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Determination of the Post-Earthquake Capacity of an Eccentrically Braced Frame Seismic Resisting System

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Abstract.

Eccentrically Braced Framed (EBF) seismic resisting systems are the most commonly used seismic resisting system in New Zealand. They are designed for controlled damage in a severe earthquake, with the damage being concentrated into specific elements of the frame, called Active Links, that ensure the frame and overall building remain stable in a severe earthquake. After the earthquake, the building must be assessed to determine whether replacement of any active links is required.

The 22 February 2011 earthquake of the 2010/2011 Christchurch earthquake series was the first worldwide to push EBF systems into the inelastic range, with most systems in Christchurch displaying active link yielding. That raised the question of how to assess the post earthquake capacity of these yielded systems in order to determine which links can be left in place, which must be replaced and in the latter case how to do that.
This report provides that guidance, based on research undertaken since 2011. That research has also raised three further questions that must be answered before the guidance can be considered complete; these questions are documented and research to answer them is due to begin in 2016.

Section 1: Overview and Scope

Background.

This report is written for application to an Eccentrically Braced Framed (EBF) seismic resisting system which has undergone inelastic demand in a severe earthquake. This EBF seismic resisting system may be one of a number of seismic resisting systems in a particular building or it might be the only form of seismic resisting system for that building.

The 2010/2011 Christchurch earthquake series were the first earthquakes worldwide to push EBFs significantly into the inelastic condition and impacted on a range of EBF buildings ranging from 2 to 22 storeys in height (Clifton, Bruneau et al. 2011). One of these, HSBC Tower, is shown in Figure 1; this picture was taken in the week following the most intense earthquake of the series, the earthquake of 22 February 2011.

That event caused yielding of most of the active links in every EBF building in the Christchurch central business district. Four examples are shown in Figure 2. These include two active links with yielding of the webs and no local buckling, cracking or fracture, Figure 2 (a) and (d); one with yielding of the webs and local buckling of the bottom flange at one end, Figure 2(b) and one which underwent web yielding and then fracture, Figure 2(c).

The scope of this report is on EBFs. Figure 3 shows the two most common form of EBF bracing layout. On the left is the K braced system, used when the ratio of frame width to storey height is close to 1. On the right is the V braced system, used when the ratio of frame width to storey height is closer to 2.

Figure 4 shows the terminology for the active link components, showing the connection of the braces to the active links, the link/collector beam panel zone and the regions where shear studs can and cannot be placed. It doesn't show the intermediate stiffeners, examples of which can be seen in Figure 5.

Prior to the Christchurch earthquake series, the EBF member containing the active link was typically continuous with the collector beam or beams and the brace was welded to these members. This made active link replacement difficult. Since then, bolted active links have become the standard detail in EBFS which are built integrally with the structural frame. Figure 5 shows examples of both sorts used in buildings.

Once an EBF building has been through a damaging earthquake, the structure must be assessed to determine the following:

(a) Can the active links be left in place or is replacement needed?
(b) Is there any damage to other parts of the structure that would require replacement or repair of those components or replacement of the whole structure?

The scope of this report is to provide that guidance. It is based principally on the research work of ((Nashid 2015), (Choi 2013), (Hyland 2008)) and is a generalised version of a confidential report written to determine the post earthquake status of the HSBC tower (Clifton and Ferguson 2014). Addressing issues around strain ageing draws on research by Pussegoda (Pussegoda 1978)

Scenario for Application of this Document

The scenario is as follows:

(1) There has been a severe earthquake, sufficient to push buildings with EBF systems into the inelastic range.
(2) Visual inspection of EBF active links has shown the following:
    a. The ground around the building shows no or negligible evidence of ground instability during the earthquakes.
    b. The foundation system shows no visible sign of failure.
    c. The building has effectively self centered to within approx 0.15% of vertical or within the construction tolerances for the building.
    d. An assessment of the non structural systems shows that damage is minor and does not preclude a rapid return to service.
    e. The key question is to determine whether the structurally damaged members need replacement.

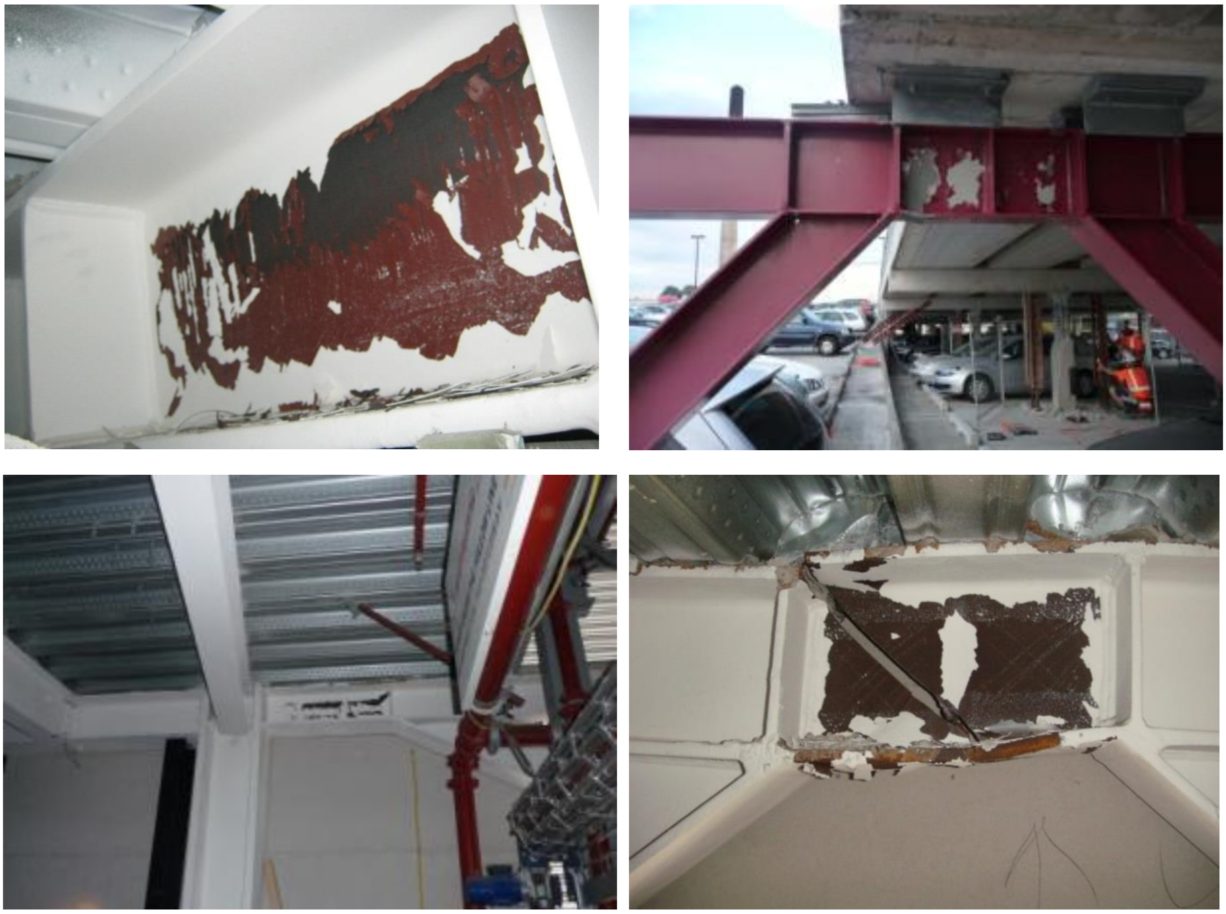
Outline

This document comprises the following parts:

Section 1: Overview and scope
Section 2: Determination of the post-earthquake capacity of the EBF system and extending this to other systems as best is possible given the current state of knowledge.
Section 3; Guidelines on replacement of damaged active links, when required
Section 4: Example of application
Further Research Required to Complete This Procedure
References

The figures and tables for each section are at the end of that section.

