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RESEARCH ON SEISMIC PERFORMANCE OF STEEL STRUCTURES

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Abstract. *This paper describes some recent and ongoing research at the civil engineering universities (Auckland and Canterbury) to produce better steel structures and to reduce the vulnerability of the built infrastructure. The types of structure considered are (i) new structures designed considering current code performance objectives, (ii) existing structures, (iii) nascent structural systems, and (iv) damaged structures. Effective decision support tools to assist in determining what approach is best are also discussed. As the different structural types above are described, research efforts, especially related to the work of the authors on the seismic performance on these structures at NZ universities are explained.*

1 INTRODUCTION

The 2010-2011 Christchurch earthquake sequence showed the performance of steel structures was generally better than anticipated especially given the levels of ground shaking which were more than twice that considered explicitly in design. Furthermore, unlike many reinforced concrete structures, a lot of damage was reparable. These earthquakes have resulted in structural steel being the construction material of choice in the Christchurch rebuild.

Due to this success, it could be considered that steel structures are a good proven solution and that further research on steel structures is not required. In fact, since the mid-1980s, when the provision of ductility and the use of capacity design became a requirement, member sizes and details in the majority of structures around the world have not changed much. This is in spite of considerable research into efforts on topics such as performance-based earthquake engineering. There have been some changes to design of steel structures though. These include the development of more ductile steel structures after the 1994 Northridge earthquake through the SAC Steel Project, and the more explicit provisions for very tall buildings especially in regard to in-service performance.

It may be argued that since recent developments in steel structures, and in earthquake engineering in general, have relatively made small impacts, that further significant changes in approach are unlikely, and that earthquake engineering research should not be a high priority for national funding. In this case, work related to structures in earthquake zones should emphasize education of practitioners to correctly use current codes, ensuring that the intent of standards is not eroded by competing interests or the passage of time since severe earthquake events, etc. Here, structural earthquake engineering may be regarded as mature, and the work of structural engineer is much like that of any tradesman, such as an electrician or a mechanic, who applies technology in a clear standardized way to obtain a standard of construction which meets the performance expectations of society - at least while those expectations are life safety. It is interesting that many institutions that were famous for their ground-breaking work in earthquake engineering in the US and Japan in the 1970s and 1980s are emphasizing this area less. These institutions are now employing academics without a strong earthquake engineering background and whose research is often focused on other topics. The consequence of the status quo is that we are likely to see more

Christchurch-type damage in future earthquakes around the world where the life-safety performance objective was met.

While the status-quo may be acceptable to some, for the people associated with the Christchurch event, it was considered that the damage level was unacceptable (Buchanan et al., 2011) [1], and that we should do better. What engineers accepted as "earthquake resistant" had a very different meaning to building owners and occupiers, most of whom following the Christchurch earthquake of 22 February 2011 had to leave their damaged buildings either temporarily or, for many commercial building owners and occupiers, permanently. Society as a whole suffered considerably from the severe disruption to the fabric of city life, which is only now starting to be reinstated.

In contrast to the argument that structural earthquake engineering methodologies are mature, there is another argument. This is that structural earthquake engineering research and techniques are essential to modify and improve structures and make them more effective and more economical. The need for better structures can be considered for the following structural types:

(i) existing structures, for better retrofit interventions;
(ii) new structures designed according to the current life safety philosophy, to ensure intended response by addressing inadequacies in current standards;
(iii) new structures designed for performance levels above the minimum defined in current standards, and
(iv) damaged structures, to allow reinstatement and reuse.

This paper outlines some issues associated with the above four types of structure, and some recent and ongoing research at the Universities of Canterbury and Auckland and elsewhere to develop better performance of steel building systems subject to earthquake shaking for a specified cost.

2 OLDER STRUCTURES

Many existing buildings were constructed before current standards were enforced. Often they use materials or techniques that are not included in current design approaches. A decision needs to be made about any vulnerable older buildings, sometimes called earthquake-prone buildings, as to whether or not people should be permitted inside, or near, them. The need for the decision, and the decision itself, are affected by the structural assessment and estimated total costs associated with structural retrofit. Different stakeholders and interest groups, such as the owner, occupier, local business groups and councils, risk analysis groups, relatives of those killed in recent events, the construction industry, and politicians may have very different vested interests and opinions about what to do with such structures (MacRae, 2015) [2]. The recommendation for a specific structure may range from doing nothing to full retrofit. There are large economic, safety, and societal implications of this. Strongly differing views of the different stakeholder groups are normal for such high impact low probability events (Mezaros, 2005) [3]. There is a role for engineering here to minimize these polarized views. This can be done by (i) performing more accurate assessments, and (ii) developing cheaper retrofit options for the life safe performance objective.

An example of one structural form where there is significant discrepancy between standard calculations and observed performance is low-rise industrial steel portal frame structures. Some of these have been assessed to have strengths as low as 25% of that to the new building standard. However, of the many frames that went through the Christchurch earthquakes, where there was over 200% of the design ultimate limit state shaking, very few came near collapse. A number or reasons have been cited for this approximately 800% difference between observed and computed behaviour. These include the influence of soil structure interaction, "non-structural" elements contributing to the response, different end fixities, the lack of consideration of nonlinear elastic buckling response of the members, and conservative assumptions of engineers undertaking the studies. In a recent undergraduate study, Makwana and Nanayakkara (2014) [4] used the computer software ABAQUS to capture elastic and inelastic out-of-plane buckling deformations and considered likely rotational stiffness and explained the observed behaviour. Practitioners generally use simple frame analysis software for their assessments so cannot capture many important effects.

A second example relates to the behaviour of multistorey steel frames in the Christchurch earthquakes. These generally behaved much better than expected. This may be due to a number of effects such as non-structural effects, higher material strengths than specified, but specific studies looking a slab effects in an eccentrically braced frame and soil-structure interaction (SSI) effects by Momtahan and Clifton [61] indicate that SSI effects in particular can significantly decrease the drift response.

The lack of consideration of the likely behaviour of actual structures can result in unnecessary retrofit or demolition.

