Some NZ Earthquake Lessons and Better Building Construction

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Abstract

Over the past few years, the South Island of New Zealand has been subject to significant sequences of earthquake shaking. In particular, 2010-2011 events affected the city of Christchurch, resulting in widespread demolition of buildings. Also, the recent and continuing 11/2016 events caused severe damage in the countryside, in small towns, and moderate damage further afield. This paper summarizes general lessons associated with these events. It also describes "low damage construction" methods being used in NZ, and especially in the Christchurch rebuild, to limit the possibility of building demolition in future large seismic events. The buildings used in the Christchurch rebuild are generally supported by structural steel framing. These steel buildings include BRB systems, EBF systems with replaceable active links, rocking systems, base isolation using friction pendulum systems and/or lead-rubber dissipaters, RBS beams, lead extrusion dissipaters, yielding flexural dissipaters, and friction connections. Concerns about a number of currently used systems are discussed. It is shown that subjective quantitative tools, rather than purely probabilistic ones, may be more useful to engineers as they decide what structural system to use.

Keywords: New Zealand, seismic, lessons, steel structures, low damage

1. INTRODUCTION

This paper discusses two issues – (i) Lessons from recent earthquakes in NZ, and (ii) Structures used in NZ after the 2010-2011 Canterbury earthquake sequence.

The 2010-2011 Canterbury earthquakes are known for their liquefaction, for the collapse of mainly older structures, and for causing demolition of the city of Christchurch. The more recent November 2016 event (Kaikoura earthquake) had long duration shaking, caused landslides, isolated a community, and caused significant damage in some modern buildings. While the November shaking is continuing, with significant aftershocks still expected, some of the preliminary findings are highlighted.

After the 2010-2011 Canterbury earthquakes, almost all unreinforced masonry buildings, and the majority of buildings over 3 stories in height have been demolished as a result of direct damage, or irregular foundation settlement. In their place a new city is emerging. A significant majority of these structures have structural steel framing and a number are using some of newer techniques to dissipate energy resulting from the earthquakes. Many of these are intended to be "low-damage" structures, without requiring replacement after a major structural event, and some are intended to be directly useable soon after a major earthquake. This reflects the current emphasis from an increased number of clients for rapid return to service following an earthquake series. A number of innovations are based on research that has been, or is being, conducted in NZ. The aims of this paper are to describe the construction, as well as some of the related research. In particular, the following questions are addressed:

1) What is legal framework by which buildings are constructed in NZ?
2) What type of systems are being built in NZ?
3) What interesting forms are being built?
4) What type of systems are not being built in NZ?
5) What design and /or construction concerns exist?
6) How are decisions made related to low damage steel buildings?

2. LESSONS FROM RECENT NZ EARTHQUAKE EVENTS

Much has been written about the 2010-2011 shaking. This includes the findings of the Royal Commission as well as many articles. Some of these findings include the following (MacRae 2013):

1. Significant shaking around Christchurch occurred on previously unknown faults. This is also true for some other recent significant earthquakes around the world. The importance of considering this possibility explicitly in earthquake design codes is therefore reemphasized.
2. Waves released from ground rupture have directionality and local site effects which can result in significantly greater shaking than that considered as a possibility (i.e. the maximum considered event, *MCE*) for a specific region. It is important that building owners and the public are aware of the level of shaking that a city is being designed for, the philosophical approach used for buildings subject to this shaking, and the fact that this may be significantly exceeded. This has also been observed in the 2016 Kaikoura earthquake.
3. Ground deformation effects, including liquefaction, rockfall, and lateral movement on hillsides and in liquefaction prone areas, may have a significant effect on both damage and economic loss in a region.
4. The possibility of significant aftershocks should be included explicitly in earthquake loss and insurance programmes.
5. The "life safety" performance objective for building structures under the design level of earthquake shaking was achieved in shaking much greater than the design level shaking. This indicates that many structures had extra factors of safety due to foundation conditions, the effect of slabs, and non-structural elements, which limited the demand.
6. Engineers are efficient at designing new structures or making additions/alterations to existing structures. However, they generally have little or no experience of damaged structure performance assessment.

Furthermore, while FEMA306 and other documents exist, they were often inappropriate or incomprehensible for the NZ situation, there is little good guidance or help with the decision that must be made for a damaged structure regarding if it should be:

a) left as is,
b) demolished and replaced, or
c) repaired (and if-so, what repair method is best?)

Further guidance and training about this decision, considering likely cost and time issues, is required if engineers are to become competent in this role.

7. While it is possible to protect life with buildings constructed following modern building codes, many structures may need to be replaced following the event(s). Unliveable houses and business premises that cannot be entered cause major social and economic implications for a region. This was a major issue in the Christchurch Central Business District where about one half of the major buildings required replacement. Future efforts should be to develop "low-damage" construction which can be usable after a major event.
8. When people have no acceptable place to live, and/or work, there is a tendency for them to move away from the affected region.
9. Insurance can be very positive bringing billions of dollars into a region. This "new income" to a region provides employment and affects house prices.
10. Insurance companies do not make money by making full payments in a timely manner. They have many incentives to delay payments and to pay the minimum amount possible.
11. Approval for changes or reconstruction from insurance companies may take many years.
12. The issues relating to land use are the same as those for other disasters. These were expressed in The Economist (2012) and are summarised below:

    The right role for government, then, is ... to minimise the consequences when disaster strikes. At present, too large a slice of disaster budgets goes on rescue and repair after a tragedy, and not enough on beefing up defences beforehand.
    Second, government should be fiercer when private individuals and firms, left to pursue their own self-interest, put all of society at risk.
    Third, governments must eliminate the perverse incentives their own policies produce.
    When governments rebuild homes repeatedly struck by disasters, they are subsidising people to live in hazardous places.

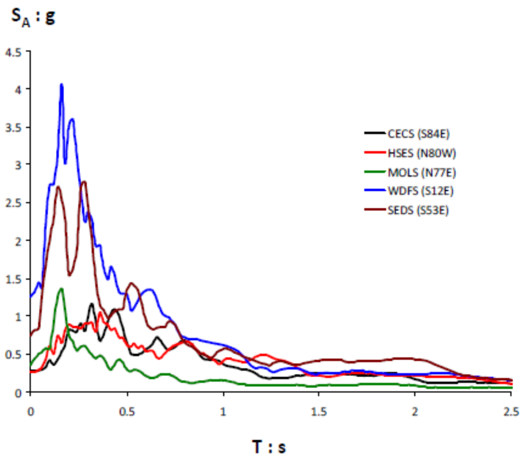
13. Much has been learnt about earthquake engineering over the past decades. It is a credit to our fathers and grandfathers that they developed techniques to prevent building collapse, thus protecting people's lives in a significant earthquake. They ensured a political process that considers this technical knowledge in legal requirements for design and construction throughout New Zealand. It has been estimated that, without these efforts, rather than 185 people dying, there would likely have been around 3000 deaths and many injuries. Gratitude is therefore due to our fathers and grandfathers for their insight, wisdom and determination in establishing these systems.