Figure 1 HSBC Building, Christchurch, March 2011 (from (Clifton, Bruneau et al. 2011))



Clockwise from top left: (a) Yielded active link level 3, 12 storey HSBC Tower, (b) Active link at edge of building with slight flange local buckle, level 1, 3 storey car parking building; (c) Fractured and yielded active link, level 6, 22 storey Pacific Tower; (d) Yielded active link, level 4, 22 storey Pacific Tower

Figure 2 Yielded Active Links from Christchurch 22/02/2011 Earthquake (from (Clifton, Bruneau et al. 2011))

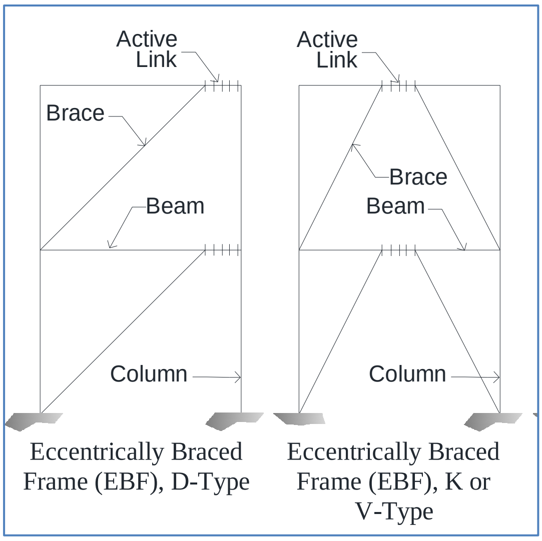


Figure 3 Two Common Types of EBFs with Member Terminology (from (Nashid 2015))

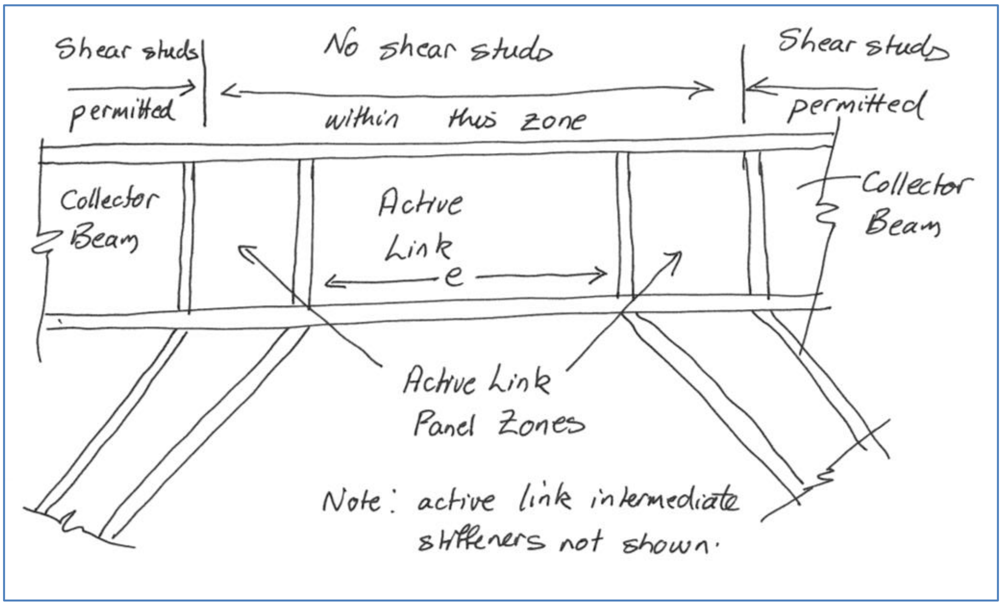


Figure 4 Terminology for EBF Active Link Components

(a) EBF with continuous active link and collector beam

(b) EBF with bolted replaceable active link

Figure 5 Two Forms of EBFs Integral with the Surrounding Structural System

Section 2: Determination of Post-earthquake capacity of the EBF system

This is presented in a sequential step by step basis.

Step 1: Initial Post-Earthquake Evaluation of Overall Building and Yielded Links

This needs to cover the following:

1. Undertake a survey of the vertical profile of the building to determine that it has self centered to within acceptable limits. The experience from Christchurch is that these limits are more severe than the 0.3% drift typically quoted in the literature. For example, the HSBC building, shown in Figure 1, returned to 0.14% maximum residual drift following the earthquake series. At this level of drift, the lift shaft guide rails needed realignment to prevent excessive wear on the lifts into the future, so it is likely this will be required for any non-low damage building following a severe earthquake. Rounding up that drift limit to 0.15% residual drift is a desirable target to meet.
2. Survey of the building services to make sure all are operational or are repaired prior to returning the building to service. Once again the experience from Christchurch is that with well designed and detailed modern steel framed buildings on stable ground this condition will be met.
3. Check the structural system for all visible evidence of inelastic demand and document its location. In painted steelwork this could be evident from cracking or spalling of paint; examples are given in Figure 2 and Figure 6, left hand side. Determine the likely cause of this deformation. In the EBF system, this should be confined to the active link web region of active links, and also to the flexible endplate in the brace/beam/column connection where the flexible endplate connecting the gusset plate to the column (or to the beam but typically to the column) accommodates the opening of the initial gap between the beam and the column as the frame deforms inelastically during the earthquake. Figure 6 left hand side shows an example of this, which is consistent with the inelastic deformation of the EBF system. Check splices for movement; no examples of this were noted in the steel framed buildings from Christchurch; Figure 6 right hand side shows an example of a flush beam splice.
4. Check the column base connections into the foundation system for any sign of inelastic response or cracking/spalling of the grout pad between the underside of the column endplate and the supporting concrete in bolted column endplate connections.
5. Undertake a Visual Examination, in accordance with AS/NZS 1554.1 (AS/NZS_1554.1 2004) of all critical locations within the EBF frame for evidence of material cracking or fracture. These critical locations are defined in NZS 3404.1: 2009 (NZS3404.1 2009). Document any instances for subsequent replacement of that component
6. Undertake a visual inspection of visible parts of the floor slab to determine the extent of cracking greater than 1.5mm and between 0.75mm and 1.5mm in width. This should initially be a rapid inspection of any visible concrete surfaces, concentrating on regions around the seismic resisting systems. As shown in Figure 7, crack widths greater than 0.75mm will typically form in lightly reinforced slabs under normal in-service conditions, so these need to be distinguished from cracks of similar width caused by earthquake. One distinguishing feature is that the earthquake cracks will typically have cleaner edges (compare Figure 8(a) with Figure 7, for example). A crack width measurement device should be used; see the example in Figure 8(a).
   The significance of these two limits is that for cracks larger than 1.5mm in width, any cold drawn mesh crossing the cracks may have fractured (Clifton 2005) while for cracks greater than 0.75mm wide, aggregate interlock is starting to be lost reducing the shear transfer capacity across the crack.
   As part of the full building post earthquake evaluation, floor slab crack maps should be compiled (see Figure 9) where the initial investigation shows earthquake induced crack widths exceeding 0.75mm. Cracks widths greater than 0.75mm should be epoxy grouted.
   The field evidence from Christchurch was that composite floor systems performed very well as general slabs and as diaphragm connections, with no crack widths greater than 1.5mm observed and very few above 0.75mm width.
7. Determine especially the condition of all diaphragm interfaces between the floor slab and seismic-resisting systems and note any cracks greater than 0.75mm in width. These will need to be repaired and for any crack widths greater than 1.5mm the concrete removed to determine the status of the reinforcement crossing the crack. Reinstatement of the interface adequacy may be required by removing concrete sufficient to put replacement reinforcing bars (Ductility S classification to AS/NZS 4671 (AS/NZS_4671 2001) into the slab and replace the concrete.
8. Finally, look carefully at the yielded webs of hot rolled active links to determine whether the shear yielding of the webs extends the full depth between the flanges or does not extend into the top and bottom quadrants of the webs. Two examples of the latter are shown in Figure 2 (a) and (c). Where the shear yielding does not extend at all into the top and bottom quadrants, it indicates that the earthquake induced plastic shear is not more than about 3%; where it extends fully into these quadrants the earthquake induced plastic shear

will be more than about 5%. This is due to the manufacturing induced plastic strain incorporated into the top and bottom quadrants of hot rolled section webs by the roller straightening of the flanges during manufacture (Nashid 2015). It offers a rapid way of determining approximately the peak earthquake induced shear demand. However it does not work on welded three plate section active links.

Step 2: Undertake Hardness Testing of the EBF Active Link Yielded Webs and Control Surfaces

The hardness testing is undertaken with the Leeb TH170 portable hardness tester or equivalent. See Figure 10 for a picture. The performance of this make and model has been extensively researched by Nashid (Nashid 2015) who has established detailed guidance on preparation of the steel surface to be hardness tested. When using this tester, select the hardness output on the Rockwell B (HRB) scale.

The provisions in this step are a summation of his key points.

1. On the basis of the visual observation, determine the number of storeys to be hardness tested. As a starting point, this should comprise:
    a. for buildings of 8 storeys up to 12 storeys, measure the hardness of the bottom level, top level and every third level between these two
    b. for buildings up to 8 storeys measure the bottom and every second level to the top
    c. for buildings over 12 storeys high, measure the bottom and every third storey to the top
    d. if there is an obvious visible difference in the inelastic demand within these levels, test each level over which that variation is occurring
2. The hardness testing must be undertaken over a region of web within the middle half height of the web and not closer than 50mm to any intermediate or end stiffeners. This is to avoid hardness testing in regions either plastically deformed during manufacture or within the heat affected zones around stiffeners.
3. Within this region, choose at least 5 sites to do the testing from.
4. Each site for surface testing must be cleaned to an average surface roughness Ra not exceeding 0.4μm. According to Nashid, this is most effectively achieved by using an electric *Powerfile* (see Figure 11) with P120 belt to remove all surface coatings and expose bare metal, following by hand sanding with a sandpaper grit number less than P180. These processes can be undertaken with either battery power or a portable generator if mains electricity is not available.
5. Undertake 7 hardness readings per site and discount the highest and the lowest readings. Record the average and the variation.
6. It is very important to establish a reliable baseline hardness from an area of the EBF that has clearly not undergone any fabrication or earthquake induced inelastic action. For an active link which is integral with the collector beams on each side, these baseline sites can be in the web of the collector beam, near the point of minimum moment. That can be taken as the ¼ point or the ¾ point along the clear length of the collector beam from the active link towards the adjacent column, but shall not be close to any incoming secondary beam onto the collector beam. Select two sites at each active link level and take the average of the two for the baseline hardness.
7. For bolted in place active link, select a brace or another member of the same grade and as close as possible to the same designation to determine the baseline hardness from.
8. Finally, determine the change in hardness for the yielded active links by the difference in hardness between the yielded region and the baseline values.