3 CURRENT DESIGN

The primary performance objective of most current standards is to limit the possibility of life loss in a structure during one design level shaking event. While this aim is clear, the degree of confidence that this will always be achieved may be low. Some examples of inadequacies in current design are the following:

3.1 Floor Diaphragms

In undergraduate structural classes around the world, the importance of load paths is emphasized. The capacity of an element meant to remain elastic is required to be greater than the expected force demand. However, the load paths between the seismic resisting system and the floor diaphragm and those through the diaphragm itself are not well considered because both demands and capacities are difficult to estimate.

Diaphragm in-plane demands may be estimated using 3-D frame non-linear dynamic analysis. However, practitioners generally do not have the will and/or capability to use such techniques. They often use elastic software such as ETABS where there is a rigid diaphragm assumption. The word rigid is sometimes taken to imply that the diaphragm is infinitely stiff and strong so that diaphragms are seldom explicitly designed and beam axial forces are seldom calculated.

i) Demand estimation

A number of methods have been proposed to estimate demands using 3-D elastic analysis software which is commonly available to designers. These include:

(i) the *pESA* method (Bull, 2004 [5], Gardiner, 2011 [6], Fenwick et al. [56]). This has been used in NZ building design. The clearest reference to this method is the appendix to reference [56]. It uses lateral forces associated with the frame overstrength shear. It considers both inertia and transfer force effects with one lateral force distribution. This is however only effective for buildings up to about 9 stories in height (Tiong and Lyes, 2014) [7].

(ii) A method by Cowie et al. (2013) [8] is based primarily on inertial forces. Force transfer between a composite floor diaphragm and the seismic resisting system frame, and design/detailing requirements are included.

(iii) Sabelli et al. (2002) [9] has also proposed a similar method which does not consider frame overstrength.

(iv) An alternative method (in Tiong and Lyes, 2014 [7]) includes transfer forces and inertial forces. It is conducted by applying the overstrength force distribution to the frame, but making sure that the forces at the level considered are no less than those resulting from the anticipated in the Parts and Components section of the loadings standard. This approximation is more conservative than some other methods.

After the analysis of the frame is conducted, a separate analysis is then generally conducted of the floor system. The forces on the diaphragm are obtained from the difference in shear above and below each vertical element passing through the floor system. These, together with the applied forces at the level, result in a floor system that is in equilibrium, and the different components can be designed.

Deep beam modelling (Cowie et al 2013) [8] and strut-and-tie/truss analysis (Bull 2004 [5] and 2014 [10], Gardiner 2011 [6], Scarry [11]) are starting to be used in New Zealand. When strut-and-tie analysis is used, the shear studs on the top of any beam can be represented by one effective stud at the *centre of* *the beam span* [64]. Resistance parallel to the beam is resisted by the beam, tension force demands perpendicular to the beam can be resisted by steel placed in the slab, and compression forces at different angles to the beam axis is resisted by the concrete. This approach also gives axial forces in the beams which also must be considered in the design. From the strut and tie approach at the beam centres, the resistance is then provided by spreading out the shear studs along the length. The reinforcing bars going to the beam edges needs a 135 degree hooks around the shear studs. Luo et al. [65] have initiated a study on the performance of the slab itself under direct in-plane and shear loadings in order to ensure that the slab is able to carry required direct and transfer forces. It is anticipated that extensions of this study will provide recommendations for minimum slab thickness, with and without reinforcing, to carry various in-plane slab actions. Preliminary findings indicate that the in-plane forces do not seem to have a significant effect on the out-of-plane gravity displacements.

Further studies are required to reasonably estimate diaphragm demands, and computer software needs to be developed to easily obtain diaphragm demands, and to analyse the diaphragm itself.

3.2 Slab In-Plane Capacity and Beam Axial Forces

Slab in-plane effects are due to i) inertia, ii) transfer, iii) slab bearing and iv) compatibility forces [64]. These are discussed in turn below.

i) Inertia Forces

Beams in moment-frames are often considered to carry bending only and it is generally assumed that the slab will transfer the lateral forces to the column through compression on the column face. However, since slab inertia forces act in the same direction as the frame sways, a gap opens at location “A” shown in Figure 1. Because of this, slab inertia forces cannot go directly into the column. Instead the forces must move into the steel beam through friction and mechanical transfer using shear studs, along the beam causing axial force, through the beam plastic hinge and connection into the column. These elements along the load path need to be designed for these forces. Also, because the inertial forces go into the shear connectors at an angle, tension reinforcement is required perpendicular to the shaking direction to enable the strut and tie forces in Figure 2 to be developed. The slab has to be strong enough to not reach a critical limit state under the applied loading.

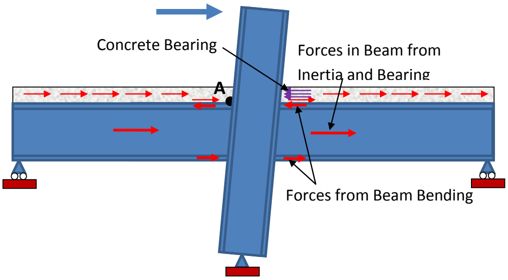


Figure 1. Inertia and Slab Bearing forces Actions

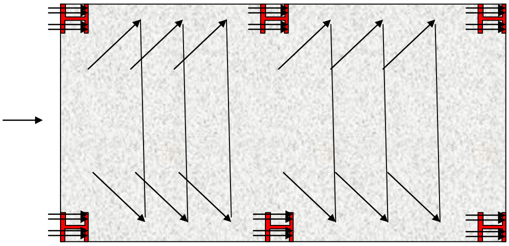


Figure 2. Inertia and Slab Bearing forces Actions

ii) Slab Bearing Forces

If there is no construction gap between the slab and column, as the column sways, it bears against the concrete slab on the far side of the column from point A in Figure 2. This causing a slab-interaction effect that increases the forces that the slab must transfer into the beam and the beam must transfer back to the columns. The magnitude of the force is also dependent on the number of shear studs. In general, the additional strength of the slab due to bearing is not sustained through large deformations. This is because the slab is not confined on the top surface so it can spall. Other mechanisms of slab strength loss, such as shear key fracture, and longitudinal splitting, can also occur (MacRae, Hobbs et al, 2013 [13], [14]).