    At the same time, the lack of insight and wisdom of our fathers and grandfathers has caused significant frustration. While they had performed work and political processes to protect life, they did not aim to protect the infrastructure in very strong shaking. Therefore, while most structures remained standing, many had to be demolished. This has resulted in major economic loss and inconvenience to the region.

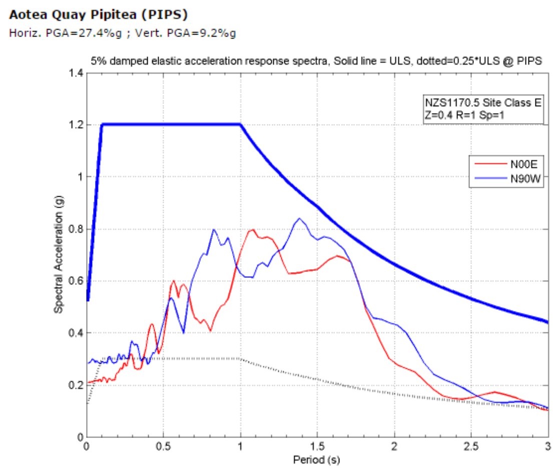
    There is therefore a challenge to this generation to develop systems that will protect our infrastructure. This involves a paradigm shift from "damage-prone" design and construction, to provide construction which will sustain little or no damage during a major event. It involves new technical developments and political changes. Resulting legal requirements must be applied not only to specific buildings, but all buildings in a city/community, as their resilience/performance/sustainability may be affected by the resilience/performance/sustainability of the weakest structural system in the community. This challenge, for the sake of our children and grandchildren, is to not only save lives, but also to economically protect our infrastructure so that our children and grandchildren look back with gratitude on our efforts.

The recent 2016 earthquakes occurred in the northeastern part of the south island of NZ. The major event, on Monday 14th November, had a release of energy much closer to Wellington than suggested by the epicenter location (NZSEE 2016). The magnitude was 7.8, and consisted of rupture on up to six faults. This was the second largest event since European settlement, and the largest since 1855. The shaking was felt around much of NZ. The rupture zone extends approximately 200km, north-north-east past Kaikoura. The shaking lasted approximately 90-seconds in some locations, which is significantly longer than the 20-seconds of the 22 February 2011 Christchurch Earthquake. The land under the sea along the coast rose about 4m over a length of about 100km. The peak ground surface rupture, a right lateral slip, of 10m went through a house. Another 2 storey house with unreinforced brick, Elms Estate, collapsed. Two people were killed as a result of the shaking.

The peak ground shaking was 1.27g in Ward, and the peak 5% spectral acceleration there reached 4g as shown in Figure 1a. While much of the region is sparsely populated, there was considerable damage to slopes and man-made structures.



(a) In High Shaking Regions (Gazetas, 2016)



(b) On a Soft Soil Site In Wellington (Ma and Wotherspoon, 2016)

Figure 1. Spectral Acceleration

GeoNet estimates (from reconnaissance flights) that there may have been from 80 000 to 100 000 landslides. Some of these are large and will take a long time to clear. These have blocked road access on the coastal routed to Kaikoura. Also, with damage along the inland route through Waiau cut off the town of Kaikoura by rail and road. Over 600 tourists caught in this town had to be released by air and boat. The Clarence and Conway rivers were blocked by landslides behind which lakes occurred. In the Clarence the water build up caused concerns for kayakers downstream who were fortunately rescued.

In Wellington, the shaking on a soft soil site is given in Figure 1b. Buildings with periods of about 0.8s-2.0s have been most significantly affected. It may be seen that the shorter period structures, such as many of the older Wellington building stock, have shaking levels similar to the 0.25ULS line (the grey line in the figure) and so did not experience extreme distress. Even for the taller structures, the shaking was less than the ULS level indicated by the thick blue line. It also indicates that the shaking that occurred there was less significant than is expected in a design level "big-one". Efforts are being made by engineers to conduct building assessments to determine if the damage has reduced the resilience of the building significantly, irrespective of whether the building may have previously been categorized as earthquake-prone (NZSEE2016). According to this document a number of issues have been observed. These include:

a) Damage to floor diaphragms in buildings with moment-resisting frames, particularly in buildings with precast floor systems, typically constructed since the late-1970s.
b) Plastic hinge elongation effects have cracked hollowcore units and reduced the seating of precast flooring. There is concern here because seat lengths, particularly in pre 2000 buildings, may not have been large initially, and this can lead to collapse. At least one relatively new structure seems to have sustained such a collapse of this sort. Also, another has an indication of a shear crack in a column (- a vertical beam in the media!), and it is being considered for demolition.
c) Single cracks forming at beam hinge zones of frame structures possibly suggesting reinforcement yielding concentrated in one location.
d) Compromised glazing systems.

Buildings recommended for special consideration are those of 8-15 storeys, on soft soils, or ridgelines. Anecdotal evidence is that many structures on the stiff soils performed well. Large content damage, stretch of carpet, spalling of concrete on supports of precast elements, large cracks remaining open, edge columns moving away from the main floor are possible indicator of structural damage.

While NZ engineers have experience in assessing building damage from previous experiences, this time it is different because no national state of emergency has been declared. Because of this, there is no ability for engineers or the government to enter any building they wish. Entry is at the discretion of the owner. It seems that many owners want buildings to be opened immediately, and want a positive inspection result with no inspection damage. Some engineers seem to be providing such a service. Other engineers are looking at the key structural components which are hidden by wall board or ceilings. A thorough inspection of this sort can result in several thousand dollars of inspection damage, but it provides an indication of the presence of structural damage.

The remainder of this paper discusses recent construction of multistory buildings within NZ.