Step 3: Determine the Change in Mechanical Properties of the Active Links

The increase in yield stress f_y and tensile strength f_u with average change in hardness are given by:

$$R_{fy} = 0.0376\Delta_{HRB}+1 \quad \text{(Eqn 1)}$$

$$R_{fu} = 0.0187\Delta_{HRB} + 1 \quad \text{(Eqn 2)}$$

where:
R_{fy} = ratio of (f_{yp}/f_{y0})
f_{yp} = the increased yield stress of the plastically deformed active link web
f_{y0} = the yield stress pre-earthquake
R_{fu} = ratio of (f_{up}/f_{u0})
f_{up}= the increased tensile strength of the plastically deformed active link web
f_{u0} = the tensile strength pre-earthquake
Δ_{HRB} = the average change in hardness from the baseline to the plastically deformed active link web

The decrease in ultimate fracture strain with average change in hardness is given by:

$R_{\varepsilon u}$ = -0.0289$\Delta_{\mathbf{HRB}}$ + 1 (Eqn 3)

where:
$R_{\varepsilon u}$ = ratio of ($\varepsilon_{up/}\varepsilon_{u0)}$
ε_{up} = the decreased fracture strain of the plastically deformed active link web
ε_{u0} = the fracture strain pre-earthquake

Step 4: Determine the Peak Plastic Shear Strain Based on the Estimated Loading History.
This step first uses the key findings from (Nashid 2015) linking change in hardness to peak plastic shear strain. The key findings from his work in that regard are first summarised, followed by the means of making that determination:

1. Six pairs of active links comprising Grade 300 L0 steel were subjected to cyclic plastic shear loading. The details are given in Table 1.
2. For the active links under constant cyclic plastic shear, the hardness reaches its maximum value over one to three cycles of loading depending on the magnitude of the plastic shear. See Figure 12.
3. For the active links under variable cyclic plastic shear, the hardness corresponds to the peak plastic shear reached in the test cycle, irrespective of where in the cyclic loading regime this peak cycle occurs. See for example , Figure 13 which is for Link No 4. This means that, irrespective of where in the earthquake loading regime the peak cycle occurs, the hardness reading will allow that peak plastic shear strain to be used with reasonable accuracy.

The step secondly uses the results from Choi (Choi 2013) which are presented in Table 2

With these two lots of information, the average peak plastic shear strain in the EBF active link is determined as follows:

(1) From the average change in hardness, determine what peak plastic shear strain would be associated with this based on the end of the first cycle for link nos 1 to 3 in Table 1.
(2) From the average change in hardness, determine what (0.8x peak plastic shear strain would be associated with this based on the end of the second cycle for link nos 1 to 3 in Table 1
(3) Use linear interpolation as appropriate to determine the value for each determination and then take the minimum value determined from these two steps to determine the peak plastic shear strain. The minimum value is used because at the peak shear strain levels up to 15% it takes more than one cycle to reach the peak strain and using the maximum would over-determine the peak strain for a loading regime involving multiple cycles of variable plastic shear strain. Round to the nearest 0.5%
(4) Check this is consistent with the rest of the results.

This is best illustrated with three examples.

Example no 1: the difference in hardness in the active link, Δ_{HRB} = 15 the link is a hot rolled link and the pattern of paint loss is similar to that shown in Figure 1(a), with paint in the top and bottom quadrants of the web reasonably but not fully intact..

1(a) For the first cycle this is between the change in hardness for 7% and for 3%. Linear interpolation gives 6.1% strain, rounded to 6%
1(b) For the second cycle this is also between 7% and 3%. Linear interpolation gives 4.5% strain for two cycles to 0.8x the peak strain, giving a peak strain of 5.6, rounded to 5.5%.
1(c) The peak strain of 5.5% is consistent with some retention of paint in the top and bottom quadrants, giving a determined plastic cyclic shear strain of 5.5%

Example no 2: the difference in hardness in the active link, Δ_{HRB} = 20, the link is a hot rolled link and paint loss extends all the way to the root radius of the section and into the root radius in places.

2(a) For the first cycle this is between the change in hardness for 7% and 15%. Linear interpolation gives 13% strain.
2(b) For the second cycle this is at 7% strain for two cycles to 0.8x the peak strain, giving a peak strain of 8.75%, rounded to 9%.
2(c) The peak strain of 9% is consistent with loss of paint in the top and bottom quadrants, giving a determined peak plastic cyclic shear strain of 9%.

Example no 3: the difference in hardness in the active link, Δ_{HRB} = 8, the link is a hot rolled link and there is no paint loss in the web of the active link, just some cracking of the paint in the middle two quadrants.

3(a) For the first cycle this is at 3% strain.
3(b) For the second cycle this is between 0% and 3% strain for two cycles to 0.8x the peak strain. Linear interpolation gives 2 cycles to 0.8x the peak strain at 2%, giving a peak strain of 2.5%
3(c) The peak strain of 2.5% is consistent with only some cracking of paint in the middle two quadrants, giving a determined peak plastic cyclic shear strain of 2.5%.

Step 5: Estimate the Loading History and the Cumulative Plastic Shear Strain Demand

The loading history as a set of (S,N) data points, where (S, N) ≡ (Plastic shear strain, number of cycles) points is determined from Table 2. This depends on the length of strong ground motion shaking, which for most sites in New Zealand will be obtainable from GNS shortly after the earthquake and before the evaluation commences. As a rough estimate, an earthquake not exceeding Richter Magnitude Ms = 7.3 will have a length of strong ground motion shaking under 30 seconds; Ms > 7.3 will have a length of strong ground motion shaking over 30 seconds (Choi 2013).

The cumulative plastic shear strain demand (CPD%) is then determined from $\sum(S,N)$. For example, for example no 1 from step 4 for a length of strong ground motion shaking under 30 seconds, CPD is;

$$\text{CPD} = 4x5.5(1 + 2x0.8 + 2x0.65 + 3x0.5 + 4x0.3) = 145\%$$

Step 6: Consider the Change in Charpy Impact Energy from the Plastic Shear Strain Demand and the Presence of Crack Initiating Sites

Following the Kobe and Northridge earthquakes, the requirements for minimum Charpy Impact energy of seismic-resisting steels was raised from the previously specified value of 27J at 0°C to the current requirement of NZS 3404 of 70J at 0°C (NZS3404.1 2009) for members expected to undergo inelastic demand in a severe earthquake. This was based on research undertaken in Japan and New Zealand; the principal source for New Zealand is (Hyland 2008). Hyland researched the effect of axial pre-strain on the transition temperature of grade 300 constructional steels; a key finding from his work is shown in Figure 14.

In the Pacific Tower building, a fractured link was discovered in the top level of an EBF system. This link is shown in Figure 2 (c). All active links in that EBF frame were cut out and replaced, with each active link being subjected to detailed structural and metallurgical examination. Details are given in Chapter 5 of (Nashid 2015) and the key points are:

(a) The material in those active links had a CVN of as low as 5J at -15°C with an average of about 12J at 0°C and a maximum of 25J at 0°C. As usual with Charpy Impact tests, there is considerable variation between individual tests. The ductile brittle transition temperature (DBTT), defined as being at 27J, is11.5°C. The expected CVN for this steel would be 27J at 0°C. The estimated temperature of that steelwork at the time of the second strongest of the earthquake series in June 2011 at 90% of the design ULS PGA. That steelwork was located on the "cold side" of the building thermal envelope, being in a car parking area with open ventilation to the outside and with the building not occupied at the time of that earthquake.
(b) The active link which fractured had two shear studs welded onto the top flange over the stiffener at the end face of the active link side of the active link/brace/collector beam panel zone (see Figure 4). This was the crack initiation site for the fracture of that link, being in the region of very high local flange curvature at the ends of the active link where the local plastic hinge lines form. The fracture is shown in Figure 15
(c) Only the fractured stud on level 6 had shear studs in that location, the active links in the lower levels were of the same property steel, showed similar visible sign of web yielding but did not undergo brittle fracture.
(d) Therefore to get brittle fracture in that example required both low CVN and a crack initiator in an adverse location. On the basis of the limited data available, and for this specific building and fracture analysis only, an average threshold CVN of below 12 J is required. This is a low value as, Charpy tests for another EBF showed average Charpy values of 70J at 0°C and 86J at 10°C.
(e) For steelwork in buildings in service that is located on the warm side of the building external envelope, a permissible service temperature of 10°C should be used as this is a lower bound; for steelwork on the cold side of the building envelope use the LODMAT temperature given by NZS 3404 Figure 2.6.3.1.

On the basis of those key points, step 6 is implemented as follows:

1. Either determine the CVN value for the steel at 0°C by testing in accordance with AS1544.2 or use the value given on the Certified Mill Test Report or Test Certificate. If neither is available then use the

minimum specified value for the grade of steel being tested. If none of these are known, then the testing option must be used. (In some buildings from the Christchurch earthquakes, records held by consultants/council were not always available due to the earthquake damage and it was not clear what the specified grade of steel was with respect to CVN value). If CVN testing must be undertaken, it must be from the same grade and designation of steel; if the active link is continuous with the collector beams, take the samples from about the web centreline of the collector beams close to the ¼ or ¾ points along the collector beam clear span. A disc of material 75mm in diameter will need removing for these tests.