The New Zealand steel standard requires that the effect of the slab on the beam overstrength be considered for design of the column (generally increasing the beam overstrength demand on the column by about 30%), but the slab effect is not considered for sizing the beam itself. This is reasonable for slabs which do not reliably maintain strength at large displacements. After the frame has moved in one direction, and the slab has spalled on one side of the column, the strength of a subassembly is different in each direction, so there is likely a tendency for larger displacements (i.e. cumulative yielding or rachetting) in one direction rather than the other. This is currently not considered in design.

In order to avoid these effects there have been efforts to more economically design frames with slabs [13, 14]. These include (i) designing the slab around the column to limit strength degradation during large deformations allowing a smaller composite beam can be used in the seismic design by special detailing around the column [13, 14]), and (ii) by providing a gap between the column and the slab to avoid bearing effects.

iii) Transfer Forces

Transfer effects occur due to vertical lateral force resisting elements within the frame having different stiffnesses. In multi-storey frames, these can result in large differential shear forces in the floor system [64]. These transfer forces can be greater when the strengths of the lateral force resisting systems are constant over the height [64]. Considerable beam axial forces may result from these forces, and they can be estimated using analysis methods such as pESA [56].

iv) Compatibility Forces

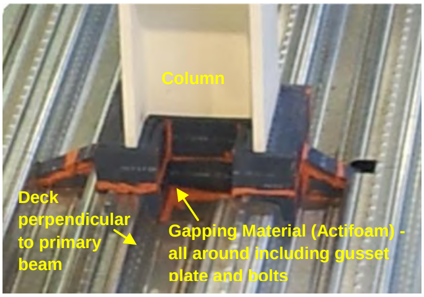
Compatibility forces occur when the neutral axis of the beam is at different heights on the different sides of the column as the ends of the beams of a subassembly are generally pushed apart. When there is no slab and no beam axial force, then the beam neutral axis is at the beam mid-height. However, if there is slab bearing, the beam ends want to move away from each other. This can be modelled simply (E.g. Umarani et al. [66, 67]), but is not considered in most analyses.

A separation gap may be placed between the column and slab so that it does not contribute to the column demands. The advantages of this are that bearing and compatibility stresses are reduced so there is little or no slab effect on beam overstrength, smaller columns and lower beam/connection axial forces. Also, because there is the same strength in each direction without degradation, ratcheting effects are less likely. The disadvantages are that the initial (first cycle) stiffness is decreased, column buckling/twisting is more likely, and there is a gap installation cost [64].

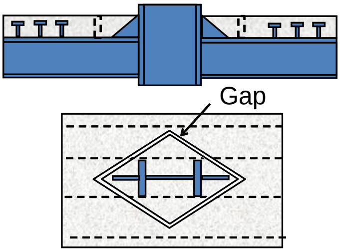
In a welded beam connection, only one layer of isolation may be required on each side of each column flange to fully isolate the slab considering strong axis bending. For a bolted end-plate connection it is necessary to isolate the column flange, end-plate, bolts/nuts, and any haunch. It is desirable, even if the flanges are isolated, that the concrete remain in contact with the web to limit column twist. Isolation may be performed in the following ways:

i) a number of layers of fire-proof isolating material [14] may be placed around the column as shown in Figure 3a.
ii) the slab may be have a diamond gap cut all around the column, end-plate, haunch, etc. as has been done in NZ and is shown in Figure 3b.
iii) Material also may be sprayed around the column. This has been used in NZ too. A strict specification is required to ensure it will behave as desired during concrete placement and during a seismic event.
iv) A ceramic fibre blanket in polyethylene (to prevent water ingress during concrete placement) may be wrapped around the column, and pinned to the column while the concrete is cast against it. This method has used on two medium-rise steel framed buildings recently at the University of Auckland.

A slab fully isolated from the column face does restrain the beams laterally thereby providing better performance than when a slab is not present [14]. However, in general a frame with an isolated column has greater displacements as its stiffness is reduced and its fundamental period longer. Also, care is needed to minimize the chance of column torsional instability due to the decrease in restraint associated with having no slab. Some torsional restraint may be provided for the situation shown in Figure 3a, but allowing the flange tips to contact the slab thereby restricting movement in the column weak-axis direction. For the diamond shaped cut shown in Figure 3b, restraint may be provided by an additional plate to the out-of-plane beams as shown in Figure 4. Similar plates on the top flange may be used as part of the drag-strut to carry axial force in the beams.



(a) Around the Column [14]



(b) Within the Slab

Figure 3. Isolating the Slab from the Column

Whatever method is chosen has implications for the column, for the slab and for beam axial forces. If beam axial forces in a moment frame are large, they may reduce the lateral force resistance of the frame, so should be limited. A limitation of 20% of the axial force capacity will not have a significant effect on the beam plastic hinge strength, so this level may be an appropriate limit, and justification would be required for greater levels of axial force.

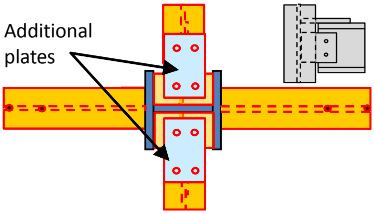


Figure 4. Additional Plates for Column Torsional Resistance with Isolated Slabs

3.3 Column Splices

Columns can generally be trucked to site in lengths of no more than about 3 stories, and therefore require connecting to the column above and below. This connection is typically referred to as a splice, and it is placed near the middle of the interstorey height in accordance with the requirements of NZS 3404. Also, because of past issues with field welding, and the desire to construct quickly and independently of the weather, bolted connections are desirable in NZ. When such connections use vertical plates, holes are made in the column thereby reducing the flexural and shear capacity of the column member at the splice. While efforts have been made to do away with bolted connections in recent versions of US codes, the economic costs of this are high and there are concerns as to whether such an approach is not overly conservative. In NZ, the splice strength in a major column is permitted to be as low as 45% (ϕ x 50%) of the member capacity for ductile systems, which is lower than the value of 55% (R_u x 50%) in older US codes. Recent analyses, as part of a programme underway at the University of Canterbury to assess the likely performance of partial strength/reduced stiffness splices in frames under seismic loading, indicate that splice strength may be exceeded in some cases (Tork Ladani et al., 2014 [15]). Since many floors may be supported above these splices, it is essential that splice failure resulting in collapse not occur. The provisions of NZS 3404 [16] have not changed since the 1992 edition of the standard and were in many modern steel framed buildings between 3 and 22 storeys in height that were pushed into the inelastic range during the Christchurch earthquake series; no adverse effects of splice behaviour were reported.