3. NZ BUILDING LEGISLATION

The NZ Building Code (1992), in Clause B1:Structure, describes some high level functionality and performance requirements to safeguard people from injury, and loss of amenity or property due to structural behavior/failure. These are written in simple, general terms. The NZ Building Act (2004) provides the legislative framework to meet the NZ Building Code. It describes how the legislative systems works, including penalties for non-compliance, as well as how Compliance Documents, which comprise Acceptable Solutions and Verification Methods, can be established and are used. These, if followed, are automatically deemed to satisfy the requirements of the Building Code and the Building Control Authority (BCA) must accept their use. Acceptable Solutions are Compliance Documents presenting simple, highly specified means of compliance, which allow very little freedom for specific design. (They are effectively “cookbook” solutions). Verification Methods are also Compliance Documents but are dependent on application of engineering procedures and some judgment. Alternatively, compliance with the Building Code may be satisfied by “Alternative Solutions”. An Alternative Solution follows provisions that in whole or part are outside the scope of the Compliance Documents. In practice, this Alternative Solution may be deemed to comply with the Building Code, if the design is approved by a Licenced Building Practitioner who is a Chartered Professional Engineer and the Building Consent Authority (BCA) is satisfied as to the procedures used. As part of this, peer review will typically be required at the discretion of the BCA. There are no Compliance Documents for most novel, or “low damage” structural systems, so they are considered as Alternative Solutions.

The Alternative Solutions approach used to satisfy the performance requirements of the NZ Building Code for low damage construction is quite flexible. It allows new solutions to be implemented in actual structures without large disincentive. Sometimes though, peer reviewers may reject a particular solution that is proposed. In other countries, such as Indonesia, Japan, and the USA, for special systems (e.g. tall. irregular or very important buildings, and some buildings using new structural systems), standing expert review panels are involved. This provides consistency over a region.

4. SYSTEMS USED

After the 2010/2011 Canterbury earthquakes, it took time for new buildings to be constructed. This was influenced by many factors, especially the extent of the damage and deconstruction required, the willingness of insurance companies to pay out on damaged buildings and to insure new buildings, and of banks to lend. Some of the earliest buildings were completed after July 2013. Significant building construction has been continuing since then. Steel Construction New Zealand (SCNZ) has collected building data from Christchurch City Council within the Central Business District of Christchurch and other main areas of re-construction, for structures of more than two stories. As at July 2015, the total number of buildings completed or under construction since July 2013, was 69. Of these, 87% of the new total floor area was supported by structural systems of steel, or steel-composite construction. Buildings with mixed construction, or unknown construction were included with reinforced concrete and timber structures as part of the remaining 13%. It is clear that Christchurch has become a structural steel city.

There is no accepted definition of low-damage construction, although proposals have been made (MacRae et al. 2013). A number of possible low damage structural systems for steel buildings were described in MacRae and Clifton (2013b). These include nominally elastic systems, EBF systems with replaceable links, BRB systems, axially yielding devices, flexurally yielding devices, viscously damped systems, base isolation (using sliding friction systems, lead-rubber dissipaters, or both), and rocking systems. These, in addition to the traditional EBFs, MRFs with yielding beams (with or without RBS sections) are being used with composite (CFT), or bare steel, columns. Some examples are given below:

a) Existing steel systems

Of the structures within Christchurch at the time of the earthquakes, only a small proportion of buildings were supported by steel framing. Steel framing was not used until around 1990 and had 50% market share within the CBD by 2010. Thus these buildings were also amongst the most recently built. These structures, except for those with significant foundation issues, were rapidly repaired when repair was required, and reused again. These include (i) the 22 story Pacific Tower on Manchester Street, where an EBF link fractured, and repair of a number of links occurred. This is the tallest structure in Christchurch after the earthquakes; (ii) the 11 storey Club Tower, which housed the Canterbury Earthquake Recovery Authority (CERA) shortly after the earthquakes, and (iii) Les Mills Gym near the location of the collapsed CTV building. It has been argued that since many steel structures are reusable after experiencing shaking which is about 200% of what they were explicitly designed for, that even traditional design of steel structures relying on ductility, may result in low-damage systems. Reasons for this good behavior have been attributed to the high stiffness to strength ratio of steel sections, the contribution of non-structural elements, the short duration of the earthquake shaking, and soil-structure interaction effects. The latter is a major contributor (Storie et al. 2014).

b) Nominally elastic systems

Effective design peak ground accelerations for the 500 year event of about 0.13g, 0.3g and 0.4g, are used in Auckland, Christchurch and Wellington respectively. For New Zealand they range from 0.10g for Kaitaia to 0.60g in Arthurs Pass. Elastic, or nominally elastic (called "nominally ductile" in NZS1170.5), design is more common in Auckland and other low seismic areas than in Christchurch or Wellington. One particular structure is the central Christchurch bus exchange on the SE corner of Lichfield and Colombo Streets. This has a truss roof as shown in Figure 2a. Other structures in Auckland, such as the new University of Auckland labs, are also nominally ductile, as shown in Figure 2b.

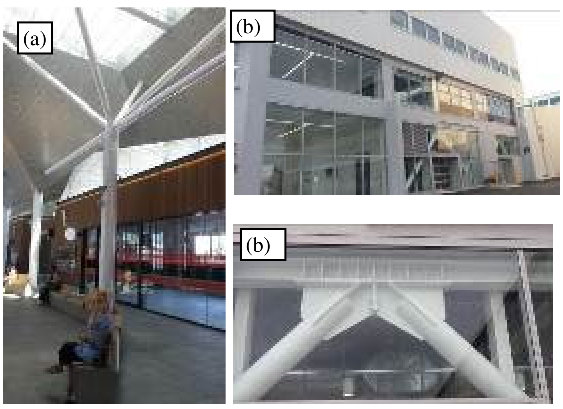


Figure 2. Nominally Ductile Systems (MacRae)