2. The 0% prestrain curve from Figure 14 is for a steel with CVN of 150J at 0°C. If the steel under consideration is a similar grade and has a CVN at 0°C different from 150J, adjust the 0% prestrain curve to the left or right as required so that it passes through the given CVN value at 0°C for the steel under consideration. (For example if the steel has 70J at 0°C the 0% prestrain curve must be shifted to the right.
3. From the peak plastic shear prestrain determined from Step 3, determine the equivalent plastic axial prestrain using Note 1 to Figure 14 and hence the amount of right hand shift required to the 0% prestrain curve and the shape of the prestrained curve.
4. The final curve is that adjusted first for the 0% prestrained CVN value at 0°C and secondly for the amount of plastic shear prestrain. See Note 3 to Figure 14 for an example from the HSBC building.
5. Determine the expected CVN value for the earthquake prestrained steel from this adjusted curve for a design operating temperature of 10°C for steelwork located on the warm side of the building envelope or for the LODAMT temperature from NZS 3404 for external steelwork or steelwork located on the cold side of the building envelope.
6. With regard to the presence of crack initiating sites, if such sites occur and have initiated cracking from the post earthquake visual inspection of Step 1 Item 6, then these active links will need replacing if the CVN is low. If there are potential crack initiating sites such as shear studs located in adverse locations identified in Figure 4, then these shear studs must be removed and the beam flange dressed in accordance with (NZS3404.1 2009) or in accordance with expert advice from eg the New Zealand Welding Centre (www.hera.org.nz)

Step 7: Determine Whether the Active Links Can be Left in Place or Require Replacement

1. With regard to their post earthquake plastic shear capacity, if the CPD from step 5 ≤ 0.55x284% = 156%, the active link can be left in place.
2. With regard to their post earthquake CVN values, the Standard AS/NZS 3404.1:2009 requires for the SeismicS0 condition that CVN be 70J at 0°C, and if this is not so then an Expert Fracture Analysis needs to be undertaken, before deciding if the active link can be left in place.
3. With regard to their post earthquake CVN values, if the S0 seismic condition is met from Step 6 and there are potential crack initiating sites but that have not induced crack initiation in the earthquake that has triggered the evaluation, the active link can be left in place.

Step 8: Determine the Number of Links Needing Replacement to Maintain an Appropriate Strength Balance Up the EBF Frame.

When the EBF frame is initially designed, the active link strength is decreased from the bottom level to the top in accordance with the decreasing seismic demand. This is typically undertaken in bands of several storeys, for example:

i. average the bottom 3 or 4 storeys of the building but not more than the bottom half
ii. average the top 2 to 4 storeys depending on the building height
iii. for buildings of 12 storeys or more average in bands of 4 storeys
iv. average the remaining storeys in bands of 2 storeys for up to 8 storeys in height and 3 storeys for up to 11 storeys in height

As determined from Step 3, the earthquake yielding of the active link webs will have raised the post-earthquake strength of the active links and reduced their fracture strain. This increase in strength will have reduced the ductility demand on the frame in future events. However, it means that if the active links in say the bottom few floors of an EBF frame are to be replaced and if they are to be replaced with active links of the same material grade and designation, then the active links left in place above those levels will now be stronger than the new links. This distribution of strength up the height of the EBF is now the opposite to what was originally designed and the extent to which that happens must be limited.

When undertaking post earthquake replacement of active links in Pacific Tower, the concepts developed, expressed in accordance with the recommendations of this guidance document, are as follows:

1. Either replace all links up the height of the EBF until the ratio R_{fy} of links to be left in place, as determined from Eqn.1, ≤ 1.25. This will mean replacing all links up the height of the frame for which the increase in hardness exceeds 7 HRB; or
2. Replace the links with a higher grade of material so that the ratio of $R_{fy}/(f_{y,new}/f_{y,old}) \leq 1.25$, where:
   $f_{y,new}$ is the nominal yield stress of the replacement active link steel
   $f_{y,old}$ is the nominal yield stress of the active link steel that is being replaced
3. The increase in grade is limited to that which will exhibit the required mechanical properties for future earthquake demand. It is recommended that if the original active links were Grade 300 steel or the equivalent to one of the material standards specified in NZS 3404, that the replacement material could be up to Grade 400.

Step 9 Determine the %NBS for the EBF Frame and For the Building

This evaluation is made for each principal axis direction and it is assumed that in that direction, the EBF system comprises 100% of the seismic resisting system. (This is typical practice).

9.1 If no active links are to be replaced, the % NBS is determined as follows:

9.1.1 Determine the increase in yield strength and decrease in fracture strain generated by the earthquake action on each level and the sum of these effects on the overall frame. These are called $R_{fy,link}$, $R_{fy,frame}$ and $R_{\varepsilon u,link}$ and $R_{\varepsilon u,frame}$, respectively

9.1.2 Determine the %NBS for strength for the whole frame and for the most heavily strained active link as follows:

$\%NBS_{,strength,frame} = (Z_{old}/Z_{new})* R_{fy,frame}*100$ Eqn 4

$\%NBS_{,strength,link} = (Z_{old}/Z_{new})* R_{fy,link}*100$ Eqn 5

where:
Z_{old} = Z factor from NZS 1170.5 prior to the earthquakes
Z_{new} = Revised Z factor from NZS 1170.5 following the earthquakes (if this has occurred)

9.1.3 Determine the %NBS for ductility capacity as follows:

$\%NBS_{,ductility,frame} = R_{\varepsilon u,frame}*100$ Eqn 6

$\%NBS_{,ductility,link} = R_{\varepsilon u,link}*100$ Eqn 7

9.1.4 Determine the %NBS for the frame and the link as follows:

$\%NBS_{,frame} = 0.5*(\%NBS_{,strength,frame} + \%NBS_{,ductility,frame})$ Eqn 8

$\%NBS_{,link} = 0.5*(\%NBS_{,strength,link} + \%NBS_{,ductility,link})$ Eqn 9

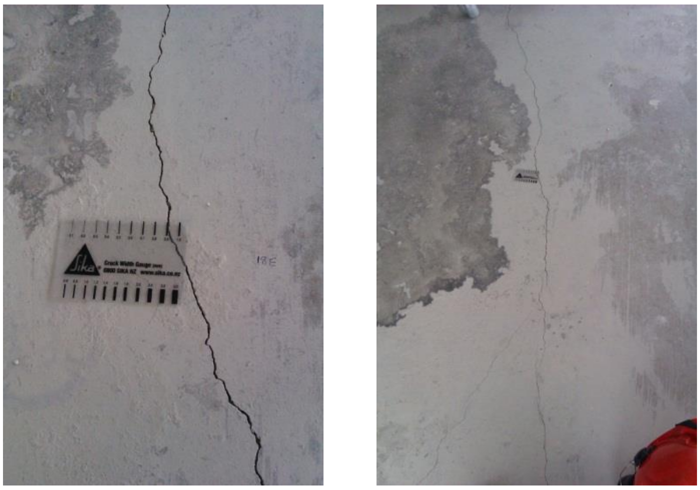
9.1.5 Determine the %NBS for the building as:
Once steps 9.1.1 to 9.1.4 are undertaken for each frame in the principal axis under consideration, the %NBS for the building about that principal axis is taken as the least of the $\%NBS_{,frame}$ and $\%NBS_{,link}$ determined for each frame acting along that principal axis.

9.2 If some active links are to be replaced, the %NBS is determined in accordance with the above procedure, but using the change in strength and the change in ductility taking into account the change in nominal specified properties of the change in grade of steel.

9.3 If all active links in an EBF are being replaced, then the %NBS can be conservatively taken as 100%. (In practice it will be at least 120% given that %NBS is typically based on average material strengths not nominal material strengths).

Connections in Buildings, Christchurch From left to right ; (a) Brace/beam/column connection showing out-of-plane yielding in endplate but no inelastic demand in gusset plate; (b) Flush moment endplate splice connection.
Figure 6 Connections in Buildings Post Earthquake (from (Clifton, Bruneau et al. 2011))

Figure 7 Crack over 0.75mm wide In Floor Slab Developed From Normal In-service Conditions



(a) 0.9mm width crack post earthquake

(b) 0.2mm crack post earthquake

Figure 8 Two Examples of Post Earthquake Cracks in Floor Slab

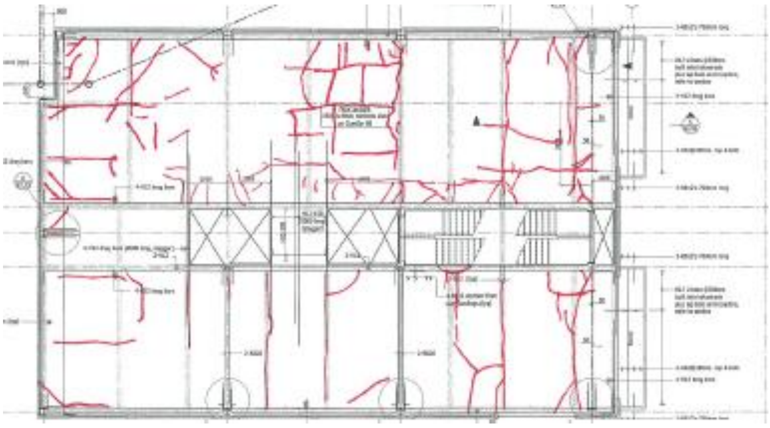


Figure 9 Floor Slab Crack Map

Figure 10 Integrated Portable Leeb Hardness Tester TH170

Figure 11 Powerfile Sander by Black&Decker

Table 1 Difference in Hardness from Cyclic Plastic Shear Testing (Nashid 2015)