3.4 Gravity Frame Issues

Gravity frames are often designed to carry axial forces only. However, they need to follow through seismic frame displacements (E.g. SNZ 2007 [16]) and may be subject to moments for the following reasons:

(i) while beam connections to such columns are nominally pinned, there is generally some moment transfer (Liu and Astaneh, 2004 [17]),

(ii) relative floor drifts also cause moments (MacRae et al, 2004 [18]; MacRae, 2011 [19]). These relative floor drifts occur in both horizontal directions.

Furthermore, it is possible in slender columns that the peak moment may not occur at the member end causing the possibility of yielding at a location where the likely response is not known (MacRae, Lu et al., 2010 [20]; MacRae et al. 2004 [8]).

Such moment demands are seldom considered explicitly in design. In addition to the demands on the column itself, there are flexural and shear demands on column splices and on the base connections (Borzouie et al. 2014 [21]). These elements must be considered for satisfactory behaviour.

3.5 Proprietary System Issues

These can be understood with relation to the following discussion about William Tell. Mr Tell is a famous archer who shot an apple off his son’s head. However, while he was successful in that particular instance, it is not clear if he could have done it again. Perhaps he was just lucky! Also, if the boy were running around, rather than standing still, his chance of success may also have been lower.

There are the same issues with many patented products such as BRBs. Tests have shown that they can perform very well. However, there is no guarantee that they would behave well if one of the dimensions or material properties were altered by a small percentage due to a change in design or statistical differences related to the same design. The overall sensitivity to construction differences is not generally known or understood (Jones et al. 2014 [22]). Also, brace tests in NZ (Holmes Solutions), the US (UC Berkeley), Japan (Tokyo Tech), and Taiwan have indicated that braces meeting current specifications can fail. Some of the test failures have been from well-known brace manufacturers using established connection details (e.g. Tsai et al, 2008 [23]).

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. From such testing, many companies have proprietary design procedures for BRBs, but specific brace design information has also been developed by Tsai’s group [57] and also by Wijanto [24]. While existing procedures may be suitable for earthquakes which only cause in-plane shaking, the earthquake does not act only in one direction! 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, it is not clear that such proprietary elements are likely to 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.

3.6 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 (Thornton, 1991 [25]), the proportioning method (Clifton, 2000 [26]), and the Generalized Uniform method (Muir, 2008 [27]), and some simplified methods - are available to consider direct forces. While frame action effects may be considered (e.g. Clifton, 2000 [26], Lin et al. 2013 [28]), 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).

Also, most procedures advocated for gusset plate design consider the gusset plate has an effective length factor for buckling of 0.65-0.70 even though sway is the predominant failure mode! Other types of axial cleat connection have exhibited problems with sway and mitigation methods have been developed

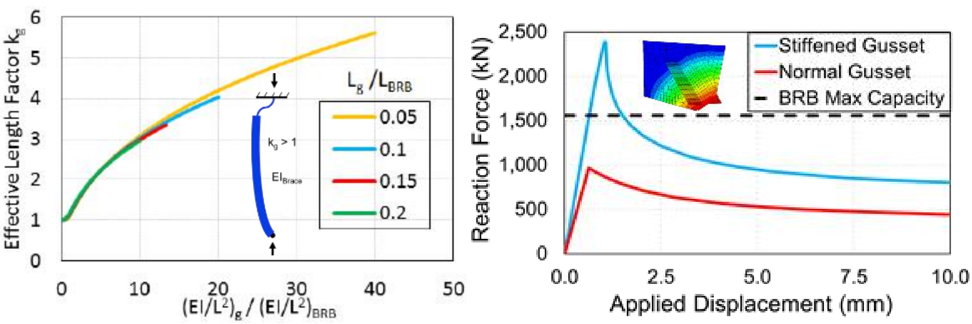
(E.g. Clifton et al, 2009 [29]; MBIE, 2010 [30]). Some researchers are making recommendations for gusset plate design based on tests with a gusset plate restrained so that sway cannot occur (E.g. Yam and Cheng 2002 [31]). While the gusset plate effective length and width may be calibrated with these short effective length factors to realistically estimate overall capacity in some cases, poor behaviour can occur. Clifton (2000) [29] and Lin et al. (2005) [32] indicate that for unstiffened gusset plates it may be appropriate to increase the effective length factor from 0.65 to 2.0.

A recent study at the University of Canterbury (Westeneng and Crake, 2014 [33]) 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 5a. 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 for a typical configuration by Tsai and Hsaio (2008) [23] and replicated by Westeneng and Crake (2014) [33] in Figure 5b. Furthermore, while gusset plate stiffeners can increase strength, care must be taken to prevent beam twist reducing gusset plate end restraint affecting brace strength.

Frame action is not considered explicitly in most design methods, although Lin et al. (2013) [28] have shown that gusset plate stiffeners assist the behaviour under cyclic loading.



(a) Effective Length Factor (b) Strength Evaluation

Figure 5. Gusset Plate Performance (Westeneng and Crake, 2014) [33]

A number of methods are available to minimize the effect of gusset plate buckling. These include:

a) Requiring additional stiffeners along the gusset plate edges, as is recommended by Tsai's group and specified in Brace-on-Demand [57] which has a cloud website specified for New Zealand.
b) Requiring design for a lateral force equal to 2.5% of the maximum axial compressive force of the brace, as specified in of the New Zealand standard (NZS3404 Clause 6.7.2).
c) A method is specified by Bruneau et al. [58] where the effective length is taken as twice the distance from the end of the end of the BRB jacket to the centre of the beam column joint. (i.e. the working point). This method is being used by CoreBRACE. The method was first referenced by Tsai's group, and seems to have been used by Nippon Steel engineers as described by Westeneng et al. [72].
d) Consideration of the strength used to cause buckling as developed by Takeuchi et al. [59]. Plastic methods have also been developed to describe the brace buckling behaviour.

If gusset plates are too slender, they may buckle. However, if they are too stocky, they may yield during out-of-plane deformation also compromising their performance in subsequent cycles. Recent tests by Bruneau (2015) 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 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. Further work is continuing on this topic at UC by Jones and Westeneng led by Dr Lee.