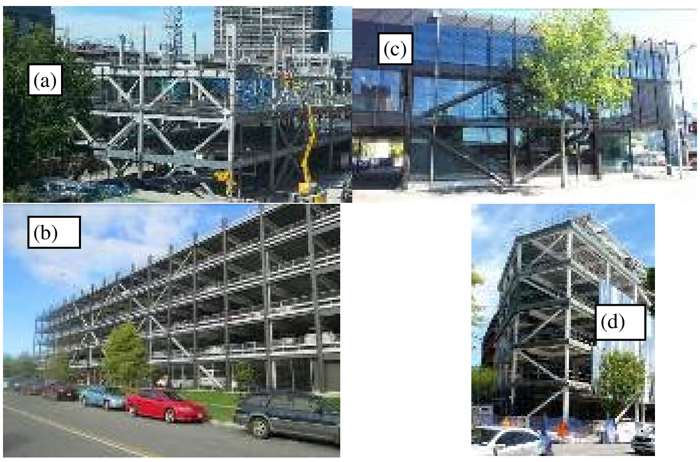


Figure 3. BRB Systems (MacRae)

c) BRB systems

While BRB systems are used widely around the world, no specific regulations exist in NZ and prior to 2011 there was only one home-designed and untested BRB retrofit on the psychology building at the University of Canterbury in Christchurch; built in 1990. Since the earthquakes, these have become very popular, being incorporated into approximately one half of the new steel buildings and often replacing traditional EBFs, which was the principal seismic-resisting system used prior to 2010. BRBs are used in diagonal, or super X, as well as in the Chevron configurations as shown in Figure 3. Some examples here include: (a) UoC Science Annex, (b)14 Hazeldean Rd, Addington, Christchurch 8024, (c) 254 Montreal Street, (d) PwC Centre, Cambridge Terrace. In order of decreasing popularity are braces from StarSeismic, Taiwan, and CoREBrace then “in-house” designs. Both StarSeismic and the Taiwanese braces are being fabricated under licence in Auckland. One frame on the location of the old police station site on Cambridge Terrace was constructed with temporary angles at the location of the BRBs until the braces arrived.

Details of the brace end connections are shown in Figure 4 from (a) 254 Montreal Street, (b) NE corner Lichfield and Colombo Street, (c) PwC centre, (d) UoC Science Annex. It may be seen that (i) columns are a mix of bare steel and CFT, (ii) connections to the gusset plates are a mix of bolted and pinned, and (iii) in general the gusset plates have no stiffeners. The advantage of the pin-end connection is that no moment is induced due to in-plane action, however there are likely to be more issues with fit, and possibly with gusset plate stability. Many NZ designers follow AISC specifications for gusset plate design, where an effective length of the gusset plate used is k = 0.65, even though the mode of failure is sway, implying that this effective length factor should be greater than unity. SCNZ is developing a design guide that is expected to be available during 2016.

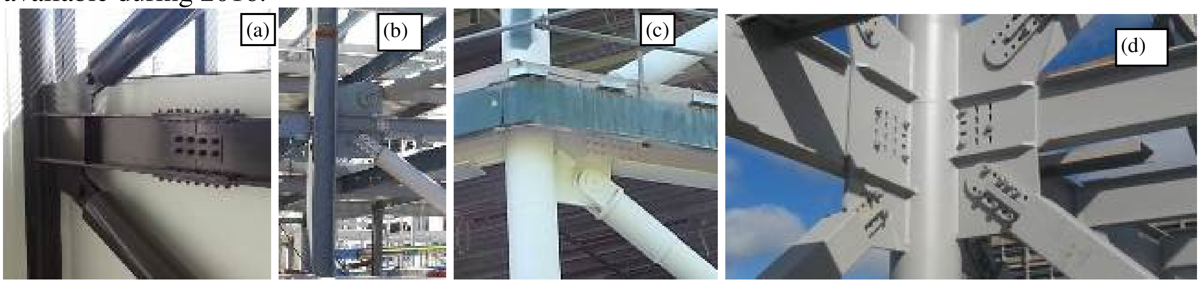


Figure 4. Gusset Plate Details (MacRae)

d) EBF systems with replaceable links

EBFs, with and without replaceable links, are being designed according to a guidance document from HERA (2013). The replaceable link is required to be smaller than the main beam to allow it to be bolted in place. It generally has about 50% of the traditional link shear force resistance. Relative advantages of the replaceable link EBFs and traditional EBFs are given in Table 1. Examples of EBF structures in Christchurch are given in Figure 5 including (a) 120 Hereford Street, (b) 329 Durham St, (c) NW Corner of Lichfield and Barbadoes St, (d) 208 Barbadoes St. The EBF link replacement used in Pacific Tower is shown in Figure 5e (Gardiner et al., 2013).

Table 1. Relative advantages of Replaceable link EBFs and Traditional EBFs

Traditional EBF Advantages:	**Replaceable EBF Advantages:**
• Overall lower detailing costs • No butt weld is required at the critical beam flange location. • The BRB has smaller continuous beams outside the link. • EBFs damaged in the 2010/2011 Canterbury earthquakes were relatively easily repaired (Gardiner et al., 2015). This was by removing a section of the EBF by making cuts in the beams midway between the link end and end of the beams, as well as midway along the braces, and then replacing this portion with a new section	• The links are more easily replaceable. • They are easily imagined/sold as a low-damage system. • The replaceable EBF link may deform in flexure, rather than shear, so no intermediate web stiffeners may be required. • Since the flexural demand at the end of the link is generally significantly less than that of the beam no special considerations are required to protect the beam from yielding.

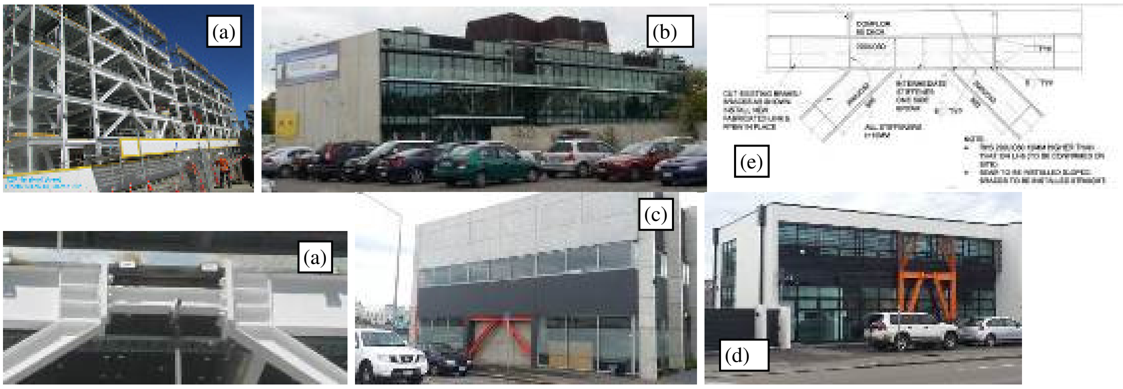


Figure 5. Examples of EBF structures in Christchurch (MacRae)

e) Base Isolation

A number of steel structures are using base isolation. This includes (a) 151 Cambridge Terrace, (b) the Justice Precinct at 121 Tuam Street shown in Figure 6a, and others. The Justice Precinct is required to perform very well and composite circular CFT column connected to beams with external diaphragm connections using bolted connections. The beams have reduced beam sections (RBS) as shown in Figure 6b. A bearing above the column is shown in Figure 6c, and low friction supports in Figure 6d. Similar framing is also used on the north side Kilmore street west of Durham Street. In addition to new design, the existing mainly concrete Art Gallery has been retrofit with base isolation. Lead-rubber as well as sliding friction systems have been used. A guideline for the design of base-isolated systems is due to be released soon by NZSEE.