Sample	Loading History	End of Cyclic Test	End of First Cycle	End of Second Cycle	Cumulative Plastic Shear Strain %	Fracture reached
		Difference (ΔHRB)	Difference (ΔHRB)	Difference (ΔHRB)		
Link 1	Uniform plastic shear strain 7% γ 5 cycles	21±2	17±3.5	20±3.5	140	No
Link 2	Uniform plastic shear strain 15% 5 cycles	22±4.5	21±4.5	22±4.5	300	No
Link 3	Uniform plastic shear strain 3% 5 cycles	13±4.5	8±5.5	12±4.5	60	No
Link 4	Variable plastic shear strain 20% max, 2% min 9 cycles	27±2.5	27±6	24±2.5	288	Yes
Link 5	Variable plastic shear strain 15% max 2% min 19 cycles	29±5	15±5	15±5	276	Yes
Link 6	Variable plastic shear strain 20% max 2% min 8.25 cycles	28±2	26±1.5	25±2.5	288	Yes

Note: the variations are between the maximum and minimum hardness readings for the particular sample; sample size typically 15 readings.

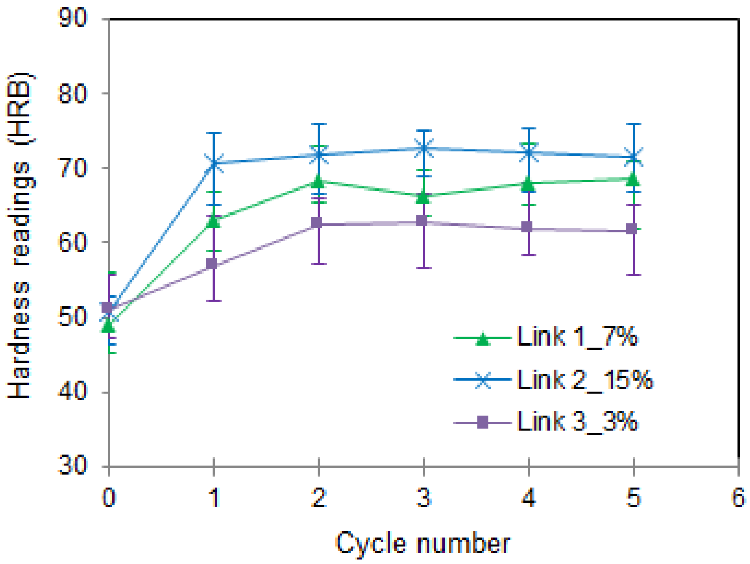


Figure 12 Hardness comparison at the centre of the webs using Leeb TH170 tester for uniform loading (Fig 7.6 from (Nashid 2015))

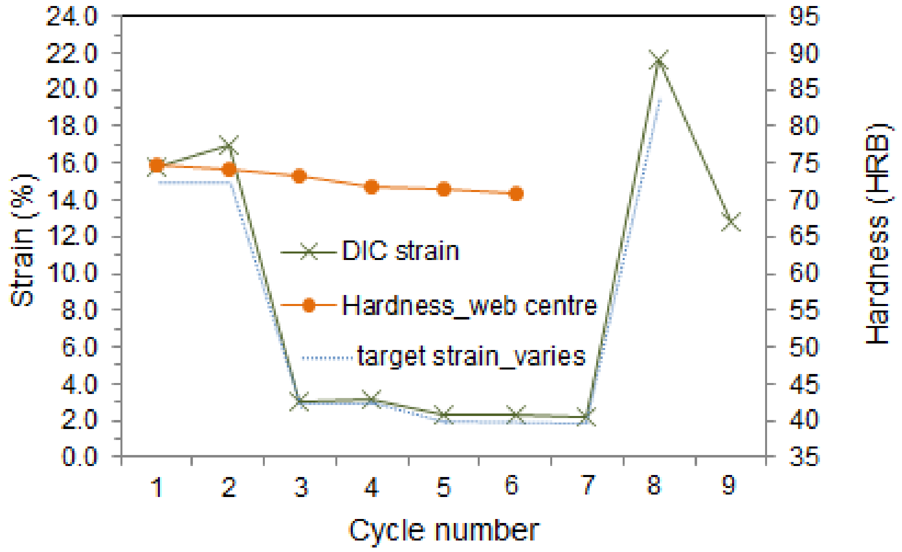


Figure 13 Target strain compared with DIC strain and hardness measured by TH170 tester (Fig 7.8 from (Nashid 2015))

Table 2 Relationship Between Number of Cycles of Inelastic Action as a Proportion of the Maximum Cycle (from (Choi 2013))

Length of Strong Motion Under 30 Seconds		Length of Strong Motion Over 30 Seconds	
No of Cycles	Proportion of Peak Cycle Plastic Shear Strain	No of Cycles	Proportion of Peak Cycle Plastic Shear Strain
1	1.00	1	1.00
2	0.80	2	0.80
2	0.65	4	0.65
3	0.50	6	0.50
4	0.30	8	0.30

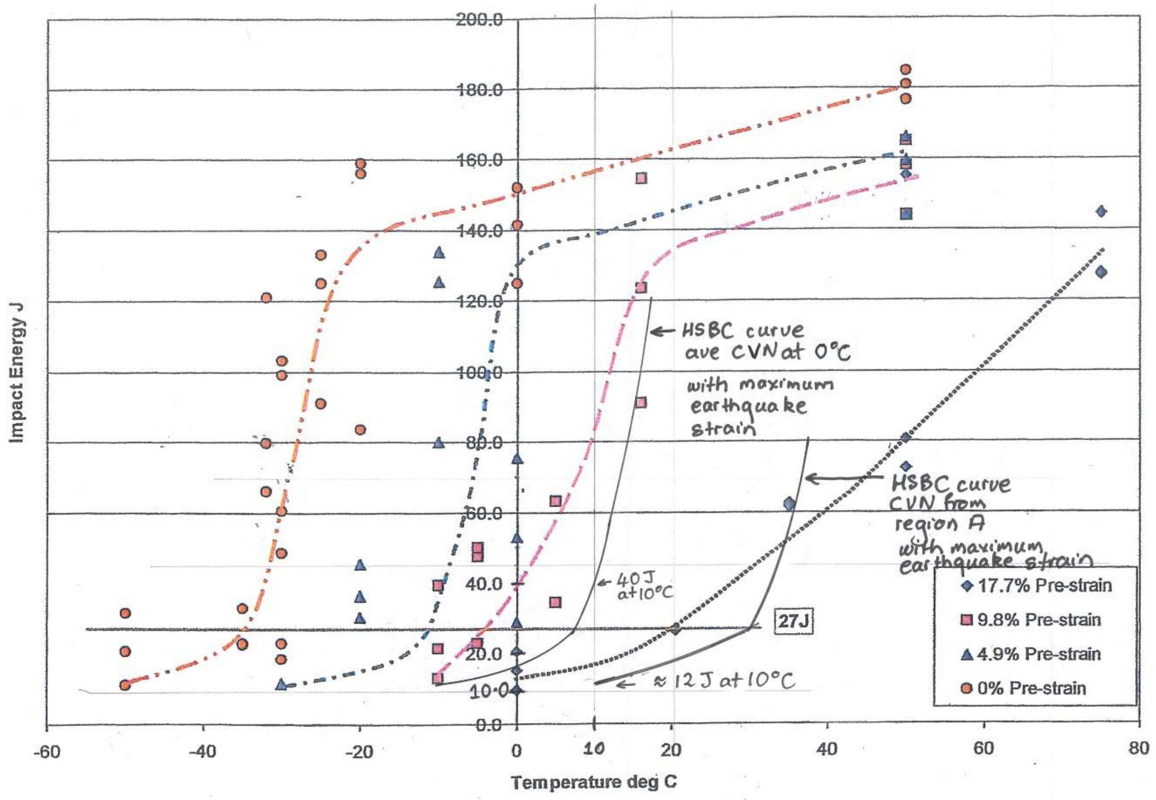


Figure 14 Charpy Impact Curves for Constructional Steels at Various Levels of Axial Prestrain (from (Hyland 2008))

Notes to Figure 14:

1. These pre-strains in this figure are plastic axial strain. To convert the plastic shear strain to plastic axial strain, in order to use this figure, the plastic shear strain must be divided by two; thus the curve for 4.9% pre-strain corresponds to a plastic shear strain of 4.9x2 = 9.8% (Nashid 2015). For example, this means that, if the non deformed steel from Example No 1, step 4, had the CVN curve for 0% prestrain shown in Figure 14, then after 5.5% plastic shear strain its CVN curve would correspond to that for 5.5/2 = 2.8% pre-strain in Figure 14, ie it would lie between the curve shown for 0% prestrain and the curve shown for 4.9% prestrain.
2. The two hand drawn in curves are those developed, on the basis of fracture analysis, for the HSBC building evaluation (Clifton and Ferguson 2014) undertaken using tested CVN values for non-prestrained samples of steel and modified for peak plastic shear demand using the process described in step 6.
3. The HSBC curve shown for the ave CVN at 0°C is for an average value of 70J at 0°C for 0% prestrain (measured from CVN tests on samples from the collector beams) and subjected to 5% maximum plastic shear strain.