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. Hewitt and Thornton (2004 [71]), specified 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 should have sufficient strength the carry the demands.

3.7 Tentative Guidance for BRB System Design

It may be seen from the above discussion that there are a number of issues related to the performance of BRB systems that are still unresolved. Two opinions on how to deal with some of these issues are given below until better information is available.

i) Clifton Considerations

Clifton proposes a design procedure in accordance with the general requirements of NZS 3404. This involves the following:

1. The out of plane design capacity is based on resisting the actions generated by an out of plane force of $0.025xN^{o}_{com,brace}$, where $N^{o}_{com,brace}$ is the overstrength compression capacity from the brace back to the intersection of the gusset plate with the supporting beam and column.
2. There are two options – either thinner edge stiffened gusset plates or thicker edge unstiffened gusset plates.
3. Especially when edge stiffened gusset plates are used, the supporting column and beam web must be stiffened under the edge stiffeners if required to avoid a column flange or web failure when the applied moment from the out of plane actions on the gusset plate come into the column
4. The effective column formed from the Whitmore section of the gusset plate must have an elastic buckling load based on an effective length factor of 1.2 on the average column length formed from the Whitmore section of at least 3.5 times $N^{o}_{com,brace}$
5. There must be a minimum clearance between the edges of the brace to gusset plate connection and the connection of the gusset plate into the supporting beam and the supporting column of between 2.5tp and 3tp.
6. The $0.025xN^{o}_{com,brace}$ force should also be applied to the connection to the end of the BRB at the junction to the gusset plate, generating a moment back to the face of the BRB restraining jacket, when the jacket is at its maximum distance from the joint considered.
7. The gusset plate out of plate resistance is determined by simplified yieldline bending.
8. To meet the requirement of NZS 3404 Clause 12.9.7.4, the gusset plate will typically be welded to one of the supporting members and connected to the other by a flexible end plate (FEP). The tension capacity of this endplate, when required, shall be calculated by yieldline theory and shall be governed by a mode 1 failure mode. The weld between the gusset plate and the endplate shall be

based on the lesser of (overstrength tension capacity of the FEP; nominal tension capacity of the endplate) in order that the endplate can deform without the weld failing.

ii) Other Considerations

Another group, consisting of Dr Chin-Long Lee, Audsley Jones, Ben Westeneng and Dr Gregory MacRae have also been independently considering how better design BRB frames until further information is available. Some of these ideas are listed at the end of this document.

It is anticipated that the ideas listed above will be part of the discussions in the development of design guidelines for BRB frames by a Steel Construction NZ (SCNZ) coordinated working group. It is currently planned that these guidelines will be available in the first half of 2016.

3.8 Column Continuity Issues

Similar to CBFs, EBFs and steel plate shear walls, there are no explicit requirements for BRBFs to have sufficient column stiffness/strength to prevent a soft storey mechanism. It is theoretically possible to design some braced frames to be totally pinned at each member end according to most current standards. However, such a design will exhibit a soft storey as soon as brace yielding occurs. The continuous column concept (MacRae and Kimura, 2004 [18]; MacRae, 2011 [19]) seems to be a good way to manage this as it provides the stiffness over the height to mitigate a large drift concentration. However, most standards worldwide do not have requirements for this.

3.9 Overturning and Torsional Effects

After the recent Canterbury earthquakes, changes to the loading standard (SNZ, 2004 [34]) were suggested by the Royal Commission for the Canterbury Earthquakes.

One of these was related to the likely greater response of structures with cantilevers. This resulted in the building having a significantly different strength in each direction. This difference is also seen in symmetrically loaded structures with different strengths in different directions. This issue has been recognized (Yeow et al., 2013 [35]) and some design recommendations do exist. For example, for structures with BRBs, the maximum strength difference permitted reverse directions is 10% (SNZ, 2007 [16]). However, clear design recommendations are not yet available for the loadings standard.

Another change was requested for buildings that have lack of torsional restraint as soon as the first lateral force resisting system yielded. While, a large amount of research has been conducted on torsion worldwide there seems to be little guidance to explicitly consider such effects. Two methods are referenced in MacRae and Deam (2009) [36], but further work needs to be conducted to obtain something simple and robust for building codes.

3.10 Other Details

Many other issues are not considered by current codes and it is not always clear what minimum provisions are required to develop systems to satisfy life safety criteria. Some of these are with the use of different combinations of materials. In the Christchurch rebuild, two-way moment frames are being used with rectangular concrete-filled tubular columns. While many techniques for constructing such columns exist in Asia, these generally often use sophisticated automatic processes to obtain good quality welds, and such automation is not available in NZ. Economical means of connecting beams to such columns have been considered (Chunhaviriyakul et al., 2015 [37]) and recent structures have been constructed using external diaphragm connections with low damage friction connections. Research work has also followed to develop simple and robust design recommendations for such structures (Tjahjanto et al., 2015 [38]) and to see if better and more economical connections could be made. Furthermore, recommendations currently available to assess composite column flexural stiffness give significantly different stiffness values. The use of the wrong stiffness can result in unsafe designs (Gharashir et al. 2015 [71]).

4 LOW DAMAGE SYSTEMS

Low-damage systems (E.g. MacRae and Clifton (2013) [39], Buchanan et al. (2012) [1]) are designed to enable the structure to be reused shortly after a major earthquake, and performance objectives can be described in different ways (E.g. MacRae, 2013 [40]). Low damage structural systems cause little damage to the structural system including the slab. Non-structural elements can also be designed for limited damage. Low damage structural systems include: elastically responding structures; specially designed base isolation systems; rocking systems; friction systems; those with replaceable elements; or special devices; such as lead and fluid dissipaters. Some but not all, have self-centring characteristics.

Not all so-called low damage systems are equal, and they may have significantly different costs. For example, post-tensioned beam systems, which were advocated strongly in the early 1990s have not become popular, because, while they behaved well in beam-column tests, they do not behave well when a slab exists, as it needs to damage the slab to perform as expected unless special and expensive mitigation measures are taken. Furthermore, these post-tensioned beam systems tend to push columns apart causing extra demands. These effects are most significant in multi bay frames with deep beams [39].

