragms tie the vertical structures to each other they are hence forced to displace to a common shape. This can result in large forces forming across the floor diaphragms as the vertical structures of different stiffness and strengths are constrained to shapes that are not the same as if they were acting in isolation.

These "diaphragm forces" are significantly easier to design and detail for when the floors are still essentially intact and well connected to the supporting frames (and walls). This is one of the major advantages of the "non-tearing floor" over the conventional and rocking structure frame systems which produce large gaps or "tears' between the floors and neighbouring frames, as described later.

6.3.2 Seating of precast floors

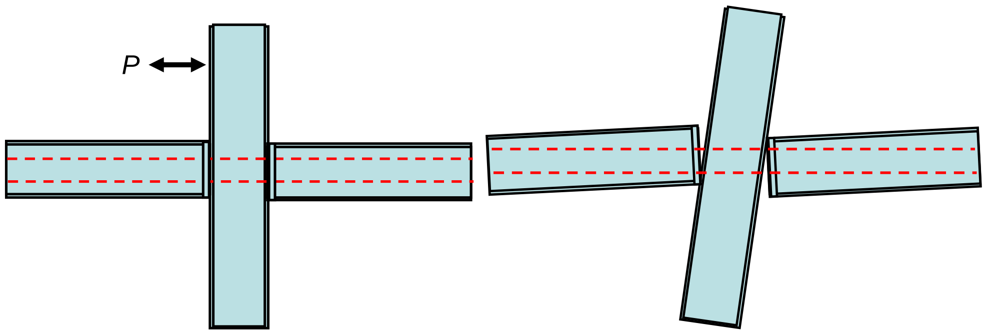
Another critical reason for avoiding damage to floors is the possible loss of gravity load seating for the floors. This can be a particular problem for one-way precast concrete floor systems. Any tearing of floors, especially at the supporting edge, can compromise occupant safety. Experimental tests on the 3-dimensional performance of precast concrete frames and hollow core floor units (Matthews et al., 2003), have further underlined issues related to the inherent displacement incompatibility between precast floors and lateral load resisting systems, including beam-elongation effects. Appropriate design criteria and detailed technical solutions should thus be adopted.

Other problems inherent with the use of precast concrete floor systems in earthquake regions have been identified (Park, Paulay, and Bull, 1997) including loss of support, analysis and design of diaphragms and their connections, transfer diaphragms, strut & tie node points, serviceability requirements and construction methods.

6.4 Frame Elongation

For traditional cast-in-place ductile frames in reinforced concrete, experimental and numerical studies (Fenwick and Fong, 1979, Douglas, 1992 and Fenwick and Megget, 1993) have shown that plastic beam hinges cause growth in the beam length, depending on the beam depth, the expected position of the neutral axis and the rotation (drift) demand. This frame elongation has been identified as a serious issue for reinforced concrete moment-resisting frame buildings, and much research has been done to try and alleviate the problem (Peng, 2009). This issue of frame elongation can also be a serious problem for a damage-resistant rocking system, as described below.

It has been argued (Pampanin, 2010) that frame elongation is less for a post-tensioned rocking frame than for a traditional cast-in-place reinforced concrete frame, because the rocking system has only geometric elongation and does not have a material contribution due to the cumulative residual strain within the steel of the plastic hinges. Frame elongation could also be less for a post-tensioned rocking frame if the beam depth is smaller than in a traditional system.



(a) At rest **(b) During lateral displacement**

Figure 6.4. Post-tensioned beam-column joint, showing frame elongation.

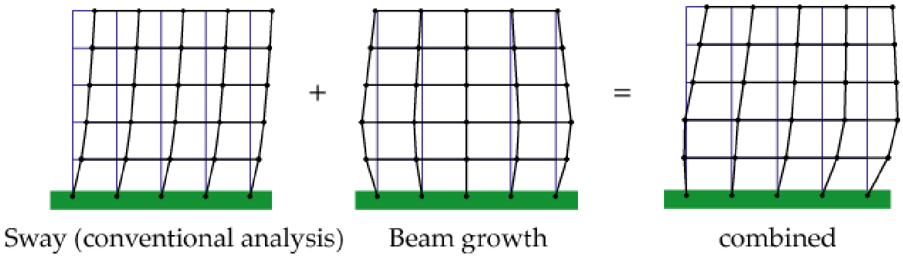
Tests of beam/column subassemblies with one column and without slabs have shown very good behaviour with no permanent displacements after the earthquake, and no significant damage. However, when the beam supports a slab, and/or when the beam is part of a frame that has more than one column, additional effects occur which may result in damage.

For frames with more than one column, the gaps which form at the beam ends cause "beam growth" or "frame elongation" (Peng, 2009). In conventional sway analysis, used in routine design, this effect is not considered as shown in Figure 6.5(a). The effect of the beam-growth itself is shown in Figure 6.5(b). It can be seen that the exterior columns are being pushed apart. The combined effect, which is the likely behaviour of an actual frame under significant seismic displacements, is shown in Figure 6.5(c). It can be seen that

i) As the number of bays in the seismic frame increases, the demands on the columns due to gap opening also increase. While this does not contribute toward the possibility of a soft-storey mechanism (as the columns are being pushed in different ways), there is more possibility that the capacities of some columns may be used up.

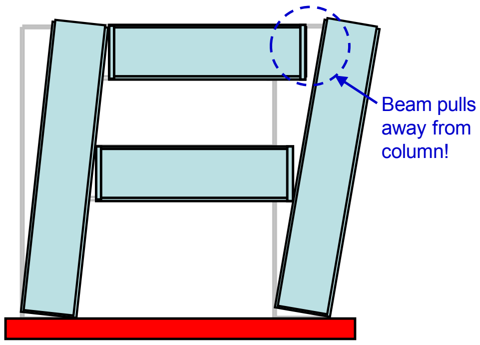
ii) The beams at the first storey are subject to compression forces. This will increase their flexural strength and increase the possibility of column yielding above that from conventional analysis.

iii) The beams in other stories will be subject to axial forces too. Above the level of maximum frame expansion, they may well be in tension. This is illustrated in Figure 6.4. Here, if there are relatively stiff columns held in place at the base, the beams and columns will want to separate at the higher levels, and this should be taken into account in the structural analysis. In a worst-case scenario, this could lead to column failure as shown in Figure 6.5, unless properly designed for.



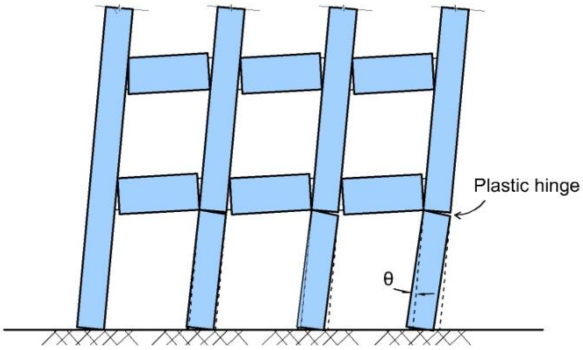
(a) (b) (c)

Figure 6.5. Frame elongation in moment-resisting frames (Kim et al., 2004).



(a)

Figure 6.6. (a) Gap Opening Effect on 2 Storey Frame with Stiff Columns (MacRae, 2010).



(b)

Figure 6.6. (b) Damage to column caused by frame elongation (Pampanin, 2010).

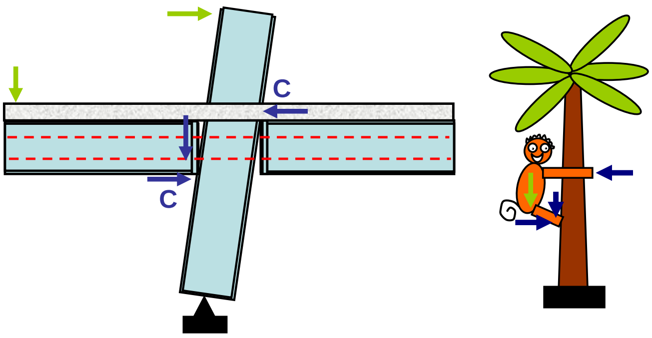
6.5 Non-Tearing Floors

For frames with slabs, where the slabs are connected to the beams, the gap opening at the end of the slab wants to cause extend the slab, as shown in Figure 6.7a). The forces applied on the joint may be understood using the idealisation in Figure 6.7(b), where the tension force in the concrete slab is equivalent to the force in the arms of the monkey. As stated above, it is essential to prevent any significant damage to the floor slabs, by designing some kind of non-tearing floor solution.

6.5.1 Damage to slabs

There are two possible situations:

a) If the slab is very strong in tension, then the gap can never open and the desired frame yielding mechanism cannot occur. This means that the moment demand from the beam and slab will be increased, and column yielding may occur unless the columns are specifically designed to resist these larger forces. Any serious column damage, such as shown in Figure 6.6, is not acceptable in a damage-resistant design.



(a) Deformation with a slab. (b) Monkey Idealisation

Figure 6.7. Slab Effects on Subassembly (from MacRae, 2010a).

b) If the slab is not strong enough in tension, then the gap can open. However, the gap opening will result in slab damage during the imposed displacements. This damage is also not acceptable in a damage-resistant design. Some damage from a test of this type, is shown in Figure 6.8. This damage is not acceptable in a damage resistant design.

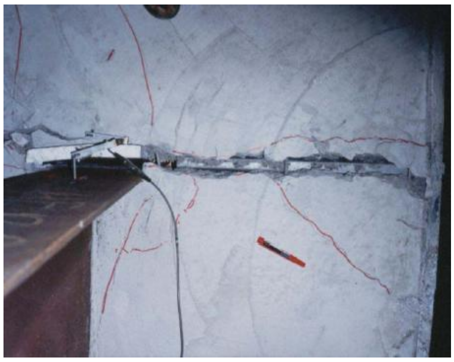


Figure 6.8. Slab damage in a post-tensioned steel beam subassembly (Clifton, 2005).

6.5.2 Methods of avoiding slab damage

There are several methods of avoiding the damage to slabs caused by frame elongation described above. The two principal methods are to use "articulated" floors or top-hinging beams.

Articulated floors

An "articulated" flooring system is built so that it is partially detached from the supporting structure, with sliding joints or other innovative details, to avoid damage to the floor but to retain the essential diaphragm action. Possible details for articulated floors are given in Chapter 7 for concrete structures and Chapter 8 for steel structures, but there is continuing debate about their effectiveness. At this point in the development of damage-resistant buildings, there are aspects of emerging technologies that may require further research, and one is the divergence of opinion on how to progress and apply the variations of "articulated floor" systems.

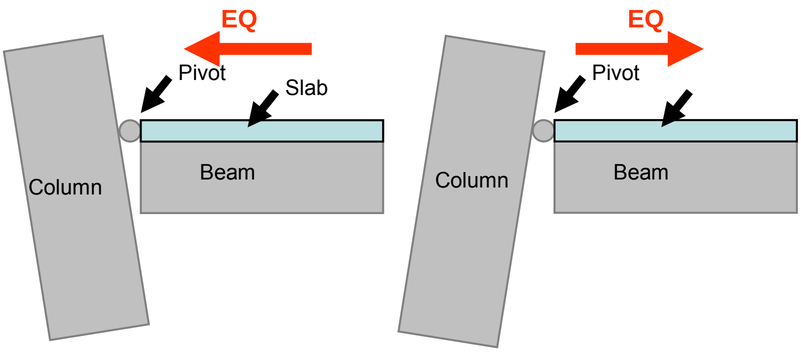
Top-hinging beams

An alternative method of preventing damage to floors due to frame elongation is to design the beam to column connections such that there is a top hinge, which will allow ductile rotation without causing damage to the slab. One variation of this is the "slotted beam" detail where a reinforced concrete beam has a slot formed into the lower half of the beam, as described further in Chapter 7. A similar detail is the sliding hinge joint for steel structures described in Chapter 8.

The term "non-tearing floors" pertains to a beam-column-floor configuration that results in very small cracks in the floor next to the beam-column junctions. Typically, the configuration involves forming a hinge or pivot at floor level. A schematic of the floor pivot is shown in Figure 6.8. The slotted beam has conventional reinforcing bars crossing the slot between the beam and column, so it generates flexural strength in a similar manner to a conventional beam. Figure 6.9(a) shows how the slot opens as a designed gap, and the reinforcement crossing the slot can be seen. Figure 6.9(b) shows the cracks that form in conventional beams. It is evident that the amount of damage in the slotted beam/ "non-tearing floor" specimen is considerably less than that seen in the conventional reinforced concrete beam (at a lesser inter-storey drift). The comparative lower damage was mirrored in the floor slabs of the two test specimens.

The very minor cracking shown in Figure 6.10(a) is considerably less disruptive to the flow of forces across the floors than the beam cracking shown in Figure 6.10(b). The minor cracking in the top-hinging floor will typically allow buildings to be reoccupied immediately after a major earthquake, with no repair required. Recent studies for concrete beams, at the University of Canterbury by (Amaris, 2007), (Au, 2010) and (Leslie, 2010), have confirmed the technical viability of this method. Au and Leslie focused on a conventional reinforced slotted beam – see Figure 6.10(a), while Amaris developed a hybrid version of slotted beam, "non-tearing floor" that incorporated aspects of the PRESSS technology, including the ability to re-centre the buildings (because of post tensioning in the beams and gravity effects on rocking columns) as described in Chapter 7.

Frames with "non-tearing floor" connections can be married with rocking wall systems, with or without post tensioning and rocking structural steel braces, as described in Chapter 6. "Non-tearing floor" solutions have been developed for buildings primary frames or walls constructed with concrete, structural steel, or timber.



(a) Right to left earthquake movement. (b) Left to right earthquake movement.

Figure 6.9. Schematic of the pivot or hinge detail.

(a) (b)

Figure 6.10. (a) Slotted reinforced concrete beam after 4.5% peak drift (Au, 2010). (b) Damage to conventional beam after 3.0% peak drift (Lindsay, 2004).

References

Amaris, A., Pampanin, S., Bull, D. and Carr, A., (2007). Development of a Non-tearing Floor Solution for Jointed Precast Frame Systems. Proceedings of The New Zealand Society of Earthquake Engineering Annual Conference, Palmerston North, New Zealand.

Au, E., (2010), "The Mechanics and Design of a Non-tearing Floor Connection using

Slotted Reinforced Concrete Beams", ME Thesis (2010), Department of Civil and Natural Resources Engineering, University of Canterbury Christchurch, New Zealand.

Clifton, G.C., (2005). Semi-Rigid joints for moment resisting steel framed seismic resisting systems. PhD Thesis, Department of Civil and Environmental Engineering, University of Auckland.

Douglas, K.T., (1992). Elongation in Reinforced Concrete Frames. PhD Thesis, Department of Civil Engineering, University of Auckland.

Fenwick, R.C. and Megget, L.M., (1993). Elongation and Load Deflection Characteristics of Reinforced Concrete Members containing Plastic Hinges. Bulletin of the New Zealand National Society for Earthquake Engineering, 26 (1). 28-41.

Fenwick, R.C. and Fong, A., (1979). The Behaviour of reinforced concrete beams under cyclic loading , University of Auckland, School of Engineering, Auckland, New Zealand

Lindsay, R., (2004). "Experiments on The Seismic Performance of Hollow-Core Floor Systems in Precast Concrete Buildings", ME Thesis (2004), Department of Civil and Natural Resources Engineering, University of Canterbury Christchurch, New Zealand.

Matthews, J., Bull, D., and Mander J., (2003). Hollowcore floor slab performance following a severe earthquake. Concrete Structures in Seismic Regions. Proceeding of the First fib Symposium. Athens, Greece.

Pampanin, S., (2010). "Damage-control self-centering structures: from laboratory testing to on-site applications", Chapter 28 in "Advancements in Performance-Based Earthquake Engineering (M FardisEditor) Publisher Springer,Part 3, pp. 297-308

Park, R., Paulay, T. and Bull, D., (1997). "Seismic Design of Reinforced Concrete Structures", *Technical Report No. 20, New Zealand Concrete Society*. 1997, 135 pp

Peng, B., (2009). Seismic Performance Assessment of Precast Concrete Buildings with Precast Concrete Floor Systems. PhD Thesis, University of Canterbury.

Priestley, M.J.N., Calvi, G.M. and Kowalski, M.J. (2007). Displacement Based Seismic Design of Structures. IUSS Press, Italy.

Priestley, M.J.N. and MacRae, G.A., (1996). Seismic Tests of Precast Beam-to-Column Joint Sub-assemblages with Unbonded Tendons, PCI Journal, January-February; 64-80.

Priestley, M.J.N., Sritharan, S., Conley, J.R. and Pampanin, S., (1999). "Preliminary results and conclusions from the PRESSS five-story precast concrete test building." PCI Journal, 44(6), 42-67.

Stanton, J.F., Stone, W.C., and Cheok, G.S., (1997). A hybrid reinforced precast frame for seismic regions, PCI Journal, Vol. 42, No.2, pp.20-32

7 DAMAGE RESISTANT DESIGN OF CONCRETE STRUCTURES

7.1 Jointed Ductile "Articulated" Systems – PRESSS-technology

In PRESSS (PREcast Seismic Structural System) frame or wall systems (Figure6.1), precast concrete elements are jointed together with unbonded post-tensioning tendons or steel bars creating a moment-resisting structure, with all the advantages associated with such a robust structural scheme. Full scale testing of frames and walls is shown in Figure 7.1.

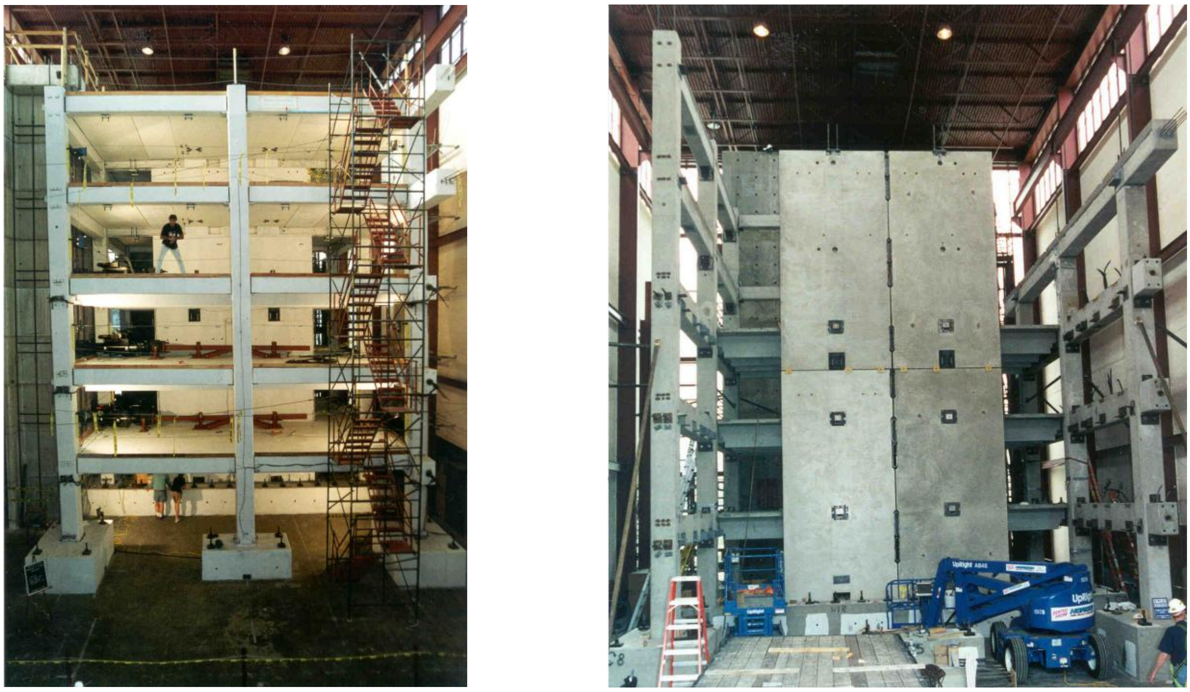


Figure 7.1. Five-Storey PRESSS Building tested at University of California, San Diego (Priestley et al., 1999).

In case of earthquake shaking, the inelastic demand is accommodated within the connection itself (beam-column, column-to-foundation or wall-to-foundation interface), through the opening and closing of existing gaps in a rocking motion. The gap opening or rocking acts as a fuse or isolation system with no damage accumulating in the main structural elements which are basically maintained in the elastic range. The basic structural skeleton of the building can thus remain undamaged after a major earthquake without any need for structural repair.

This is a major improvement when compared to cast-in-situ reinforced concrete solutions where, as mentioned, damage is expected to occur in the plastic hinge regions, leading to substantial costs of repairing and major business interruption.

7.2 The Hybrid System: Concept and Mechanism

A particularly promising and efficient solution within the family of jointed ductile connections is given by the "hybrid" system (Stanton et al. 1997, Fig. 7.2), where the connection reinforcement is given by the combination of unbonded post-tensioned bars or tendons and non-pre-stressed mild steel (or similarly additional external energy dissipation devices as discussed in the previous sections), inserted in corrugated metallic ducts and grouted to achieve fully bond conditions.

Under static loading considerations (no seismic actions), these two types of reinforcement can guarantee a high level of connection strength and stiffness, with reduced congestion of the joint-connection region and easy installation process. Clearly, as for any structural system relying upon a moment resisting connection at the beam-column interface and taking advantage of the benefits of pre-stressing, longer span lengths can be achieved with reduced beam depths.

Under wind loading and low-seismic actions the clamping action of the post-tensioned bars/tendons guarantee a very stiff initial condition (gross section) when compared to a typical cast-in-situ solutions (cracked sections), thus resulting into immediate benefits under serviceability loading conditions.

