Low Damage Design of Steel Structures

by

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Steel Innovations 2013 Workshop, Christchurch,
21-22 February 2013 organized by Steel Construction New Zealand

1. *Background*

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

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

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

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

2. *Definition of Damage-Resistant Design*

Before discussing the damage-resistant techniques, it is first necessary to define terms. It is actually not possible to design and build structures which are damage-resistant under all earthquakes, so the term "damage-resistant" should be used with care. In the context of this document, it simply means that there should be less damage than in existing construction during design level earthquake excitation. A structure which satisfies this criteria should be occupyable immediately after experiencing large shaking (design level) and might be occupyable in a short time-frame after very large shaking. This target objective is compared with performance objectives for existing construction in Figure 1. In this figure, Group I buildings are those for ordinary occupancy, such as an office building, while Group III buildings are essential facilities, such as a hospital.

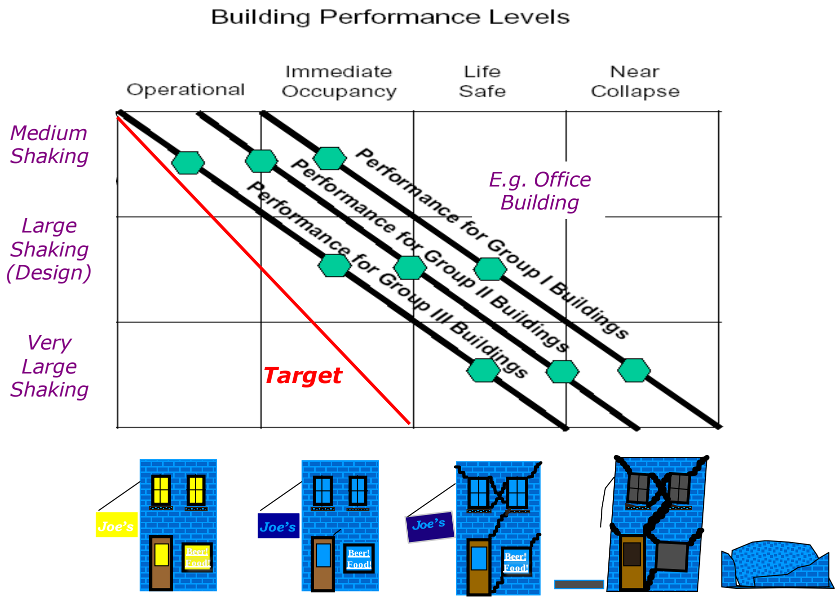


Figure 1. Possible Target Performance Objective for Damage-Resistant Structures (MacRae, 2010, based on Hamburger, PEER)

3. *Reasons for this development*

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

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

1) Elastic Structures (Section 4)
2) Moment-frame structures (Section 5)
    a) Post-tensioned beams (PTB) (Section 5.1)
    b) Asymmetric friction connection (AFC) (Section 5.2)
    c) High-force-to-volume lead extrusion dissipater (HF2V) (Section 5.3)
3) Concentrically Braced Structures (Section 6)
    a) Traditional (Section 6.1)
    b) BRB (buckling restrained braces) (Section 6.2)
    c) Friction Brace – SFC (symmetric friction connection) (Section 6.3)
    d) Friction Brace – AFC (asymmetric friction connection) (Section 6.4)
    e) HF2V Brace (Section 6.5)
    f) Self-Centering Braces (Section 6.6)
4) Eccentrically Braced Structures (eccentrically braced frames, EBF) (Section 7)
    a) Replaceable Link (Section 7.1)
    b) AFC Link (Section 7.2)
    c) AFC Brace (Section 7.3)
5) Rocking Structures (Section 8)
6) Base-isolated structures (Section 9)
7) Supplementary damped structures (Section 10)
8) Base Connections (Section 11)

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

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

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

4. *Elastic Structures*

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

5. *Moment-frame structures*

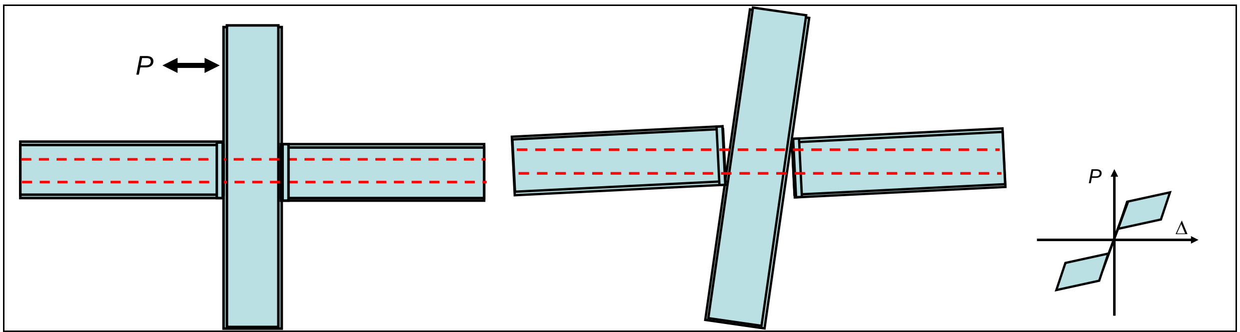
Many modern multi-storey steel structures in New Zealand are designed using moment frames. These are described below in several categories.

5.1 Frames with Post-Tensioned Beams or Spring Loaded Joints

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

The post-tensioned beam technology involves prestressing/post-tensioning prefabricated beams to the column face, as shown in Figure 2a. During large lateral deformations, as may be expected from severe earthquake shaking, a gap opens between the end of the beam and the column face as shown in Figure 2b. As the gap opens, the post-tensioning tendon extends providing additional force to close the gap. Different dissipaters may be placed over the gap to dissipate energy. The strength of the dissipaters should be small enough that after the earthquake shaking, the tendon pulls the structure back to its initial at-rest position. This is shown by the displacement at zero force always being zero, as shown in the hysteresis loop of Figure 2c. Spring loaded joints work in a similar way, with the beams clamped to the columns with flat endplate connections and pre-compressed ring spring joints. When the gap opens, the beam rotates about the point of compression contact between the endplate and the column flange and the springs are further compressed, generating increasing moment with increasing rotation.

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



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

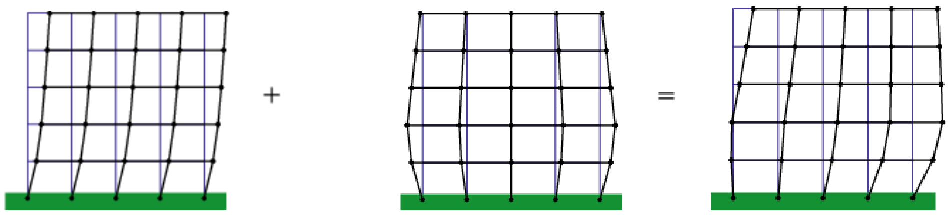
Figure 2. Post-Tensioned Beam Deformation and Hysteretic Behaviour (MacRae, 2010)

For frames with more than one column, the gaps which form at the beam ends cause "beam growth" or "frame expansion". In conventional sway analysis, used in routine design, this effect is not considered, as shown in Figure 3a. The effect of the beam-growth itself is shown in Figure 3b. It can be seen that the exterior columns are being pushed apart. The combined effect, which is the likely behaviour of an actual frame under significant seismic displacements, is shown in Figure 3c. It can be seen that:

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

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

iii) the beams in other stories will be subject to axial forces too. Above the level of maximum frame expansion, they may well be in tension. This is illustrated in Figure 4. Here, if there are relatively stiff columns held in place at the base, the beams and columns will want to separate at the higher levels, and this should be taken into account in the analyses, which can be difficult. Trying to avoid this problem by using more flexible/weaker columns, makes the frame more susceptible to a soft-storey mechanism, so care needs to be taken in sizing these columns.