Figure 6. Some details from the Justice Precinct (MacRae)

f) Friction Moment Directions

These were developed in NZ and used initially in the Te Puni Apartment building at Victoria University in Wellington in 2007. In this building, sliding occurred at the columns at the base of the structure and in the beam to column connections of the perimeter frame in the 2013 Grassmere earthquake. Friction dissipation has been recently used in (a) Forte Health Building on Kilmore Street, and (b) as the main energy dissipation system in the Terraces Project on Oxford Terrace in Figure 7. Oxford Terrace was unique in that Friction connection were used in two directions using external diaphragm connections on a RCFT column as shown in Figure 7b. The base connection was a two way connection with the column sitting on a central pin and oversized holes placed in the baseplate to allow flexural movement in both directions as shown in Figure 7c. No codified design guidelines currently exist for the design of friction connections.

Figure 7. Details of the Friction Connections on the Terrace Project (MacRae)

g) Rocking frame systems

Rocking steel concentrically braced frames with ring spring base hold down systems were also used in the Te Puni Apartment building at Victoria University in Wellington in 2007 and exhibited uplift of some 4 to 6mm in the 2013 Grassmere earthquake. A number of other rocking frames have been constructed since then around the country. One rocking concrete wall was in Christchurch at the time of the Canterbury earthquake. While it exhibited some movement, it was on the north side of Christchurch were excitations were considerably less than in the central city. The first new rocking steel frame buildings in Christchurch after the earthquakes was the Forte Health building, 132 Peterborough Street, which used many interesting systems including friction dissipaters. The rocking walls were coupled and energy is dissipated during relative movement between the walls by lead extrusion dissipaters as shown in Figure 8a. Here, the rocking frames are held down by tendons extending over their height. Vertical deformations of the rocking frame are also decoupled from the slab of the building while allowing horizontal force transfer. An alternative means to hold down the building is used in 141 Cambridge Terrace. Here, the large springs at the base of the structure limit the onset of rocking as shown in Figure 8b, flexural yielding U-shape dissipaters are placed up the side of the frame. The advantage of this system is that the frame members over the height do not need to be sized for the post-tensioning forces, and the spring size can be selected for the appropriate deformations. However, the cost of the springs must be considered.

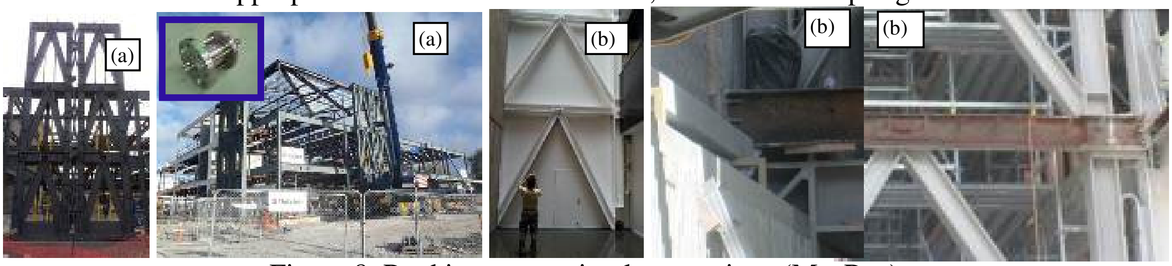


Figure 8. Rocking system implementations (MacRae)

h) Viscously damped systems

A few buildings are including these systems. Devices are either imported (e.g. Taylor devices), or constructed in NZ. Figure 9 shows devices implemented in 12c Moorehouse Avenue.

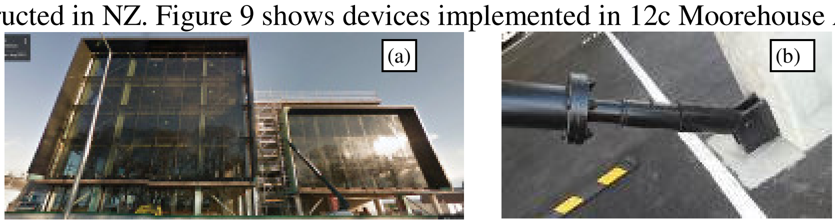


Figure 9. Viscous devices implementation (Google Maps and MacRae)

Figure 10. CBF Application (MacRae)

i) CBF systems

CBFs may be designed according to NZS3404. An example at 124 Kilmore Street is given in Figure 10.

5. INTERESTING STRUCTURAL FORMS

There have been other cases where interesting structural forms have been used. These include:

i) CBFs being placed over 3 stories in an unbalanced manner at the Univ. of Canterbury engineering hub as shown in Figure 11a. The frame design ductility demand here is low.

ii) Beams carrying gravity force which go through the centre of, and are supported by the active link on a EBF building (136 Moorehouse Avenue) are shown in Figure 11b. Again, the ductility demand is stated as being low. There is nothing specific in current NZ standards to prevent this, it was not envisgaed such an appliation would be used and according to SCNZ it will be prevented in the next version of the standard.

iii) An IL3 (moderately important) building, located at the corner of Cambridge Terrace and Armagh Street, has CBF braces on the 2nd, 3rd, 4th and 5th stories, but no braces in the ground (first) storey as shown in Figure 11c. This building is base isolated.

iv) A BRB extension to 32 Oxford Terrace (shown in Figure 11d from Tuam Street) has gravity beams framing into the BRB bay with small (perhaps 2 M16) bolts. Here, it is assumed that the slab will transfer all the force into the BRB bay so that drag structs are not needed. This is quite a different philosophy than that used in the UoC Science Annex (Figure 4d), for example, where drag struts were used to carry beam axial forces. It was not clear whether web doubler plates were needed, or provided, in the beam and column to carry the brace forces from the thick gusset plate into these members from the views obtained.

v) A friction connection seems to be placed in columns of a building in Boulcott St, between St Mary of the Angels Church/O'Reily Ave and Willis Street, Wellington, as shown in Figure 11e (provided by John Scarry). If it is indeed a friction connection, it is not clear how it would work considering the column must carry axial force, and flexure in two directions.

vi) An eccentric cleat connection is provided in a brace at a PakNSave store in Auckland. It is not clear how this will perform under earthquake induced compression.