At the University of Canterbury, recent studies have been conducted on friction braces (Chanchi et al. 2015 [41]), and base connections (Borzouie et al. 2015 [21]) and desirable behavior has been obtained. Similar work at the University of Auckland has been conducted on frictional rotational links (Leung et al., 2015 [42]), rotating frames (Djojo et al. 2015 [43, 62]) and frictional connections with Belleville springs (Ramhormozian et al. 2015 [44, 63]). 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 of actual tests. It included the variation between initial and peak sliding strength for bolts tightened one way under laboratory conditions (giving a factor of approximately 1.25 according to Figure 7 in reference [68]), the average tightening effect variation from the computed nominal value [68], the variation in initial sliding strength between the design equation and bolts tightened approximately the same way of 1.24 [69], and the difference between likely laboratory and field conditions of 1.1 [12]. While these values multiplied together give a value significantly greater than 1.40, the square root of the sum of the squared values, which is more appropriate since all of these effects are unlikely to occur at the same time, is close to 1.40. Furthermore, in one of the tests from one blasted and coated tests, gave a peak overstrength of about this value. It should be noted that for NZS3404, the likely increase in strength is multiplied by 0.9 to obtain the overstrength factor. This means that the nominal, rather than the dependable, strength of the protected members (i.e. “brittle link”) is used.

A device to prevent buckling of tension dissipative devices in compression was recently described by Gunning and Weston (2013) [45]. The device 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 Figures 6 and 7. 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. 2015) [46].

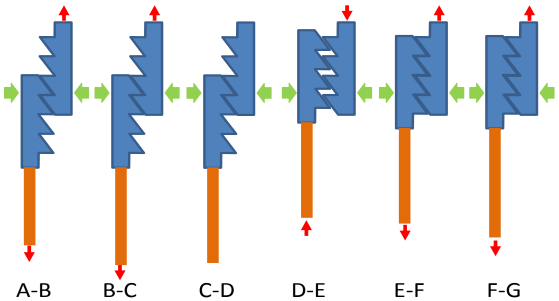
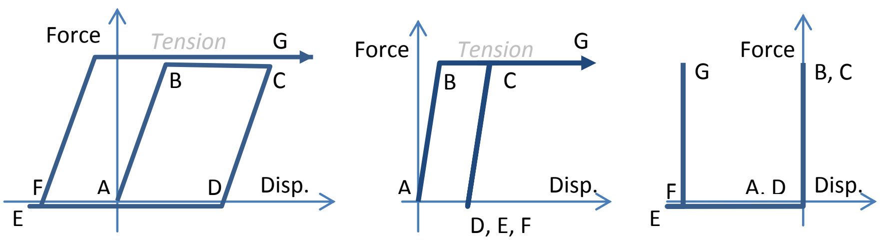


Figure 6. Tension-Only System Push-Pull Behaviour



(a) Full System (b) Dissipative Element (c) Device
Figure 7. Tension-Only System Hysteretic Behaviour

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.

Typical low-damage systems offer a significant performance enhancement for additional costs generally less than 5% (MBIE, 2015) [47]. 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) [47]. Also, unfamiliarity of different construction processes by the construction community can lead to higher costs.

5 REPAIR

After a major event it is generally desirable to reinstate the building as fast as possible. In the best case, if there is no structural damage, nothing needs to be replaced. In other cases, it may be necessary to replace parts of the structural system. It is preferably if these parts are small, not part of the gravity force resisting system, and are at one location or level in the structure to minimize access time.

When an item is damaged, a decision needs to be made as to whether the damage is so minimal that it can be left in place to resist a larger event, whether it can be repaired, or if replacement is needed. This decision requires assessment tools. After the Canterbury earthquakes it was found that appropriate assessment tools are not available for many of practical situations.

In reinforced concrete buildings subject to the Christchurch earthquakes often showed a few large cracks near the column face. These large cracks were different from the large number of distributed cracks often seen in laboratory tests because of the dynamic pulse type loading in one direction, the presence of the slab, and the concrete strength being significantly greater than that specified at 28 days. Since the cracks cause large strains in the reinforcing steel which passes into the column joint, repair is difficult. As a result many of the Park and Paulay type reinforced concrete structures, which satisfied the life safety performance objective well, were replaced after the Canterbury earthquakes.

For steel structures, success has been obtained in determining the level of damage in eccentrically braced frame active links by Nashid et al. (2013) [48] using the field based hardness. This method takes into account the considerable variation of hardness readings through requirements for surface preparation of the surface to be hardness tested, where and how to take the measurements, how to establish a baseline for the unyielded material, the assessed loading regime etc. It has been applied to the assessment of a 2010 built 12 storey EBF structure in Christchurch to determine what yielded active links could be left in place. It is only applicable to the webs of shear links that have yielded in a principally shear mode.

An example of an EBF link replacement for Pacific Tower is given in Figure 8.

It is possible that structures without any permanent or non-replaceable damage may have residual displacements after a major event. These displacements may result in further damage during aftershocks (e.g. Abdolahirad et al. 2015) [50]. When straightening is conducted manually, some sequences of placing straightening cables/members, disconnecting elements, and performing the straightening, may leave the structure more vulnerable to an aftershock than others. Also, immediately after a structure has been pushed over, some temporary stabilization may be required to limit the possibility of the structure deforming further in the same direction during an aftershock. Such stabilization may be performed using very simple techniques, such as tension cables, or compression elements. If it is done well, when an aftershock comes, the aftershock may be used to straighten the structure. Some techniques may also stop the structure moving too far in the opposite direction. One of these simple techniques being investigated by Abdolahirad involves the use of Tension Only Devices in two directions. Those in one direction are done up tight to limit the possibility of further significant movement in that direction, while those in the opposite direction may be provided with initial slackness so that do not grip until the structure starts to move in the opposite direction.