(a) Sway (Conventional Analyses) (b) Beam growth (c) Combined

Figure 3. Gaping Effects on Seismic Frame Behaviour (Kim et al., 2004)

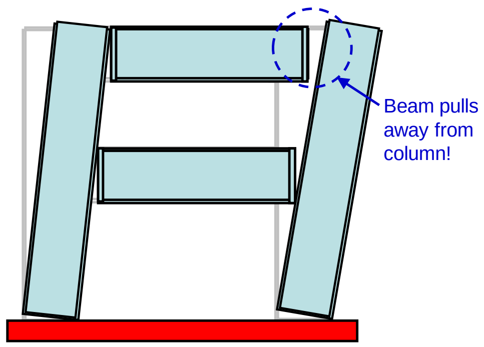
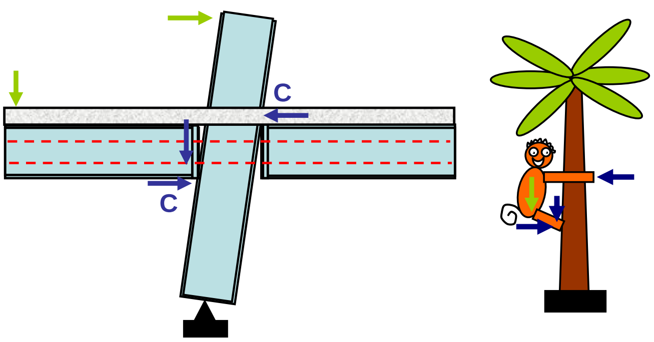


Figure 4. Gap Opening Effect on 2 Storey Frame with Stiff Columns (MacRae, 2010)

For frames with slabs, where the slabs are connected to the beams, the gap opening at the end of the slab wants to extend the slab, as shown in Figure 5a. The forces applied on the joint may be understood using the idealization in Figure 5b, where the tension force in the concrete slab is equivalent to the force in the arms of the monkey. If:

a) the slab is very strong in tension, then the gap can never open and the desired mechanism cannot occur. This means that the moment demand from the beam and slab will be increased, and column yielding may occur. This results in column damage, which is not acceptable in a damage resistant design.



(a) Deformation with a Slab (b) Monkey Idealization

Figure 5. Slab Effects on Subassembly (from MacRae, 2010)

b) the slab is not strong in tension, then the gap can open. However, the gap opening will result in slab damage during the imposed displacements. Some, from a test of this type, is shown in Figure 6. This damage is not acceptable in a damage resistant design.

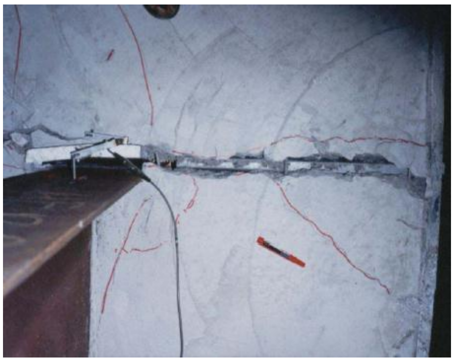


Figure 6. Slab Damage in a Post-Tensioned Beam Subassembly (Clifton, 2005)

It should be noted that some efforts to solve the issues associated with the slab have been proposed. These have their own limitations as described below.

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

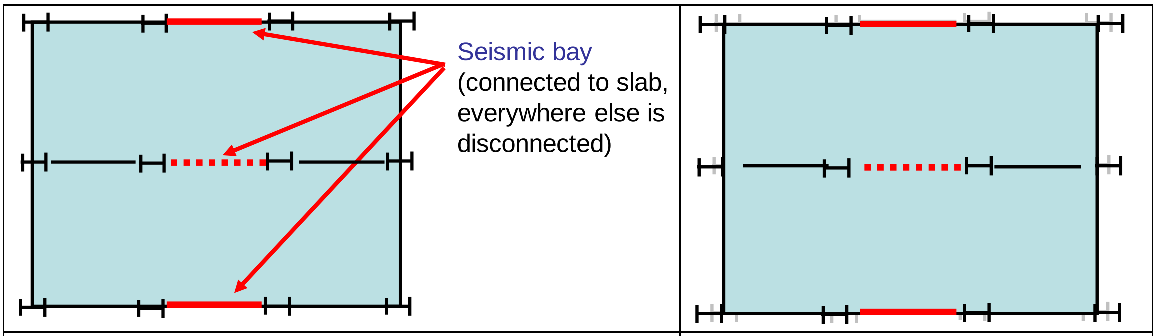
In order to prevent slab damage due to gap opening, it has been proposed that the slab be connected to only one beam in frame as shown in Figure 7a. Here, the slab slides over non-seismic beams. While this seems attractive, and has been shown to work in the push-pull analyses of a 2-D frame, there are a number of issues including:

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

ii) The exterior cladding has to be able to expand in the direction of shaking as shown in Figure 7b. Special detailing of cladding would be required.

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

iv) The system cannot be easily applied in 2 horizontal directions using traditional approaches. The connections between the slab and the beam would need to allow sliding of the slab in the direction perpendicular to the direction of the beams, which is even more problematical than sliding parallel to the beams.

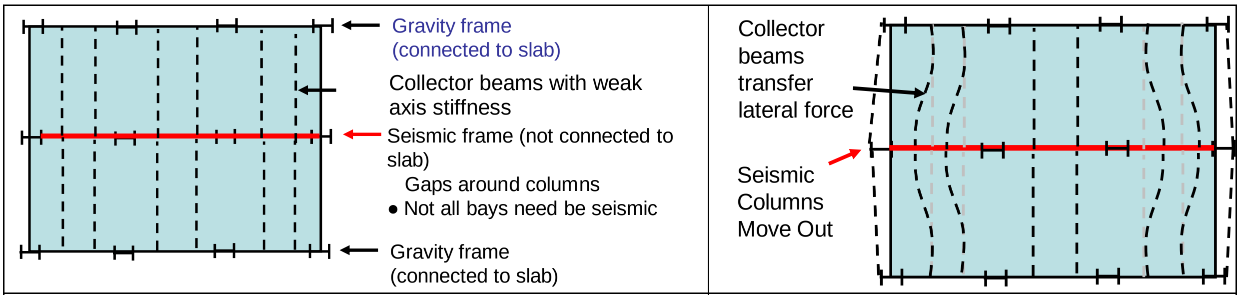


(a) Connections of Slab to Beams | (b) Deformations of Frame

Figure 7. PT Beam Connected to Slab over One Bay

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

It has been suggested that the slab be connected only to the gravity frames in a building as shown in Figure 8a. Gaps are provided between the slab and the seismic columns, and the slab slides on the seismic beams. Collector beams are provided with stiffness which transfer the lateral forces between the seismic and gravity frames. The deformations of the frame are shown in Figure 8b.