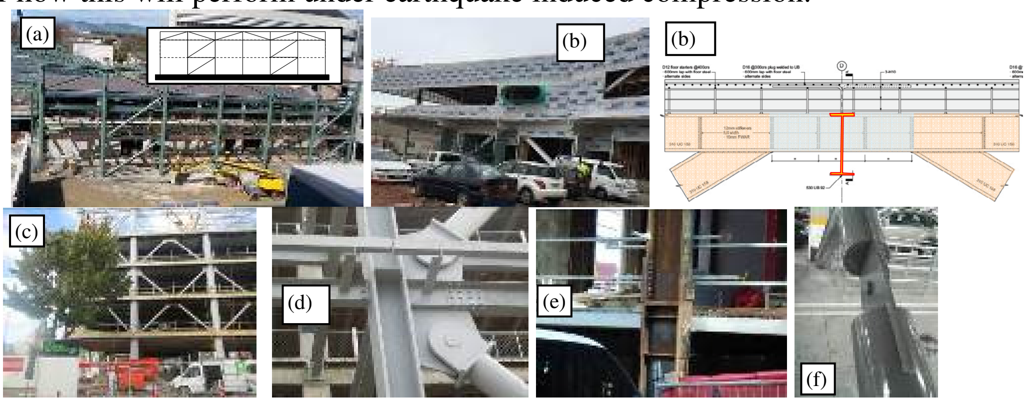


Figure 11. Some Interesting Structural Systems

6. STRUCTURAL STEEL SYSTEMS NOT USED

It is interesting to note, given the significant amount of research that has been undertaken around the world, what systems have not been used in Christchurch. Some are new, like the GripNGrab (discussed later) and friction braces (MacRae and Clifton, 2013). Further studies are continuing on these topics. Another one that is not being used is the post-tensioned beam system. This is because such a system seems to work well on simple subassemblies which are not part of a frame, and which do not have a slab (MacRae and Clifton, 2013). It is interesting to note though, that even though the same issues exist with these systems with other materials, they have been used in buildings with other materials in NZ.

7. ISSUES WITH CURRENT CONSTRUCTION

Since low damage construction is a relatively new concept, many of the new systems being designed and built don't yet have robust design guidelines. While it has been illustrated that these devices be effective in some cases, it does not mean that they will necessarily perform well is one of the design parameters is slightly changed, or if the boundary conditions or loading is more realistic that than considered in the experiments. This has been discussed with respect to the William Tell illustration (MacRae and Clifton, 2015). A number of topical issues are discussed below, many of which are in MacRae and Clifton (2015).

a) BRB issues

While it has been shown that BRBs can perform very well the overall sensitivity to construction tolerances is not generally known or understood. Furthermore, the vast majority of BRB testing around the world considers brace axial load only, or brace axial force together with some in-plane frame action. Since, earthquake cause deformation in two horizontal directions, there is concern that this should be considered when selecting a brace for a particular situation. Furthermore, brace inertial effects, which may be significant on long braces, such as those which are now being built around the world with lengths exceeding 50 feet (about 15m), are not considered. For these reasons, sufficient evidence does not exist to indicate that the BRBs will behave well in all earthquake situations. Other BRB issues are related to the connections at the end of the BRBs, which are generally gusset plates, as described below.

b) BRB gusset plate issues

Gusset plates are attached to the ends of compression/tension members in frames subject to seismic and non-seismic loading. A number of design recommendations – including the Uniform Force Method of Thornton, the proportioning method, and the Generalized Uniform method, and some simplified methods - are available to consider direct forces (MacRae and Clifton, 2013). While frame action effects may be considered standardized generally accepted methods are not available. Methods to explicitly consider the following actions on the gusset plates are not generally available:

i) In-plane moments from brace in-plane bending when brace connections are not totally pinned, and
ii) Out-of-plane moments (e.g. from frame out-of-plane deformation, brace buckling or inertia).

Most gusset plate design procedures (E.g. AISC) consider that the gusset plate has a buckling effective length factor of 0.65-0.70 even though sway is the predominant failure mode. This issues is similar to issues with axial cleat connections which sway and for which mitigation methods have been developed (E.g. MBIE, 2010). Westeneng et al. (2015) used stability functions to obtain the actual gusset plate effective length factor corresponding to overall buckling failure of an elastic gusset plate connected to an elastic BRB brace member as shown in Figure 12a. The upper line on this graph can be approximated by the equation $k_{gusset} = 1 + 0.55\alpha^{0.58}$, where $\alpha = (EI/L)_g{}^2/(EI/L)_{\mathrm{BRB}}{}^2$. They showed that the effective length factor is (i) dependent of the stiffness of the brace, (ii) greater than unity as would be expected for a sway element, and (iii) may be greater than 3.0 for high gusset plate stiffnesses.

An implication of this is that gusset plates designed according to standard methods, with small effective length factors, may fail under compressive load alone at strengths lower than that specified in the standards. This has been shown and replicated by Westeneng et al. (2015) as shown in Figure 12b. Westeneng et al. (2016) also demonstrated by calibrated FEM analysis that current recommendations are not conservative. Furthermore Sitler et al. (2017) have shown that a number of failures of BRB systems have occurred in the past and Takeuchi et al. (2016) have proposed a general method to consider these things. Simple guidance from this method is not yet available in English. There is also the concern that if gusset plates are too slender, they may buckle. However, if they are too stocky, they may yield during out-of-plane deformation compromising their performance in subsequent cycles.