Under moderate to high seismic actions, the traditional plastic hinge mechanism is replaced by this controlled rocking mechanism (gap-opening and closing) at the critical interface without any structural damage (Fig. 7.3) in the structural elements. While the tendons provide self-centering and restoring actions, the mild steel bars or other similar devices (for example coupling steel bars or U-shaped flexural plates between adjacent rocking walls) act as energy dissipaters and shock absorbers for the structure under seismic loading. This particular dissipative and re-centering mechanism is described by "flag-shape" hysteresis behaviour (force-displacement or moment-rotation cyclic behaviour) as shown in Figure7.5.

A damage-control limit state can thus be achieved under a design level earthquake (typically that set at a 500 years return period, refer to Performance Objective Matrix described above) leading to an intrinsically high-performance seismic system almost regardless of the seismic intensity .

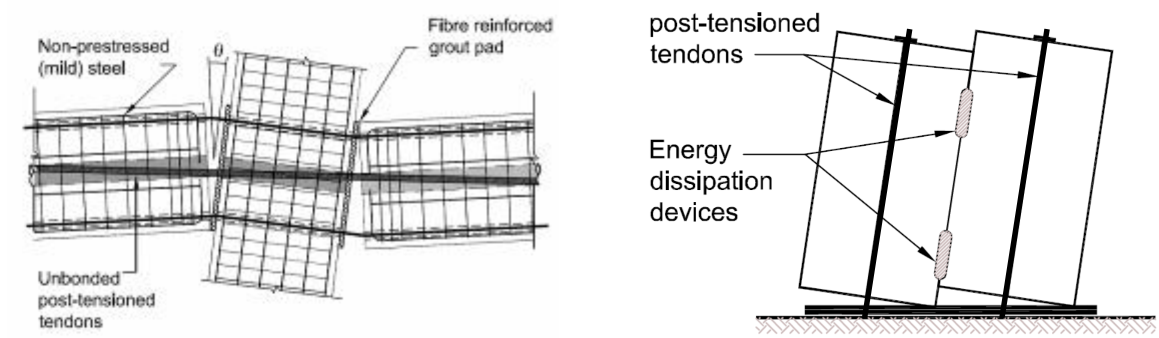


Figure 7.2. Jointed precast "hybrid" frame and wall systems developed in the PRESSS-Program (modified from fib, 2003; NZS3101:2006).

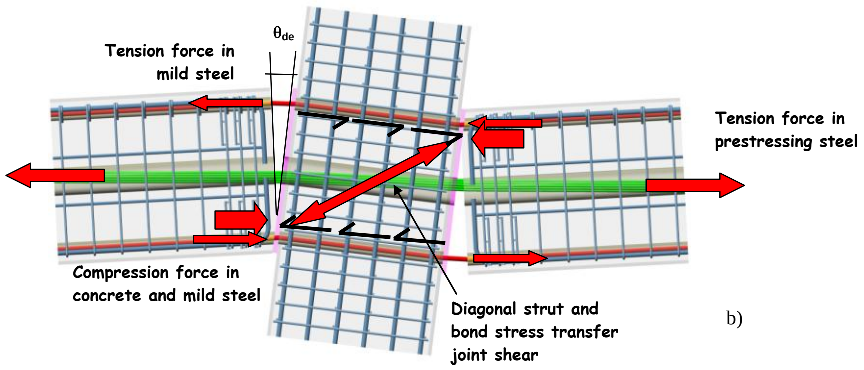


Figure 7.3. Hybrid beam-column connection: controlled rocking mechanism (courtesy of Mrs. Suzanne Nakaki).

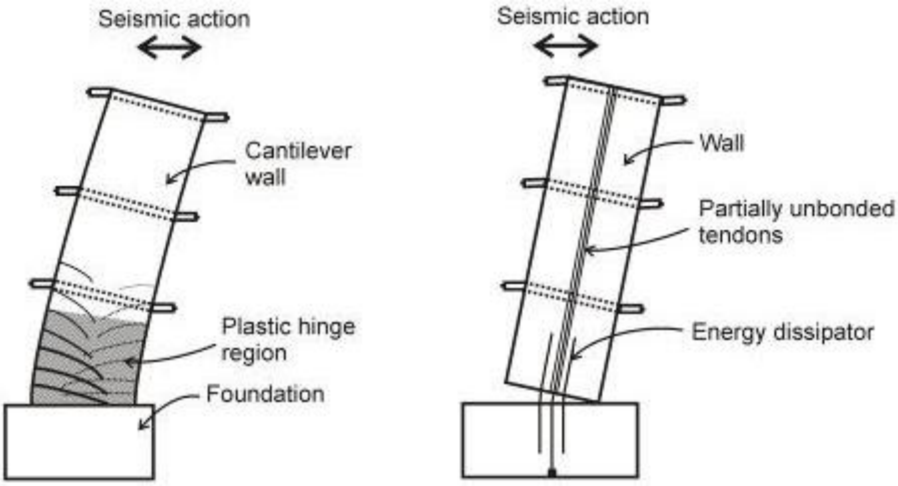


Figure 7.4. Comparison of monolithic wall and jointed precast rocking wall (after fib, 2003).

For a lateral load resisting system of structural walls, Figure 7.4 shows the comparative response of a traditional monolithic wall system (damage in the plastic hinge) and a jointed precast rocking wall solution (no damage and negligible residual deformations).

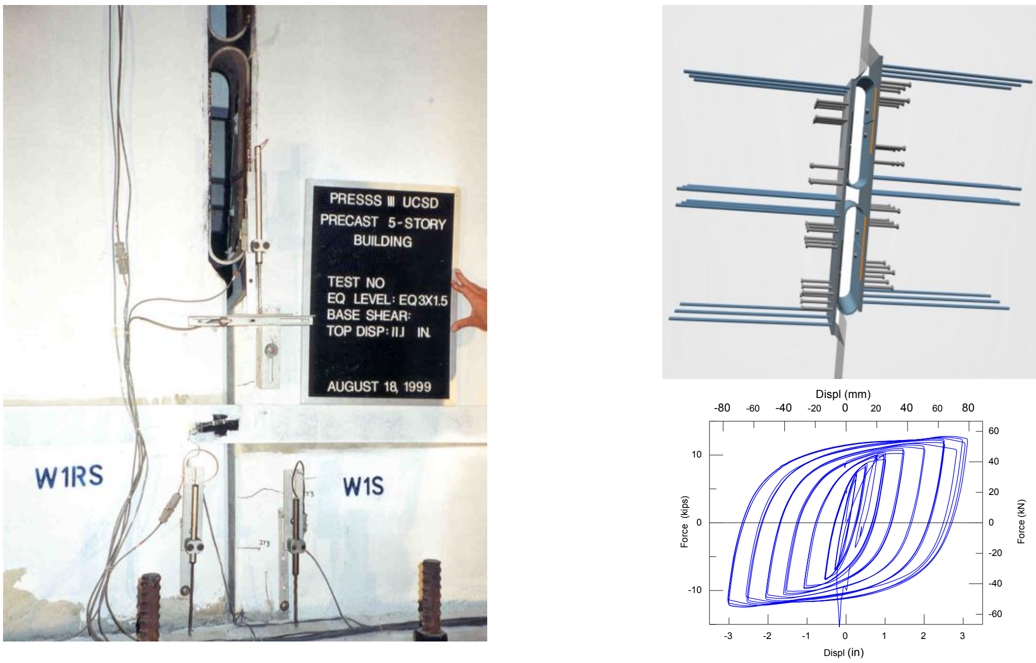


Figure 7.5. Rolling displacement behaviour of U-shape Flexural Plate Dissipaters between rocking walls

7.3 Replaceable Fuses – External Plug & Play Dissipaters

Following the declared target of a no-damage structural system (or at least a very low-damage system), significant effort has been dedicated in the past few years at the University of Canterbury to the development of cost-efficient externally located (*"Plug & Play"*) dissipaters, which can be easily demounted and replaced after an earthquake event, if required (Pampanin, 2005). This option allows a modular system with replaceable sacrificial fuses at the rocking connection, acting as the "weakest link of the chain", according to capacity design principles. Figures 7.6, 7.7.

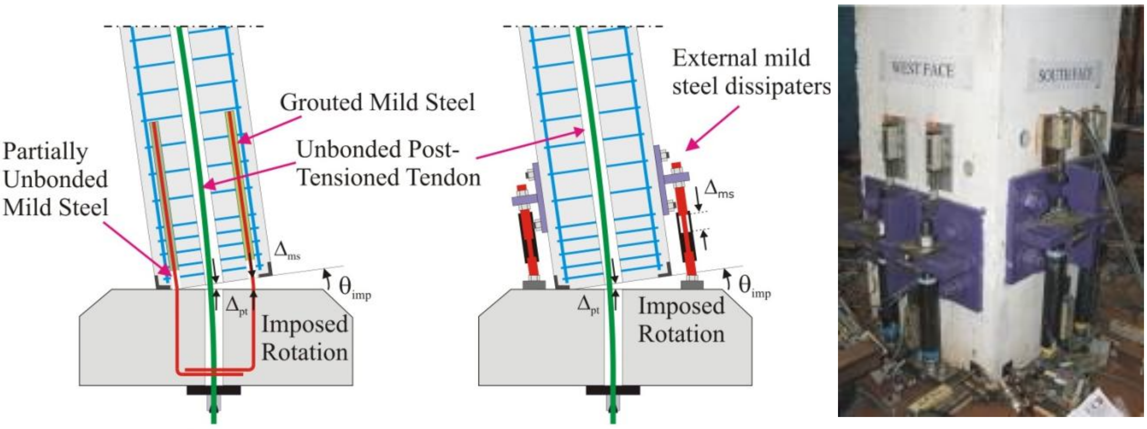


Figure 7.6. Internal versus external replaceable dissipaters at the base connections (after Marriott et al. 2008a).

(a) (b) (c)

Figure 7.7. Alternative configurations of external dissipaters for hybrid systems: (a) and (b) beam-column connections, with recesses in the beam (from Pampanin et al., 2006); (c) wall to foundation connection (from Marriott et al., 200b).

It is worth noting that the controlled rocking mechanism used in PRESSS-technology can be further improved by merging the advantages of the best energy dissipation and supplemental damping devices options. In terms of material and type of dissipation, either metallic or other advanced materials (e.g., shape memory alloys or visco-elastic systems) can be used to provide alternative dissipation mechanisms including elasto-plastic axial or flexural yielding, or friction devices.

7.4 Preventing Damage to Floors

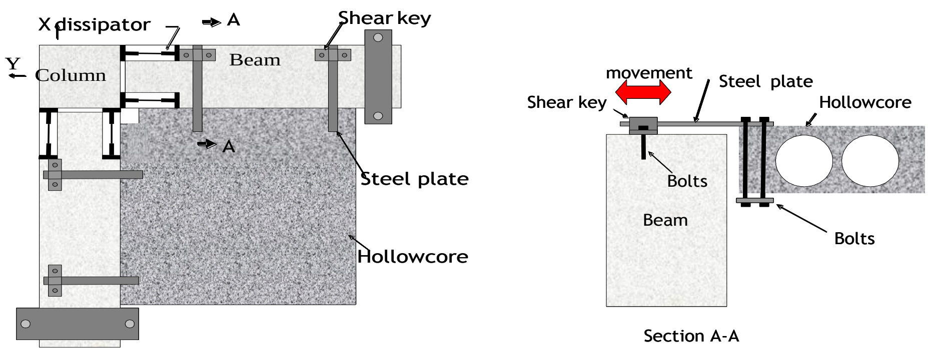
As described in Chapter 5, there are several methods of avoiding the damage to slabs caused by frame elongation of rocking systems. The two principal methods are “articulated” floors or top-hinging beams.

7.4.1 Articulated floors

An “articulated” flooring system is built so that it is partially detached from the supporting structure, with sliding joints or other innovative details, to avoid damage to the floor but to retain the essential diaphragm action.

According to this proposed solution, the floor is connected to the lateral beams by slider/shear mechanical connectors, acting as shear keys when the floor moves (relatively) in the direction orthogonal to the beam and as sliders when the floor moves in the direction parallel to the beam. In theory, the system is able to accommodate the displacement incompatibility between floor and frame by creating an articulated or jointed mechanism, which is effectively decoupled in the two directions. A similar solution was used in the five-storey PRESSS building tested at the University of California, San Diego in 1999 (Priestley et al., 1993) in the form of welded X-plate mechanical connectors (shown in Fig. 7.8) between the double-tee floor members and the frame beams. See Pampanin et al., (2006) andAmaris et al., (2007) for more details of possible articulated flooring systems, including the possible system illustrated in Figure 7.9.

Figure 7.8. Discrete X-plate mechanical connectors used in the PRESSS five storey test building at San Diego (after Priestley et al., 1999).



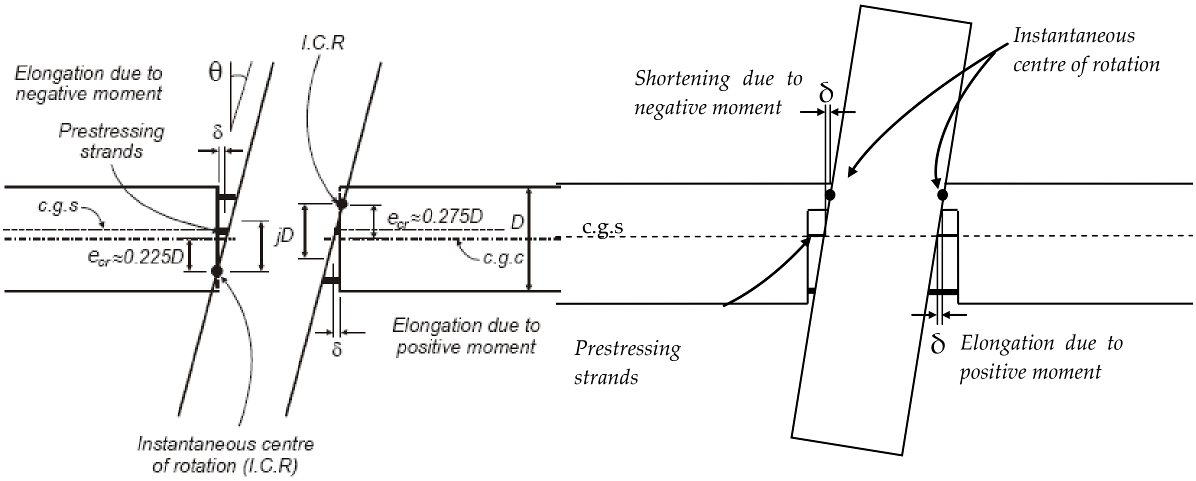
(a) Plan view. **(b) cross section at edge of slab.**

Figure 7.9. Beam-column joint with articulated floor unit at a corner of a reinforced concrete frame building (Amaris et al., 2007).

Another design option which will minimise floor damage 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 are minimised. This can be combined with partial de-bonding of the reinforcing bars in the concrete topping, and a thin cast-in-situ slab in the critical areas, to further increase the deformation compatibility.

7.4.2 Top-hinging beams

An alternative method of preventing damage to floors due to frame elongation is to design the beam to column connections such that there is a top hinge at the beam to column connection, which will allow some ductile rotation without causing damage to the slab. One variation of this is the "slotted beam" detail where a reinforced concrete beam has a slot formed into the lower half of the beam. This is shown in its simplest form in Figure 7.10.



(a) Interior plastic hinge lever arms for a conventional connection (Lindsay 2004).

(b) Interior plastic hinge lever arms for a non-tearing connection

Figure 7.10. Comparison of (a) Gapping connection and (b) Top-hinge solution.

The development of this concept starts from the evolution of the Tension-Compression Yield–Gap connection (TCY-Gap), developed during the PRESSS-Program, using mild-steel bars on the top (inserted into grouted sleeves) and unbonded post-tensioned tendons at the bottom. The peculiarity of this system was that beams and columns were separated by a small gap, partially grouted at the bottom to avoid the primary beam-elongation effects, thus not affecting the centre-to-centre distance between columns. However, such a solution would not prevent the tearing action in the floor due to the opening of the gap at the top of the beam, and no re-centering contribution is provided by the tendons, which are located with a straight profile in the centre of the compression grout and are thus not elongating.

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 has led to the development of the "non-tearing floor" beam-column connection developed and tested at the University of Canterbury (Amaris et al., 2007), which could be combined with any traditional floor system. Solutions are available with or without post-tensioning, providing an alternative "non-tearing" floor system able to exploit the advantages of the PRESSS-technology while still relying on more traditional floor-to-frame connections (i.e., topping and continuous starter bars). See Figure 7.11 for more details of alternative seating details for top-hung beam-column joint mechanisms.

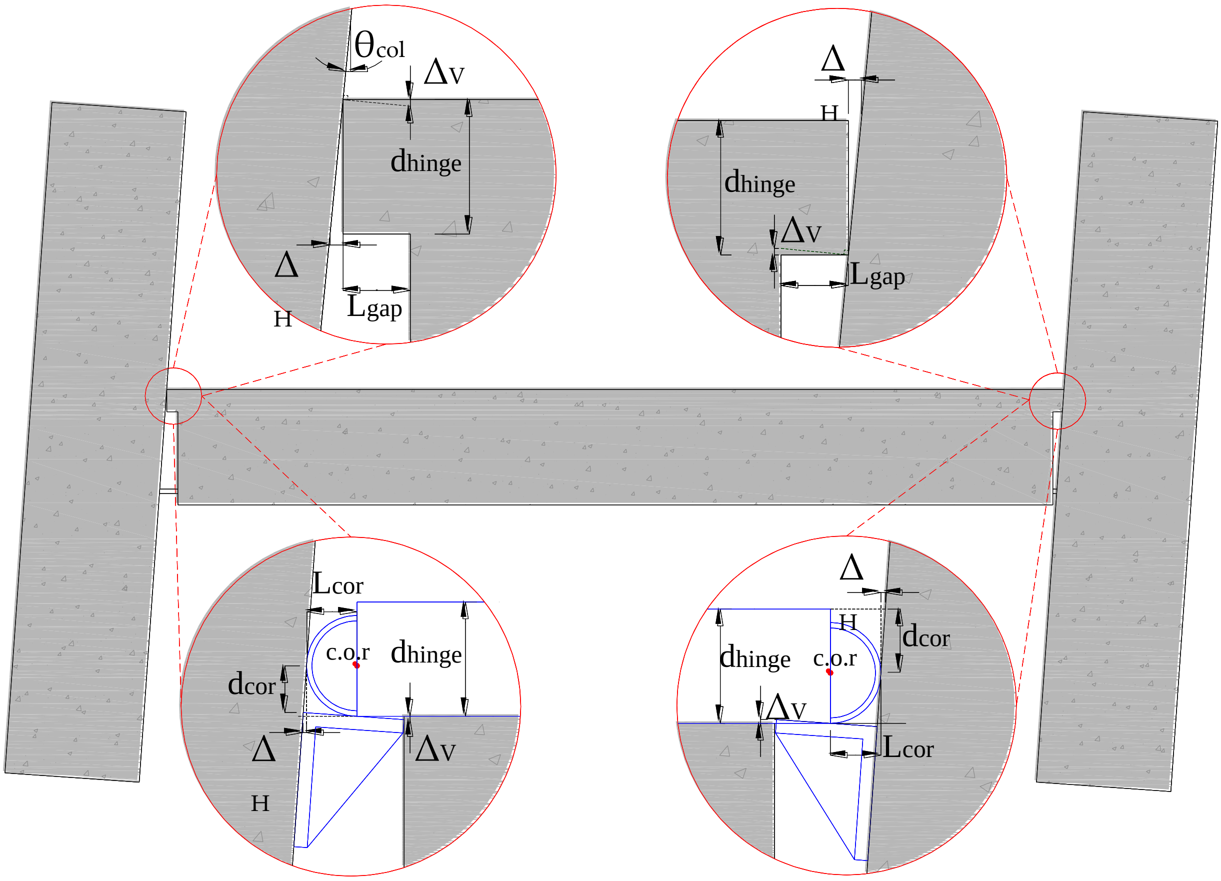


Figure 7.11. Deformation components of rectangular and circular hinge profiles under opening and closing rotations. Leslie, B., Bull, D., Pampanin, S. 2009

7.5 Examples of On-Site Implementations of PRESSS-Technology

Several on site applications of PRESSS-technology systems based on jointed ductile connections have been implemented in several earthquake-prone countries around the world.

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

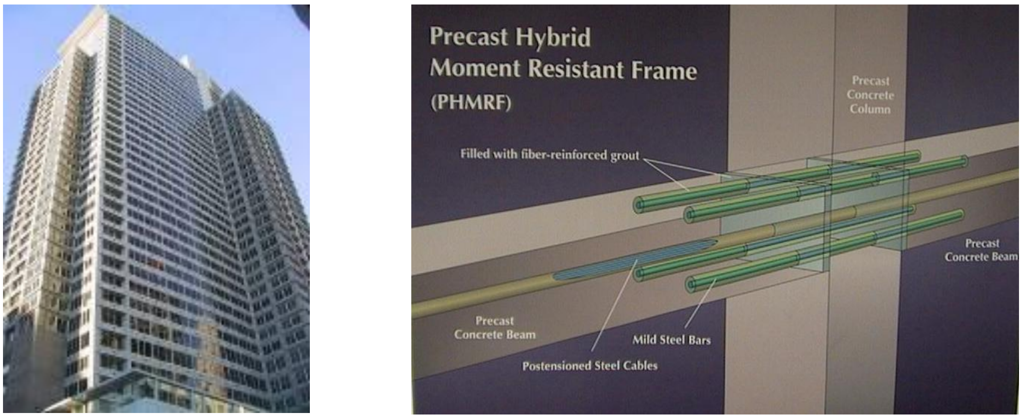


Figure 7.12. 39-storey Paramount Building, San Francisco (Englerkirk, 2002, photos courtesy of Pankow Builders, E. Miranda, Len McSaveney).