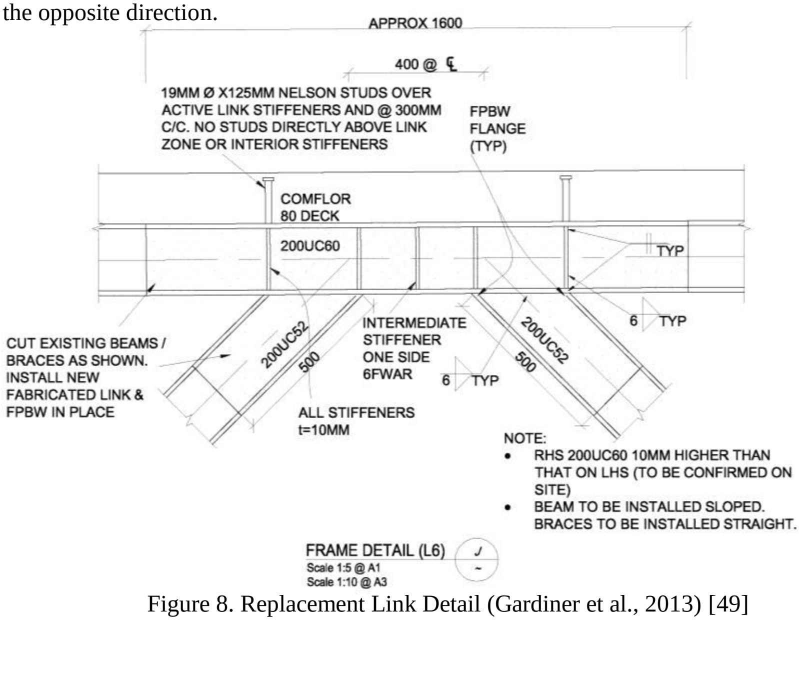


Figure 8. Replacement Link Detail (Gardiner et al., 2013) [49]

6 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 2006 [51], Bradley et al. 2009 [52], Yeow et al. 2012 [53]), such as that shown in Figure 9 below, 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.

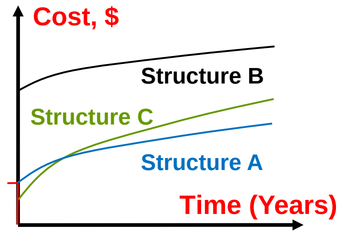


Figure 9. Comparison of Different Retrofit Approaches - Break-Even Analysis (MacRae, 2006) [51]

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 (Chanchi et al. 2012 [54]) 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, 2014) [55] 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.

7 CONCLUSIONS

This paper describes recent and ongoing research on the seismic performance and design of steel structures at NZ universities. It is shown that while steel structures generally exceeded their life-safety performance objectives in recent earthquakes, significant research is required to address:

(i) inadequacies with current standards that require addressing, including issues with diaphragm design, beam axial force design, column splices, gravity frames, proprietary systems (such as BRBs), gravity frames, gusset plates for BRBs, column continuity, frame ratcheting, frame torsion, and details for composite construction,

(ii) a lack of information about the real likely behaviour of older steel structures, including older portal frame structures,

(iii) a need for robust design techniques that can provide structures with a higher performance than that provided in current standards. A new tension-only system is introduced here.
(iv) a need for better assessment and repair techniques for damaged structures.

Research relating to all these topics in NZ is discussed. In addition, decision support tools to assist in selecting the best structural system, is described. Further research into these areas will result in better steel structures and more seismically sustainable cities.

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APPENDIX

The following thoughts have been developed from a group consisting of Dr Chin-Long Lee, Audsley Jones, Ben Westeneng and Dr Gregory MacRae. Helpful comments on a preliminary version of this have also been received from Kevin Cowie of SCNZ without necessarily any endorsement. These are provided for possible consideration in the discussion of the development of SCNZ interim design guidelines.

1) Demonstrated Performance

Requirement: BRBs are not permitted to have strength degradation greater than 80% of the peak strength in tension or compression during cyclic displacements in the two horizontal directions of at least twice the displacement expected in a ULS event. Frames with BRBs are not permitted to have strength degradation greater than 70% of the peak strength in tension or compression during cyclic displacements in the two horizontal directions of at least twice the displacement expected in a ULS event.

Means of compliance: The experimental verification of part of this is:

1) BRB braces having satisfied the AISC experimental test requirements in *two directions of loading* only for (i) the brace alone, and (ii) for the brace as part of a subassembly to a displacement of $2/S_p$ times the ULS displacement including simultaneously. Braces constructed on a different construction line to those tested shall have a representative sample tested to ensure satisfactory performance. A representative sample of braces in frames shall also be tested.
2) BRB braces having satisfied the AISC experimental test requirements in *one direction of loading* only for (i) the brace alone, and (ii) for the brace as part of a subassembly to a displacement of $2/S_p$ times the ULS displacement. These tests may be undertaken considering unidirectional load only, and shall consider the lateral force regime and required test scale. Braces constructed on a different construction line to those tested shall have a representative sample tested to ensure satisfactory performance. A representative sample of braces in frames shall also be tested. Because they are tested on one direction only, they must satisfy additional criteria.

Comments:

1. If 2-D horizontal testing is the only acceptable verification method, then this would severely limit the number and type of BRB that can be used as very few large scale 2-D tests have been conducted around the world.
2. The $2/S_p$ is consistent with is suitable for all buildings, not just the subset which may be on soft soil, or have significant floor or non-structural element participation in which the displacement under the 500 year shaking may be less than S_p times the ULS displacement. It should be noted that this displacement is similar for long period buildings with different values of design ductilities and the same period. Incoming NZS1170.5 requirements for stair seating length have similar criteria.
3. Using twice the displacement associated with design level displacement is used even though we do not anticipate demands greater than 1.5 to 1.8 times the design level displacement for a return period of 2500 years. The reason for this is that it:
   - is similar to the $2\square_{bm}$ (or twice the design displacement) in the US standard
   - is consistent with the inability to predict displacement accurately from analysis of models
   - considers the possibility slightly larger displacement demands than anticipated and avoids the possibility of sudden impact if the brace were to have a smaller design displacement in compression
   - considers the statistical variation of the constructed element properties from that of the standard test element
   - provides some level of confidence that frame can reach the design displacement in 2 directions if testing is only conducted in one direction.
4. Quality control is essential and BRBs made on different fabrication lines may have different performance.
5. No consideration is made of brace inertial effects. These may have a high frequency response which does not adversely affect performance, but studies on this are needed.

2) Subassemblies/Braces not tested to twice the design displacement simultaneously in two directions

Requirements: BRBs, and systems with BRBs, not satisfying the means of compliance in 1a above, are required to meet the additional criteria listed below to obtain the same performance.