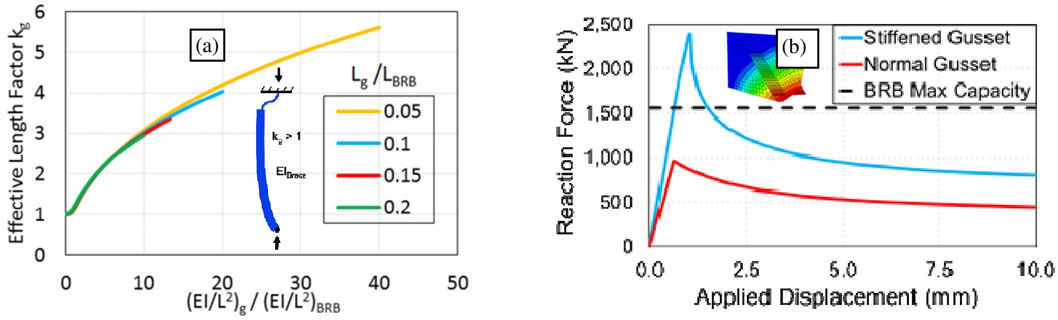


Figure 12. Gusset Plate Performance (Westeneng et al., 2015)

To prevent failure of the gusset plate, the weld may be sized for the capacity of the plate using capacity design principles. While many engineers do size the welds this way, some use weaker welds, or simply consider the capacity of the brace and to determine a uniform weld size. This may be problematic if there are non-uniform distributions of force in the gusset plate due to element flexibility, frame (beam-column) opening effects, brace in-plane moments, brace force eccentricity, out-of-plane bending of the plate from member out-of-plane deformation etc. A weld distribution factor which may account for some of these actions, but it is not clear how robust it is in general. In addition to the weld, the elements in to which the gusset plate is connected, i.e. the beam and columns, should have sufficient strength the carry the demands as described by Palmer et al. (2016).

Recent tests by Bruneau indicate good performance of BRBs in 2-D horizontal loading with drifts of 6% in each direction indicating that systems can, with care, provide satisfactory performance.

A group of concerned BRB researchers met at the Shanghai STESSA conference in July 2015 and agreed to work together to solve the issues associated with gusset plates. These include key researchers from Italy, Taiwan, USA as well as NZ and the committee chair is Kevin Cowie from SCNZ. A further informal meeting was held at the 2016 NZSEE conference with representatives of CoreBrace, StarSeismic, Takeuchi group, a consultant, a BRB fabricator, and research groups from Auckland and Canterbury universities.

Development of design guidelines for BRB frames has been initiated by a Steel Construction NZ (SCNZ) coordinated working group and the guidelines are expected in 2016.

c) Externally Mounted Tension-Compression Yielding Devices (EMTCYD)

These are mini BRBs used by Marriott (2009) in tests with concrete rocking systems. They have been implemented mainly in concrete and timber structures around NZ as a means to dissipate energy. Implementation in a timber structure in 11 Birmingham Drive is shown in Figure 13a and b (MacRae). The majority of the testing of these devices in their development was conducted under unidirectional loading. However, in recent bi-directional testing by Gultom and Ma (2015), it was shown that they buckled and lost strength. The test setup and buckled dissipater are shown in Figure 13c and d (Gultom and Ma, 2015). The comment was also made that if the design had been different they may have worked.

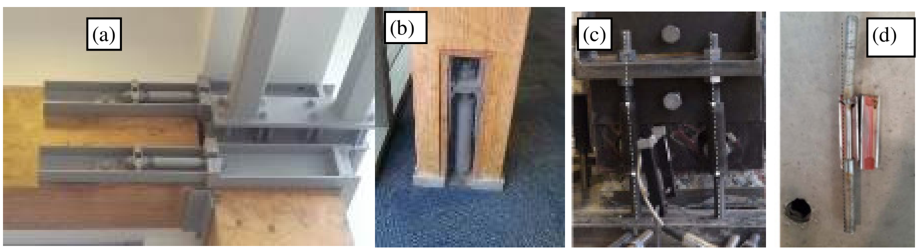


Figure 13. Tests of Externally Mounted Tension-Compression Yielding Devices (MacRae, Gultom and Ma)

(d) Friction dissipaters

Friction dissipation has been shown to have promise as a means of dissipating energy. However, some recent findings indicate that sliding force coefficient (i) has some dependency on bearing area, (ii) and is sensitive to the (a) bolt characteristics (size, material props etc.), (b) tightening and surface friction, (c) possible heating during the excitation, (d) out-of-plane force causing prying on the connection. For these reasons, a robust method for developing friction which is relevant for design is not available. Also, there is not yet a consistent method to relate symmetric friction connections (SFC) and asymmetric friction connection (AFC) characteristics.

(e) Lead core in base isolation devices

Lead dissipaters used for base-isolation, which were expected to remain intact have shown cracks. This was attributed to be possibly due to the frequent / small amplitude cyclic deformation such as that from wind (Kasai et al. 2012, 2013). Cracks of up to 32mm depth were reported. However no cracks or any signs of distress were noticed in lead rubber base isolation units recently replaced in the William Clayton Building in Wellington, which was the world's first base isolated building completed in 1974. The system was being refurbished to accommodate near fault displacements, which were not recognised in the original design and had the potential to seriously damage the originally designed system.

7. RELATED RESEARCH

Low-damage system research underway at NZ universities has been described by MacRae and Clifton (2013) in order to develop robust design guidelines and address some of the issues described above. This includes:

a) Friction

At the University of Canterbury, recent studies have been conducted on friction braces, and base connections and desirable behaviour has been obtained. Similar work at the University of Auckland has been conducted on frictional rotational links, rotating frames and frictional connections with Belleville springs. References to these are in MacRae and Clifton (2015). The use of a series of stacked Belleville Springs with each bolt has the advantages that (i) after a bolt has elongated due to MPV yielding, it still carries more load than if BSWs are not present. This means that the unloading curve is stiffer, (ii) the bolt load is spread out over a larger area than with a traditional washer and the effective friction factor is slightly increased, and (iii) by partially tightening the bolt, bolt yielding may be minimized and increasing the self-centring from the hysteresis loop. The disadvantages are the cost of the washers and the extra time required to obtain good quality control.

Current design recommendations for asymmetric friction connections with hard (e.g. Bisalloy 400 or harder) shims is to use a sliding force equal to 0.25 multiplied by the number of surfaces in sliding (normally 2) multiplied by the number of bolts multiplied by the proof load per bolt. A strength reduction factor, ϕ, of 0.7 for friction is used, and the overstrength for the connection, ϕ_o, considering bolt, and surface variations is 1.40. The value of 0.70 is consistent with the steel standard value for friction, and the value of 1.40 was obtained from observations from actual tests (MacRae and Clifton, 2015). These need further validation.

