ASSESSMENT AND REPAIR OF DAMAGED ECCENTRICALLY BRACED FRAMED BUILDINGS FOLLOWING A SEVERE EARTHQUAKE

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Abstract

The Christchurch earthquake series of 2010/2011 was the first event worldwide to cause Eccentrically Braced Frame (EBF) buildings to have fractures in their links. The response of these most systems was good, with a high damage threshold. However, the earthquakes were sufficiently strong to cause partial to full plastic mechanisms in EBF systems, with significant yielding of EBF active link webs over many to all levels in buildings from 3 to 22 storeys in height.

In the damaged buildings, it was relatively easy to access the yielded webs of links in order to perform evaluation of their residual capacity. It was also straightforward to check whether any other elements of the EBF system had undergone yield.

In all cases, yielding had been confined to the active link webs. A research programme was undertaken in order to develop a method of determining the post-earthquake capacity of an EBF system with yielded active link webs, using hardness testing carried out in-situ with a portable Leeb TH170 hardness tester.

This research programme provided answers to the following;

1. What is the pattern of shear yielding expected in webs of EBF active links?
2. What surface preparation should be undertaken for replicable hardness testing?
3. What number of hardness tests should be undertaken at what locations?
4. How do changes in mechanical properties relate to changes in hardness?
5. How can the web cumulative plastic displacement (CPD) be estimated based on the hardness change?
6. What is the influence of strain ageing on the hardness readings?
7. What is the influence of plastic strain on the web Charpy Impact value?
8. How should the decision regarding *replace* or *leave in place* be made given test results?
9. If some active links have to be replaced, how strong should the replacement links be?
10. What is the post-earthquake capacity of a yielded EBF system with no, or with partial, active link replacement?

The paper provides an overview of the evaluation procedure developed and its implementation to an EBF system with post-earthquake yielded active links.

Keywords: seismic damage; assessment; repair;eccentrically braced frames.

1. Introduction and Background

The 2010/2011 Christchurch earthquake series were the first earthquakes worldwide to push EBFs so that some fractured. Many other links in a range of EBF buildings ranging from 2 to 22 storeys in height yielded [1]. 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 active links in every Christchurch central business district EBF building. 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 urgent need for evaluation of the EBF system for post-earthquake inspection and repair/replacement of active links led to a significant research project, aimed at developing a dependable, non-destructive method of determining the cumulative and the peak plastic demand on EBF active links. The fundamental reserach work was undertaken by Nashid [2], followed by Currie [3]. A design procedure [4] was developed from this work by the first two authors of this paper for the New Zealand regulators. It covers the V-brace and the D-brace active links, shown in Figure 3.

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 a common detail in EBFs which are built integrally with the structural frame. Figure 5 shows examples of both sorts used in buildings.

This paper presents a summary of the key details from the evaluation procedure, followed by application to a hypothetical building. It has to be a hypothetical building as the evaluation of a real building has been shown to be sensitive to public identification of details pertaining to an actual building.

2. Scenario for Application of the Evaluation Procedure

The scenario is as follows:

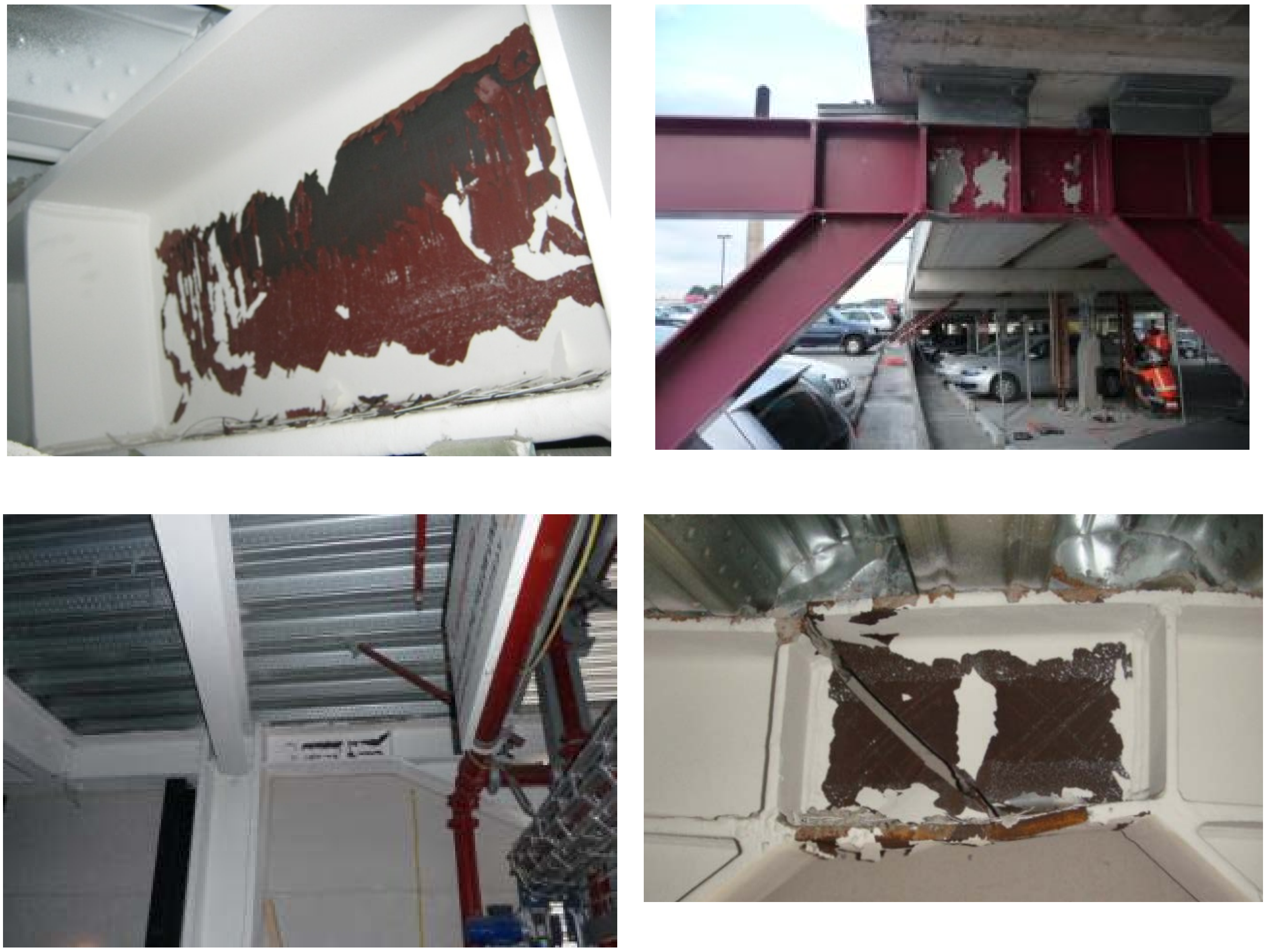
(1) There has been a severe earthquake, sufficient to push buildings with EBF systems into the inelastic range. The magnitude and depth of this earthquake are known.

(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, meaning there is no loss of stability due to liquefaction or other forms of ground instability. This includes minimal change in height of the ground surface under the structure.
   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.

(3) Access to the building is available and especially to the EBF seismic resisting systems within the building. The evaluation procedure uses a portable hardness measurer and methods of surface preparation at the proposed points of measurement so is not dependent on power being available.

The key question is to determine whether the structurally damaged members need replacement.