(a) Connections of Slab to Beams | (b) Deformations of Frame

Figure 8. PT Beam Connected to Slab over One Bay

This system also has a number of issues:

a) It is sensitive to the beam weak axis stiffness. It the collector beams are too stiff, then there will be no gap opening. If they are too flexible, there will be no force transfer.
b) Axial forces on the beams will be greater than if all beams were connected to the slab. Seismic frame beams will generally be in compression, while the gravity frame beams will tend to tension.
c) The system is dependent on accuracy of construction to ensure the design slip planes are achieved and undesired slip planes suppressed
d) The exterior of the structure is deformed, and this needs special consideration in design.
e) Three-dimensional systems, using the approach described in 2 horizontal directions, is difficult.

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

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

i) They may be used simply to replace the traditional steel beam with a rigid connection. In this case, some column yielding may be expected and slab damage may be expected during design level shaking. However, the total damage may be less than that expected in a traditional NZ rigid weak beam - strong column moment frame buildings significant beam yielding may be expected.

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

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

5.2. Asymmetric Friction Connection (AFC) in Steel Moment Frames

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

The sliding hinge joint has the components shown in Figure 9. The beam end is placed a distance equal to the "beam clearance" away from the column face. The beam top flange is connected to the column by means of the top flange plate. Rotation of the beam end occurs about the connection of the top flange plate to the column flange as shown. Because no sliding or gapping is expected between the beam, top flange plate and column, beam growth and slab damage are minimized. The shear force in the beam is carried by the top web bolts. Horizontally slotted holes are provided in the bottom flange plate and in the bottom holes of the column web plate to allow significant rotations of the beam end relative to the column face. A gap is provided between the end of the beam bottom flange and the column face. This gap is required to be large enough that the demand in and beside the weld connection to the column face is not too large. Below the bottom flange plate is the bottom flange cap plate. It may be described as a floating plate because it has no physical connection to the rest of the joint apart from through the bolts. A web cap plate is similarly placed on the outside of the web plate. On all surfaces where sliding may possibly occur, shims are placed. These shims may be manufactured of steel, brass or other materials. These have standard sized holes so sliding occurs on the side of the shim in contact with the bottom flange plate or web plate. High quality control may be maintained using shop welding site bolting techniques.

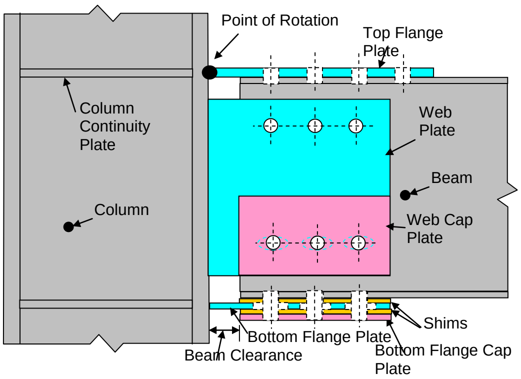
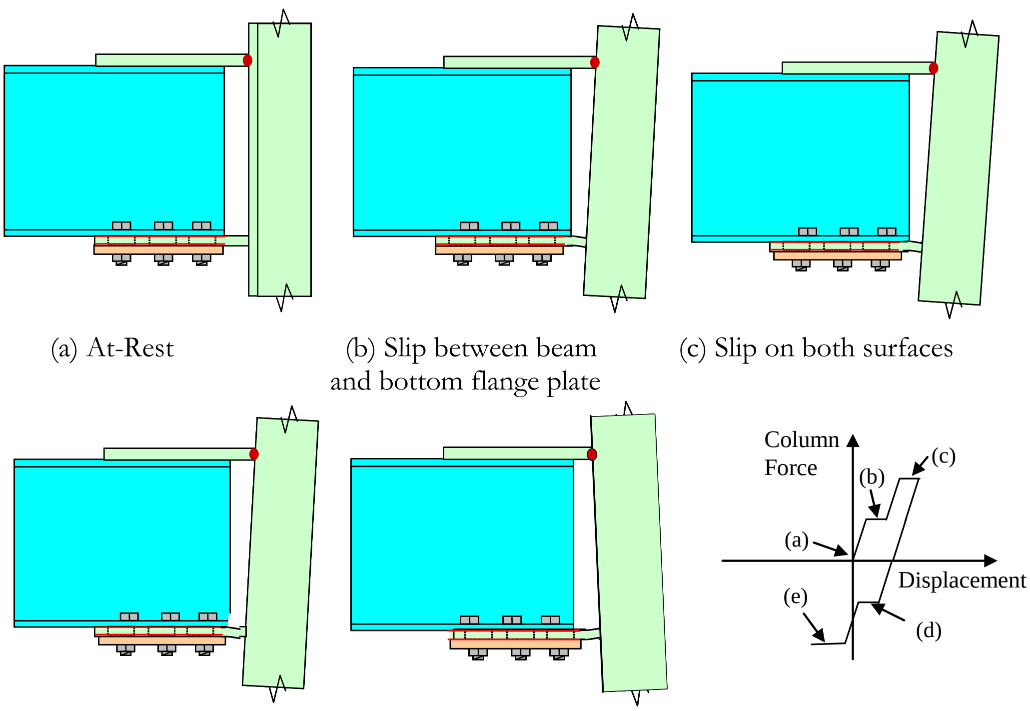


Figure 9. Flexural AFC SHJ Connection (MacRae and Clifton, 2010)

Figure 10 illustrates how the sliding hinge joint works (MacRae, Clifton and Butterworth, 2009). Only the bottom flange friction surfaces are shown for simplicity. The column starts from rest as shown in Figure 10a. As the top of the column moves to the right, slip occurs between the bottom of the beam flange and the bottom flange plate as shown in Figure 10b. At this stage the bottom flange cap plate is not sliding because the shear force imposed on it is relatively small. As the deformations become greater, the bolts in the bottom flange move to such an angle that they provide sufficient force for slip to also occur between the bottom flange plate and the bottom flange cap plate as shown in Figure 10c. Because the peak friction forces on either side of the bottom flange plate occur at different displacements, it is referred to as an "Asymmetric Friction Connection (AFC)". The slip on both surfaces causes approximately twice the resistance than from one surface as shown as (c) in Figure 10f. When loading reverses, slip initially occurs only between the bottom of the beam flange and the bottom flange plate as shown in Figure 10d, and at large displacements in the opposite direction, the bolts are pulling the floating plate in the opposite direction than before as shown in Figure 10e, again causing an increase in lateral resistance as shown in Figure 10f.