Figure 15 Fracture of Active Link Flange Adjacent to Shear Stud (from (Gardiner, Clifton et al. 2013)

Section 3: Guidelines on Replacement of Damaged Active Links, Where Required.

Very brief guidelines are given only as the exact process to follow will be building specific and require expert input. Active links were replaced in Pacific Tower, with details of the replacement process given in (Gardiner, Clifton et al. 2013). That paper should be read by any engineers contemplating link replacement following the post-earthquake assessment of the building.

If an active link is determined as not meeting the criteria from Step 7 and Step 8 to remain in place, two options are available, namely:

Option 1: Cut out the active link, taking the cut back from the potential yielding region in the top of the braces and the ends of the collector beam adjacent to the active link, fabricate and weld into place a new member. Figure 16 shows this being done on Pacific Tower for a V brace active link and Figure 17 shows a D brace active link welded in place. The welding of the active link into the existing frame generates residual stresses from welding induced distortion and shrinkage and the welding sequence must be carefully planned to minimise these stresses. The NZ Welding Centre can advise on this; they provided this advice for the links shown in those two figures.

Option 2: Cut out the active link , as closely as practicable to the ends of the active link behind the end stiffener, prepare the end and weld a new stiffener in place onto the existing active panel zone. Measure the gap between the two ends into which the active link endplates will be bolted, fabricate a new active link with endplates that is 1mm shorter than the gap and bolt into place. Figure 18 shows the drawings for replacement of a D brace active link, while Figure 19 shows an installed D brace active link.

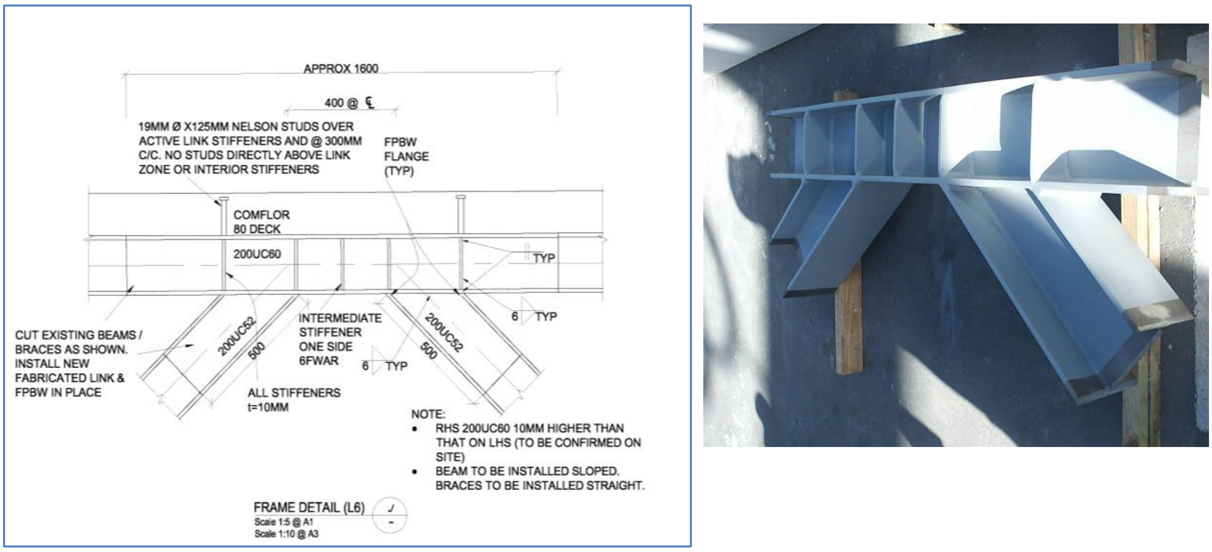
This option is particularly attractive for a D brace active link which was bolted to the column with a moment resisting bolted endplate (MEP) connection in the original fabrication as there is only one side of the active link to cut out of the existing frame.

In either case of replacement, where the active link has a slab over the top of it, the region of slab around the active link will need to be cut out for the replacement and reinstated. Figure 20 shows an example from Pacific Tower. Typically the slab will not need temporary support during this process.

To ensure the overall stability of the building during active link replacement, in the event of another significant earthquake, one level should be worked on at a time and only one active link replaced at a time. That will ensure the building overall continues to perform well under lateral loading during the link replacement period.

With the above guidance and the information given in (Gardiner, Clifton et al. 2013) engineers planning link replacement will be able to determine the appropriate type and processes to follow.

Note if replacing a link with one of a higher grade, when required from Step 8, ensure that the welding consumables and welding procedures used are appropriate for the higher grade steel. Guidance on this is in NZS 3404 and AS/NZS 1554.1 (AS/NZS_1554.1 2004).



(a) Drawing of replacement active link

(b) Fabricated link prior to installation on site

Figure 16 Welded In Replacement V Brace Active Link (from(Gardiner, Clifton et al. 2013))

Figure 17 Welded In Place D Brace Active Link (from(Gardiner, Clifton et al. 2013))

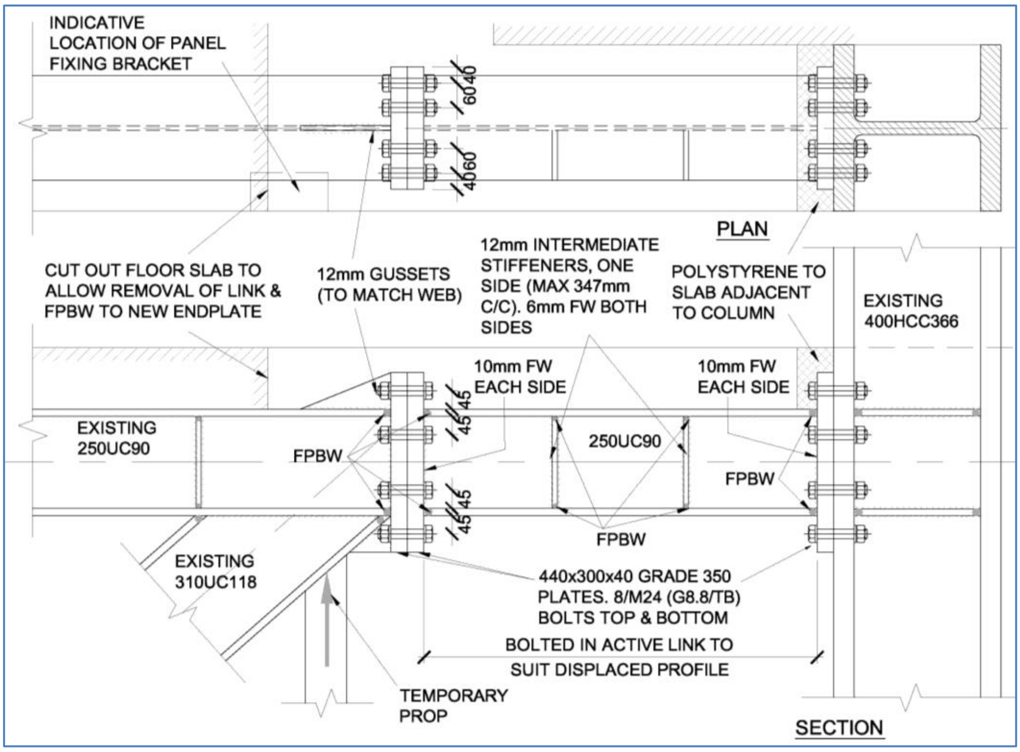


Figure 18 Bolted in Active Link Drawings (from(Gardiner, Clifton et al. 2013))

Figure 19 Bolted in Active Link Installed (from (Gardiner, Clifton et al. 2013))