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 have been implemented in earthquake-prone countries around the world, including but not limited to USA, central and South America, Europe and New Zealand. One of the first and most glamorous applications of hybrid systems in high seismic regions was the Paramount Building in San Francisco (Figure 7.12) consisting of a 39-storey apartment building, the tallest precast concrete structure in a high seismic zone (Englerkirk, 2002). Perimeter seismic PRESSS frames were used in both directions.

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" (Figure 7.13), developed by BS Italia, Bergamo, Italy, with draped tendons for longer spans and a hidden steel corbel (Pampanin et al., 2004).

More than ten buildings, up to six storeys, have been designed and constructed in regions of low seismicity (gravity-load dominated frames). These buildings have different uses (commercial, exposition, industrial, hospital), plan configurations, and floor spans. A brief description has been given in Pagani (2002), Pampanin et al. (2004).

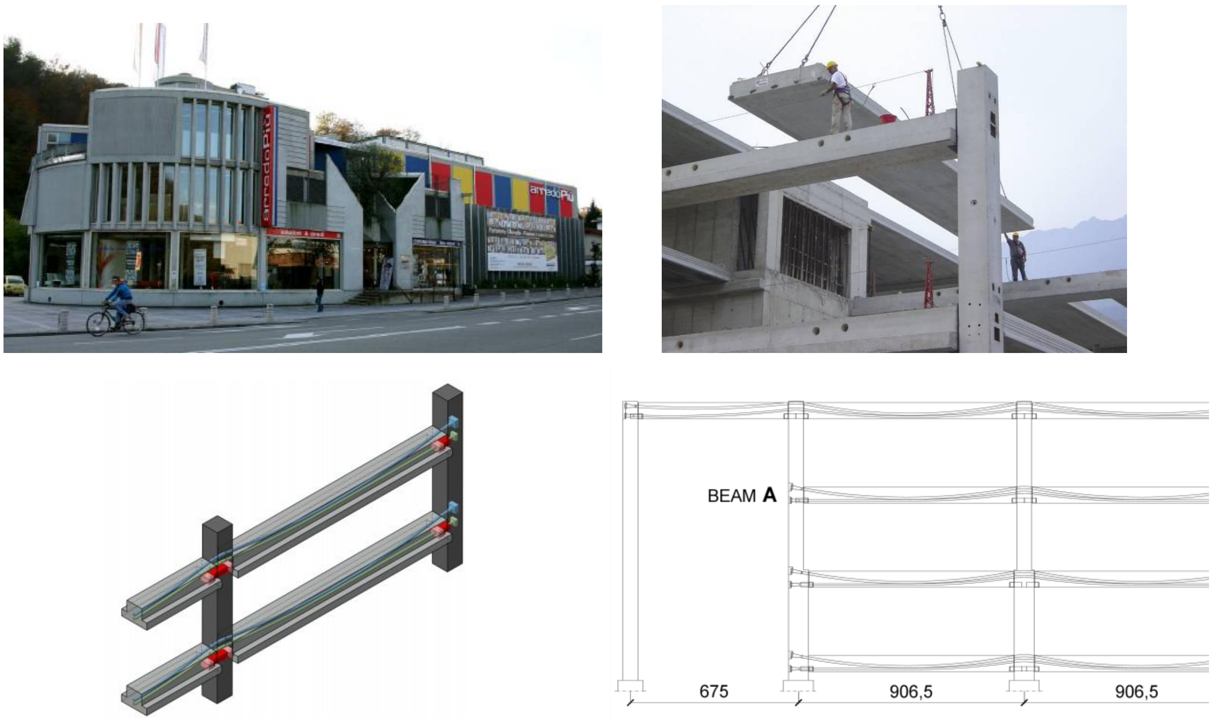


Figure 7.13. Photographs and sketches of the post-tensioned "Brooklyn System" (Pampanin et al., 2004).

Alan MacDiarmid Building, Wellington

The first multi-storey PRESSS-building in New Zealand is the Alan MacDiarmid Building at Victoria University of Wellington (Figure 7.14), 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 (Cattanach and Pampanin, 2008). This building was awarded the NZ Concrete Society's Supreme Award in 2009.

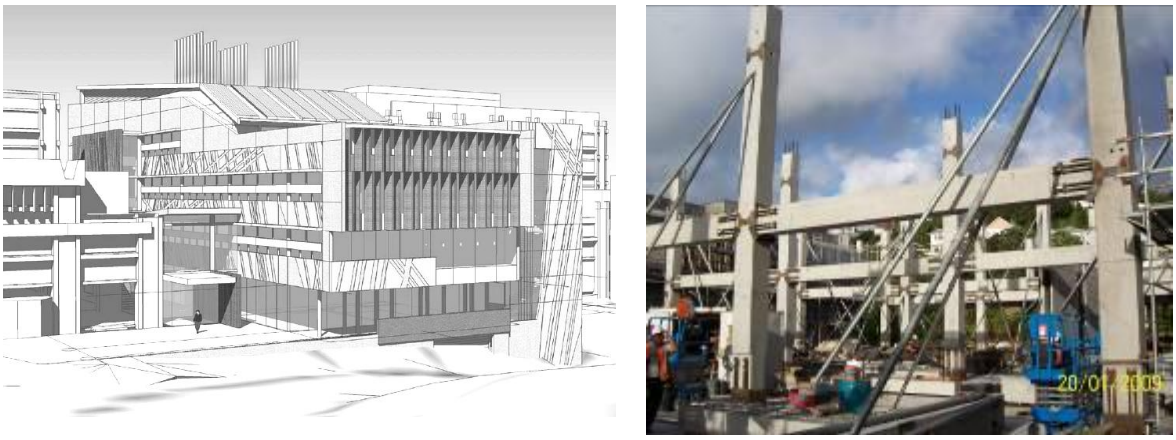
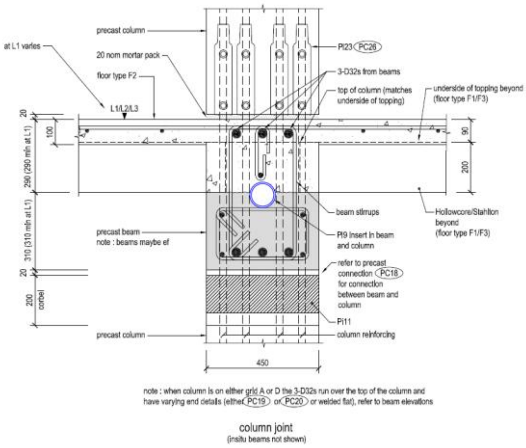


Figure 7.14. First multi-storey PRESSS-Building in New Zealand (Cattanach and Pampanin, 2008).

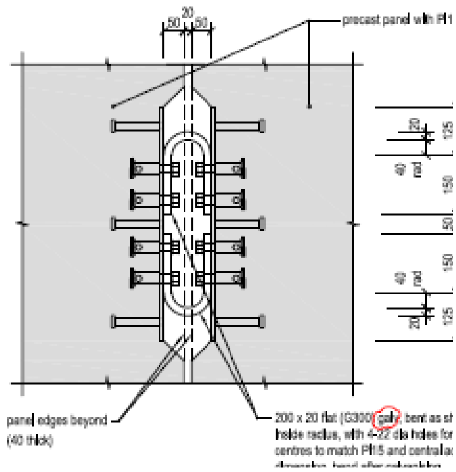
Southern Cross Hospital, Christchurch

(a)



(b)

(c)



(d)

Figure 7.15. Southern Cross Hospital, Christchurch (a) Finished building, (b) Beam-column detail,(c) Frame during construction, (d) dissipaters between coupled walls(Pampanin et al., 2011).

The design and construction of the second PRESSS-Building in New Zealand and the first in the South Island is the Endoscopy Consultants' Building in Christchurch, designed for Southern Cross Hospitals Ltd by Structex Metro Ltd (Figure 7.15). Again in this case both frames and coupled walls have been used in the two orthogonal directions. The unbonded post-tensioned walls are coupled by using the aforementioned U-Shape Flexural Plates dissipaters.

7.6 Testing of Seismic Performance in the Christchurch Earthquakes

Figure 7.16. Negligible damage, to both structural and non-structural components, in the Southern Cross Hospital after the earthquake of 22 February.

The Southern Cross hospital PRESSS-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.

As a comparison, Figure 7.16shows the minor/cosmetic level of damage sustained by the structural systems (post-tensioned hybrid frames in one directions with post-tensioned hybrid walls coupled with U-shape Flexural Plate Dissipaters) in the Southern Cross

Hospital Endoscopy Building. Furthermore, the medical theatres with very sophisticated and expensive machinery were basically operational the day after the earthquake.

One of the main feature in the design of a rocking-dissipative solution is in fact the possibility to tune the level of floor accelerations (not only drift) to protect both structural and non-structural elements including content and acceleration-sensitive equipment. More information on the design concept and performance criteria, modelling and analysis, construction and observed behaviour of the building can be found in Pampanin et al., (2011).

References

Amaris, A. Pampanin, S., Bull, D & Carr, A., (2007). Development of a Non-tearing Floor Solution for Jointed Precast Frame Systems. Proceedings of The New Zealand Society of Earthquake Engineering Annual Conference, Palmerston North, New Zealand.

Au, E., (2010). "The Mechanics and Design of a Non-tearing Floor Connection using Slotted Reinforced Concrete Beams", ME Thesis (2010), Department of Civil and Natural Resources Engineering, University of Canterbury Christchurch, New Zealand.

Bradley B. A., Dhakal, R. P., Mander J. B., and Li L., (2008). "Experimental multi-level seismic performance assessment of 3D RC frame designed for damage avoidance". *Earthquake Engineering and Structural Dynamics*, Vol. 37, No. 1, pp. 1-20.

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

Englerkirk, R., (2002). "Design-Construction of the Paramount – a 39 story Precast Prestressed Concrete apartment Building" PCI Journal, Vol. 47, No. 4

fib, (2003). International Federation for Structural Concrete, 2003. Seismic Design of Precast Concrete Building Structures, Bulletin 27, Lausanne, 254 pp

Kam, W.Y., Pampanin, S., (2011). " General Performance of Buildings in Christchurch CDB after the 22 Feb 2011 Earthquake: a Contextual Report", Department of Civil and Natural Resources Engineering, University of Canterbury

Leslie, B.J., Bull, D.K. & Pampanin, S., (2010). "The Seismic Performance of a Non-Tearing Floor Precast Concrete Structural System", in *Proceedings, New Zealand Society for Earthquake Engineering Conference*, Wellington, New Zealand, 2010.

Li L., Mander J. B., and Dhakal R. P., (2008). "Bi-directional cyclic loading experiment on a 3-D beam-column joint designed for damage avoidance". *ASCE Journal of Structural Engineering,* Vol. 134, No. 11, pp. 1733-1742.

Lindsay, R., (2004). "Experiments on The Seismic Performance of Hollow-Core Floor Systems in Precast Concrete Buildings", ME Thesis (2004), Department of Civil and Natural Resources Engineering, University of Canterbury Christchurch, New Zealand.

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

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

NZS 3101, (2006). "Appendix B: Special Provisions for the Seismic Design of Ductile Jointed Precast Concrete Structural Systems" Standards New Zealand, Wellington.

Ohkubo, M., and Hamamoto, T., (2004). "Developing Reinforced Concrete Slotted Beam Structures to Reduce Earthquake Damage and to Enhance Seismic Structural Performance." 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada.

Pampanin, S., Amaris, A. and Palermo, A., (2006).Implementation and Testing of Advanced Solutions for Jointed Ductile Seismic Resisting Frames.FédérationInternationale du Béton - Proceedings of the 2nd International Congress, June 5-8, 2006 – Naples, Italy.

Pampanin S., (2005). Emerging Solutions for High Seismic Performance of Precast - Prestressed Concrete Buildings, Journal of Advanced Concrete Technology (ACT), invited paper, Vol. 3 (2), pp. 202-222.

Pampanin, (2010). "Damage-control self-centering structures: from laboratory testing to on-site applications", Series "Geotechnical, Geological, and Earthquake Engineering, Volume 13"; Chapter 28 in the book "Advancements in Performance-Based Earthquake Engineering (M Fardis Editor) Publisher Springer, ISBN: 978-90-481-8745-4 (Print) 978-90-481-8746-1 (Online), Part 3, pp. 297-308

Pampanin, S., Kam W., Haverland, G., Gardiner, S., (2011). Expectation Meets Reality: Seismic Performance of Post-Tensioned Precast Concrete Southern Cross Endoscopy Building During the 22nd Feb 2011 Christchurch Earthquake" NZ Concrete Industry Conference, Rotorua, August

Pampanin, S., Kam, W.Y., Akguzel, U., Tasligedik, S., Quintana Gallo, P, (2011). " Seismic Performance of Reinforced Concrete Buildings in the Christchurch CBD after the 22 February2011 Earthquake", Department of Civil and Natural Resources Engineering, University of Canterbury

Pampanin, S., Pagani, C., Zambelli, S., (2004). "Cable Stayed and Suspended Solution for Precast Concrete Frames: the Brooklyn System", Proceedings of the New Zealand Concrete Industry Conference, Queenstown, New Zealand, September 16-18.

Priestley, M. J. N., Sritharan, S., Conley, J. R. and Pampanin, S., (1999). "Preliminary results and conclusions from the PRESSS five-story precast concrete test building." PCI Journal, 44(6), 42-67.

8 DAMAGE RESISTANT DESIGN OF STEEL STRUCTURES

8.1 Background

Most modern multi-storey steel structures in New Zealand are designed using moment frames, eccentrically braced frames or a combination of both. There is also a significant number of low and medium rise concentrically braced frame buildings. They typically use shop welding and site bolting construction. While they are permitted to be designed for high ductility (up to a structural ductility factor of 4), many structures are designed for low ductility, or even elastic response. This is because considerations other than earthquake strength, such as gravity load effects, wind loading, or earthquake inter-storey drift limitations, often control the member sizes. Member over strength including slab effects, and the presence of non-structural elements in the structure, which are not considered directly in design, can also increase the strength to the extent that the multi storey steel buildings will almost respond elastically, with no damage during a design level earthquake.

As a result, in the 2010 and 2011 Canterbury earthquakes, even though the shaking was significantly greater than the design level, steel buildings on the whole behaved very well by not only satisfying their "life safety" mandate (Bruneau et al., 2010), but also by being able to be occupied immediately after the earthquake. Minor yielding did occur in most structures, there was some gypsum board damage, and some elevators needed to be realigned, but the buildings continue to be used. This is a tribute to the modern design of well-detailed steel structures. There were no deaths or major injuries reported from these buildings. It is clear from the performance in Christchurch that, for design level shaking, modern well designed and built steel construction may well be described as being "damage resistant".

There were isolated examples of more significant damage, but these could be specifically traced back to poor detailing or construction leading to compromised load paths. There were no examples of unexpected poor behaviour from a detail or structural system. One example was due to fracture of the weld at the end of a brace. Another, in a parking structure, was due to braces not lining up with stiffeners, causing fracture of an eccentrically braced link, as shown earlier. Others suffered foundation damage and the ground floor slab was broken, or the footings moved, but the frame itself did not lose its integrity.

Even though steel structures did behave remarkably well, the New Zealand steel industry has been aware that we can do better, and either proactively reduce the possibility of significant damage to major steel members or design and detail these members for rapid replacement. For example, the NZ Heavy Engineering Research Association Structural Steel Research Panel has been considering the possibility of designing all multi-storey steel buildings as"damage-resistant" in the next ten years (Mackinven et al. 2007).

8.1 Definition of Damage-Resistant Design

Before discussing the damage-resistant techniques, it is first necessary to define terms. It is actually not possible to design and build structures which are damage-resistant under all earthquakes, so the term "damage-resistant" should be used with care. In the context of this document, it simply means that there should be less damage than in existing construction during design level earthquake excitation. A structure which satisfies this criteria should be available for occupation immediately after experiencing large shaking (at design level) and might be ready for occupation in a short time after very large shaking, as discussed in Chapter 3.

8.2 Reasons for this Development

Specifically designed damage-resistant steel structures are being developed in New Zealand in order to increase the seismic sustainability of steel structures and to minimise losses due to (i) damage, and (ii) downtime. The importance of this was clearly illustrated by the 1994 Northridge earthquake in the USA, which caused significant damage to the welded connections of multi-storey, moment-resisting steel framed buildings. These buildings used techniques that are quite different to those used in New Zealand, with welding a limited number of large beam sized steel frames constructed with site welding of the beams to the columns. Japanese experience (Yamada et al. 2010) has shown that even code compliant frames can suffer undesirable failure modes in strong earthquakes.

There are many ways possibly ways of creating damage-resistance with structural steel. Some of the structures incorporating these devices are described below:

1. Elastic Structures (Section 8.4)
2. Moment-frame structures (Section 8.5)
   (a) Post-tensioned beams (PTB) (Section 8.5.1)
   (b) Asymmetric friction connection (AFC) (Section 8.5.2)
   (c) High-force-to-volume lead extrusion dissipater (HF2V) (Section 8.5.3)
3. Concentrically Braced Structures (Section 8.6)
   (a) Traditional (Section 8.6.1)
   (b) BRB (buckling restrained braces) (Section 8.6.2)
   (c) Friction Brace – SFC (symmetric friction connection) (Section 8.6.3)
   (d) Friction Brace – AFC (asymmetric friction connection) (Section 8.6.4)
   (e) HF2V Brace (Section 8.6.5)
   (f) Self-Centering Braces (Section 8.6.6)
4. Eccentrically Braced Structures (eccentrically braced frames, EBF) (Section 8.7)
   (a) Replaceable Link (Section 8.7.1)
   (b) AFC Link (Section 8.7.2)
   (c) AFC Brace (Section 8.7.3)
5. Rocking Structures (Section 8.8)
6. Base-isolated structures (Section 8.9)
7. Supplementary damped structures (Section 8.10)
8. Base Connections (Section 8.11)

These systems will be discussed in detail in the remainder of the chapter. In order to describe the relative performance of each of these systems, aspects relating to their 'seismic sustainability" are described. It recognises that all "damage-resistant" structures are not equal. The approach taken by Chanchiet al. (2010) is followed where seismic sustainability was characterised qualitatively by:

(a) Structural damage
(b) Element replaceability
(c) Floor damage
(d) Permanent displacements

Damage to non-structural elements resulting from large drifts, displacements, or accelerations were not considered, as it is possible to detail these elements to either suffer significant damage, or no damage, in a traditional frame or in a "damage-resistant" frame.

8.3 Elastic Structures

Because of the high strength of steel, it is possible to design multi-storey steel structures to behave in an elastic manner in a design level earthquake. This is easier in zones of low seismicity, where the strength demands reduce more rapidly than the stiffness demands. They should still possess sufficient ductility to prevent a brittle failure in maximum credible event (MCE) shaking. These structures are likely to have no structural damage so there is no need to have replaceable elements. Residual displacements will also be very low.

8.4 Moment-Frame Structures

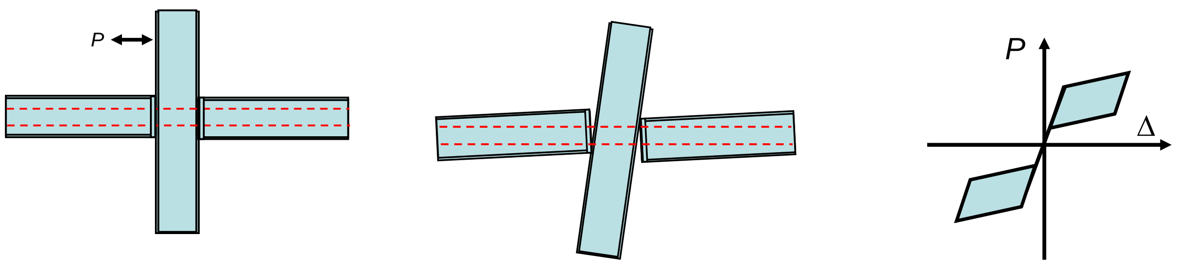
Many modern multi-storey steel structures in New Zealand are designed using moment frames. These are described below in several categories.

8.4.1 Frames with Post-Tensioned Beams or Spring Loaded Joints

One of the earliest systems using post-tensioned beams (PTB) was the PRESSS (PREcast Structural Seismic System) developed for concrete frames (Priestley and MacRae, 1996; Priestley 1997). This system has also been applied to steel frames (Danner and Clifton, 1994), (Clifton, 2005), and (Christopoulos, 2006).

The post-tensioned beam technology involves pre-stressing/post-tensioning prefabricated beams to the column face, as shown in Figure 8.1(a). During large lateral deformations, as may be expected from severe earthquake shaking, a gap opens between the end of the beam and the column face as shown in Figure 8.1(b). As the gap opens, the post-tensioning tendon extends providing additional force to close the gap. Different dissipaters may be placed over the gap to dissipate energy. The strength of the dissipaters should be small enough that after the earthquake shaking, the tendon pulls the structure back to its initial at-rest position. This is shown by the displacement at zero force always being zero, as shown in the hysteresis loop of Figure 8.1(c). Spring loaded joints work in a similar way, with the beams clamped to the columns with flat endplate connections and pre-compressed ring spring joints. When the gap opens, the beam rotates about the point of compression contact between the endplate and the column flange and the springs are further compressed, generating increasing moment with increasing rotation.

Tests of beam/column subassemblies with one column and without slabs have shown very good behaviour with no permanent displacements after the earthquake, and no significant damage. However, when the beam supports a slab, and/or when the beam is part of a frame that has more than one column, additional effects occur which may result in damage.



(a) Beam Column Joint **(b) Deformed Shape** **(c) Hysteresis**

Figure 8.1. Post-tensioned beam deformation and hysteretic behaviour (MacRae, 2010).