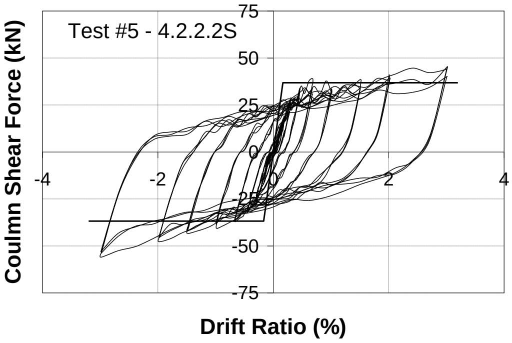
(a) At-Rest (b) Slip between beam and bottom flange plate (c) Slip on both surfaces

(d) Reverse Loading (e) Far in opposite direction (f) Force-Displacement Curve

Figure 10. Simplified Lateral Force-Displacement Mechanism

The hysteretic loops for the beam-column subassembly test in Figure 11a are shown in Figure 11b. It can be seen that the hysteretic loop shape is not that of a traditional friction device (rectangular) but it is like a smeared out version of Figure 10f. Such a curve has dynamic self-centring characteristics, so the permanent displacement would not be expected to be large.

(a) Test Frame



(b) Hysteretic Behaviour

Figure 11. Test Configuration and SHJ Hysteresis with Steel (S) Shims

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

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

Sliding hinge joint AFC construction has been used in at least five multi-storey steel buildings in New Zealand. Details of one of these are shown in Figure 12. Research is continuing at the Universities of Canterbury and Auckland on the friction forces, construction tolerances, the loss of stiffness that occurs when the joint is pushed into the active sliding state, and the durability. Durability issues may be in terms of cold welding and corrosion. Corrosion is most likely to be significant when the link is placed in an exterior environment, such as beneath a bridge. Work is also being conducted at the University of Auckland, with additional devices, in order to improve the self-centring ability of the joint.

(a) Sliding Hinge AFC Joint

(b) Completed Structure

Figure 12. Te Puni Village Buildings using AFC and Rocking Technology (Sidwell, 2010)

Figure 13 shows a variation to the sliding hinge AFC joint proposed by Chanchi (MacRae and Clifton, 2010). Here, the sliding mechanism is placed perpendicular to the point of rotation. The advantage of this is that the demands on the bottom flange plate are predominantly axial, and the flexural component is minimized. Modifications to this, such as making an arc-shaped sliding mechanism, are also possible. Sliding hinge AFC joints which are may provide greater self centring characteristics are also being developed (e.g. Khoo et al. 2011).

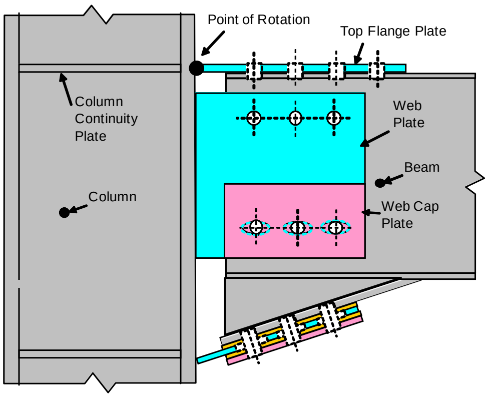
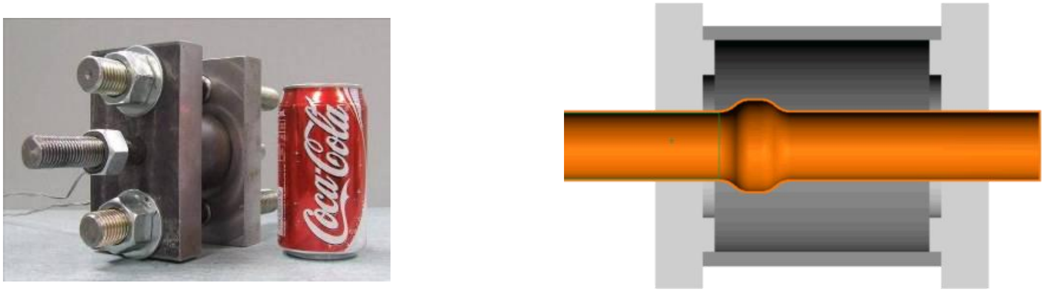


Figure 13. Alternative SHJ Connection (MacRae and Clifton, 2010)

5.3. HF2V devices in steel moment frames

One of the devices tested at the University of Canterbury on steel frames is the high-force-to-volume (HF2V) lead extrusion dissipater (e.g. Rodgers et al. 2010)).The device can be relatively small, as shown in Figure 14a. It resists force as a bulge on the shaft pushes through lead as shown in Figure 14b. The lead recrystalizes after the deformation thereby decreasing the likely permanent displacement (Desombre et al., 2011)

The device allows structures to sustain large displacements without any damage. It can be used in a steel moment-frame in much the same way as an AFC system, as shown in Figure 15.



(a) Size of the HF2V device (b) Shaft with bulge that passes through lead

Figure 14. HF2V Devices in Steel Moment Frames (Rodgers et al. 2010)

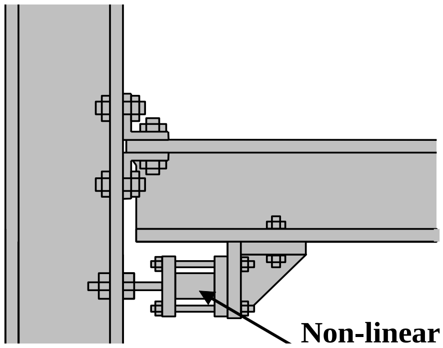


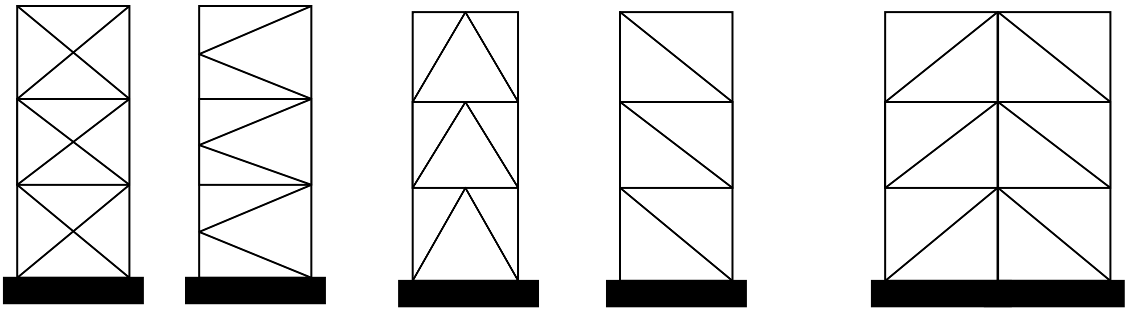
Figure 15. Schematic of HF2V Devices Below Beam in Steel Moment Frames (Mander et al. 2009)

6. *Concentrically Braced Structures*

6.1. Traditional Brace Dissipators

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

The bracing may be placed in different configurations, such as X, K, inverted V, or diagonal bracing as shown in Figure 16. Balanced diagonal bracing is the most common for moderate rise structures because it provides the same strength in both directions.