Means of compliance: BRBs, and systems with BRBs, are to tested to twice the NZS1170.5 ULS displacement in one direction and satisfy the AISC criteria for this testing. Furthermore, because they have only been tested in one direction, they are to satisfy the following:

a) Gusset plates must meet the AISC conditions considering the Whitmore section and a gusset plate length of 1.2.

b) Using an equivalent length of gusset plate, L_g, calculated as twice the distance between the near end of the BRB casing and the working point (at the intersection of the beam and column centrelines) when the casing is extended to its furthest distance from the joint considered under $2/S_p$ multiplied by the NZS1170.5 ultimate displacement, and $(EI)_g$ is calculated for the transition region of the brace section consider the axis resulting in the minimum I value, then the effective length of the gusset plate, k_g, may be computed from Equation A1, where $\alpha = (EI/L)_g^2/(EI/L)_{BRB}^2$. The compressive strength is computed using k_gL_g, the transition section I and A, and the residual stress parameter, $\alpha_b = +0.5$. This compressive strength shall be at least 10% greater than the measured axial compressive strength of the brace.

$$k_g = 1 + 0.55\alpha^{0.58} \quad \text{(A1)}$$

c) Gusset plate stiffeners are to be provided following the method by Tsai.

d) Welds and frame elements anchoring the gusset plates and stiffeners are to be provided with sufficient strength to carry the loads associated with gusset plate yield.

<u>Comments</u>:

1. There is currently no rigorous method to size gusset plates
2. Both buckling over the Whitmore area using the Whitmore criteria, and buckling of the overall brace and gusset plate need to be considered.
3. The Whitmore method has been calibrated when full sway is not likely. However, it too is a sway failure, and the effective length of 1.2 suggested by Clifton is included here.
4. Overall buckling has been seen in tests such as those conducted by Chou. Also, as BRBs are becoming longer than 15m it is important that the effect of the brace slenderness be considered in the stability calculation. Westeneng and Crake (2014) have shown that the gusset plate effective length should be greater than unity.
5. Equation 1 is an empirical curve fit from the relationship between the parameters shown from Westeneng and Crake (2014).
6. The Working Point Method, where twice the distance from between the near end of the BRB casing and the working point (at the intersection of the beam and column centrelines) when the casing is extended to its furthest distance from the joint, has the following basis. (a) It was first described by Tsai et al. (2002) but probably came from Nippon Steel designers. (b) It accounts for flexibility of the frame at the joint and the lack of full restraint at the edge of the jacket. It also accounts for some accidental out-of straightness.
7. Westeneng et al. [72] indicate that all existing design methods for BRB end connections with gusset plate stiffeners are non-conservative with respect to estimation of failure strength. Equation 1 above is also non-conservative for a normal gusset connection. However, finite element studies, which match experimental behaviour, indicate that gusset plate stiffeners do result in significantly improved performance. Based on this, satisfying Equation 1, together with providing gusset plates, may result in acceptable behaviour.
8. While some standards around the world allow the gusset plate weld size to be determined by capacity design of the brace, sometimes with a distribution factor (e.g. Hewitt and Thornton), this approach is not considered to be sufficiently robust given the different actions on the gusset plate. Capacity design should be based on the plate, not the brace.
9. Larger storey lateral drifts do not always cause brace out-of-plane deformation. This depends on the slopes of the levels above and below, and the stiffness of beams framing into the joint. It is undesirable for braces to yield so needs to be checked.
10. Brace connections and the braces themselves may be subject to initial axial force eccentricity, or eccentricity associated with out-of-plane deformation, causing moments on the brace. This creates a moment at the connection or in the member itself which is magnified by the effect of axial force. Such effects shall be considered in the brace/connection design.

3) Column strength

Requirements: Columns are not to become primary energy dissipating elements during the response to ULS shaking.

Means of compliance:

a) Pushover analysis using overstrength actions may be used to compute the required column strength when the column shears are magnified by 1.2 times.

b) Elastic analysis may also be used for regular structures where:
   a. Capacity design requirements from NZS3404 may be used for axial force.
   b. In the case that all beams and braces have perfect pins to the column, the columns should be provided with flexural strength associated with the continuous column concept [19].
   c. In the case that all beams and braces have moment connections to the column, in addition to the moments from (b) above, the columns shall be designed to resist the minimum of:
      (i) the overstrength moments these members framing into the column, or
      (ii) the sum of $6(EI/L)_{member.}\theta$ for every member into the joint. Here, θ is the corresponding interstorey drift.

   The column moment demand may be obtained by dividing the input moment from the joints and braces equally to the column above and below the joint considered.

c) The flexural strength of any column splices shall be greater than the demand. Column splice shear resistance shall be equal to the sum of the column flexural strengths at the top and bottom of the column divided by the clear column length.

Comments:

a) Fussell in 2015 described the need for columns to be designed for flexure as well as axial force in braced frames, and the difficulty of doing this with elastic analysis. Pushover analysis is one solution, and the requirements are similar to that for brace design. For elastic analysis, moments are generally small. The value of $6(EI/L)\theta$ is for a member subject to double curvature, as would be expected for the beams and braces in this sway frame. While not all this drift occurs in this member (so the result is conservative), there are also non-conservatisms such as ignoring the moments from out-of-plane shaking, and the ignoring of dynamic magnification factors.

b) The requirements for column splice shear are from AISC 2005. In this document, the minimum splice flexural strength was R_y*50% = 1.1 *50% = 55% for the smaller section capacity.

4) Soft Storey Mitigation

Requirements:

Drift concentrations in stories of the frame are to be controlled to ensure that the capacity is not greater than the demand during a ULS event.

Means of compliance:

Columns, or stiff lateral force resisting systems providing continuity over height, must have sufficient stiffness to ensure a continuous column stiffness factor of at least 0.20. Also, the strength of splices in any continuous columns should be sufficient to not yield under anticipated moment demands.

Comments:

A number of different techniques have been used to limit drift concentration in frames. These include the use of pagoda columns, rotating walls, strongbacks/stiffbacks, or the columns within a structure. By meeting this requirement with a continuous column stiffness factor of at least 0.20 the Drift Concentration Factor, DCF, for regular structures may be approximated by $DCF = 1 + 0.132(N - 1)$ [18].