b) Rocking Systems

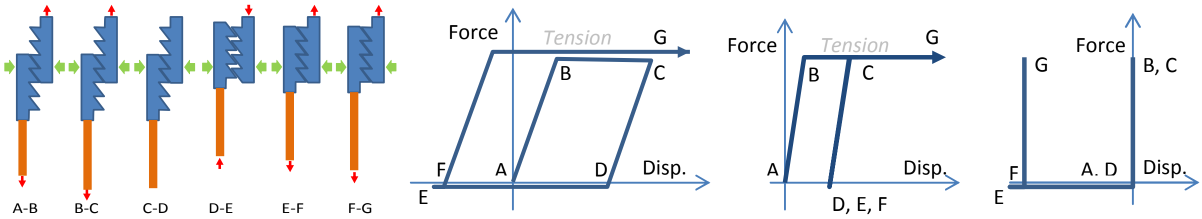
Studies are underway to determine the frame displacements, moments and shears, due to rocking (E.g. Kordani et al, 2016).

c) Building Straightening

Since low damage structures may have some residual displacement after a shaking event, studies are being conducted about the best way of straightening a building manually as well as modifying the building and using subsequent events (Rad et al. 2016).

d) Tension Only Device

A device to prevent buckling of tension dissipative devices in compression was inspired by plastic cable ties which can carry tension force, but carry little force in compression as they are pushed through the hole. It has similar, but opposite, characteristics to a car axle jack which is a compression only device. The tension only device has the behaviour described in Figure 14. The device itself is shown with the teeth in blue. A small lateral compressive force is required to encourage the two parts not to fall away from each other, so that the teeth engage. The dissipate element is shown in brown. Dissipation may occur due to yielding, frictional sliding or other means. Initially the device is loaded elastically in tension (A-B) then yielding/frictional sliding occurs in the dissipative element increasing its length (B-C). When the force is taken off (C-D) there is some elastic shortening of the dissipative element. When compression force is applied, the device carries very little compression but slides (D-E). When tension force is applied again (E-F), the device slips until the teeth are engaged but the dissipative element does not change in length since the axial force in this stage is very small. The maximum possible E-F distance is the tooth pitch. For greater tensions (F-G), displacement increases in the elastic range and then causes dissipation in the dissipative element as before. Preliminary tests of small scale devices indicate excellent behaviour with monotonic dissipation only of the yielding element (Cook et al. 2016). Such a device has the potential to be used on the outside of rocking walls, in brace, and in other applications requiring energy dissipation. The dissipative element would need to be replaced, and the device reset, after every major event.



(a) Push-Pull Behaviour (b) Full System (c) Dissipative Element (d) Device

Figure 12. Tension-Only System (GripNGrab) Behaviour

8. DECISION

The best structure, or best retrofit, in a given situation may depend on many factors such as the viewpoint of the stakeholder, their vested interests and their insurance policy.

Two methods to support decision-making are described below. They are (i) probabilistic loss assessment (PLA), and (ii) subjective quantitative analysis (SQA).

In the first, all probabilistic information is combined using convolution integrals to obtain scenarios losses (for a particular event), or probabilistic loss (estimating the loss over time). While this type of analysis can be used to quantify dollar losses due to damage, death (and injury), and downtime, many assumptions are required as input information of good quality is seldom available and it is difficult to include all factors. When more accuracy is desired, and more parameters and factors are considered in the analysis, there is also an increase in uncertainty. This uncertainty may swamp the analyses. PLA may be useful though. For example, a break-even analysis (MacRae et al. 2014), can be used to evaluate the "best option" considering both initial (or retrofit) cost, as well as loss smeared out over time including discount rate as a result of natural hazards or other effects. This is the line with lowest total loss at the time of interest.

In SQA, the decision is made based on the outcomes that are seen to be important. For each option, a rating is given for each outcome, and then these are combined to obtain the final rating in a way that seems appropriate to the decision makers using a decision matrix. The "best option" is the one with the highest rating. In a recent study of some building structures (MacRae et al. 2014) factors considered included (i) frame damage, (ii) slab damage, (iii) element replaceability and (iv) permanent displacement. While this approach is subjective, it allows factors which are not quantified easily in a probabilistic approach to be directly included empowering the decision makers. It also includes the information in a way that allows it to be easily communicated to other audiences. A show of hands at the 2014 ASEC conference (MacRae et al., 2014) indicated that the vast majority of consultants prefer this SQA approach to the PLA approach which is regarded as being a black box, difficult to perform, and difficult to check. PLA and SQA may be used together in the decision making process.

Typical low-damage systems offer a significant performance enhancement for additional costs generally less than 5% of the structure cost (MBIE, 2015). However, other disincentives also exist. Many low-damage systems do not satisfy the acceptable solution of the NZ Building Code. They are therefore considered as alternative solutions to meet the minimum performance requirements of the Building Code. This can involve increased design and consenting costs, with peer review as well as testing or test records demanded by the Building Consent Authority (MBIE, 2015). Also, unfamiliarity of different construction processes by the construction community can lead to higher costs.

When the first author was discussing these issues with Minister of Building and Construction Nick Smith at the 2016 NZSEE dinner, the Minister's comments were that, (i) Structural elements and systems used in NZ should be robust for the 3-D shaking expected in an earthquake. If not, they should not be used. (ii) As new structural elements and systems are developed they should be incorporated into NZ standards, (iii)

Standards need to be frequently and regularly updated to allow incorporation of new and innovative systems and therefore, (iv) as a consequence, the “Alternative Solution” should be only seldom used. There is an increased emphasis on funding and resourcing Standards Development to achieve this outcome including the establishment of standing technical committees to facilitate this. (v) If there are problems still, an enquiry may be requested.

9. CONCLUSIONS

This paper shows that while many lessons have been learnt from the 2010-2011 Canterbury earthquakes, the recent 2016 earthquakes have created different challenges. The paper also describes NZ construction and research related to low damage construction. It was shown that:

1) NZ allows design of structures which are demonstrated to meet the Building Code performance requirements. Since there is no NZ specification for low damage structures, most are designed as Alternative Solutions.
2) Many of the buildings in the Christchurch rebuild are supported by structural steel framing. These steel buildings include BRB systems, EBF systems with replaceable links, rocking systems, base isolation using friction pendulum systems, lead-rubber dissipaters or both, RBS beams, lead extrusion dissipaters, yielding flexural dissipaters, and friction connections. Examples of these are provided.
3) A number of structural systems use interesting systems which are useful to initiate discussion about possible seismic performance.
4) Post-tensioned beam systems are not being constructed in steel because they do not always behave as low damage systems.
5) It is shown that significant further work is required on a number of systems before they can be considered to be robust. Some of the work being conducted was briefly described.
6) It was shown that while probabilistic tools may be useful to aid in decision making, most engineers prefer to have a simpler method that they can understand and trust. Furthermore, some suggestions to make NZ low damage structures more robust were described.

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