



New Zealand Research Applications of, and Developments in, Low Damage Technology for Steel Structures

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Abstract

This paper describes New Zealand construction and research related to low damage construction. Following the large scale demolition of buildings in Christchurch after the 2010/2011 earthquake series, many of the buildings in the Christchurch rebuild are 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. Some interesting structural systems are described, and concerns about a number of currently used systems are discussed. Research being conducted in NZ to address some of these issues is listed. Also, it is shown that subjective quantitative tools, rather than purely probabilistic ones, may be more useful to engineers. Also, some suggestions to make NZ low damage structures more robust are described.

Keywords: Steel Structures, Low Damage, Research, New Zealand

1. Introduction

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, which are intended not require 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 capability following the 1+ years operational disruption following that earthquake series. A number of these 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) What research is currently underway related to low damage systems?
- 7) How are decisions made related to low damage steel buildings?

2. 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 judgement. 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.

3. 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 1a. Other structures in Auckland, such as the new University of Auckland labs, are also nominally ductile, as shown in Figure 1b.

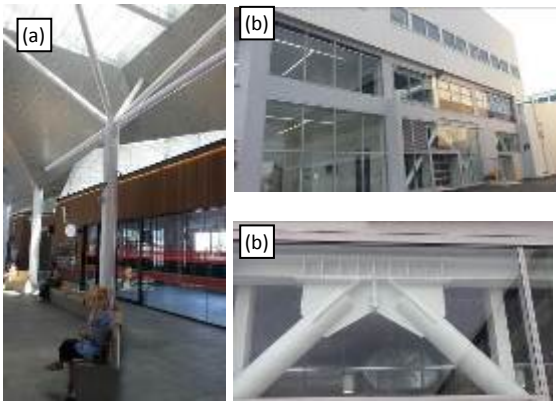


Figure 1. Nominally Ductile Systems (MacRae)

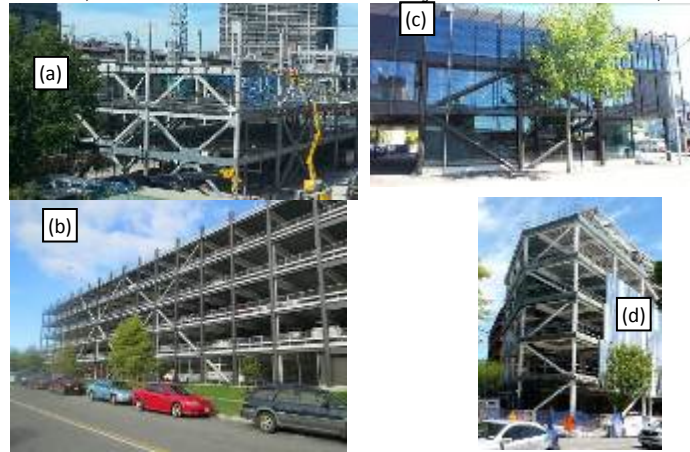


Figure 2. BRB Systems (MacRae)

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 2. 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 3 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.

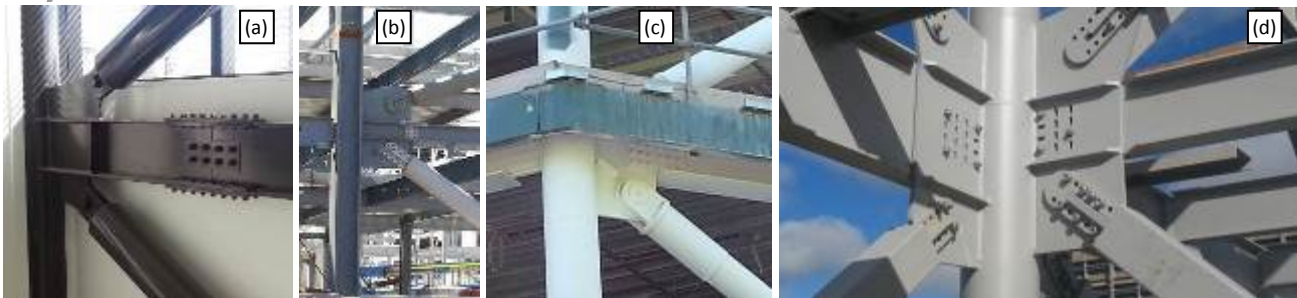


Figure 3. Gusset Plate Details (MacRae)

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 4 including (a) 120 Hereford Street, (b) 329 Durham St, (c) NW Corner of Lichfield and Barbadoes St, (d) 208 Barbadoes St. The EBF like replacement used in Pacific Tower is shown in Figure 4e (Gardiner et al., 2013).

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

Traditional EBF Advantages:	Replaceable EBF Advantages:
<ul style="list-style-type: none"> Overall lower detailing costs No butt weld is required at the critical beam flange location. Because the shear resistance of a traditional link is about twice that of a replaceable EBF link, half as many EBF bays are required. 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. 	<ul style="list-style-type: none"> The links are more easily replaceable. They are easily imagined/sold as a low-damage system. Since the replaceable EBF link is smaller, it is often more slender, deforming 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.