For frames with more than one column, the gaps which form at the beam ends cause "beam growth" or "frame expansion" as described in Chapter 6. Some efforts have been proposed to solve the issues associated with slab damage due to beam elongation. These have their own limitations as described below.

(a) Connecting the Beams to the Slab in One Bay Only (e.g., Lin et al., 2009)

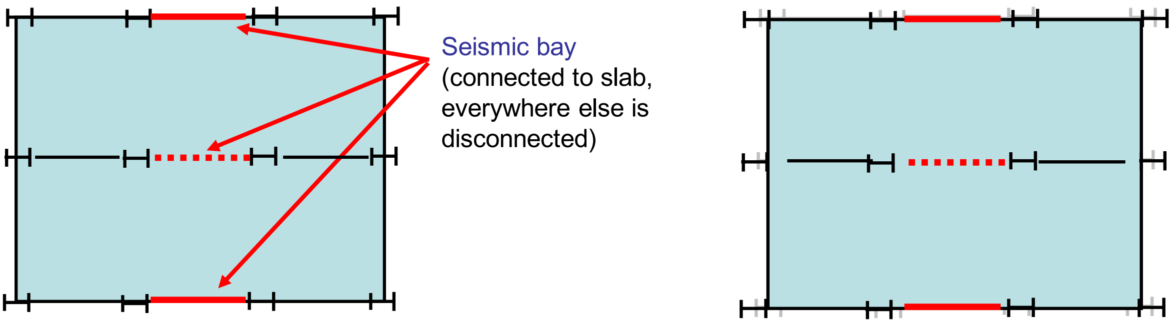
In order to prevent slab damage due to gap opening, it has been proposed that the slab be connected to only one beam in frame as shown in Figure 8.2(a). Here, the slab slides over non-seismic beams. While this seems attractive, and has been shown to work in the push-pull analyses of a 2-D frame, there are a number of issues including:

i) All of the diaphragm force is transferred to the beam over one bay. This means that axial force is imparted to one beam, rather than the full number of beams in the frame. As a consequence, beam axial forces will be significantly greater than in a traditional frame. Also, this effect will limit the number of bays in the frame.

ii) The exterior cladding has to be able to expand in the direction of shaking as shown in Figure 8.2(b). Special detailing of cladding would be required.

iii) The system requires the slab to slide dependably over the beams from which it is in theory "isolated". That requires careful attention to design and construction and to a good knowledge of the actual loading that will be on the regions of separated slab and beam. These are all factors difficult to accurately control thus making this concept difficult to accurately implement.

iv) The system cannot be easily applied in two horizontal directions using traditional approaches. The connections between the slab and the beam would need to allow sliding of the slab in the direction perpendicular to the direction of the beams, which is even more problematical than sliding parallel to the beams.

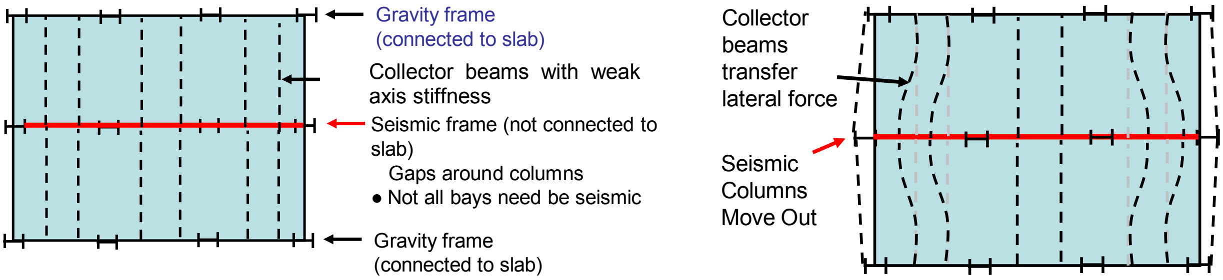


(a) Connections of slab to beams. **(b) Deformations of frame.**

Figure 8.2. PT beam connected to slab over one bay.

(b) Connecting the Beams to Gravity Frames Only (e.g., Garlock, 2009)

It has been suggested that the slab be connected only to the gravity frames in a building as shown in Figure 8.3(a). Gaps are provided between the slab and the seismic columns, and the slab slides on the seismic beams. Collector beams are provided with stiffness which transfer the lateral forces between the seismic and gravity frames. The deformations of the frame are shown in Figure 8.3(b).



(a) Connections of slab to beams. **(b) Deformations of frame.**

Figure 8.3. PT beam connected to slab over one bay.

This system also has a number of issues:

(a) It is sensitive to the beam weak axis stiffness. It the collector beams are too stiff, then there will be no gap opening. If they are too flexible, there will be no force transfer.
(b) Axial forces on the beams will be greater than if all beams were connected to the slab. Seismic frame beams will generally be in compression, while the gravity frame beams will tend to tension.
(c) The system is dependent on accuracy of construction to ensure the design slip planes are achieved and undesired slip planes suppressed
(d) The exterior of the structure is deformed, and this needs special consideration in design.
(e) Three-dimensional systems, using the approach described in two horizontal directions, is difficult.

It has been argued "post-tensioned steel systems can work well for buildings without floor slabs if the difficulty with the columns being pushed apart is ignored"! While this statement is an exaggeration, it does emphasise the need for very careful design/detailing of these frames if they really are to be considered as damage resistant systems.

Post-tensioned beam systems can be used in frames two ways:

i) They may be used simply to replace the traditional steel beam with a rigid connection. In this case, some column yielding may be expected and slab damage may be expected during design level shaking. However, the total damage may be less than that expected in a traditional New Zealand rigid weak beam - strong column moment frame buildings significant beam yielding may be expected.
ii) They may be used to minimise damage on the overall structure including slab and column damage. However, because of the issues described above associated with gapping, these frames will be boutique tailored structures, rather than general solutions. This is because they may only be a few bays wide and a few levels high. In the design, considerably more care is required than in the design of standard steel structures. Issues that should be considered include:
    - proper analysis to capture the likely effects of beam growth on the whole frame,
    - the influence on framing in the perpendicular direction
    - the detailing of the connections between the lateral systems and floor slabs for shaking in two horizontal directions.
    - detailing of the connections between the cladding and the structure to ensure that the cladding can accommodate the likely displacements.
    - ensuring the structure is built as detailed.

In addition to the issues described above, the structure may have a high increase in stiffness at high velocity while it is unloading and this can potentially cause other issues as described in the section on "rocking structures".

8.4.2 Asymmetric friction connection (AFC) in steel moment frames

The sliding hinge joint is an asymmetric friction connection (AFC) which was developed by Clifton while at the NZ Heavy Engineering Research Association. Initial tests were conducted at the University of Auckland (Danner and Clifton 1994; Clifton 2005) and further studies were conducted at the University of Canterbury (Mackinven et al., 2008). Asymmetric friction connections are considered to have considerable potential for damage-resistant design of steel moment-frame structures.

The sliding hinge joint has the components shown in Figure 8.4. The beam end is placed a distance equal to the "beam clearance" away from the column face. The beam top flange is connected to the column by means of the top flange plate. Rotation of the beam end occurs about the connection of the top flange plate to the column flange as shown. Because no sliding or gapping is expected between the beam, top flange plate and column, beam growth and slab damage are minimised. The shear force in the beam is carried by the top web bolts. Horizontally slotted holes are provided in the bottom flange plate and in the bottom holes of the column web plate to allow significant rotations of the beam end relative to the column face. A gap is provided between the end of the beam bottom flange and the column face. This gap is required to be large enough that the demand in and beside the weld connection to the column face is not too large. Below the bottom flange plate is the bottom flange cap plate. It may be described as a floating plate because it has no physical connection to the rest of the joint apart from through the bolts. A web cap plate is similarly placed on the outside of the web plate. On all surfaces where sliding may possibly occur, shims are placed. These shims may be manufactured of steel, brass or other materials. These have standard sized holes so sliding occurs on the side of the shim in contact with the bottom flange plate or web plate. High quality control may be maintained using shop welding site bolting techniques.

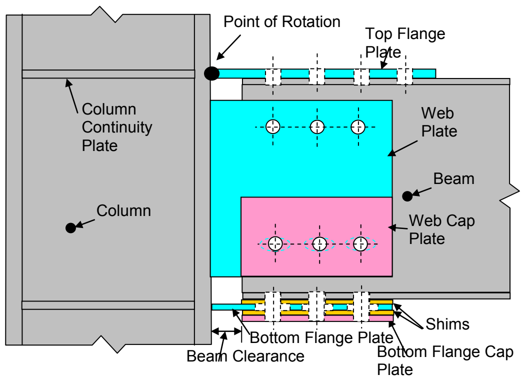
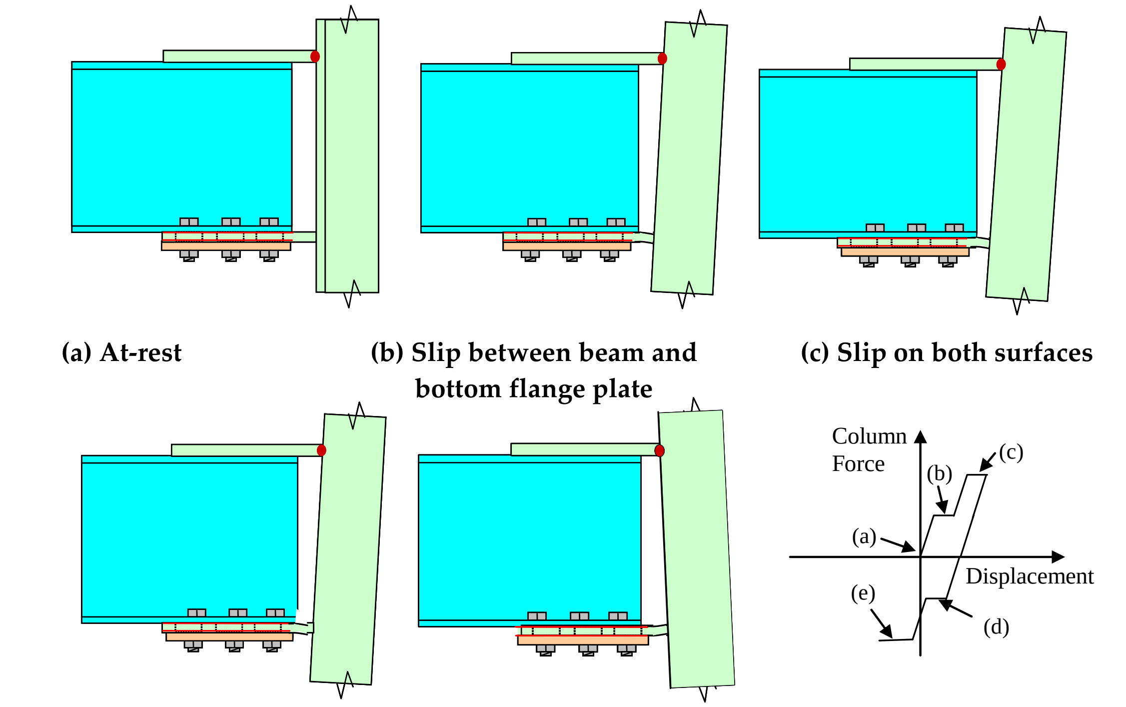


Figure 8.4. Flexural AFC SHJ Connection (MacRae and Clifton, 2010).

Figure 8.5illustrates how the sliding hinge joint works (MacRae, Clifton and Butterworth, 2009). Only the bottom flange friction surfaces are shown for simplicity. The column starts from rest as shown in Figure 8.5(a). As the top of the column moves to the right, slip occurs between the bottom of the beam flange and the bottom flange plate as shown in Figure 8.5(b). At this stage the bottom flange cap plate is not sliding because the shear force imposed on it is relatively small. As the deformations become greater, the bolts in the bottom flange move to such an angle that they provide sufficient force for slip to also occur between the bottom flange plate and the bottom flange cap plate as shown in Figure 8.5(c). Because the peak friction forces on either side of the bottom flange plate occur at different displacements, it is referred to as an "Asymmetric Friction Connection (AFC)". The slip on both surfaces causes approximately twice the resistance than from one surface as shown as (c) in Figure 8.5(f). When loading reverses, slip initially occurs only between the bottom of the beam flange and the bottom flange plate as shown in Figure 8.5(d), and at large displacements in the opposite direction, the bolts are pulling the floating plate in the opposite direction than before as shown in Figure 8.10e, again causing an increase in lateral resistance as shown in Figure 8.5(f).



(a) At-rest **(b) Slip between beam and bottom flange plate** **(c) Slip on both surfaces**

(d) Reverse loading **(e) Far in opposite direction** **(f) Force-displacement curve**

Figure 8.5. Simplified lateral force-displacement mechanism.

The hysteretic loops for the beam-column subassembly test in Figure 8.6(a) are shown in Figure 8.6(b). It can be seen that the hysteretic loop shape is not that of a traditional friction device (rectangular) but it is like a smeared out version of Figure 8.6(f). Such a curve has dynamic self-centring characteristics, so the permanent displacement would not be expected to be large.

(a) Test frame. **(b) Hysteretic behaviour.**

Figure 8.6. Test configuration and SHJ hysteresis with steel (S) shims.

The sliding hinge joint AFC system possesses the following desirable characteristics:

(a) by using elongated holes a large deformation capacity can be obtained,
(b) no patent fee is required,
(c) by using different numbers and sizes of bolts the strength can be controlled,
(d) the systems do not produce gap opening at the column face (as the post-tensioned beam (PTB) does) and it therefore avoids the related undesirable issues of the PTB system,
(e) the post-desirable loop reduces permanent displacement compared to a structure with a elasto-plastic hysteretic loop (resulting from beam yielding or symmetric friction connections, say),
(f) while the strength of the sliding hinge joint AFC connection is less than that of conventional bolted-end plate construction, this is not economically disadvantageous because most building member sizes are based on stiffness, rather than strength, and the friction connection provides high stiffness.
(g) any damage to the bolts can be remedied by replacing the bolts if need be, and
(h) costs are approximately the same as regular construction. The exact cost of the structure in the Victoria University Wellington Campus building was 0.5% more than the price with conventional connections.

Sliding hinge joint AFC construction has been used in at least five multi-storey steel buildings in New Zealand. Details of one of these are shown in Figure 8.7. Research is continuing at the Universities of Canterbury and Auckland on the friction forces, construction tolerances, the loss of stiffness that occurs when the joint is pushed into the active sliding state, and the durability. Durability issues may be in terms of cold welding and corrosion. Corrosion is most likely to be significant when the link is placed in an exterior environment, such as beneath a bridge.Work is also being conducted at the University of Auckland, with additional devices, in order to improve the self-centring ability of the joint.

(a) Sliding hinge AFC

(b) Completed structure

Figure 8.7. TePuni Village Buildings using AFC and Rocking Technology (Sidwell, 2010).

Figure 8.8 shows a variation to the sliding hinge AFC joint proposed by Chanchi (MacRae and Clifton, 2010). Here, the sliding mechanism is placed perpendicular to the point of rotation. The advantage of this is that the demands on the bottom flange plate are predominantly axial, and the flexural component is minimised. Modifications to this, such as making an arc-shaped sliding mechanism, are also possible. Sliding hinge AFC joints which are may provide greater self centring characteristics are also being developed (e.g., Khoo et al. 2011).

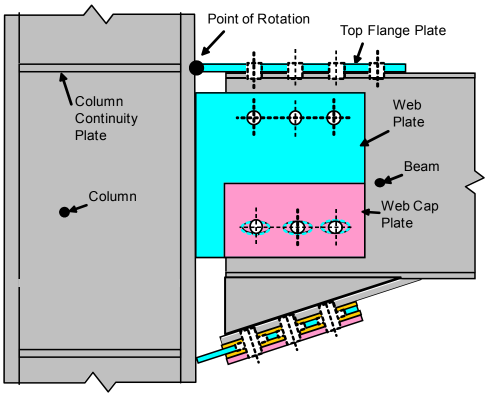


Figure 8.8. Alternative SHJ Connection (MacRae and Clifton, 2010.

8.4.3 HF2V devices in steel moment frames

One of the devices tested at the University of Canterbury on steel frames is the high-force-to-volume (HF2V) lead extrusion dissipater (Rodgers et al., 2010), illustrated in Chapter 5. Figure 8.9 shows its application in a top-hinging steel frame beam joint.

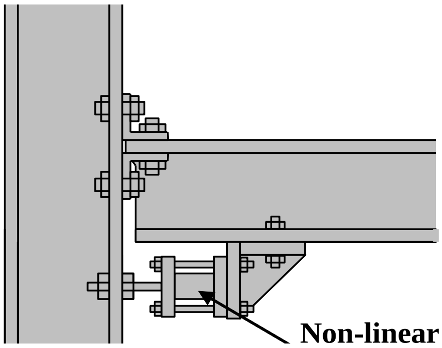


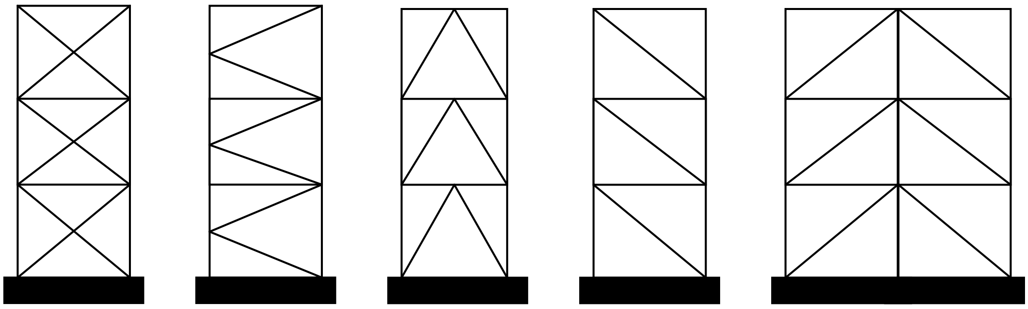
Figure 8.9. Schematic of HF2V devices below beam in steel moment frames (Mander et al., 2009).

8.5 Concentrically Braced Structures

8.5.1 Traditional brace dissipaters

Traditional concentric braces dissipate energy by yielding in tension and buckling and yielding in compression. Because of the different strengths in tension and compression, and their susceptibility to low cycle fatigue fracture due to the formation and straightening out of large local curvatures generated by buckling in compression, they are generally not permitted to be major energy dissipating element in tall structures according to worldwide codes. They also need to be designed for significantly greater strength than other systems showing more ductility.

The bracing may be placed in different configurations, such as X, K, inverted V, or diagonal bracing as shown in Figure 8.10. Balanced diagonal bracing is the most common for moderate rise structures because it provides the same strength in both directions.



(a) X (b) K (c) Inverted V (d) Diagonal (e) Balanced diagonal bracing

Figure 8.10. Different bracing configurations for concentrically braced frames.

Frames with balanced diagonal bracing sustain buckling to the braces, but with appropriate bolted connections to the frame, the braces can be replaced after a major earthquake. The practical benefits of this concept were well seen in the 1987 Edgecumbe earthquake, with one major industrial complex storing spare braces for its braced frame main production process buildings and being able to replace damaged braces and restore full structural function within 24 hours of the earthquake.

8.5.2 Buckling restrained braces (BRB)

Buckling restrained braces (BRB) are restrained from buckling by means of a casing, as shown in Chapter 5. The steel is debonded from the casing material so that it can freely slide in the sheath. They can be used in concentrically braced frames as shown in Figure 8.11. The Psychology building at the University of Canterbury was retrofitted using this technology, as illustrated in Chapter 5.

The development path taken for BRB braces and systems in North America and Japan has been to specify an experimental testing regime and to require providers of braces to show compliance with this regime. This has led to the development of a range of proprietary, patented braces. However, the small size of the New Zealand market and distance from Northern Hemisphere brace suppliers means that a generic set of design and detailing requirements is a highly desirable outcome for New Zealand. A research project based around two such systems is currently underway at the University of Auckland with results expected by the end of 2011.

Figure 8.11. Diagonal and inverted V-braced BRB concentrically braced frame structures (AISC, 2007).

8.5.3 Friction braces– SFC– inconcentrically braced structures

Concentric bracing can also be used with friction devices for energy dissipation. These friction devices may be of two sorts as shown in Figure 8.12

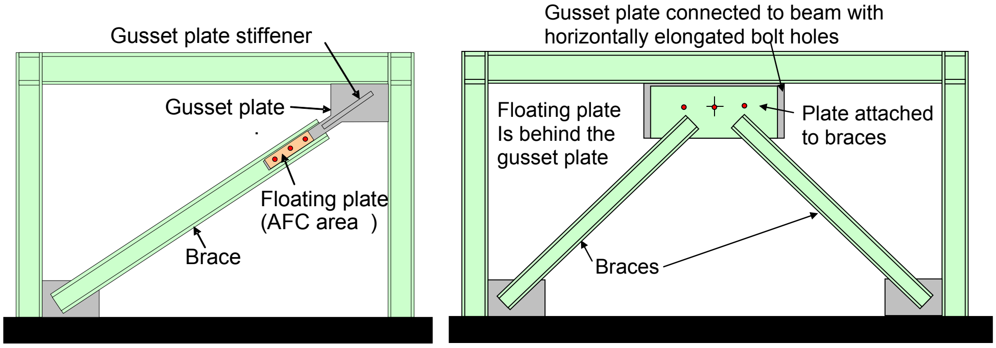
(a) Asymmetric friction connection (AFC). (b) Symmetric friction connection (SFC).

Figure 8.12. Friction connection types (from Chanchi et al., 2010).