Figure 1 HSBC Building, Christchurch, March 2011 (from [1])



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 Feb 2011 Earthquake (from [1])

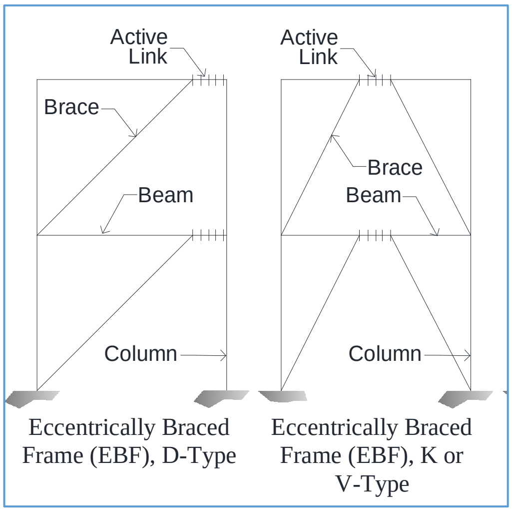


Figure 3 Two Common Types of BEFs with Member Terminology (from [2])

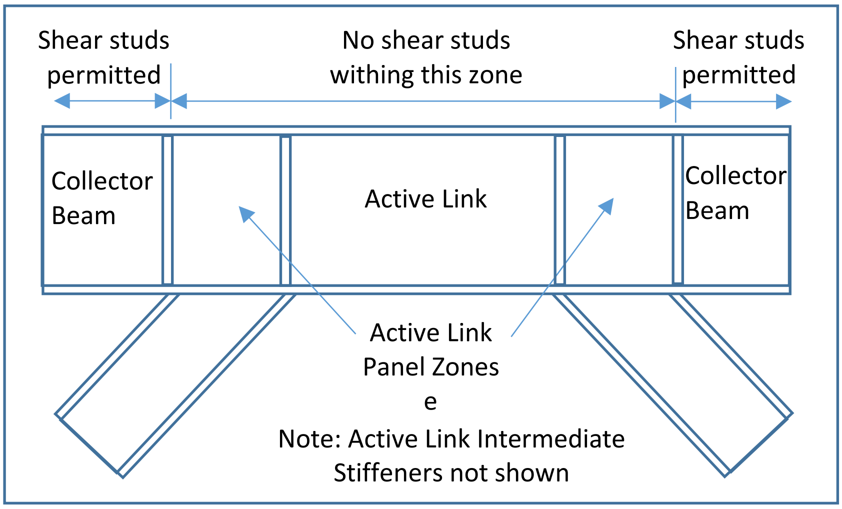
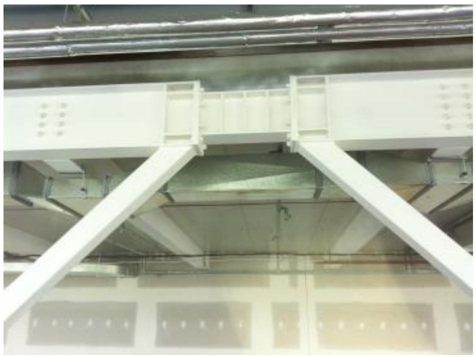


Figure 4 Terminology for Active Link Components

(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

3. Determining the Post Earthquake Capacity of the EBF System

The post-earthquake capacity is evaluated in the 9 step process summarised below [4]:

Step 1: Initial post-earthquake evaluation of overall building and yielded links. This involves:

- Determination of building self-centering such that interstorey lateral drifts are 0.15% or lower.
- Ensuring services are operational, or non-structural repairs (e.g. to lift shaft guide rails)are easily conducted
- Determination of locations of visible inelastic demand in the superstructure and visible foundations from Luder lines, or paint cracking.
- Undertaking weld visual examination for evidence of material cracking or fracture according to the welding standard, especially in the EBF frame.
- Undertaking visual inspection of floor slab crack widths. Those greater than 0.75mm wide shall be repaired with epoxy injection. Visible floor slab, or slab with hard coverings is first inspected. Then, if this shows such cracking, or there are other reasons, other floor coverings can be removed.
- Undertaking a close inspection of the diaphragm interfaces to determine any inelastic action. More recent research [5] is helpful in this assessment.

Step 2: Undertaking of hardness testing of EBF active link yielded webs and control surfaces. This requires selection of the regions of the yielded webs to be tested, surface preparation of these regions, and testing. A large number of tests should be undertaken and finally the change in hardness between the yielded regions and the control regions. Further details of this step are in [4].