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

Figure 16. Different Bracing Configurations for Concentrically Braced Frames

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

6.2. Buckling restrained braces (BRB)

Buckling restrained braces (BRB) are restrained from buckling by means of a casing, as shown in Figure 17. The steel is debonded from the casing material so that it can freely slide in the sheath. They can be used in concentrically braced frames.

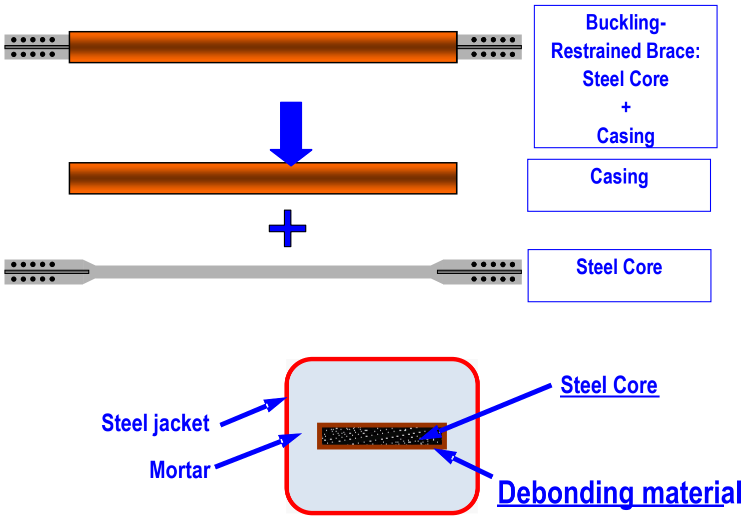


Figure 17. Schematic of BRB (AISC, 2007)

Because the brace does not buckle, it has a similar force-displacement hysteresis curve in both the tension and compression directions. The force displacement curve (shown in the continuous line) is compared with that of a traditional buckling brace (shown in the dashed line) in Figure 18. While the BRB sustains damage, the displacements are spread over a long length so the strains are generally small for significant earthquakes. This means that it may not need to be replaced after a major earthquake as long as the permanent displacements are small. The hysteresis loop for the BRB is less pinched than for a traditional tension-compression brace, so higher permanent displacements are a possibility. By using replaceable connections, the BRBs can be replaced.

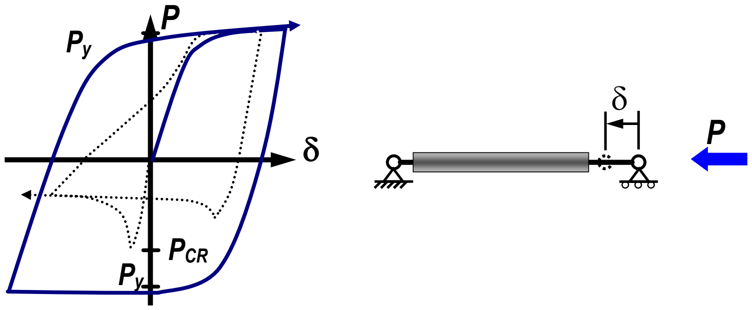


Figure 18. Schematic of BRB (AISC, 2007)

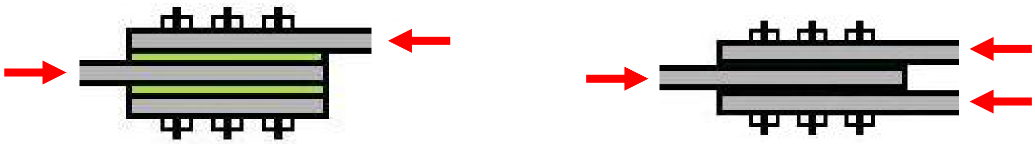
Some structure using BRB technology are shown in Figure 19. Also, the University of Canterbury Psychology building was retrofitted using this technology.

Figure 19. Diagonal and Inverted V-braced BRB Concentrically Braced Frame Structures (AISC, 2007)

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

6.3. Friction Braces – SFC - in concentrically braced structures

Concentric bracing can also be used with friction devices for energy dissipation. These friction devices may be of two sorts as shown in Figure 20

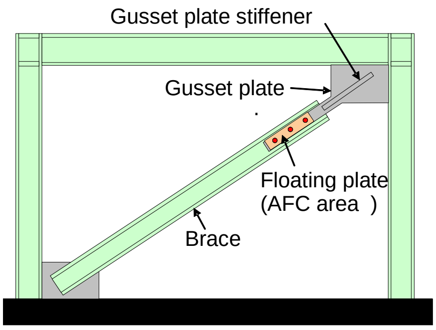


(a) Asymmetric Friction Connection (AFC) (b) Symmetric Friction Connection (SFC)
Figure 20. Friction Connection Types (from Chanchi et al. 2010)

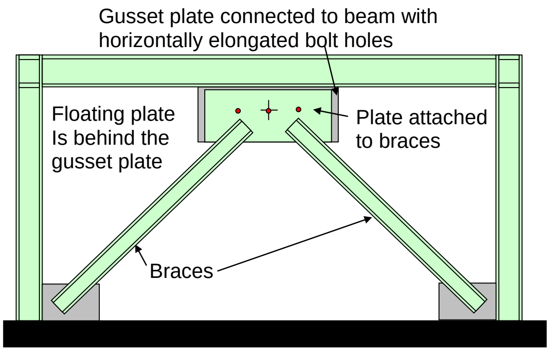
The AFC has a slightly more pinched hysteresis loop than the SFC, when large sliding deformations are considered. This is because the bolts in the AFC must first move on an angle to activate the floating plate (i.e. the bottom plate in Figure 20a). This will generally result in slightly lower permanent displacements.

6.4 Friction Brace – AFC - in concentrically braced structures

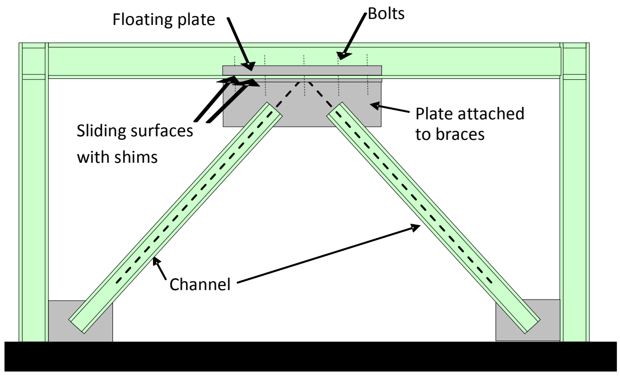
Figures 21 shows concentrically braced systems which dissipate energy by means of the AFC brace systems. In Figure 21a, the AFC is within the brace. Because elongated bolt holes can be long, large deformations may occur in the brace. Special care needs to be made with near the AFC area that an out-of-plane bending failure cannot occur. Also, the end connections of to the gusset plates must be detailed to ensure that in-plane bending of the brace does not cause any major problems. In Figure 21b, horizontal sliding occurs in the gusset plate below the beam. In Figure 21c, horizontal sliding occurs in below the beam bottom flange. Figures 21b and c impose additional bending may be applied to the beam and this should be considered in design.