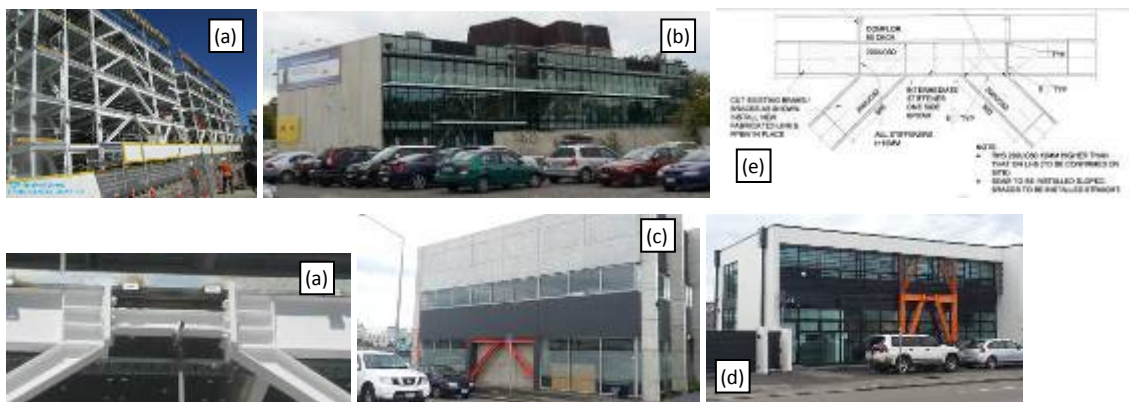


Figure 4. Examples of EBF structures in Christchurch (MacRae)

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 5a, and others. The Justice Precinct is required to perform very well and composite circular CFT columns connected to beams with external diaphragm connections using bolted connections. The beams have reduced beam sections (RBS) as shown in Figure 5b. A bearing above the column

is shown in Figure 5c, and low friction supports in Figure 5d. 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 5. 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. 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 6b. 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 6c. No codified design guidelines currently exist for the design of friction connections.



Figure 6. 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 where 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 7a. 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 7b, 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.

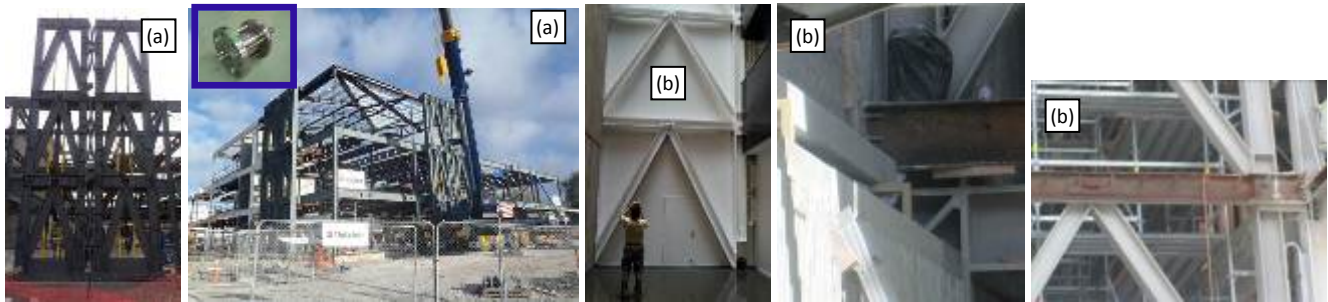


Figure 7. Rocking system implementations (MacRae)

h) Viscously damped systems

A few buildings are including these systems. Devices are either imported (e.g. Taylor devices), or constructed in NZ. Figure 8 shows devices implemented in the in 12c Moorehouse Avenue.



Figure 8. Viscous devices implementation (Google Maps and MacRae) Figure 9. CBF Application (MacRae)

i) CBF systems

CBFs may be designed according to NZS3404. An example at 124 Kilmore Street is given in Figure 9.

4. 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 9a. 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 9b. Again, the ductility demand is stated as being low. There is nothing specific in current NZ standards to prevent this, it was not envisaged such an application 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 9c. This building is base isolated.
- iv) A BRB extension to 32 Oxford Terrace (shown in Figure 9d 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 struts are not needed. This is quite a different philosophy than that used in the UoC Science Annex (Figure 3d), 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'Reilly Ave and Willis Street, Wellington, as shown in Figure 9e (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.