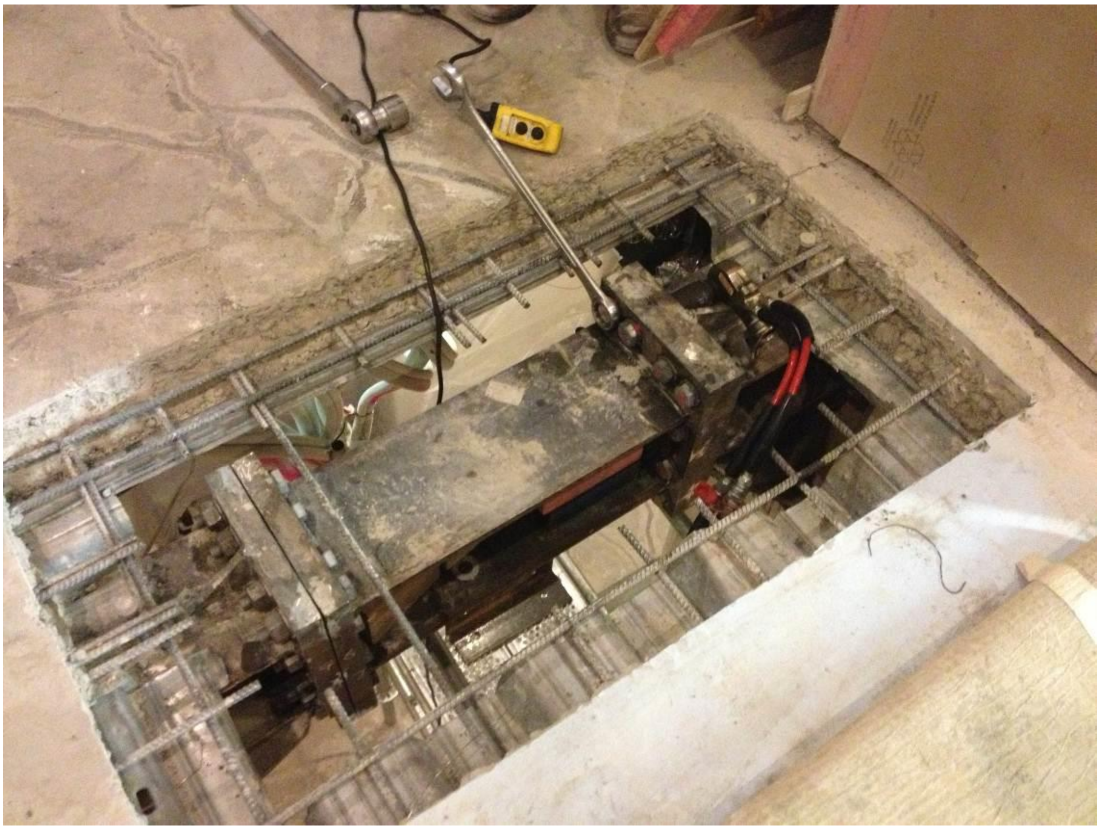


Figure 20 Slab Removed to Allow Bolted in Replacement Active Link to be Installed (from (Gardiner, Clifton et al. 2013))

Section 4: Example of Application of the Assessment Procedure.

This section presents an example of the assessment procedure given in Step 2. The building it is applied to is a hypothetical one, adapted from a building prepared for Steel Construction New Zealand by Aurecon Ltd (Aurecon 2011). An artist's impression of the building is given in Figure 21, while a typical floor plan view is given in Figure 22.

The building is 4 storeys high, has a heavy roof and for the purposes of this application there are two V braced EBFs, one at each end of the building in the short plan dimension. These are designated the Left Hand Frame (LHF) and the Right Hand Frame (RHF) respectively. They are located inside the thermal break of the external envelope, ie in the warm region of the building.

The details given in this example are for a hypothetical building, however post earthquake condition of the building and the hardness readings and the visible condition of the active link web are consistent with what was recorded and observed in yielded EBF systems from the 2010/2011 Christchurch earthquake series.

The steel in the Active Links is Grade 300 S0 steel in accordance with NZS 3404.1 (NZS3404.1 2009). The CVN of this steel is listed on the mill test certificate as 145J at 0°C.

The scenario is that the building active links have been pushed into the inelastic range by an earthquake of Ms 7.5 located 15 km from the site and 10km deep.

The step by step assessment process given in Section 2 is applied as follows:

Step 1: Initial Post Earthquake Evaluation of Overall Building and Yielded Links

1. The building has self centered to within 0.1% residual drift.
2. All services are functioning; the lifts can be operated by the lift shaft guide rails will require realignment to avoid future excessive wear on the lift system
3. A detailed check of the structural system has shown that all visible inelastic demand is confined to the active link webs. There is no visible damage to the gravity load carrying system
4. Column base connections show no damage to the column base or the hold down system into the foundations. There is no visible sign of ground instability in the proximity of the building, nor sign of inelastic demand in the foundation structural system (as much as can be identified from a detailed inspection of the visible components; it was designed for the overstrength actions from the seismic resisting systems so hidden inelastic demand is not expected.
5. A Visual Examination has been undertaken of the EBF system with no cracking reported. This was done by a Welding Inspection company.
6. A visual examination of the floor slabs and the slab to seismic resisting system diaphragm interfaces has been undertaken. This shows no damage to the diaphragm interface has been reported nor any seismic induced cracks larger than 0.75mm in width. There are some cracks larger than this width noted but their condition shows them to be pre-existing cracks caused by concrete shrinkage.
7. See 6
8. The shear yielding in the active link webs does not extend full depth for any link, but does extend partially into the top and bottom quadrants of the active link webs for level 1 in each frame.

Step 2: Undertake Hardness Testing of the EBF Active Link Yielded Webs and Control Surfaces.

This has been undertaken by a specialist inspection company; the results are in Table 3

Step 3: Determine Change in Mechanical Properties of the Active Links

The results are in Table 4

Step 4: Determine the Peak Plastic Shear Strain Based on the Estimated Loading History

The results are in Table 5

Step 5: Estimate the Loading History and the Cumulative Plastic Shear Demand

For the Ms = 7.5 earthquake, the (S,N) cycles from Table 2 for the length of strong ground motion over 30 seconds is used. The results are in Table 5.

Step 6: Consider the Change in Charpy Impact Energy from the Plastic Shear Strain Demand and the Presence of Crack Initiating Sites

Determine the change in CVN for the most heavily loaded link; if that is satisfactory then all others will be satisfactory.

The most heavily loaded link is at level 1 in the RHF, with a peak plastic shear strain of 4.5%. This corresponds to an equivalent cyclic prestrain of . 2.3% which means the shift in CVN due to cyclic loading is approximately midway between the 0% prestrain curve and the 4.9% prestrain curve shown in Figure 14.

The 0% prestrain curve in that figure is for steel with CVN = 150J at 0°C. The steel in these links has 145J at 0°C, which means that the curves must be shifted approx 10°C to the right. Then making allowance for the cyclic plastic shear strain shifts the curve for this link a further 10°C to the right, a total shift of some 20 °C to the right. However, the total shift is not as much as that shown in Figure 14 for the 4.9% axially prestrained curve. That shifted curve has a CVN of almost 135J at 0°C which is greater than the seismic S0 condition of 70J at 0°C.

This is well above the minimum required for the steel from Step 6, so no further investigation is required. As this applies to the most heavily loaded link it means all the others are satisfactory.

Note that if the S0 condition had not been met, then a more detailed Expert Fracture Analysis would be required to determine the fracture performance of the active link. This is worth undertaking as lower values may be shown to be satisfactory.

Step 7: Consider Whether the Active Links Can be Left in Place or Require Replacement

1. With regard to the post earthquake plastic shear capacity, the links at level 1 in both frames require replacement; all others can be left in place.
2. With regard to post earthquake CVN values, all links are satisfactory.

Step 8: Determine the Number of Active Links Requiring Replacement to Maintain an Appropriate Strength Balance Up the EBF Frame

From item 1 in step 8, the increase in hardness exceeds 7 HRB at every level, so if the links at level 1 are to be replaced with the same grade and designation then the links above will require replacement to maintain the strength balance up the two frames.

However, replacing all the links above to achieve that is unreasonably expensive, so the replacement active links will be made a higher grade to compensate. From step 8 item 2, the nominal fy required for the replacement links to satisfy that provision is 348MPa for the LHF link and 360MPa for the RHF link. The best solution is to use a hot rolled Grade 350 material to the designation, with S0 CVN classification. This is slightly under the strength limit for the RHF, but will be acceptable in terms of preserving the desired frame behaviour.

This means that the increase in R_{fy} for the links on level 1 now becomes 1.17 for the two links and the total R_{fy} for the frames becomes 1.3 for the LHF and 1.35 for the RHF.

The change in $R_{\varepsilon u}$ becomes 0.8 (= 20%/25%) for the two level 1 links and that for each frame becomes 0.75 for the LHF and the RHF.

Step 9: Determine the %NBS for the EBF Frame and for the Building

Because the links on level 1 are to be replaced, the procedure for %NBS determination follows that in step 9.2.

Given there is no change in Z factor, then for the LHF frame and for the most heavily loaded link in that frame (which is now that on level 2):

$\%NBS_{,strength,frame} = (Z_{old}/Z_{new})^* R_{fy,frame}{*}100 = 1.0{*}1.3{*}100 = 130\%$
$\%NBS_{,strength,link} = (Z_{old}/Z_{new})^* R_{fy,link}{*}100 = 1.0{*}1.40{*}100 = 140\%$
$\%NBS_{,ductility,frame} = R_{\varepsilon u,frame}{*}100 = 0.75{*}100 = 75\%$
$\%NBS_{,ductility,link} = R_{\varepsilon u,link}{*}100 = 0.70{*}100 = 70\%$

For the RHF frame and for the most heavily loaded link in that frame (which is now that on level 2):

$\%NBS_{,strength,frame} = (Z_{old}/Z_{new}) * R_{fy,frame} * 100 = 1.0 * 1.35 * 100 = 135\%$
$\%NBS_{,strength,link} = (Z_{old}/Z_{new}) * R_{fy,link} * 100 = 1.0 * 1.45 * 100 = 145\%$
$\%NBS_{,ductility,frame} = R_{\varepsilon u,frame} * 100 = 0.75 * 100 = 75\%$
$\%NBS_{,ductility,link} = R_{\varepsilon u,link} * 100 = 0.70 * 100 = 70\%$

Finally, the %NBS is given by:
$\%NBS_{,frame} = 0.5*(\%NBS_{,strength,frame} + \%NBS_{,ductility,frame})$ = 103% for the LHF frame and 105% for the RHF frame, therefore 104% for the overall building in the direction of seismic loading parallel to the short span.
$\%NBS_{,link} = 0.5*(\%NBS_{,strength,link} + \%NBS_{,ductility,link})$ = 105% for the LHF critical link and 108% for the RHF critical link; meaning the LHF critical link governs.