(a) AFC Brace



(b) AFC Attachment



(c) AFC on Beam Bottom Flange (MacRae, 2011)

Figure 21. Some AFC Brace Configurations (MacRae and Clifton, 2010)

The AFC brace systems have the following desirable characteristics:

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

6.5 HF2V dissipaters in concentrically braced structures

HF2V dissipaters may be used in braced frames as shown in Figure 22. Here, brace buckling issues need to be addressed as well.

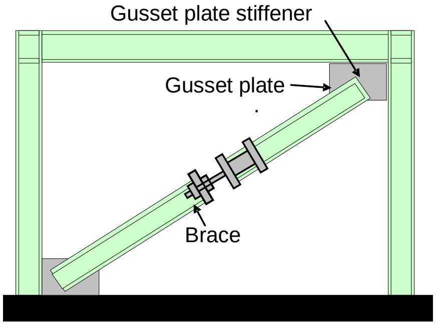


Figure 22. Braced Frame with HF2V Device

6.6 Self-Centring Braces in concentrically braced structures

Innovative braces have been developed by Christopoulous et al. (2008). These have a flag-shapped hysteresis similar to that in Figure 3c. This results in energy dissipated and no permanent displacement at the end of an earthquake. These desirable characteristics do come at a cost.

7. *Eccentrically Braced Frame (EBF) Structures*

7.1 Eccentrically Braced Structures with Replaceable components

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

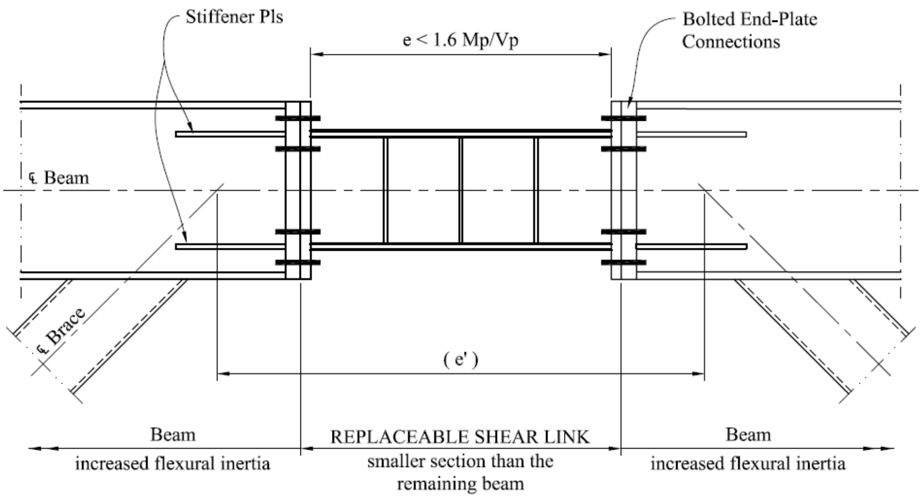
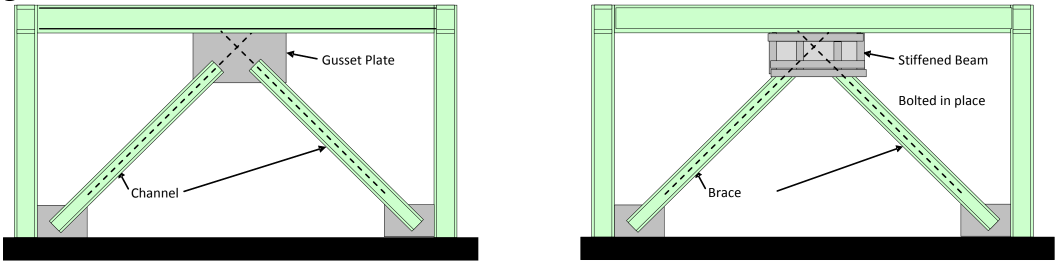


Figure 23. Replaceable Link in EBF (Mansour et al, 2009)

An alternative to this is to use the gusset plate below the beam to dissipate energy. This has been shown to work effectively (e.g. Astaneh 1990). This minimizes floor damage as shown in Figure 24a. Presumably a replaceable T-section bolted to the bottom of the beam could be used for the gusset plate. Alternatively, a replaceable link beam could be used as shown in Figure 24b.

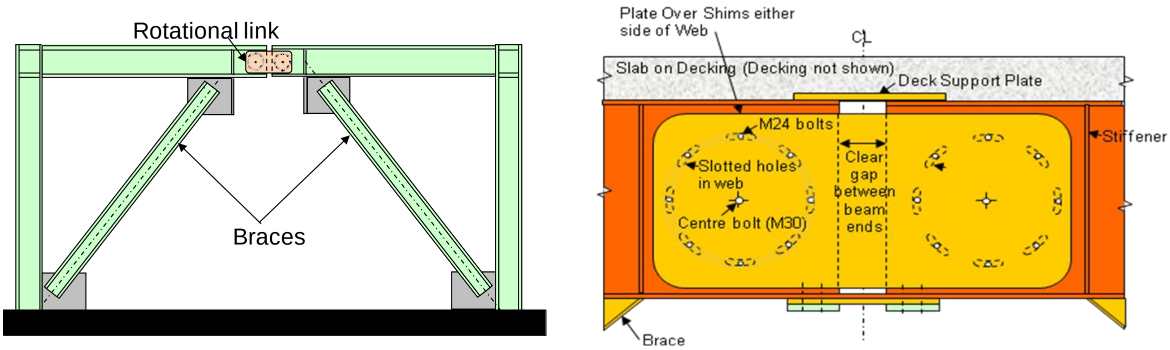


(a) Use of Gusset Plate (Astaneh, 1992) (b) Replaceable link below beam (MacRae, 2011)

Figure 24. Energy Dissipation below Beam in EBF

7.2. Eccentrically Braced Structures with AFC Link

Figure 25 show an innovative link connection for an eccentrically braced frame conceived by Clifton (Khan and Clifton, 2011). Rotation occurs about the centre bolts. The other bolts provide a clamping force for dissipating energy in friction. While the connection has almost no damage, the configuration has the same problem as traditional EBF design, or EBF design with replaceable links. That is, when the shear link deforms to its design inelastic deformation capacity, any slab sitting on top of the link may be damaged and need replacing. In the rotational AFC, the slab demands may be greater than in the traditional EBF because the differential movements and angle of deformation of the beam beneath the slab is greater. This means that while the frame is not expected to suffer significant damage, the structural system, which includes the slab, may.



(a) EBF with Rotational AFC Link (b) Rotational AFC link

Figure 25. Rotational AFC Link in EBFs (Khan and Clifton, 2011)

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

7.3. Eccentrically Braced Structures with AFC Braces

The same eccentrically braced configuration may be obtained without the need for inelastic deformation in the link. The braces can be used to dissipate the energy, in the same way that they can be used for concentrically braced frames, in Figure 26. While the means of energy dissipation is much less elegant than the rotational link, this concept has the advantage that it is not likely to result in significant slab damage. Again, care needs to be taken to prevent brace buckling.