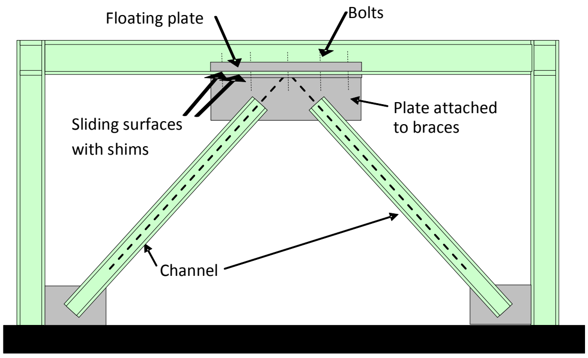
The AFC has a slightly more pinched hysteresis loop than the SFC, when large sliding deformations are considered. This is because the bolts in the AFC must first move on an angle to activate the floating plate (i.e., the bottom plate in Figure 8.12(a). This will generally result in slightly lower permanent displacements.

8.5.4 Friction brace – AFC - in concentrically braced structures

Figures 8.13shows concentrically braced systems which dissipate energy by means of the AFC brace systems. In Figure 8.13(a), the AFC is within the brace. Because elongated bolt holes can be long, large deformations may occur in the brace. Special care needs to be made with near the AFC area that an out-of-plane bending failure cannot occur. Also, the end connections of to the gusset plates must be detailed to ensure that in-plane bending of the brace does not cause any major problems. In Figure 8.13(b), horizontal sliding occurs in the gusset plate below the beam. In Figure 8.13(c), horizontal sliding occurs in below the beam bottom flange. Figures 8.13(b) and (c) impose additional bending may be applied to the beam and this should be considered in design.



(a) AFC brace. **(b) AFC attachment.**



(c) AFC on beam bottom flang

e (MacRae, 2011).

Figure 8.13. Some AFC brace configurations (MacRae and Clifton, 2010).

The AFC brace systems have the following desirable characteristics:

i) there is no significant damage to the frame (except perhaps to some bolts which may require tightening or replacement),
ii) the system has similar behaviour in both directions of loading,
iii) the hysteretic loop of the AFC together with the elastic response of the moment frame will have a significant post-elastic stiffness which encourages re-centring of the structure after an earthquake, and
iv) the technology developed does not require patents for use.

8.5.5 HF2V dissipaters in concentrically braced structures

HF2V dissipaters may be used in braced frames as shown in Figure 8.14. Here, brace buckling issues need to be addressed as well.

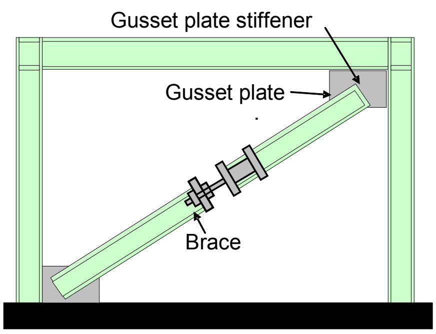


Figure 8.14. Braced frame with HF2V device.

8.5.6 Self-centring braces in concentrically braced structures

Innovative braces have been developed by Christopoulos et al. (2008). These have a flag-shaped hysteresis similar to that in Figure 8.3(c). This results in energy dissipated and no permanent displacement at the end of an earthquake. These desirable characteristics do come at a cost.

8.6 Eccentrically Braced Frame (EBF) Structures

8.6.1 Eccentrically braced structures with replaceable components

EBF frames with replaceable components have been tested by Mansour et al. (2009). It was found that replaceable links could perform very well. However, because of the large inelastic deformations required, the floor slab needed to be replaced. It should be noted that in the Christchurch earthquake, minimal slab damage was seen (Bruneau et al., 2011) possibly because of the increased strength of the link-slab system which resulted in low link deformations. Investigations to quantify slab effects on the strength, stiffness and overstrength of EBFs are currently underway at the University of Auckland.

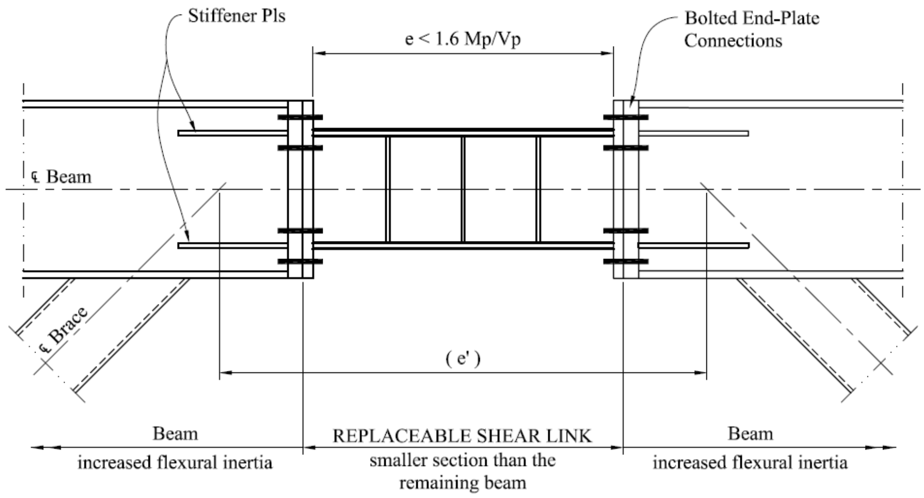
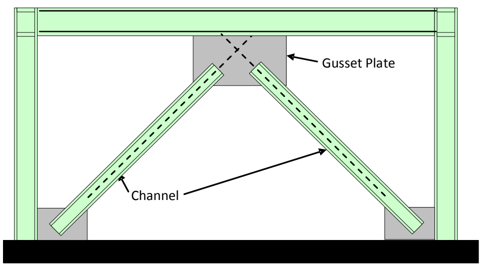
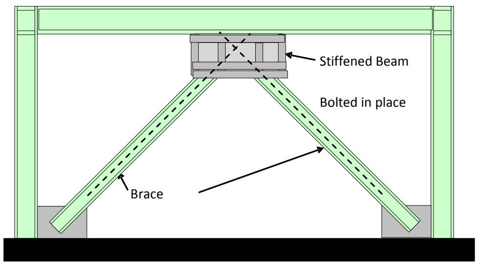


Figure 8.15. Replaceable link in EBF (Mansour et al, 2009).

An alternative to this is to use the gusset plate below the beam to dissipate energy. This has been shown to work effectively (e.g., Astaneh, 1990). This minimises floor damage as shown in Figure 8.16(a). Presumably a replaceable T-section bolted to the bottom of the beam could be used for the gusset plate. Alternatively, a replaceable link beam could be used as shown in Figure 8.16(b).



(a) Use of gusset plate (Astaneh, 1992).

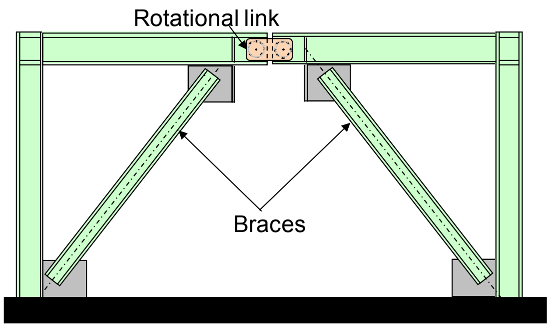


(b) Replaceable link below beam (MacRae, 2011).

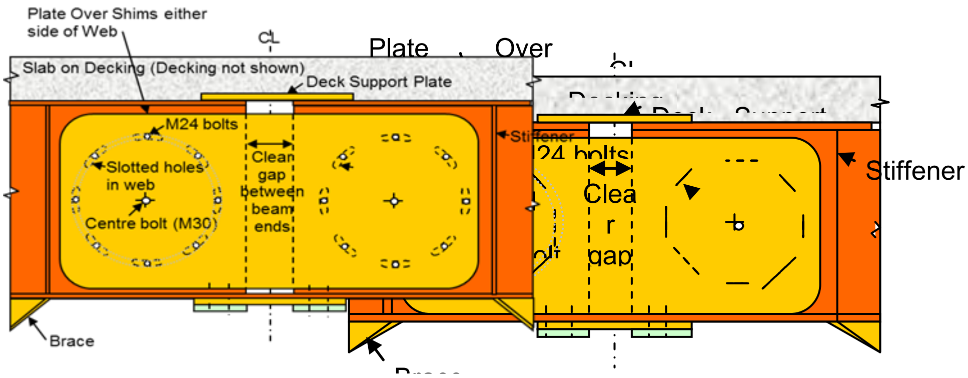
Figure 8.16. Energy dissipation below beam in EBF.

8.6.2 Eccentrically braced structures with AFC link

Figure 8.17 show an innovative link connection for an eccentrically braced frame conceived by Clifton (Khan and Clifton, 2011). Rotation occurs about the centre bolts. The other bolts provide a clamping force for dissipating energy in friction. While the connection has almost no damage, the configuration has the same problem as traditional EBF design, or EBF design with replaceable links. That is, when the shear link deforms to its design inelastic deformation capacity, any slab sitting on top of the link may be damaged and need replacing. In the rotational AFC, the slab demands may be greater than in the traditional EBF because the differential movements and angle of deformation of the beam beneath the slab is greater. This means that while the frame is not expected to suffer significant damage, the structural system, which includes the slab, may.



(a) EBF with rotational AFC link.



(b) Rotational AFC link.

Figure 8.17. Rotational AFC Link in EBFs (Khan and Clifton, 2011).

Currently studies at the University of Auckland are being conducted to evaluate whether, by separation the slab over the link region, it is possible to keep the slab elastic and undamaged, and to increase the restoring characteristics of the structure.

8.6.3 Eccentrically braced structures with AFC braces

The same eccentrically braced configuration may be obtained without the need for inelastic deformation in the link. The braces can be used to dissipate the energy, in the same way that they can be used for concentrically braced frames, in Figure 8.18. While the means of energy dissipation is much less elegant than the rotational link, this concept has the advantage that it is not likely to result in significant slab damage. Again, care needs to be taken to prevent brace buckling.

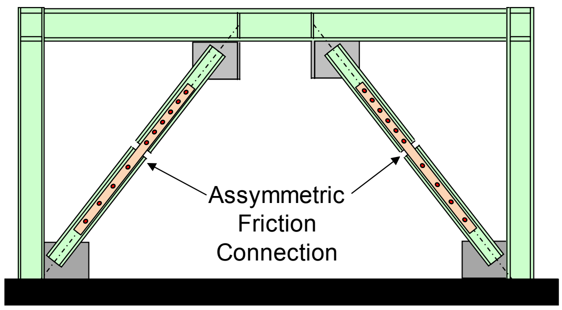


Figure 8.18. Schematic of EBF with AFC braces (MacRae, 2010).

8.7 Rocking Structures

Rocking structures are uplift under severe lateral seismic accelerations as shown in Figure 8.19. New Zealand has a legacy of designing rocking structures as shown from 1981 South Rangitikei Rail Bridge. The first steel structure designed to rock was built in Wellington in

2007 (Gledhill et al. 2008) idealised in Figure 8.20(a). Here the self-centring cables are attached to springs at the bottom of the legs in Figure 8.20(b). These springs increase the level of earthquake inertia force under which uplift occurs, thereby increasing the secant stiffness and reducing the expected frame displacements.

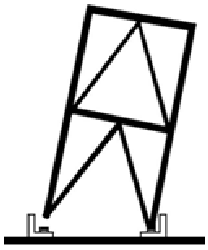
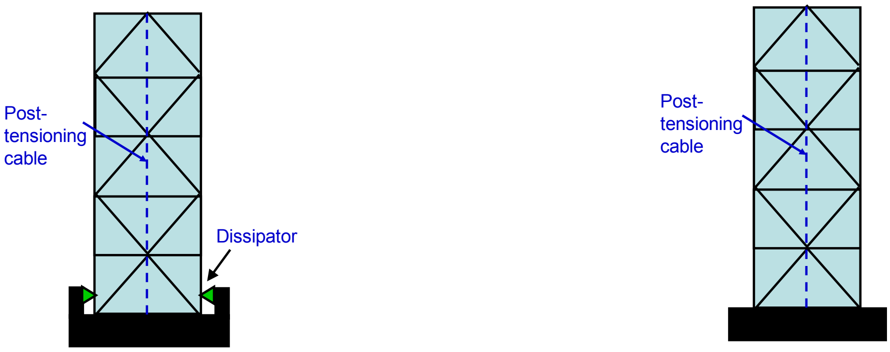


Figure 8.19. Schematic of rocking steel structure (Chanchi et al., 2010).

(a) Schematic **(b) Photo of Springs in the Legs (Sidwell, 2010)**

Figure 8.20. Rocking structure in Wellington.

More recently other rocking systems have been proposed and testing has been conducted by groups based at i) Lehigh (Roke et al., 2009) and ii) Stanford-Illinois-TIT (Deierlein et al., 2010) as shown in Figure 8.21. The Stanford-Illinois-TIT frame costs more because of the dissipater, and the dissipater reduces the response. Here, post-tensioned cables extend to the top of the structures. This results in larger member sizes throughout the frame than in the New Zealand approach, but it obviates the need for the springs.



(a) Lehigh Proposal (Sause et al., 2010) **(b) Stanford et al. Proposal (Deierlein et al., 2009)**

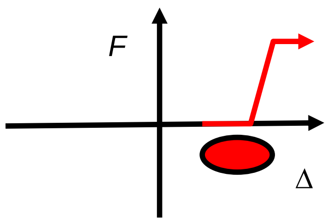
Figure 8.21. Different configurations for rocking structures.

A number of issues with rocking structures have not been fully addressed (MacRae, 2010). These include:

a) Vertical accelerations resulting from impact on the foundation as the frame returns to its initial position. This may adversely affect the non-structural elements, contents and the occupants. Restrepo (2010) indicated vertical accelerations as great as 4g in a concrete rocking wall in a recent test. These vertical accelerations will affect the non-structural elements, such as the ceiling tiles. The impact forces are likely to occur when the frame is at zero displacement, so it is out-of-phase with the maximum forces expected in the frame. Impact forces are likely to be reduced when dissipaters are present.

b) Horizontal accelerations resulting from the impact. This is also likely in all "clickety-clack" systems. These systems are those that have a rapid increase in stiffness when the structure is travelling at high velocity. Such buildings include those with traditional (buckling) concentric braces with medium to high slenderness ratios, pure steel plate shear walls, post-tensioned beams, rocking structures, concrete walls and others. This issue was first raised by MacRae (2010) where a motorbike was shown travelling at constant velocity. Because it is at constant velocity, the horizontal forces and accelerations on the motorcyclist are zero. However, the when the motorbike suddenly hits a wall, the forces on the bike suddenly increase, until the wall is pushed over. This is illustrated in Figure 8.21. The hysteresis loop for the motorcycle in Figure 8.21(b) is similar to that for many clickety-clack structures. The following provocative question was raised: "Is the difference between the motorcyclist, and a person in a "clickety-clack" building during an earthquake, only the amount of protection they are wearing?". Anecdotal evidence (e.g., Bull 2011, Clifton 2011) indicates that in buildings of this type, including concrete shearwall buildings, due to the high increase in stiffness at high velocity, many items and people were thrown across rooms during the 22 February 2011 Christchurch earthquake. This is similar to the way the motorcyclist may be expected to be thrown of their motorcycle. Research is continuing at the University of Canterbury to quantify these effects.

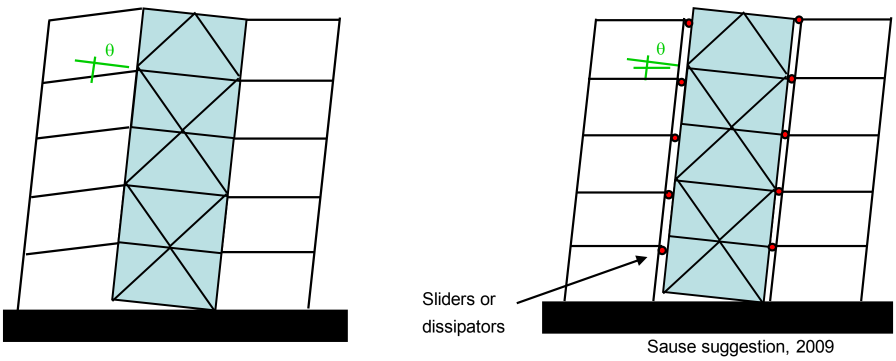
(a) Motorbike and wall



(b) Hysteresis curve

Figure 8.22. Hysteresis for Sudden Stiffness Change at High Velocity (WWW, 2010)

c) Vertical deformations on the side of the frame may result in large demands to the floor slab as it needs to kink through the angle θ in Figure 8.22(a). This interaction with the rest of the frame may limit the rocking that occurs, and it may cause damage in the frame. Sause et al. (2010) proposed separating the rocking frame from the rest of the structure as shown in Figure 8.22(b). Here, dissipaters between the frame and the rest of the structure may be placed to dissipate energy. These may be AFC dissipaters as shown in Figure 8.23 (MacRae, 2010). Also, horizontal plates between the rocking frame and the structure behind may be used to transfer lateral force but not vertical forces.



(a) Deformation if attached to frame. **(b) Separation of frames.**

Figure 8.23. Rocking frame – gravity frame interaction.

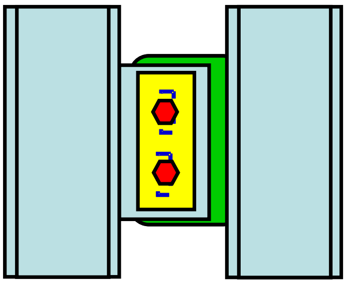


Figure 8.24. AFC Dissipation for

8.8 Base-Isolated Structures

As described in the base isolated buildings chapter, base-isolation causes the buildings to have lower fundamental periods than those on a fixed base. In most earthquake design spectra, this results in lower design forces. While special dissipaters are required at the base of the structure, and special details are required to connect building services, the lower forces have resulted in base isolation creating more economical structures (Wada, 2010).

8.9 Supplemental Damped Structures

As described in Chapter 5, the response of a structure may be considerably reduced by the placement of viscous, or near viscous, dissipaters. Because the peak dissipater force occurs at the peak velocity, which is out of phase with the peak structural force/displacement, well designed dampers do not increases the forces on the frame members. They may also be one of the only ways of minimising the effects of very large near-field pulse type accelerations (Bertero et al., 1999). However, the cost of viscous dampers is generally considerable.

8.10 Base Connections for Structures

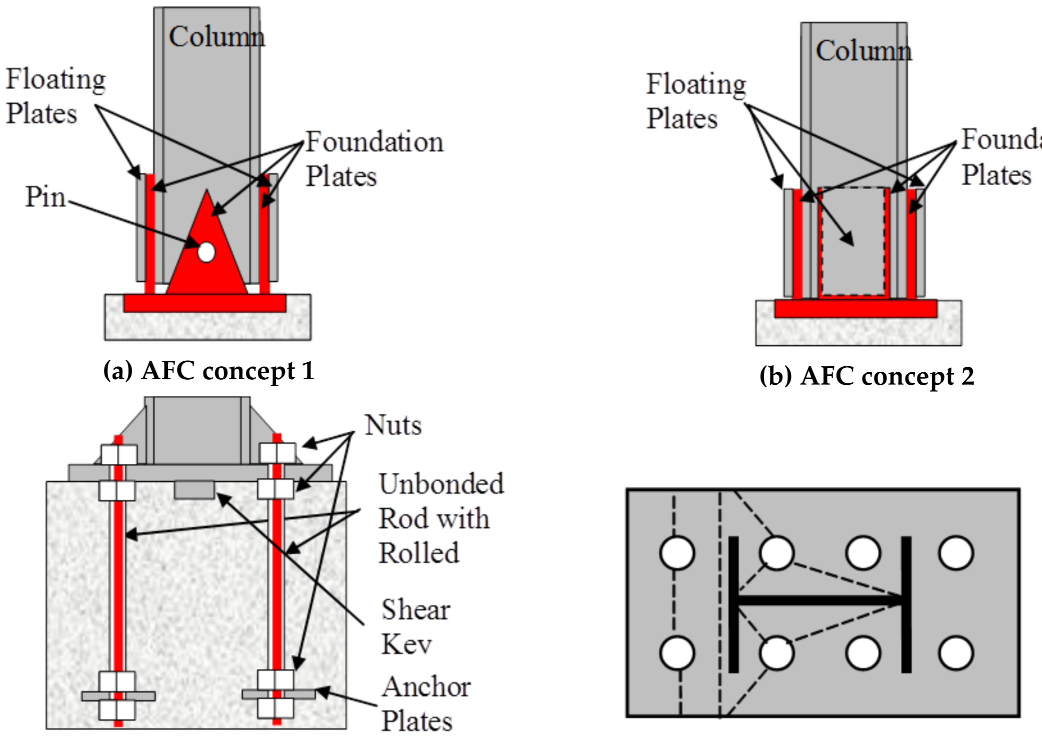
A number of different base connections to minimise damage at the column brace are described in MacRae et al. (2009).

A conceptual drawing of a AFC base detail is given in Figure 8.25(a). Here, axial force is transferred directly from the column to the pin at the centre of the column to the foundation. Shear force is carried the same way. Flexure is carried by means of asymmetric friction action in the flanges.

Figure 8.25(b) illustrates asymmetric action on both flanges and webs. Column axial compression goes directly from the column into the foundation and shear is carried through the bolts in the web. If the column is subject to large axial tension, it will be designed to stop moving when the bolts hit the top end of the elongated holes in the foundation plates. This detail is easier to construct than the that in Figure 8.25(a), but one side of the column has to move up (much like a concrete column) to allow flexural deformation to occur. This changes the height of the centre of the column. There is also the possibility that after a major earthquake that the column may not have returned to its initial position, so the bolts may need to be loosened and tightened again.