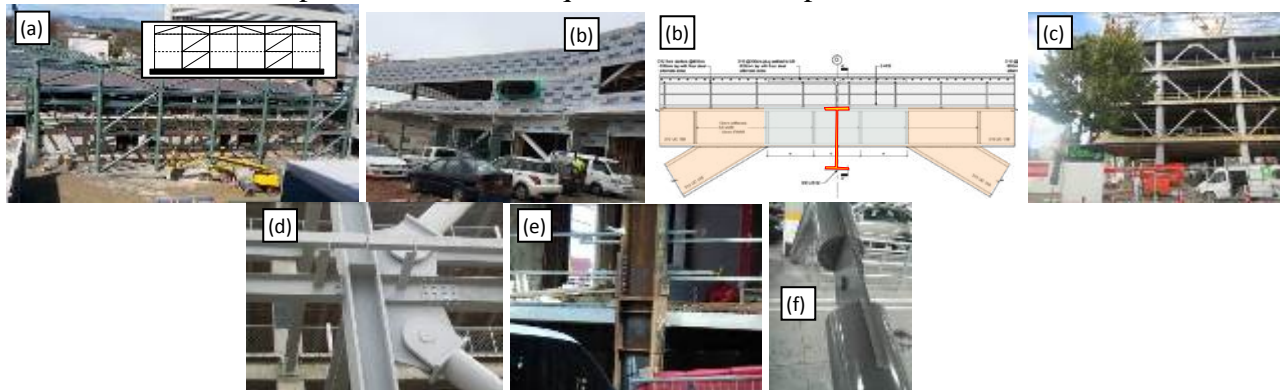


Figure 9. Some Interesting Structural Systems

5. 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.

6. 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 if one of the design parameters is slightly changed, or if the boundary conditions or loading is more realistic 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 described 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 10a. The upper line on this graph can be approximated by the equation $k_{gusset} = 1 + 0.55\alpha^{0.58}$, where $\alpha = (EI/L)_g^2 / (EI/L)_{BRB}^2$. 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 10b. 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.

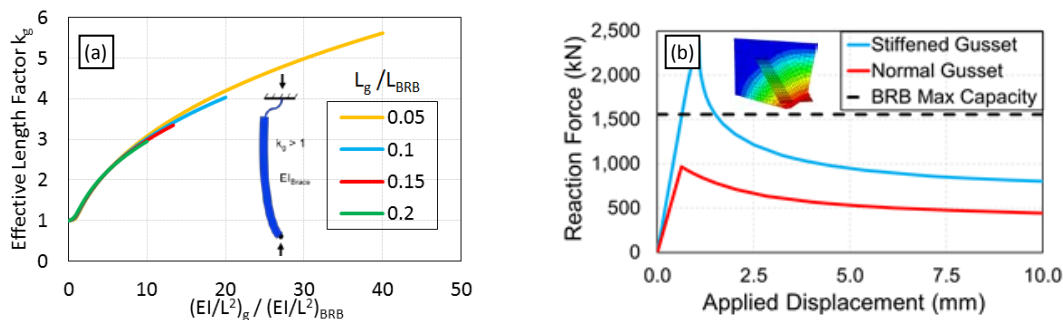


Figure 10. Gusset Plate Performance (Westeneng et al., 2015)

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 11a 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 11c and d (Gultom and Ma, 2015). The comment was also made that if the design had been different they may have worked.

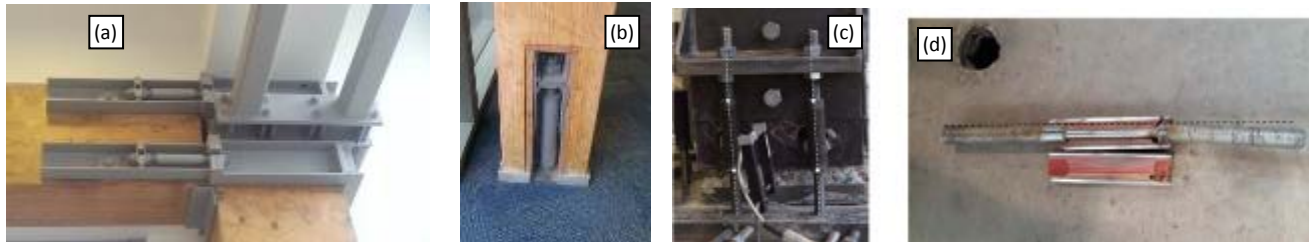


Figure 11. Tests of Externally Mounted Tension-Compression Yielding Devices (MacRae, Gultom and Ma)

(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 12. 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.

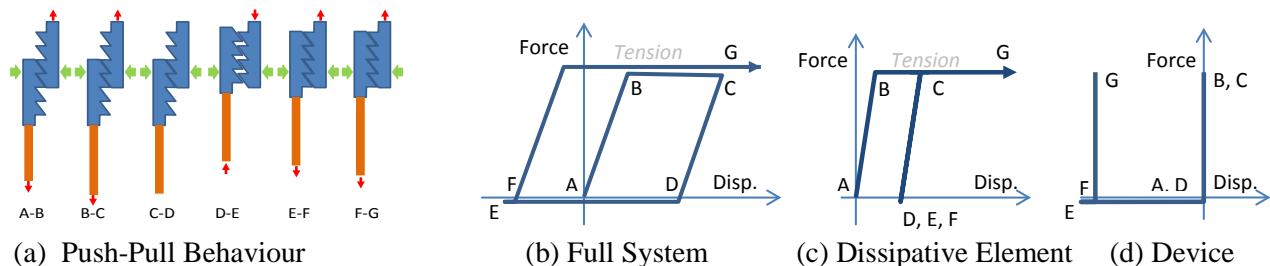


Figure 12. Tension-Only System (GripNGrab) Behaviour

7. 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.

7. Conclusions

This paper 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.
- 6) Some of the work being conducted was briefly described.
- 7) 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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