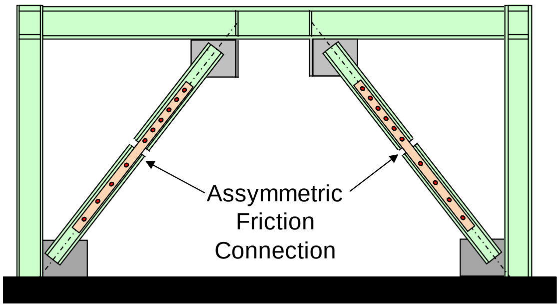


Figure 26. Schematic of EBF with AFC Braces (MacRae, 2010)

8. *Rocking Structures*

Rocking structures are uplift under severe lateral seismic accelerations as shown in Figure 27. New Zealand has a legacy of designing rocking structures as shown from 1981 South Rangitikei Rail Bridge. The first steel structure designed to rock was built in Wellington in 2007 (Gledhill et al. 2008) idealized in Figure 28a. Here the self-centring cables are attached to springs at the bottom of the legs in Figure 28b. These springs increase the level of earthquake inertia force under which uplift occurs, thereby increasing the secant stiffness and reducing the expected frame displacements.

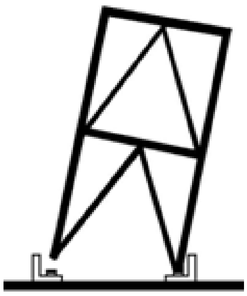
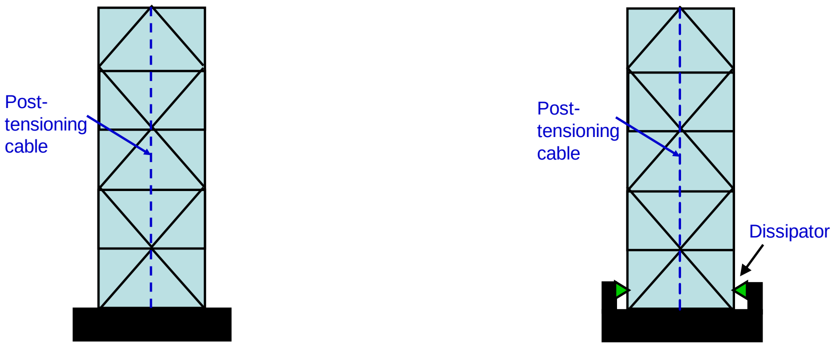


Figure 27. Schematic of Rocking Steel Structure (Chanchi et al. 2010)

(a) Schematic (b) Photo of Springs in the Legs (Sidwell, 2010)
Figure 28. Rocking Structure in Wellington

More recently other rocking systems have been proposed and testing has been conducted by groups based at i) Lehigh (Roke et al. 2009) and ii) Stanford-Illinois-TIT (Deierlein et al. 2010) as shown in Figure 29. The Stanford-Illinois-TIT frame costs more because of the dissipater, and the dissipater reduces the response. Here, post-tensioned cables extend to the top of the structures. This results in larger member sizes throughout the frame than in the NZ approach, but it obviates the need for the springs.



(a) Lehigh Proposal (Sause et al. 2010) (b) Stanford et al. Proposal (Deierlein et al. 2009)
Figure 28. Different Configurations for Rocking Structures

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

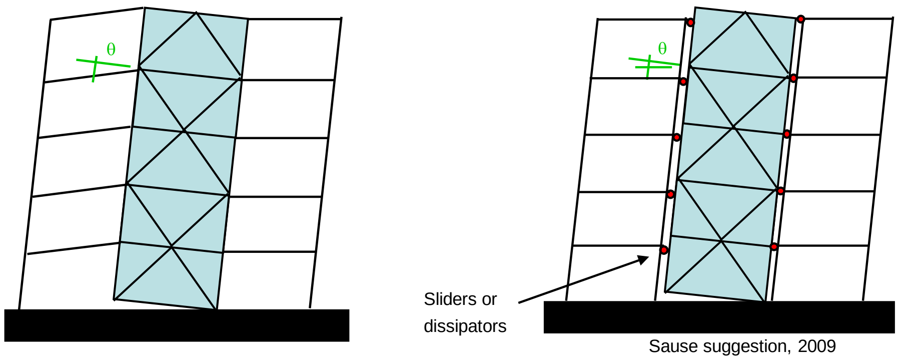
a) Vertical accelerations resulting from impact on the foundation as the frame returns to its initial position. This may adversely affect the non-structural elements, contents and the occupants. Restrepo (2010) indicated vertical accelerations as great as 4g in a concrete rocking wall in a recent test.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

b) Horizontal accelerations resulting from the impact. This is also likely in all "clickety-clack" systems. These systems are those that have a rapid increase in stiffness when the structure is travelling at high velocity. Such buildings include those with traditional (buckling) concentric braces with medium to high slenderness ratios, pure steel plate shear walls, post-tensioned beams, rocking structures, concrete walls and others. This issue was first raised by MacRae (2010) where a motorbike was shown travelling at constant velocity. Because it is at constant velocity, the horizontal forces and accelerations on the motorcyclist are zero. However, the when the motorbike suddenly hits a wall, the forces on the bike suddenly increase, until the wall is pushed over. This is illustrated in Figure 29. The hysteresis loop for the motorcycle in Figure 29b is similar to that for many clickety-clack structures. The following provocative question was raised: "Is the difference between the motorcyclist, and a person in a "clickety-clack" building during an earthquake, only the amount of protection they are wearing?". Anecdotal evidence (e.g. Bull 2011, Clifton 2011) indicates that in buildings of this type, including concrete shearwall buildings, due to the high increase in stiffness at high velocity, many items and people were thrown across rooms during the 22 February 2011 Christchurch earthquake. This is similar to the way the motorcyclist may be expected to be thrown of their motorcycle. Research is continuing at the University of Canterbury to quantify these effects.

(a) Motorbike and Wall (b) Hysteresis Curve

Figure 29. Hysteresis for Sudden Stiffness Change at High Velocity (WWW, 2010)

c) Vertical deformations on the side of the frame may result in large demands to the floor slab as it needs to kink through the angle θ in Figure 30a. This interaction with the rest of the frame may limit the rocking that occurs, and it may cause damage in the frame. Sause et al. (2010) proposed separating the rocking frame from the rest of the structure as shown in Figure 30b. Here, dissipaters between the frame and the rest of the structure may be placed to dissipate energy. These may be AFC dissipators as shown in Figure 31 (MacRae, 2010). Also, horizontal plates between the rocking frame and the structure behind may be use to transfer lateral force but not vertical forces.