Figure 8.25(c) (Mackinven et al., 2007) involves the use of unbonded steel rods to act as re-centring devices while the steel column rocks under lateral loads. The unbonded length of the rods is sufficient to allow elastic extension to re-centre the rocking column. The rod has a rolled thread passing through it and a nut above and below the end plate. This rolled thread seems to be able to withstand many cycles without fracturing. As above, the absence of yielding in the column results in the elimination of inelastic axial shortening. Some industrial complexes in the Edgecumbe earthquake of 1987 with this detail performed well.

Figure 8.25(d) illustrates a yielding endplate connection.



(a) AFC concept 1 (b) AFC concept 2

(c) Rod concept (d) Yielding endplate connection

Figure 8.25. Some Possible Methods for Preventing Column Yielding

8.11 Acknowledgements

This chapter was reviewed by Dr. Charles Clifton.

References

AISC, (2007). Design of Seismic-Resistant Steel Building Structures, Seismic Design Modules in PowerPoint.American Institute of Steel Construction.

Astaneh-Asl, H., (1992). "Cyclic Behaviour of Gusset Plates in V-Braced Steel Frames", *in Stability and Ductility of Steel Structures under Cyclic Loading,* Edited by Fukumoto Y. and Lee G., CRC Press Inc..

Bertero, V.V., Anderson, J.C. and Sasani, M., (1999). Importance of Impulse Ground Motions on Performance-Based Engineering: Historical and Critical Review of Major Issues and Future Directions,*ASCE Structures Congress*, New Orleans.

Bruneau, M., Anagnostopoulou, M., MacRae, G., Clifton, G.C. and Fussell, A., (2010). Preliminary Report on Steel Building Damage from the Darfield Earthquake of September 4, 2010. Bulletin of the New Zealand, Society for Earthquake Engineering, December.

Bruneau, M., Clifton, G.C., MacRae G., Leon, R. and Fussell, A., (2011). Preliminary Report on Steel Building Damage from the Christchurch Earthquake of 22 February 2011, Earthquake Engineering Research Institute website.

BullD., (2011), Personal communication.

Chanchí, J.C., MacRae, G.A., Clifton, G.C. and Chase, G., (2010). "Quantifying Seismic Sustainability Of Steel Framed Structures", in Proceedings of the Steel Structures Workshop 2010, Research Directions for Steel Structures, compiled by MacRae G. A. and Clifton G. C., University of Canterbury, 13-14 April.

Christopoulos, C. and Filiatrault, A., (2006). Principles of Passive Supplemental Damping and Seismic Isolation. IUSS Press. First edition. Pavia, Italia.

Christopoulos, C., Tremblay, R., Kim, H.J. and Lacerte M., (2008). "Self-Centering Energy Dissipative Bracing System for the Seismic Resistance of Structures: Development and Validation," ASCE Journal of Structural Engineering, 134(1), 96–107.

Clifton, G.C., (2005). Semi-Rigid joints for moment resisting steel framed seismic resisting systems. PhD Thesis, Department of Civil and Environmental Engineering, University of Auckland.

Clifton, G.C., (2011). Personal Communication.

Danner, M., Clifton, G.C., (1995). Development of Moment-Resisting Steel Frames Incorporating Semi-Rigid Elastic Joints: Research Report; HERA, Manukau City, New Zealand, Report R4-87.

Deierlein, G.G., Ma, X., Hajjar, J.F., Eatherton, M., Krawinkler, H., Takeuchi, T., Midorikawa, M., Hikino, T., and Kasai, K., (2010). Seismic Resilience of Self-Centering Steel Braced Frames with ReplaceableEnergy-Dissipating Fuses ; Part 2: E-Defense Shake Table Test,7th International Conference on Urban Earthquake Engineering & 5th International Conference on Earthquake Engineering Joint Conference March 3-5, 2010, Tokyo, April 2010.

Desombre, J., Rodgers, G.W., MacRae, G.A., Rabczuk, T., Dhakal, R.P., Chase, J.G., (2011). Experimentally Validated FEA Models of HF2V Damage Free Steel Connections For Use In Full Structural Analyses, "Structural Engineering and Mechanics, *An international Journal*", SEM9N626C, Structural Engineering and Mechanics, 37(4).

Garlock, M.E., Li, J., and Vanmarke, E.H., (2010). "Floor Diaphragm Design of Steel Self-Centering Moment Frames", STESSA-09, Lehigh, USA, p939-944.

Gledhill, S.M., Sidwell, G.K., Bell, D.K., (2008). The Damage Avoidance Design of Tall Steel Frame Buildings - Fairlie Terrace Student Accommodation Project, Victoria University of Wellington, New Zealand Society of Earthquake Engineering Annual Conference, Wairakei.

Kim, J., Stanton, J., and MacRae, G.A., (2004). "Effect of Beam Growth on Reinforced Concrete Frames", Journal of Structural Engineering, ASCE, 130(9), pp. 1333-1342.

Khan, M.H. and Clifton, G.C., (2011). "Proposed Development of a Damage-Resisting Eccentrically Braced Frame with Rotational Active Links, Bulletin of the New Zealand Society for Earthquake Engineering, Vol 44, No 2.

Khoo, H.H., Clifton, G.C., Butterworth, J., Mathieson, C.D. and MacRae, G.A., (2011). "Development of the Self-Centering Sliding Hinge Joint", Proceedings of the Ninth Pacific Conference on Earthquake Engineering, Building an Earthquake-Resilient Society, 14-16 April, 2011, Auckland, New Zealand. Oral presentation. Paper 106.

Lin, Y.C., Ricles, J., Sause, R. And Seo, C.Y., (2009). Experimental assessment of the seismic performance ofsteel MRF system with beam web friction devices, STESSA, Philadelphia. Taylor & Francis Group, London, ISBN 978-0-415-56326-0.

Mackinven, H., MacRae, G.A., Pampanin, S., Clifton, G.C., and Butterworth, J., (2007). "Sliding Hinge Joint Tolerances for Steel Moment Frames", Proceedings of the 8th Pacific Conference On Earthquake Engineering (PCEE), Paper 200, Singapore, 05 – 07 December.

MacRae, G.A., Urmson, C.R., Walpole, W.R., Moss, P., Hyde, K. and Clifton, G.C., (2009). “Axial Shortening of Steel Columns in Buildings Subjected to Earthquakes”, Bulletin of the NZ Society of Earthquake Engineering.

MacRae, G.A., (2010). “Some Steel Seismic Research Issues”, in Proceedings of the Steel Structures Workshop 2010, Research Directions for Steel Structures, compiled by MacRae G. A. and Clifton G. C., University of Canterbury, 13-14 April.

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

MacRae, G.A, Clifton, G.C., and Butterworth, J.W., (2009). “Some Recent New Zealand Research on Seismic Steel Structures”, STESSA, Philadelphia, August 2009. Taylor & Francis Group, London, ISBN 978-0-415-56326-0.

MacRae, G.A. and Clifton, G.C., (2010). New Technology Applications, Recent Developments and Research Directions for Seismic Steel Structures in New Zealand, Asian Conference on Earthquake Engineering, Bangkok, Thailand, December.

MacRae, G.A., (2011). AFC Sliding Below Beam, Steel Research Panel Idea Registration Form, 30 June 2011, No: IR GAM3. Heavy Engineering Research Association, Manukau City, Auckland, New Zealand.

Mander, T.J., Rodgers, G.W., Chase, J.G., Mander, J.B. MacRae, G.A. and Dhakal, R., (2009). “A Damage Avoidance Design Steel Beam-Column Moment Connection Using High-Force-To-Volume Dissipaters”, American Society of Civil Engineering. *Journal of Structural Engineering*, 135(11), November 1, 2009, pp1390–1397. ISSN 0733-9445.

Mansour, N., Christopoulos, C., and Tremblay, R., (2009). Experimental performance of full-scale eccentrically braced frames with replaceable shear links, STESSA, Philadelphia, August 2009. Taylor & Francis Group, London, ISBN 978-0-415-56326-0.

Rodgers, G.W., Chase, J.G., MacRae, G.A., Bacht, T., Dhakal, R. P., and Desombre, J., (2010). “Influence of HF2V Damping Devices On The Performance Of The SAC3 Building Subjected To The SAC Ground Motion Suites”, 9USN-10CCEE, Toronto, July 25-29.

Priestley, M.J.N., and MacRae, G.A., (1996). Seismic Tests of Precast Beam-to-Column Joint Sub-assemblages with Unbonded Tendons, PCI Journal, January-February; 64-80.

Priestley, M.J.N., (1996). The PRESSS program - current status and proposed plans for Phase III, PCI Journal. Mar.-Apr, 41(2):22-40.

Restrepo-Posada, J., 2010. “The Chile Earthquake”, NZSEE Conference, Wellington, 2010.

Roke, D., Sause, R., Ricles, J.M., and Gonner, N., (2009). “Damage-free seismic-resistant self-centering steel concentrically-braced frames”, STESSA, Philadelphia, August 2009. Taylor & Francis Group, London, ISBN 978-0-415-56326-0.

Sause, R., Ricles, J. M., Lin, Y. C., Seo, C. Y., Roke, D., and Chancellor, B., (2010). Self-Centering Damage-Free Seismic-Resistant Steel Frame Systems, 7th International Conference on Urban Earthquake Engineering & 5th International Conference on Earthquake Engineering Joint Conference March 3-5, 2010, Tokyo, April 2010.

Sidwell, G., (2010). “Low Damage Buildings - the Realisation”, in Proceedings of the Steel Structures Workshop 2010, Research Directions for Steel Structures, compiled by MacRae G. A. and Clifton G. C., University of Canterbury, 13-14 April.

Tsai, K.C., Lee, C.H., Tsai, C.Y., Lin, C.H., (2010). Large Scale Seismic Testing of Steel-Framed Structures at NCREE , in Advances in Performance-Based Earthquake Engineering, Chapter 42, Volume 13, 451-460, Springer, ISBN Number 978-90-481-8745-4

Wada, A., (2010). "Changes of Seismic Design after 1995 Kobe Earthquake", in Proceedings of the Steel Structures Workshop 2010, Research Directions for Steel Structures, compiled by MacRae G. A. and Clifton G. C., University of Canterbury, 13-14 April.

WWW (2010).
http://upload.wikimedia.org/wikipedia/commons/7/79/Motorbike_rider_mono.jp and http://neilhellmanlibrary.files.wordpress.com/2007/07/wall.jpg

Yamada et al., (2010). Full Scale Shaking Table Collapse Experiment on 4-Story Steel Moment Frame, 7th International Conference on Urban Earthquake Engineering & 5th International Conference on Earthquake Engineering Joint Conference March 3-5, 2010, Tokyo, April 2010.

9 DAMAGE-RESISTANT DESIGN OF TIMBER STRUCTURES

This section describes developments in damage-resistant design of rocking post-tensioned multi-storey timber buildings. The seismic structural system can be either timber post-tensioned frames and/or walls. Braced dissipative systems already introduced in chapter 8 for steel structures can be easily applied to multi-storey timber buildings.

9.1 Concept and Mechanism

As described in previous chapters, rocking solutions were firstly developed in precast concrete moment resisting frames or interconnected shear walls (Priestley at al., 1999) then subsequently in steel moment-resisting frames (Christopoulos et al., 2008), (Danner and Clifton, 1995), (Clifton, 2005). They have resulted in structural systems which can undergo inelastic displacements similar to their traditional counterparts, while limiting the structural damage and assuring full re-centring capability. This concept is therefore material independent and since 2004 it has been implemented and transferred to timber structures (Palermo et al., 2005a, Buchanan et al., 2008) at University of Canterbury. Particular attention has been given to the hybrid connections which combine post-tensioning bars with internal or external steel dissipaters. The basic premise is that seismic movements are accommodated through a controlled rocking mechanism between prefabricated elements, developing elastic elongation of long lengths of unbonded high-strength steel tendons, with energy dissipation provided by the yielding of short lengths of replaceable mild steel (or other types of) energy dissipation devices.

Additional joint reinforcing can be provided near the top and bottom of the beam, (or wall/column) to increase the bending moment capacity of the beam-to-column connection. This supplementary reinforcing can be provided in the form of ductile steel dissipaters which will help to provide energy dissipation under extreme seismic loading. The combination of post-tensioning and steel reinforcement in the connection gives "flag-shape" hysteresis loops under seismic loading as shown in Figure 9.1. This hysteresis loop is ideal since it provides sufficient energy absorption with a self-centering capacity, i.e. no residual displacements, as for the PRESSS system concrete structures described above.

The rocking timber system has been named Pres-Lam and covers both seismic resistant frames and walls. The system is internationally protected and the inventors are University of Canterbury researchers, respectively Alessandro Palermo, Stefano Pampanin and Andy Buchanan (Palermo et al., 2006).

Similarly to concrete and steel structures, the Pres-Lam timber system uses controlled rocking mechanisms at the connection points within the structure to deliberately allow seismic movement with no residual damage. The post-tensioned tendons elongate and provide clamping force and self-centering capacity while the reinforcing bars or dissipaters yield giving energy dissipation to the system (Figure 9.1).

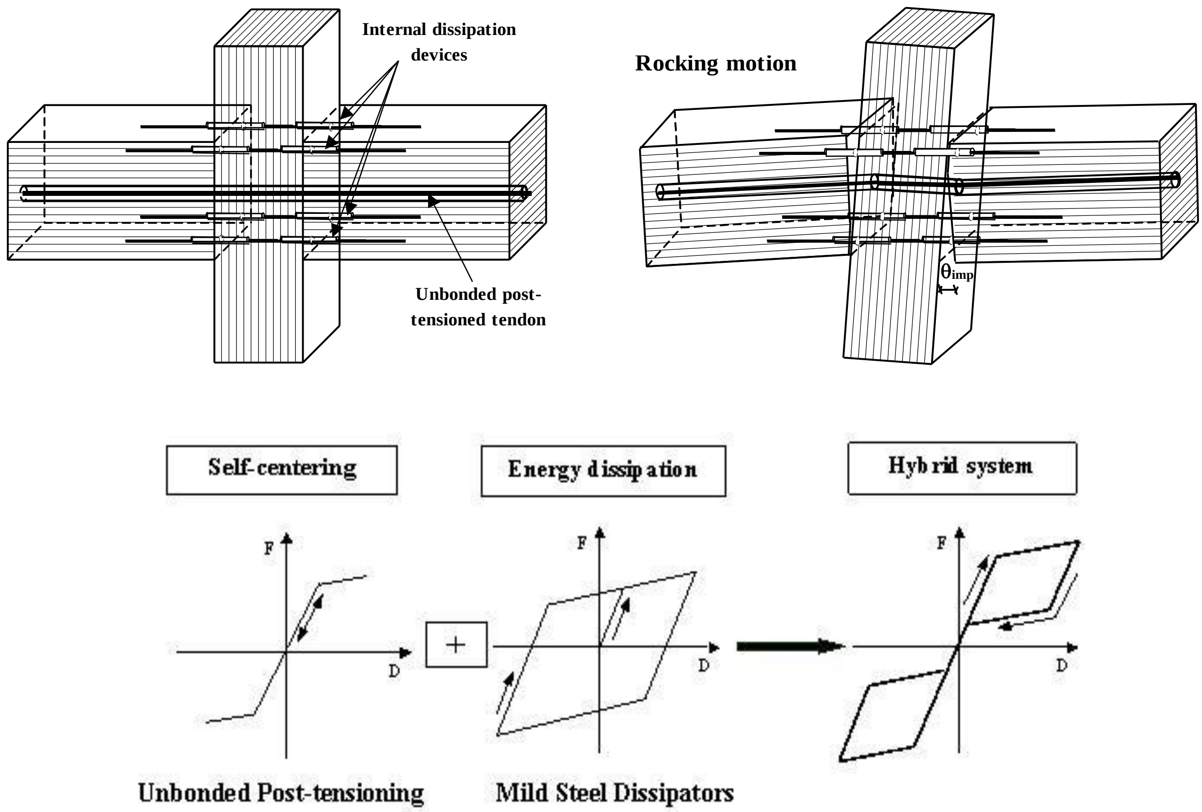


Figure 9.1. Hybrid concept for moment resisting frames and walls (Palermo et al., 2005).

9.2 Research Implementation

The Pres-Lam timber system was developed at the University of Canterbury in collaboration with the engineered wood industry and was primarily focused on LVL (Laminated Veneer Lumber) made from radiata pine but recent research has confirmed the feasibility of the system for other types of engineered wood (cross-lam, glulam etc.), (Smith et al., 2009). The experimental campaign started in 2004 and was later supported by the Structural Timber Innovation Company Ltd (STIC), a research consortium funded by industry and the New Zealand government since 2008. STIC is promoting these buildings as the "EXPAN building system incorporating Pres-Lam technology".

Rocking post-tensioned multi-storey timber buildings are constructed from engineered wood, from renewable plantation forests of radiata pine. The timber beams and columns are manufactured from Laminated Veneer Lumber (LVL) which has high and reliable strength properties for an affordable price. Manufacturing of LVL is carried out by peeling logs into 3mm veneers and gluing into billets 1.2m wide and 40-100mm thick in a mechanised continuous process which allows delivery of very long lengths, limited only by transportation. The LVL billets are then hand cut and glued into prefabricated beams, columns, floor joists and structural walls, all precisely finished to very tight tolerances. Most post-tensioned LVL members have all of the wood veneers parallel to the longitudinal axis so that the wood is only stressed parallel to the grain, but it is possible to make cross-banded LVL with some perpendicular veneers (like thick plywood) when necessary. The compression strength of LVL loaded parallel to the grain is around 40 MPa, about the same as normal strength concrete, but the perpendicular to the grain compression strength is only about 10 MPa. Cross-laminated timber (CLT), manufactured from timber boards glued together like thick plywood, is an efficient and viable alternative especially for rocking panel/wall systems. Production and use of CLT for multi-storey timber buildings is growing rapidly, especially in Europe.

9.3 Development of Connection Technology

The major benefit of post-tensioned connections is that a large number of strong ductile, moment-resisting connections can be made in one stressing operation, avoiding the need for many bolts or hundreds of nails or screws in traditional timber connections. In frames, the full-length horizontal tendons run through ducts along the beams, passing through holes in the columns, providing moment-resistance at each beam-column joint. For low seismicity regions, the connections of the structural members might be limited to unbonded post-tensioning with no external reinforcing or dissipaters.

Extensive laboratory tests on sub-assemblies adopting this technique have shown exceptional results, with no residual damage in the supporting structure, even after extreme earthquake loading (Palermo et al., 2006a), (Smith et al., 2008) and (Newcombe et al., 2008). Figure 9.2 shows test specimens for quasi-static cyclic testing, simulating earthquake condition at slow velocity rate, on beam-column, wall-to-foundation, and column-to-foundation sub-assemblies. The specimens were tested with unbonded post-tensioning tendons (as currently used for concrete structures) combined with internal or external dissipaters. Most of the concepts for dissipative fuses or dissipaters in this section are similar to those in chapter 7 (see paragraph 7.3) confirming the material independency of the hybrid or controlled rocking concept.

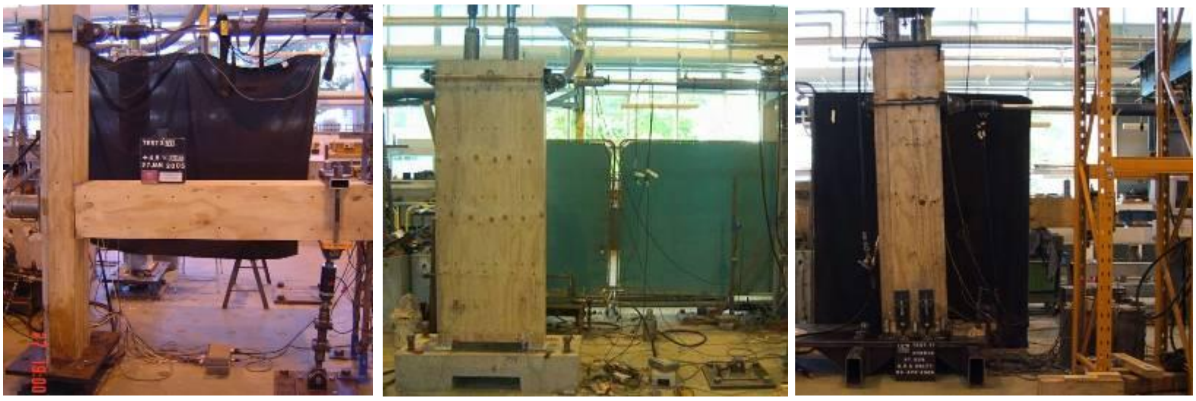


Figure 9.2. Exterior beam-column, wall-to-foundation, column-to-foundation specimens (Palermo et al. 2006, WCTE).