That is the end of the assessment.

Figure 21 Example Four Storey Building For Application of Assessment Process

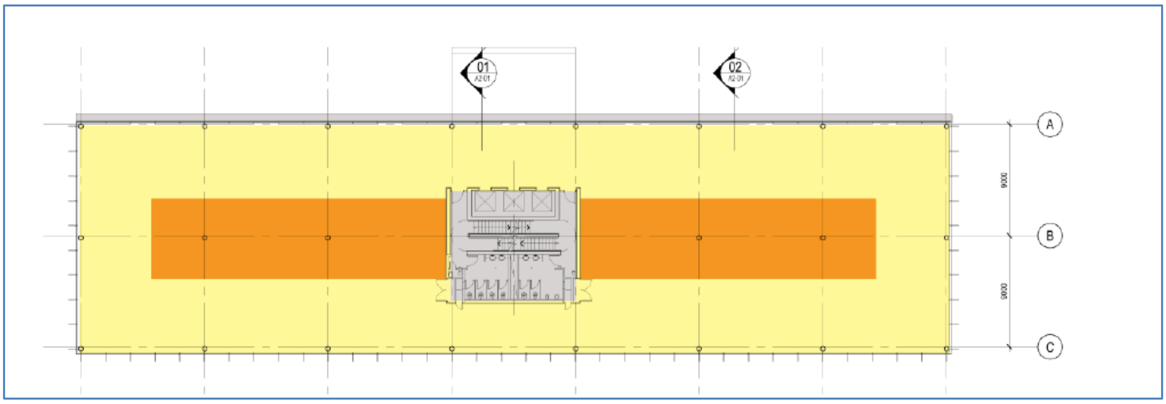


Figure 22 Typical Floor Plan of Building

Table 3 Active Link Hardness Readings For Example Building

Item No	Location Note 2	Floor	Active Link Web Hardness Readings Note 1	Baseline Hardness Readings from Collector Beam Note 1	Increase in Hardness	Visible Condition of Active Link Web
			HRB	HRB	ΔHRB	
1.1	LHF	1	81	69	12	Significant paint loss top coat and some undercoat not quite full web depth
1.2	LHF	2	81	70	11	Some paint loss top coat and some undercoat over middle half web depth.
1.3	LHF	3	78	69	9	Very minor paint loss from middle of web
1.4	LHF	4	76	68	8	No paint loss; minor cracking of paint
2.1	RHF	1	82	69	13	Significant paint loss top coat and some undercoat not quite full web depth
2.2	RHF	2	81	70	11	Some paint loss top coat and some undercoat over middle half web depth.
2.3	RHF	3	78	68	10	Minor paint loss from middle of web
2.4	RHF	4	77	69	8	No paint loss; minor cracking of paint

Notes :
1. From Section 2 Step 2 these will be average readings from up to 4 readings per active link and between 2 and 4 in the control zone. The variation from the average is typically ±2 to ±2.5 HRB
2. LHF ≡ Left Hand Frame; RHF ≡ Right Hand Frame
3.The baseline readings are taken in accordance with Step 2 item 6

Table 4 Example Building Changes in Mechanical Properties of The Active Links Due to the Earthquake Yielding

Item No	Location Note 2	Floor	Multiplier on pre-earthquake f_y and f_u		Multiplier on pre-earthquake ε_u	Visible Condition of Active Link Web
			R_{fy}	R_{fu}	$R_{\varepsilon u}$	
1.1	LHF	1	1.45	1.20	0.65	Significant paint loss top coat and some undercoat not quite full web depth
1.2	LHF	2	1.40	1.20	0.70	Some paint loss top coat and some undercoat over middle half web depth.
1.3	LHF	3	1.35	1.15	0.75	Very minor paint loss from middle of web
1.4	LHF	4	1.30	1.15	0.75	No paint loss; minor cracking of paint
1.5	LHF	All	1.40	1.20	0.70	
2.1	RHF	1	1.50	1.25	0.60	Significant paint loss top coat and some undercoat not quite full web depth
2.2	RHF	2	1.45	1.20	0.70	Some paint loss top coat and some undercoat over middle half web depth.
2.3	RHF	3	1.40	1.20	0.70	Minor paint loss from middle of web
2.4	RHF	4	1.30	1.15	0.75	No paint loss; minor cracking of paint
2.5	RHF	All	1.40	1.20	0.70	

Note:
1. The values given are rounded to the nearest 0.05 value
2. The frame values are the average of the individual level values for each frame

Table 5 Peak Plastic Shear Strain and Cumulative Plastic Shear Demand

Item No	Location Note 2	Floor	Peak Plastic Shear Strain	Cumulative Plastic Shear Strain	Visible Condition of Active Link Web
			%	%	
1.1	LHF	1	4.0	170	Significant paint loss top coat and some undercoat not quite full web depth
1.2	LHF	2	3.5	148	Some paint loss top coat and some undercoat over middle half web depth.
1.3	LHF	3	3.0	127	Very minor paint loss from middle of web
1.4	LHF	4	2.5	106	No paint loss; minor cracking of paint
2.1	RHF	1	4.5	191	Significant paint loss top coat and some undercoat not quite full web depth
2.2	RHF	2	3.5	148	Some paint loss top coat and some undercoat over middle half web depth.
2.3	RHF	3	3.0	127	Minor paint loss from middle of web
2.4	RHF	4	2.5	106	No paint loss; minor cracking of paint

Note:
1. The values given are rounded to the nearest 0.5% value

Section 5: Further Research Required to Complete this Procedure

The application of Nashid's (Nashid 2015) and Choi's (Choi 2013) research to the HSBC Tower evaluation (Clifton and Ferguson 2014) has shown up the following unanswered questions:

1. What is the effect of strain ageing of the shear deformed active link on the future deformation capacity? We know the effect of strain ageing from inelastic shear deformation on the tensile performance, which is incorporated into the change in mechanical properties in Step 3, but not on the cyclic shear performance. This is because the active links tested by Nashid had to be destroyed to cut the tensile test samples from them which were vital to completing his studies
2. How robust are the cumulative plastic shear deformation limits that have been set? These are given in step 7 item 1 and are based on Nashid's testing. However there are only three samples which fractured under variable loading and this included under cycles of plasticity very much higher than would be seen in a link being assessed to remain in place. When testing under constant cycles of lower strain, higher CPD values were obtained without fracture; see for example the CPD strain of link no 2 in Table 1. If the loading is variable with a peak plastic shear strain of around 7%, can the CPD values be higher? That test result would suggest they can.
3. What is the influence of the Charpy Impact energy of the steel on the inelastic cyclic performance?

The plan in 2016 is to undertake a ME project aimed at getting some robust answers to questions 1 and 2 above after which this procedure will be updated and then the third question addressed.

Acknowledgements.

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References

AS/NZS_1554.1 (2004). Structural steel welding Part 1: Welding of steel structures. Wellington, New Zealand, Standards New Zealand.

AS/NZS_4671 (2001). Steel Reinforcing Materials. Wellington, New Zealand, Standards New Zealand.

Aurecon (2011). SCNZ Material Comparison Study Design Features Report. Wellington, New Zealand, Aurecon Ltd.

Choi, J.-H. (2013). Evaluation of Cyclic Plastic Deformation of Active Links in Eccentrically Braced Frames. Auckland, New Zealand, University of Auckland.

Clifton, G. (2005). Semi-Rigid Joints for Moment-Resisting Steel Framed Seismic-Resisting Systems. PhD Thesis Report, University of Auckland.

Clifton, G. C., M. Bruneau, et al. (2011). "Steel structures from the Christchurch Earthquake series of 2010 and 2011." Bulletin of the New Zealand Society for Earthquake Engineering **44**(4): 297-318.

Clifton, G. C. and G. Ferguson (2014). Confidential Document: Peer Review DEE of HSBC Tower. Auckland, New Zealand, The University of Auckland.

Gardiner, S., G. C. Clifton, et al. (2013). Performance, Damage Assessment and Repair of a Multistorey Eccentrically Braced Framed Building Following the Christchurch Earthquake Series. Steel Innovations 2013. SCNZ. Christchurch, New Zealand, Steel Construction New Zealand.

Hyland, C. (2008). Assessment of Ductile Endurance of Earthquake Resisting Steel Members. PhD, The University of Auckland.

Nashid, H. (2015). Development of a Non-Destructive Field Based Method to Determine the Relationship Between Hardness and Plastic Strain in Cyclically Deformed Eccentrically Braced Frame Active Links. PhD, University of Auckland.

NZS3404.1 (2009). NZS 3404: 2009: Part 1: Materials, Fabrication and Erection. Wellington, New Zealand, Standards New Zealand.

Pussegoda, L. N. (1978). Strain Age Embrittlement in Reinforcing Steels. Doctor of Philosophy, University of Canterbury.