(a) Deformation if Attached to Frame (b) Separation of Frames

Figure 30. Rocking Frame – Gravity Frame Interaction

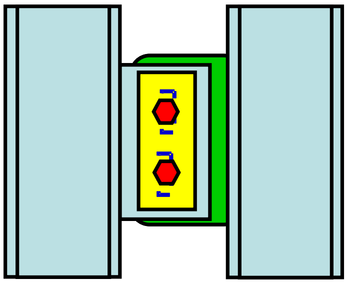


Figure 31. AFC Dissipation for Rocking Frames

9. *Base-isolated structures*

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

10. *Supplementally damped structures*

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

11. *Base Connections for Structures*

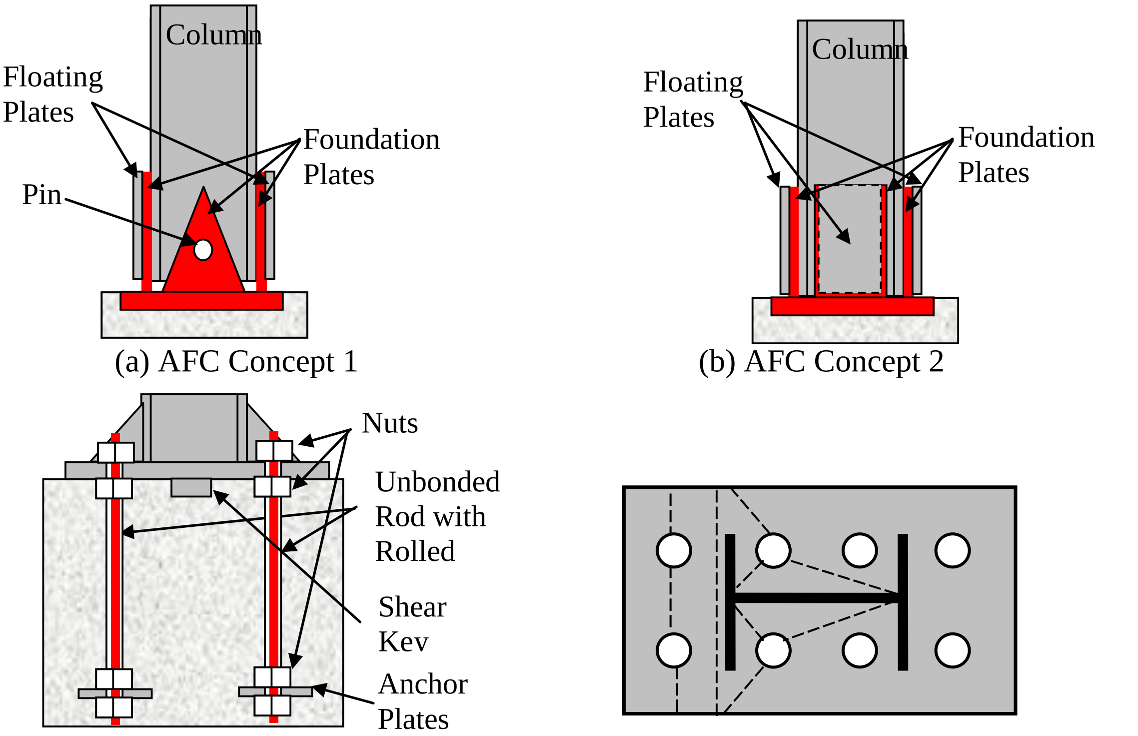
A number of different base connections to minimize damage at the column brace are described in MacRae et al. (2009).

A conceptual drawing of a AFC base detail is given in Figure 32a. Here, axial force is transferred directly from the column to the pin at the centre of the column to the foundation. Shear force is carried the same way. Flexure is carried by means of asymmetric friction action in the flanges.

Figure 32b illustrates asymmetric action on both flanges and webs. Column axial compression goes directly from the column into the foundation and shear is carried through the bolts in the web. If the column is subject to large axial tension, it will be designed to stop moving when the bolts hit the top end of the elongated holes in the foundation plates. This detail is easier to construct than the that in Figure 32a, but one side of the column has to move up (much like a concrete column) to allow flexural deformation to occur. This changes the height of the centre of the column. There is also the possibility that after a major earthquake that the column may not have returned to its initial position, so the bolts may need to be loosened and tightened again.

Figure 32c (Mackinven et al., 2007) involves the use of unbonded steel rods to act as re-centring devices while the steel column rocks under lateral loads. The unbonded length of the rods is sufficient to allow elastic extension to re-centre the rocking column. The rod has a rolled thread passing through it and a nut above and below the end plate. This rolled thread seems to be able to withstand many cycles without fracturing. As above, the absence of yielding in the column results in the elimination of inelastic axial shortening. Some industrial complexes in the Edgecumbe earthquake of 1987 with this detail performed well.

Figure 32d illustrates a yielding endplate connection.



(a) AFC Concept 1 (b) AFC Concept 2

(c) Rod concept (d) Yielding Endplate Connection

Figure 32. Some Possible Methods for Preventing Column Yielding

Acknowledgements

This paper is from “Damage Resistant Design of Steel Structures” by Gregory MacRae submitted to the team 20/8/2012 which was reviewed by Dr. Charles Clifton.

References

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

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

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

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

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

6. Bull D., (2011), Personal communication.

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

8. Christopoulus, C. and Filiatraut, A. (2006). *Principles of Passive Supplemental Damping and Seismic Isolation.* IUSS Press. First edition. Pavia, Italia.

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

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

11. Clifton GC. Personal Communication, 2011.

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

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

14. Desombre J., Rodgers G. W., MacRae G. A., Rabczuk T., Dhakal R. P., Chase J. G., Experimentally Validated FEA Models of HF2V Damage Free Steel Connections For Use In Full Structural Analyses, "Structural Engineering and Mechanics, *An international Journal*", SEM9N626C, Structural Engineering and Mechanics, 37(4), March 10, 2011.

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

16. Gledhill SM, Sidwell GK, Bell DK. The Damage Avoidance Design of Tall Steel Frame Buildings - Fairlie Terrace Student Accommodation Project, Victoria University of Wellington, New Zealand Society of Earthquake Engineering Annual Conference, Wairakei, April 2008.

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

2004.

18. Khan, MH and Clifton, GC, “Proposed Development of a Damage-Resisting Eccentrically Braced Frame with Rotational Active Links, Bulletin of the New Zealand Society for Earthquake Engineering, Vol 44, No 2, June 2011

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

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

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

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

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

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

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

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

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

28. MacRae G. A., “Damage resistant design of steel structures”, “Chapter 7 of Base Isolation and Damage-Resistant Technologies for Improved Seismic Performance of Buildings, by Andrew H. Buchanan, Des Bull, Rajesh Dhakal, Greg MacRae, Alessandro Palermo, Stefano Pampanin , Report to the Royal Commission for the Canterbury Earthquakes, New Zealand, August 2011. http://canterbury.royalcommission.govt.nz/

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

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

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

32. Priestley MJN and MacRae GA, 1996. Seismic Tests of Precast Beam-to-Column Joint Subassemblages with Unbonded Tendons, PCI Journal, January-February; 64-80.

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

34. Restrepo-Posada J., 2010. "The Chile Earthquake", NZSEE Conference, Wellington, 2010.

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

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

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

38. Tsai, KC, Lee CH, Tsai CY, Lin CH, 2010. Large Scale Seismic Testing of Steel-Framed Structures at NCREE , in Advances in Performance-Based Earthquake Engineering, Chapter 42, Volume 13, 451-460, Springer, ISBN Number 978-90-481-8745-4

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

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

41. Yamada et al. 2010. Full Scale Shaking Table Collapse Experiment on 4-Story Steel Moment Frame, 7th International Conference on Urban Earthquake Engineering & 5th

International Conference on Earthquake Engineering Joint Conference March 3-5, 2010, Tokyo, April 2010.