Step 3: Determination of the change in mechanical properties of the yielded links using the equations presented in [4]. The increase in yield stress f_y and tensile strength f_u with average change in hardness are given by:

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

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

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:

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

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: Determination of the peak plastic shear strain based on the change in hardness and the estimated loading history. This is based on the work of Nashid [2], Currie [3] and Choi [6]; as summarised in [7].

Step 5: Estimation of loading history and cumulative plastic shear strain demand following the procedure in [4], developed from [6]. The (S,N) data points, where (S, N) ≡ (Plastic shear strain, number of cycles), are length of strong ground motion shaking dependent in accordance with Table 2 of [4].

Step 6: Consider the change in Charpy V Notch Impact Energy (CVN) from the plastic shear strain demand and the presence of crack initation sites. This follows the procedure in [4] and the work of Hyland [8].

Step 7: On the bases of peak plastic strain, cumulative plastic strain and change in CVN values, determine whether each active link needs replacement or can be left in place.

Step 8: If some active links require replacement in a given EBF frame, the number to be replaced should be determined considering strength hierarchy up the EBF frame.

Step 9: If required for reporting, determine the percentage New Building Strength (%NBS) for the EBF and for the building. The procedure is given in [4].

3. Replacement of any Active Links

If an active link does not meet the criteria from Step 7 and Step 8 above, several options are available for replacement, including:

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. Welding induced residual stresses must be carefully controlled through appropriate procedures.
Option 2: Cut out the active link slightly into the panel zone, prepare the ends and weld an endplate in place onto the existing active panel zone. Fabricate a new active link with endplates, that is slightly shorter than the gap, and bolt into place with fully tensioned bolts..

Examples of both options, with pictures, are given in [4].

4. Example of Application to a Hypothetical Building

This section presents an example of the assessment procedure to a hypothetical building, adapted from a building prepared by for Steel Construction New Zealand by a New Zealand consulting engineering firm [9].

Figure 6 Example Four Storey Building for Applicaiton of the Assessment Process from [4].

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, i.e. in the warm region of the building. See Figure 6 and Figure 7.

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

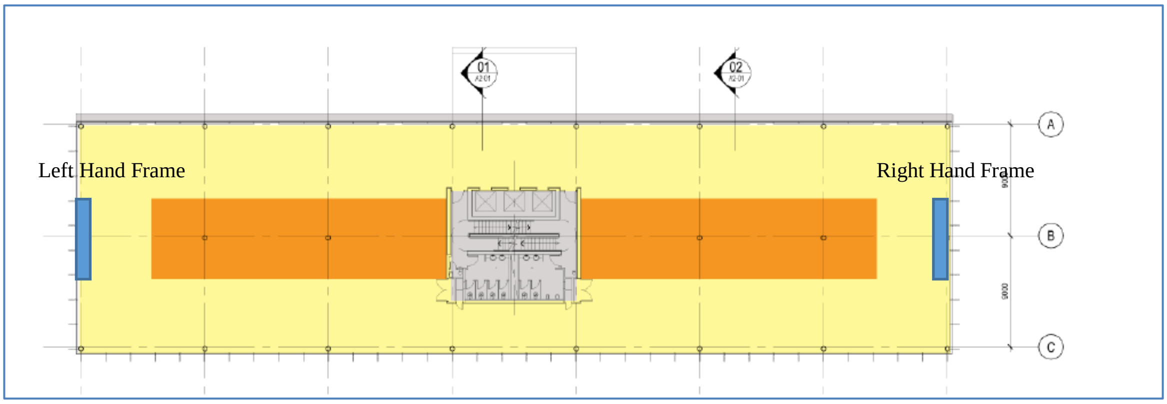


Figure 7 Typical Floor Plan of Building

was recorded and observed in yielded EBF systems from the 2010/2011 Christchurch earthquake series to which the evaluation procedure has been applied.

The steel in the Active Links is Grade 300 S0 steel in accordance with NZS 3404.1 [10]. 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 magnitude 7.5 located 15 km from the site generating strong ground motion shaking for over 30 seconds, as obtained from the national seismological institute.