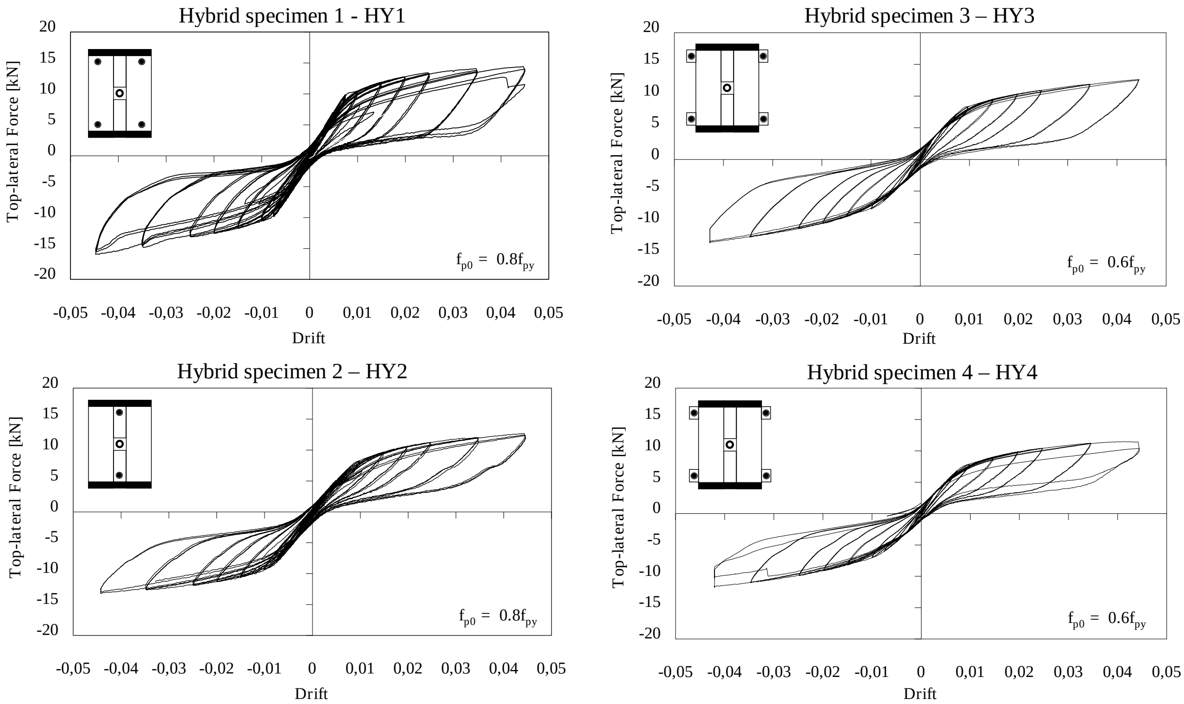
9.3.1 Beam-column connections

For example, Figure 9.3(b) shows the typical flag-shaped moment-rotation response of a hybrid beam-to-column connection with elastic unbonded post-tensioning and yielding steel dissipaters (internal and external), a tested beam-column joint with horizontal prestressing and external energy dissipaters, and the resulting hysteresis curves from this test (Smith et al., 2008). The energy dissipaters are simply a form of additional reinforcing and can be either external "plug and play" devices or internally located and epoxied into the timber. If internal epoxied dissipaters are adopted some stiffness degradation has to be accepted due to strain penetration while replaceable external dissipaters guarantee a more stable hysteresis loop and the opportunity for replacement after an earthquake. The initial high stiffness results from full contact between the end of the beam and the face of the column, and the reduction in stiffness occurs when full contact is lost, as a gap begins to open, precisely as in concrete structures, as shown in Figure 9.3(c).

INTERNAL DISSIPATERS:
epoxied mild steel bars with unbonded length

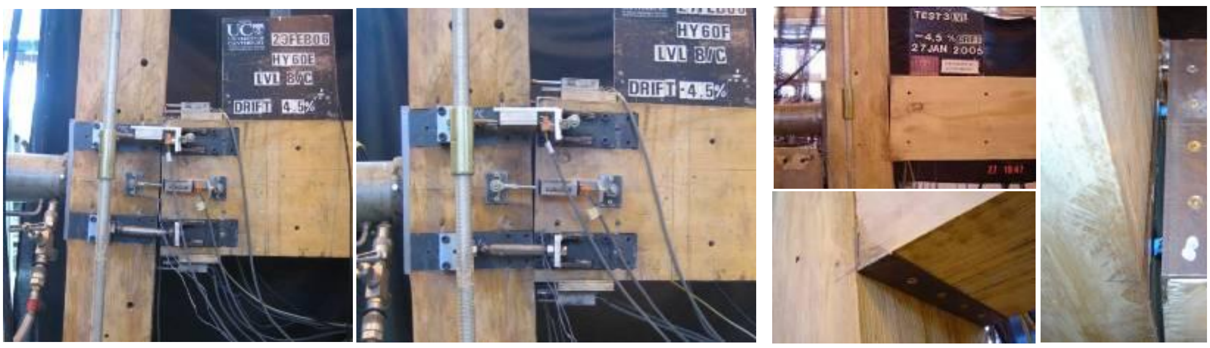
EXTERNAL DISSIPATERS:
mild steel rods with epoxied encased steel tubes

(a) Internal and external dissipaters and construction details.



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

External dissipaters **Internal dissipaters**



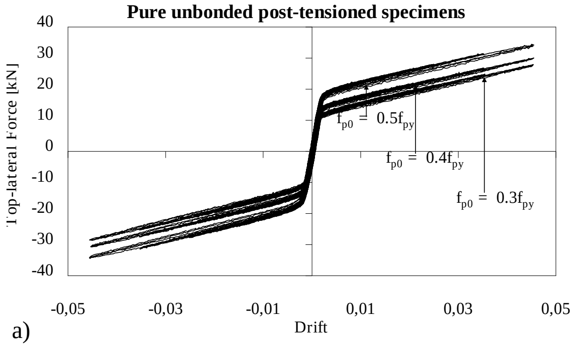
(c) Observed gap opening at 4.5% drift.

Figure 9.3. Details and performance of post-tensioned timber beam column joints.

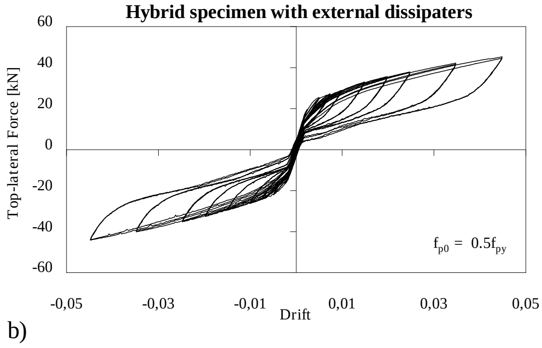
As stated earlier in paragraph 9.1, a proper design combination of the moment capacity given by unbonded post-tensioned tendons and dissipaters will provide adequate dissipation and guaranteed self-centering properties. The latter is crucial, especially if residual displacements are to be limited in order to reduce the cost of post-earthquake repairs.

9.3.2 Columns and walls

Similar seismic performance can be achieved for columns and walls. The advantage of rocking walls compared with rocking beam-column joints is that the rocking contact point in walls is between timber loaded parallel to the grain and the concrete foundation.



a)



b)

(a) Force-drift relationship for unbounded solution with post-tensioning and dissipaters.

(b) Force-drift relationship for the hybridpost-tensioning only.

(c) Observed seismic performance at 4.5% drift.

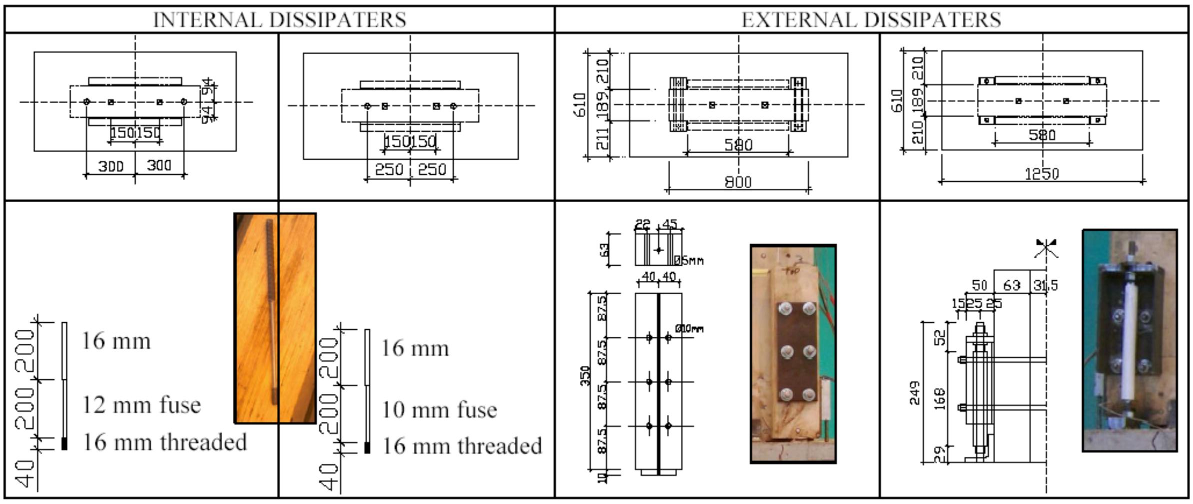
Figure 9.4. Details and performance of post-tensioned timber-to-foundation connections.

Figure 9.4 (a) shows the typical force-displacement loop with post-tensioning only. No dissipation is provided by the connection and the non-linearity comes from gap opening between the column and the foundation. In all cases the post-tensioning tendons remain in the elastic range.

An extensive experimental campaign also showed the viability of the hybrid concept for timber walls. Similar "flag-shaped" force-displacement hysteresis loops were obtained for beam-to-column and column-to-foundation specimens. Figure 9.5(a) clearly indicates a gap opening between the wall and concrete foundation induced by the lateral loading. For this particular arrangement three different types of dissipaters have been considered: internal fused dissipaters, external timber-steel epoxied dissipaters and steel encased dissipaters as shown in Figure 9.5(b).

Stressing for all the tested specimens has been carried out using high-strength 7-wire tendons. For wall foundations, a cavity in the foundation has been created in order to clamp the tendons. However, as described in the next sections alternative high-strength threaded steel bars (Macalloy or similar), can also be used and despite the lower deformability than 7-wire tendons, threaded bars present easier connection arrangements, such as threaded couplers or threaded connections embedded in the concrete foundation. Both options are readily available from the concrete prestressing industry.

(a) Hybrid walls with three different arrangements of dissipaters.



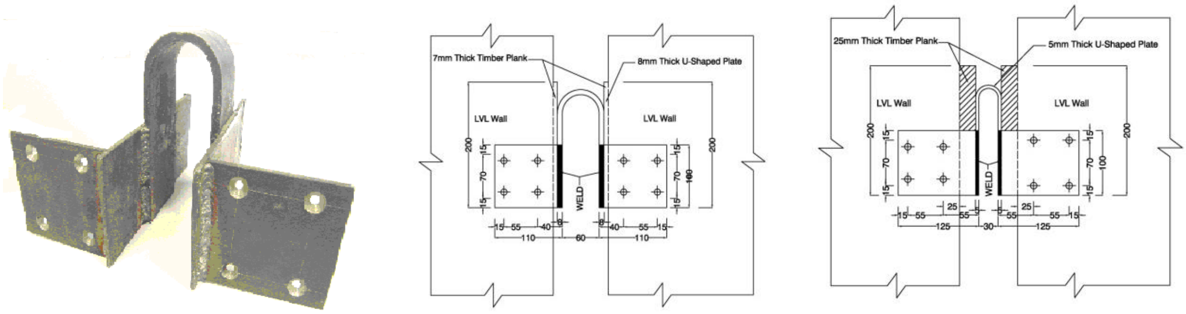
(b) Details of different dissipative arrangements.

Figure 9.5. Different dissipative arrangements for post-tensioned hybrid timber walls (Palermo et al. 2006b).

9.3.3 Coupled timber wall systems

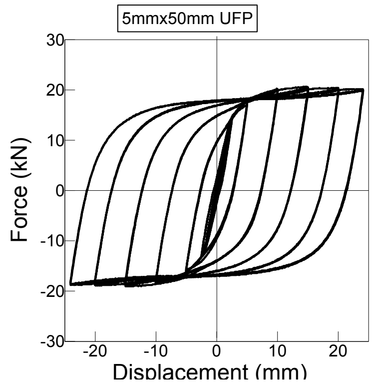
Coupled timber wall systems can be an efficient alternative to single walls for seismic resistance. If a wall is formed by two vertical cantilevers joined by coupling beams, the same lateral forces produce axial forces in the two cantilever walls in addition to the flexural stresses. This means that the net required section size is reduced in the coupled walls compared to a single cantilever wall for resisting the same lateral load.

For energy dissipation between the coupled walls, U-shaped Flexural Plates (UFPs), already described in chapter 7, have been implemented for rocking LVL timber walls. Two different fixing arrangements have been investigated as shown in Figure 9.6. The UFP devices were attached to the walls with welded brackets on the face of each wall (Figure 9.7). The brackets had holes for self-drilling screws which were fixed to the face of the wall as shown in Figure 9.6. A disadvantage of this fixing method is that the bracket on one side of the wall had to be welded in place after inserting the UFPs. Alternative details of fixing the UFPs for rapid construction can be considered, as shown in the next sections.



(a) Welded steel UFP device with two different testarrangements.

(b) Photographs of UFPs in place.



(c) Force-displacement plot for asingle UFP device.

Figure 9.6. UFP devices for coupled timber walls(Ibqal et al., 2007).

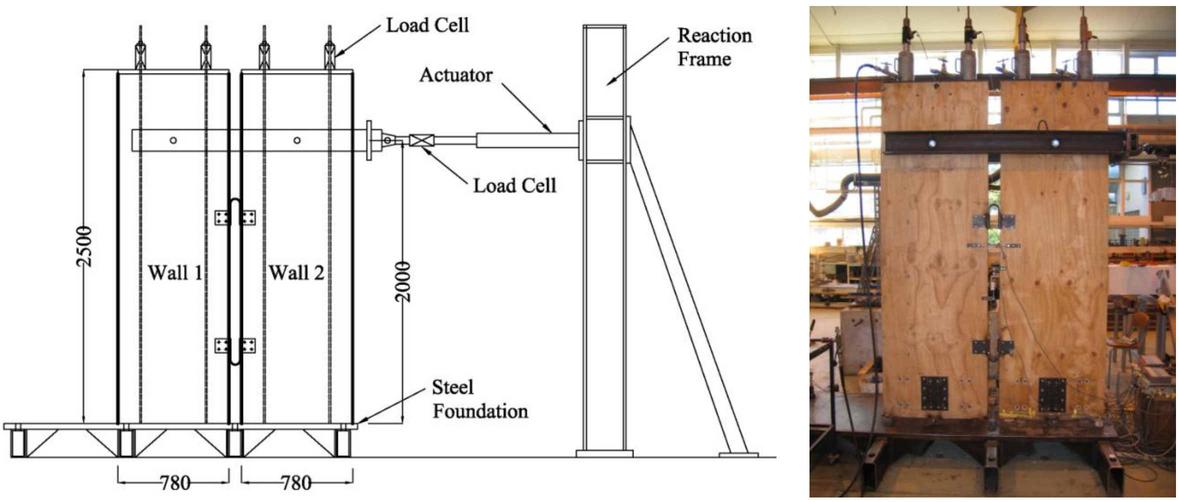


Figure 9.7. Test set-up and coupled wall specimen (Ibqal et al., 2007).

Similar to the previous specimens, post-tensioning and UFP devices can be combined in order to achieve a desirable flag-shaped hysteresis loop. The advantage of the UFP devices, as shown in Figure 9.6(c), is their instant activation induced by the relative displacement between the walls. The UFPs act as dissipative fuses and are deformed by the relative displacement between the two walls as shown in Figure 9.8. After the earthquake or, as in this case after the testing simulation, the UFP device is the only part of the system which needs to be replaced.

(b) Detail of single UFP.

(a) Coupled walls with UFPs.

(c) Visible rocking at base of the right hand wall.

Figure 9.8. Seismic performance of coupled walls during testing (Ibqal et al., 2007).

9.4 System Performance

The development of the connection technology has been tested in large scale specimens at the University of Canterbury, including full-scale beam-column assemblies (Ibqal et al., 2010) and a 2/3 scale two-storey building which combines lateral resisting walls in one direction and coupled walls in the other direction. The walls, the seismic frames and the interaction with the slab diaphragm are key aspects which drive the seismic performance of a building.

9.4.1 Moment resisting frames

The major benefit of post-tensioned frames is the low cost and rapid construction. Moment-resisting connections have been difficult in traditional timber buildings because timber cannot be welded like steel or cast like concrete, so the post-tensioned construction provides an economical solution. The prestressing loads are similar to concrete structures because the compressive strength of LVL is similar to normal strength concrete (30 to 40MPa). The same anchorage details are used as for external anchorages in prestressed concrete structures, with special bearing plates as necessary to restrict local compressive stresses (see Figure 9.9). Other than a small key or corbel to resist vertical shear forces, the entire connection consists of internal tendons which clamp the beams to the column, also providing moment-resistance. The design requirements are different for gravity frames and seismic frames.

In the design of a frame for seismic loading, roughly equal positive and negative moment capacity is required at the beam-column connection. The simplest option is to provide tendons side-by-side at mid-height. In either case, the centroid of the tendon force is at mid-depth of the beam, for both positive and negative moments; however if seismic-gravity resisting frames are considered draped tendon profiles might be implemented.

The effect of creep in dry wood stressed parallel to the grain is similar to that experienced in concrete structures, so it is not a major design issue, but special precautions such as local reinforcement must be taken when wood is stressed perpendicular to the grain by the post-tensioning (Davies, 2007, Giorgini et al., 2010). More importantly excessive shear joint flexibility of the column might compromise the overall seismic performance of the system “delaying the beam-to-column gap opening” as investigated in (Newcombe et al., 2008). Some joint reinforcement (using steel armouring, or special long self-drilling screws, for example as shown in Figure 9.9) is suggested. Alternatively cross-banded LVL, or outer layers of LVL rotated 90 degrees in the joint region, can be used to increase the joint stiffness (Van Beerschoten, 2011) and to eliminate areas of wood stressed perpendicular to the grain for long periods of time.

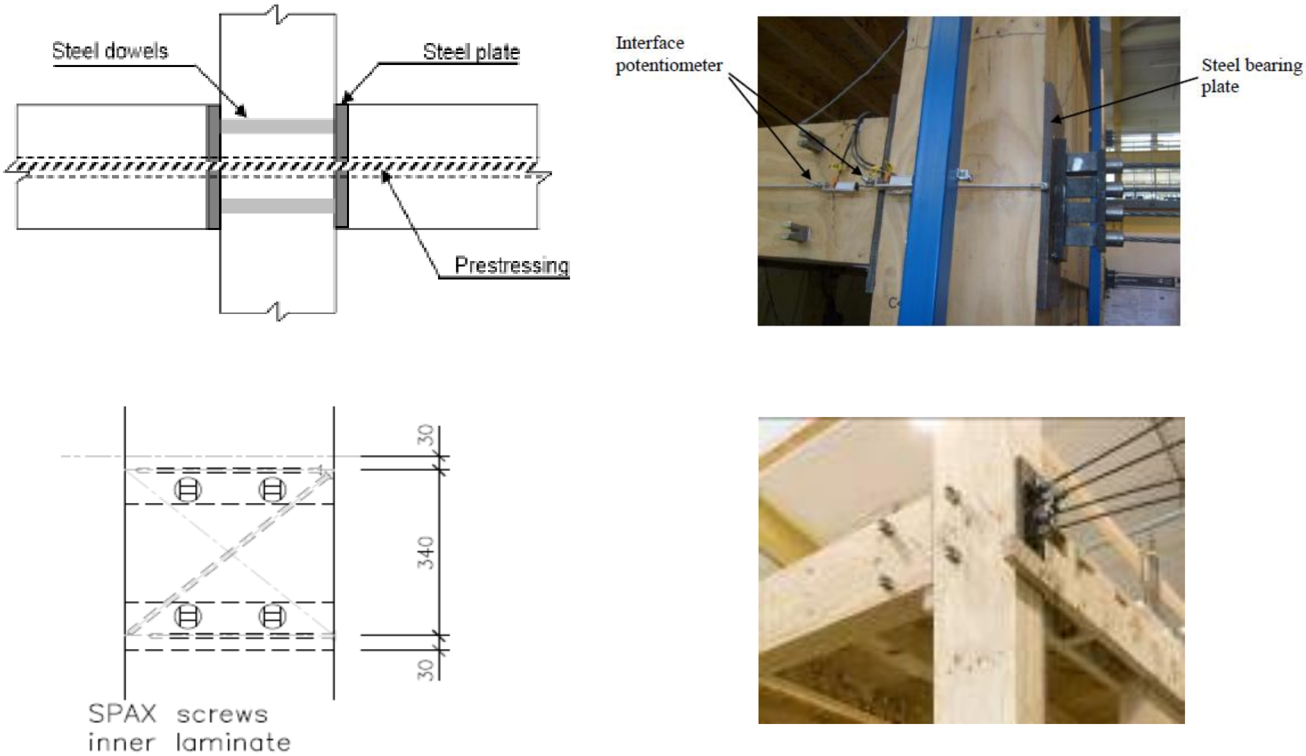


Figure 9.9. Examples of joint reinforcement (Newcombe et al., 2010a).

9.4.2 Walls

Prefabricated cantilever timber shear walls with vertical post-tensioning can provide excellent lateral load resistance. Many sizes and arrangements are possible. A wall width of 1.2m is suitable because LVL is manufactured in sheets 1.2m wide. It is possible to fabricate longer widths of wall from standard LVL sheets. Walls 3.0m wide were used at the NMIT building in Nelson. The height of pre-fabricated walls can be up to 20m or more, depending on facilities for fabrication and transport. A disadvantage of wide walls is that the vertical displacement at the ends of the wall can become very large for even a modest level of horizontal seismic movement at each floor level. Connections between walls and slabs or beams must be carefully detailed to avoid damage under seismic loading.