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 but 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 with no local buckling. 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 of the EBF system by a welding inspection company indicates no cracking.
6. A visual examination of the floor slabs shows no damage with new seismic induced crack widths larger than 0.75mm. (Any cracks larger than this width were pre-existing cracks caused by concrete shrinkage).
7. A visual examination of the slab-to-vertical lateral force resisting system interfaces indicates no new visible sign of damage with cracks larger than 0.75mm wide.
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: Undertaking of hardness testing

This has been undertaken by a specialist inspection company; the results are in Table 1.

Step 3: Determination of the change in mechanical properties

This uses Eqns 1 to 3, developed from [2, 3]. Results are in Table 2.

Step 4: Determination of Peak Plastic Shear Strain

This uses the procedure given in Section 2, Step 4 of [4]. Refer to the three examples shown in that document on how to apply the procedure; space limitations mean it cannot be shown herein. Results are in Table 3.

Step 5: Estimation of Loading History and the Cumulative Plastic Shear Demand

For the magnitude 7.5 earthquake, the (S-N) cycles from Table 2 of [4] for the length of strong ground motion over 30 seconds is used. The results are in Table 3.

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

Determine the change in CVN for the most heavily loaded link. If that is satisfactory then all others will be satisfactory. Space restrictions mean the process cannot be given in this paper. It was found that the unstrained value of CVN = 150 J at 0 Deg C is reduced to 135J by the seismic induced peak plastic strain which is still well above the minimum required of 70 J at 0 Deg 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 other links are also satisfactory.

Table 1 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

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 the hardness survey, given in Table 1, 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, due to the strength increase in those links.

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. To keep the ratio of $R_{fy}/(f_{y,new}/f_{y,old}) \leq 1.25$, as recommended in [4], the minimum specified f_y 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 (70J at 0 Deg C). This is slightly under the strength limit for the RHF, but will be acceptable in terms of preserving the desired frame behaviour.

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. However there is not space in this paper to present the details, which are given in [4].

Table 2 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 3 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: The values given are rounded to the nearest 0.5% value

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6. References

1. Clifton, G.C., et al., *Steel structures from the Christchurch Earthquake series of 2010 and 2011*. Bulletin of the New Zealand Society for Earthquake Engineering, 2011. **44**(4): p. 297-318.
2. Nashid, H., *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*, in *Department of Civil and Environmental Engineering*. 2015, University of Auckland: Auckland, New Zealand.
3. Currie, R.T., *Cyclic Plastic Shear and the Effects of Strain Ageing on Eccentrically Braced Frame Active Links*, in *Civil and Environmental Engineering*. 2017, University of Auckland: Auckland, New Zealand.
4. Clifton, G.C. and G. Ferguson, *Determination of the Post-Earthquake Capacity of an Eccentrically Braced Frame Seismic Resisting System*, in *ATC-15-15 U.S. - Japan - New Zealand Workshop on the Improvement of Structural Engineering and Resiliency*, A.T. Council, Editor. 2016, ATC: Nara, Japan.
5. Rezaeian, H., G.C. Clifton, and J. Lim. *Compatibility Forces in Floor Diaphragms of Steel Braced Multi-storey Buildings*. in *STESSA2018*. 2018. Christchurch, New Zealand: Trans Tech Publications.
6. Choi, J.-H., *Evaluation of Cyclic Plastic Deformation of Active Links in Eccentrically Braced Frames*. 2013, University of Auckland: Auckland, New Zealand.
7. Nashid, H., et al., *Relationship Between Hardness and Plastically Deformed Structural Steel Elements*. Earthquakes and Structures, 2015. **8**(3): p. 617-635.
8. Hyland, C., *Assessment of Ductile Endurance of Earthquake Resisting Steel Members*, in *Civil and Environmental Engineering*. 2008, The University of Auckland: Auckland, New Zealand.
9. Aurecon, *SCNZ Material Comparison Study Design Features Report*. 2011, Aurecon Ltd: Wellington, New Zealand.
10. NZS3404.1, *NZS 3404: 2009: Part 1: Materials, Fabrication and Erection*. 2009, Standards New Zealand: Wellington, New Zealand.