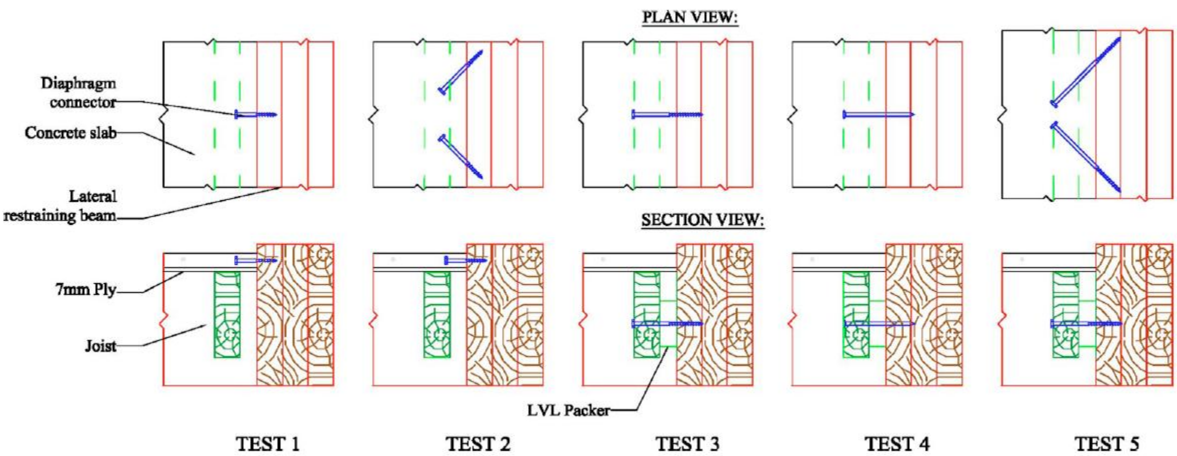
The prefabricated walls are made with full length vertical ducts for the post-tensioning tendons. Walls can be coupled to each other with the U-shaped flexural plates described earlier. Another source of energy dissipation can be ductile steel rods connecting the base of the walls to the foundation, either with replaceable external steel rods or with hidden steel rods epoxied into the walls.

9.4.3 Floors and connections to seismic resistant systems

Many options are available for floors in multi storey timber buildings. The most popular floor system is a timber-concrete composite (TCC) floor, with LVL joists supporting plywood decking and concrete topping, as shown in Figure 9.10(a). The shear connection between the concrete topping and the LVL web can be concrete in notches (as shown) or many different possible arrangements of screwed fasteners. Prefabricated panels of decking and joists can be delivered to site for rapid erection, with the concrete topping cast in-situ. Alternatively the TCC floor units can be precast in a factory. The main reason for the concrete topping is to improve the acoustic performance of the floor, with secondary benefits of increasing thermal mass, fire resistance, and providing a rigid diaphragm for lateral forces. Screws are becoming the preferred type of fastener because they are inexpensive, self drilling and easy to insert, and they can be removed later if necessary. Screws can transfer forces through dowel action in shear, or more efficiently through tension if they are inserted at 45 degrees to the wood surfaces.

(a) Prefabricated timber-concrete composite floor.

(b) Test apparatus of slab-to-frame connections.



(c) Some possible slab-to-beam connection arrangements (Newcombe et al., 2010b).

Figure 9.10. Timber-concrete-composite floors and attachment of topping concrete to the lateral load resisting system.

Most slabs are supported on beams, with the concrete topping usually running over the beams or sometimes running into the sides of the edge beams. In all cases, suitable connections are necessary to transfer horizontal diaphragm forces from the slab topping into the lateral load resisting system. This can be done with studs on top of the beam, or into the sides of the beam. The fixing system may be different depending on whether the concrete is precast or cast-in-place. However, as stated in chapter 7, beam elongation effects always need to be addressed to avoid any significant cracks in the concrete topping. An alternative design philosophy is to create an "articulated slab" which is connected through fasteners to the seismic resistant systems. Figure 9.10(b) shows some typical slab-to-beam connections. Experimental investigations described by Newcombe et al. (2010a) confirmed the enhanced performance of the timber-to-timber solution for both constructability and post-earthquake reparability.

9.4.4 Two-thirds scale LVL test building

Numerical investigations which comprise simplified modelling techniques, already implemented for concrete (Palermo et al., 2005) and design procedures (Newcombe et al., 2010) have been continuously implemented since 2005.

After a preliminary phase of extensive experimental testing of components, a two-thirds scale test building has been constructed under commercial conditions, and tested under simulated seismic loads. The building incorporated all the technical solution preliminary developed and faced also important issues related to slab-to-seismic resistant system (Newcombe et al., 2010). Timber composite slabs have been used with the concrete topping cast in site. The building has two storey-two bay frames in one direction and two coupled walls in the opposite direction. Several quasi-static tests in both directions and combined confirmed that the damage might be only limited to the external dissipaters located between the beam and the column.

(a) Two-storey post-tensioned frame building after stressing. **(b) Post-tensioning in progress.**

Figure 9.11. Post-tensioned beam column connections in the University of Canterbury test building.

9.5 Recent New Zealand buildings

Following the research described on post-tensioned timber 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 this technology was the NMIT building, constructed in Nelson. This building has vertically post-tensioned timber walls resisting all lateral loads as shown in Figure 9.12 (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.

(a) The building under construction. Post-tensioned walls to the right.

(b) The building under construction. Post-tensioned walls to the right.

(c) Finished building.

(d) Threaded couplers for Macalloy bars at base of wall.

Figure 9.12. Three storey Nelson building (NMIT) with post-tensioned walls.

The Carterton Events Centre, located 100km north of Wellington, is the second building in the world to adopt the Pres-Lam concept, as shown in Figure 9.13. 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 9.13(e)). The structural part has been completed and the building will be opened in October 2011.

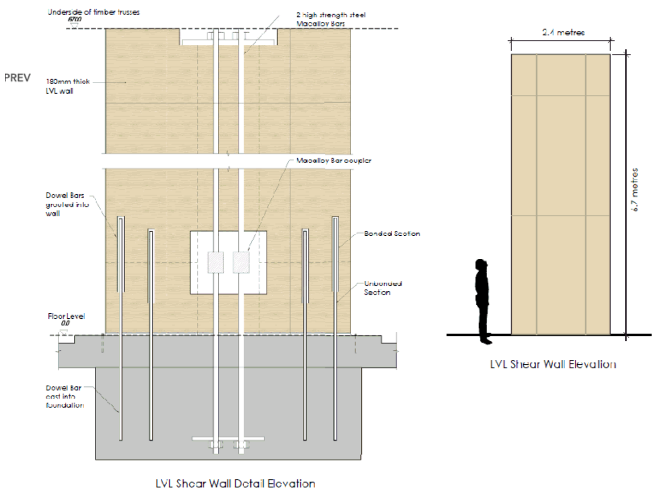
The University of Canterbury EXPAN building (Figure 9.14) is the same two-thirds scale test building as described in section 9.4. After a successful testing programme in the laboratory, the building was dismantled and re-erected as the head office for STIC (Structural Timber Innovation Company Ltd). 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 9.14(a).

(a) Architect's rendering.

(b) Interior construction view.

(c) Exterior construction view.

(d) Erecting wall panel.



(e) Wall details.

Figure 9.13 Carterton Events Centre. Single-storey building with LVL truss roof.

(a) Post-tensioned frame being erected.

(b) Structural framework in place.

(c) Finished building.

(d) Finished building.

Figure 9.14. EXPAN timber building on University of Canterbury campus.

The new College of Creative Arts building for Massey University's Wellington campus is now under construction. The building is the first to combine the post-tensioned timber frame described earlier with innovative draped post-tensioning profiles to reduce deflections under vertical loading. This is a composite material damage-resistant building which relies on rocking precast concrete walls in one direction and Pres-Lam timber frames in the other direction.

(a) Three storey structural frames.

(b) Detail of post-tensioned anchorage

(c) Beam-column joint test at the University of Canterbury.

(d) Draped tendons between beams.

Figure 9.15 Massey University building, Wellington.

References

Buchanan, A.H., Deam, B.L., Fragiacomo, M., Pampanin, S. and Palermo, A. (2008). Multi-Storey Prestressed Timber Buildings in New Zealand. Structural Engineering International. Journal of the International Association for Bridge and Structural Engineering (IABSE). Vol. 18, No. 2. pp 166-173. May.

Christopoulos, C., Tremblay, R., Kim, H. J. and Lacerte, M., (2008). "Self-Centering Energy Dissipative Bracing System for the Seismic Resistance of Structures: Development and Validation," ASCE Journal of Structural Engineering, 134(1), 96–107.

Clifton, G.C., (2005). Semi-Rigid joints for moment resisting steel framed seismic resisting systems. PhD Thesis, Department of Civil and Environmental Engineering, University of Auckland.

Danner, M., Clifton, G.C., (1995). Development of Moment-Resisting Steel Frames Incorporating Semi-Rigid Elastic Joints: Research Report; HERA, Manukau City, New Zealand, Report R4-87.

Davies, M., (2007). Long Term Behaviour of Laminated Veneer Lumber (LVL) Members Prestressed with Unbonded Tendons. Department of Civil and Natural Resources Engineering, University of Canterbury.

Devereux, C.X., Holden, T.J., Buchanan, A.H. and Pampanin, S., (2011). NMIT Arts & Media Building- Damage Mitigation Utilising Post-tensioned Laminated (Pres-Lam) Timber Walls. Proceedings, 9th Pacific Conference on Earthquake Engineering, Auckland. Paper 90.

Giorgini, S., Neale, A., Palermo, A., Carradine, D., Pampanin, S. and Buchanan, A.H., (2010). Predicting Time Dependent Effects in Unbonded Post-tensioned Timber Beams and Frames, CIB-W18 Proceedings: Paper 43-7-7, Nelson, New Zealand.

Iqbal, A., Pampanin, S., Buchanan, A.H. and Palermo, A., (2007). "Improved Seismic Performance of LVL Post-tensioned Walls Coupled with UFP devices", Proceedings, 8th Pacific Conference on Earthquake Engineering, Singapore.

Iqbal, A., Pampanin, S., Buchanan, A.H. and Palermo, A., (2010). "Seismic Performance of Full-Scale Post-Tensioned Timber Beam-Column Joints", Proc. of the 11th World Conference on Timber Engineering, Riva del Garda, Italy.

Newcombe, M.P., Pampanin, S., Buchanan, A.H. and Palermo, A., (2008). "Section Analysis and Cyclic Behaviour of Post-tensioned jointed Ductile Connections for Multi-storey Timber Buildings", Journal of Earthquake Engineering, Vol. 12:1, pp. 83-110, 2008.

Newcombe, M.P., Pampanin, S., and Buchanan, A.H., (2010a). Design, Fabrication and Assembly of a Two-Storey Post-Tensioned Timber Building. Proceedings, 11th World Conference on Timber Engineering, Riva del Garda, Italy, 2010.

Newcombe, M.P., Van Beerschoten, W., Carradine, D., Pampanin, S. and Buchanan, A.H. (2010b). "In Plane Experimental Testing of Timber – Concrete Composite Floor Diaphragms", Journal of Structural Engineering 136, pp. 19-27, 2010.

Palermo, A., Pampanin, S., Buchanan, A.H., and Newcombe, M.P., (2005a). "Seismic Design of Multi-storey Buildings using Laminated Veneer Lumber (LVL)," Proceedings, New Zealand Society for Earthquake Engineering Conference, Wairakei.

Palermo, A., Pampanin, S. and Carr, A.J., (2005b). Efficiency of Simplified Alternative Modelling Approaches to Predict the Seismic Response of Precast Concrete Hybrid Systems. fib symposium, Budapest.

Palermo, A., Pampanin, S., Fragiacomo, M., Buchanan, A.H. and Deam, B., (2006a). "Innovative Seismic Solutions for Multi-Storey LVL Timber Buildings", Proceedings, 9th World Conference on Timber Engineering, Portland, Oregon, USA.

Palermo, A., Pampanin, S. and Buchanan, A.H., (2006b). "Experimental Investigations on LVL Seismic Resistant Wall and Frame Subassemblies", Proceedings, First European Conference in Seismic Engineering and Seismology, Geneva, Switzerland. Paper 983.

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

Smith, T., Pampanin, S., Fragiacomo, M. and Buchanan, A.H., (2008). Design and Construction of Prestressed Timber Buildings for Seismic Areas. Proceedings, 10th World Conference on Timber Engineering, Miyazaki, Japan.

Smith, T., Fragiacomo, M., Pampanin, S. and Buchanan, A.H., (2009). Construction Time and Cost for Post-Tensioned Timber Buildings. Construction Materials. Institution of Civil Engineers, UK. Vol 162, No CM4, 2009. pp. 141-149.

Van Beerschoten, W., (2011). Experimental Investigation on the Stiffness of Beam – Column Connections in Post-Tensioned Timber Frames”, Structural Engineers World Congress SEWC, Como, Italy.

10 STEPS TO ACHIEVE THESE SOLUTIONS

10.1 Possible changes to Building Code, NZ Standards

The previous chapters confirmed that damage-resistant technical solutions are available and more importantly have been implemented in real buildings. These successful applications to real case studies are mainly due to the strong interaction between practitioners and researchers through the help of associations like CCANZ (Cement Concrete Association), HERA (Heavy Engineering Research Association), and the New Zealand Timber Design Society.

Figure 10.1gives an overview of building regulation in New Zealand as it relates to the New Zealand Building Code System (From the DBH report on the building regulatory framework to the Royal Commission, May 2011).

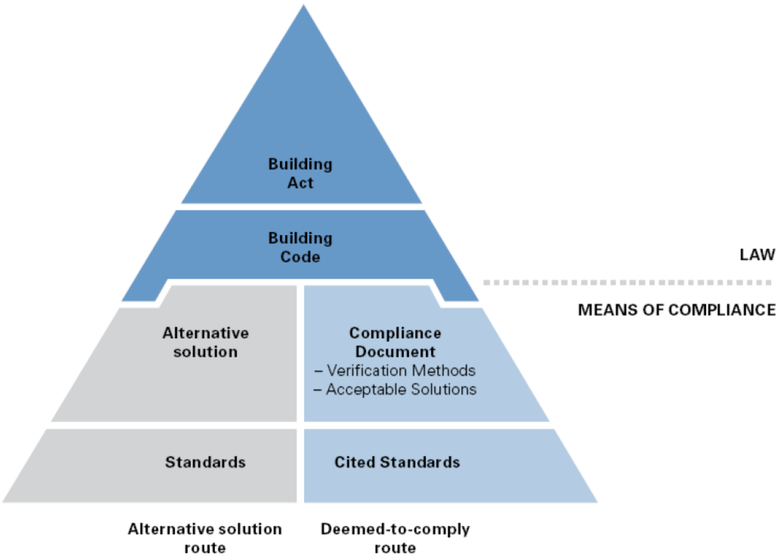


Figure 10.1 Overview of building regulation in New Zealand.http://canterbury.royalcommission.govt.nz/vwluResources/DBH2/$file/SystemofBuildingControls2.pdf**)**

The Building Act provides the legislative framework for administration of building controls in New Zealand. The New Zealand Building Code (a schedule to the Building Regulations) provides performance statements for all aspects of building design. The Building Code needs a review in the area of damage-resistant seismic design.

Figure 10.1 shows three different pathways for building owners and their structural designers to comply with the New Zealand Building Code. Compliance can be demonstrated using either an **Acceptable Solution** or a **Verification Method** in the "Deemed-to-comply" route, or by using an **Alternative Solution** in the "Alternative Solution route".

Most new buildings are designed in accordance with approved Verification Methods, those generally being the loadings standard (AS/NZS 1170 and NZS 1170.5), together with the material design standards (NZS 3101 Concrete Structures, NZS 3404 Steel Structures, and NZS 3603 Timber Structures). Many new or innovative damage-resistant building systems are not called up with specific design methods in these Verification Methods, in which case a full or partial "Alternative Solution" must be offered. The same applies to base-isolated buildings.

Some practitioners feel less confident to design through an "alternative" route, rather than conventional design using an accepted Verification Method. In order to have a stronger impact on practitioners a proper regulatory process needs to be introduced for the design of buildings protected by base isolation and/or damage-resistant design procedures. This will require some changes to NZS 1170.5 and new technical solutions in the cited material standards (NZS 3101 Concrete Structures, NZS 3404 Steel Structures, and NZS 3603 Timber Structures). Only NZS 3101 currently has an appendix (Appendix B) which gives a design procedure for PRESSS-technology in reinforced concrete structures.

Researchers actively involved in the development of damage-resistant systems have already disseminated design procedures through workshops and seminars. For example, last year a PRESSS design handbook, prepared by University of Canterbury, was disseminated through CCANZ to structural engineering practitioners. For timber structures, design seminars are frequently run and within the next year, specific design guidelines will be issued through STIC (the Structural Timber Innovation Company Ltd). Steel industry seminars are also held on a regular basis.

It is clear that design procedures and technologies are already viable options. In order to have damage-resistant systems becoming "ordinary structural systems" a dedicated section for damage-resistant design needs to be incorporated into each material design standard. New educational support for structural designers, building inspectors and review engineers is covered in the next section.

Modern performance-based design concepts have to be properly specified and documented in NZS 1170 Part 5. This may need an introductory document as a preliminary step. More importantly, promotion of these damage-resistant systems should be accompanied by new innovative design philosophies which are more strongly "displacement oriented", in order to reduce structural and non-structural damage in future earthquakes. Both alternatives, the newer **Displacement-Based** design methods and more traditional **Force-Based** design methods, should be properly detailed in the standards and an "open choice" should be left to the practitioners, provided that seismic performance and building displacements are adequately addressed.

10.2 Educational Needs

By and large, members of the structural engineering profession in New Zealand are well educated in seismic engineering compared with other earthquake-prone countries, given that most only have a four-year B.E. degree. Structural engineering education at New Zealand universities has had a significant emphasis on earthquake engineering, but the structural engineering content of the B.E. degrees at the University of Canterbury and the University of Auckland has been steadily dropping over the past few decades.

For some years there has been recognition that a four year B.E. degree is insufficient to give practising structural engineers all the education and skills they need to design sophisticated modern buildings for earthquake resistance. This is certainly the case if the structural engineering profession is to be up-skilled across New Zealand to design a new generation of damage-resistant buildings.

For some years the University of Canterbury has been recommending that a Masters degree should become the accepted entry point to the engineering profession. This would require additional study (an extra 1 to 1.5 years) beyond the current B.E. degree. In the aftermath of the 2010 and 2011 Christchurch earthquakes, this is seen as an essential change to the future education of professional structural engineers.

Another major advantage of a new Masters Degree in earthquake engineering would be the opportunity for practising structural engineers to up-skill their knowledge on a course-by-course basis, leading to a new qualification for those that complete enough courses. Flexible delivery methods could allow practising engineers to attend block courses as required at the University of Canterbury and the University of Auckland.

At a lower more urgent level, the universities need to work with IPENZ and the other learned societies to provide short-courses for new graduates and practising professional engineers. Short introductory courses in earthquake engineering should also be made available to architects, quantity surveyors, the insurance industry, building inspectors, and many others in the building industry. Such short courses could be accompanied by much wider publicity to the general public about the high level of awareness of engineering education, and the responsibility of building owners to safeguard their buildings against earthquakes.

The government needs to see an investment in earthquake engineering education and research as a long-term investment in a safer New Zealand, with much reduced loss of life and property damage in future earthquakes.

A widely discussed idea in the structural engineering community is the introduction of a "seismic star-rating" for earthquake performance of individual buildings. This could be a suitable way of raising awareness of earthquake safety among building; owners and occupiers.

10.3 Research Needs

Research and education must go hand-in-hand if new cost-effective design methods are to be introduced for a whole new generation of damage-resistant buildings.

At the research level, the resources must be provided to allow continued growth of the innovative Kiwi technology which has led to many of the design strategies described in this report. Implementation will require a modest investment in people, resources and facilities, mainly at the Universities of Canterbury and Auckland, but also at associated research hubs such as BRANZ, GNS Science, and other organisations responsible for research and testing of structural materials.

On the world stage, the structural testing facilities in New Zealand are woefully inadequate. There is no need to duplicate the huge and very expensive shaking tables in Japan, US and Europe, but a few million dollars of appropriate investment could provide world class experimental testing facilities and the associated computational analysis tools.

Testing facilities are needed, not only for academic researchers to test innovative new building structures, but also to allow practising structural engineers to create trial prototypes of their new designs. The New Zealand construction industry has a unique opportunity to lead the world in the design and construction of damage-resistant buildings, but that will only be possible if the New Zealand research community can back them up with design tools, expert advice, and testing facilities.

New testing facilities are necessary not only for the development of new techniques and materials, but also to give confidence to practising engineers, and allow them to test innovative ideas before using them in new damage-resistant buildings.

11 CONCLUSIONS

- A large number of modern buildings in Christchurch are facing demolition because of the very high costs of repairing structural damage, compared with the insurance cover.
- The high cost of damage in the Christchurch earthquakes has shown that future New Zealand buildings should be designed to be much more damage-resistant than the current building stock.
- There are three basic alternatives for designing buildings which will suffer much less damage in earthquakes. These three can be combined in many different ways. The three basic alternatives are:
    - Design for much higher levels of earthquake loading (overdesign).
    - Provide base isolation to reduce the earthquake response of critical buildings.
    - Use new damage-resistant design methods.
- Techniques for base isolation and damage-resistant design are available, from New Zealand and from around the world.
- With good design, base isolation and damage-resistance does not need to result in more expensive new buildings.
- A modest investment in education and research is necessary for rapid and effective implementation of new design methods.
- Financial incentives (such as tax incentives for research and development) will encourage innovation in modern earthquake engineering.
- There is potential for New Zealand to become a world leader in the export of earthquake engineering design services.
- Modern earthquake testing equipment is urgently required in New Zealand, for the development of new techniques and materials, and to give confidence to practising engineers.

11.1 Summary

The recent Christchurch earthquakes present a huge challenge and a huge opportunity to New Zealand's professional engineers. Now is the time to show how Kiwi structural engineers and geotechnical engineers can contribute to a sustainable cityscape for the new Christchurch, designing attractive and safe modern buildings which will not suffer the fate of today's older buildings in future earthquakes. The tools are available, with only a modest investment in building codes, education, and research necessary to make it happen.