DETERMINATION OF THE POST-EARTHQUAKE CAPACITY OF AN ECCENTRICALLY BRACED FRAME SEISMIC RESISTING SYSTEM

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Abstract

Up to the time of the 2010/2011 Christchurch earthquake series, Eccentrically Braced Framed (EBF) seismic resisting systems have been 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 the 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 that earthquake series was the first worldwide to push EBF systems well into the inelastic range, with most systems in Christchurch displaying active link yielding on many to all levels. 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. Research has been undertaken to address these questions and this paper provides an overview of that research, which culminated in a report to the Natural Hazards Research Platform in 2015. It has also raised three further questions that must be answered before the guidance can be considered complete; the current status of research to answer these questions is presented.

Overview and Scope

Background. The procedure presented in this paper (Clifton and Ferguson 2015) has been developed 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).

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

Figure 2. Yielded active links from Christchurch 22/02/2011 Earthquake (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.

The focus 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 this ratio 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.

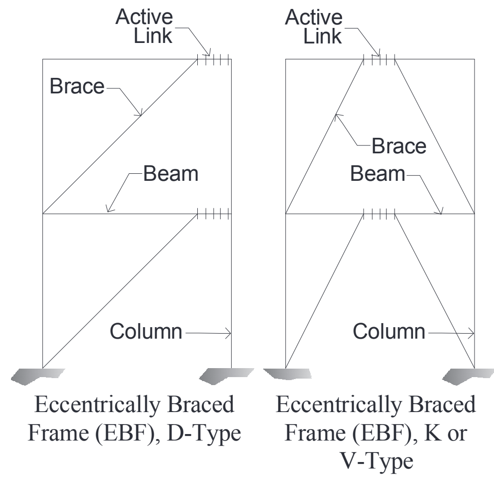


Figure 3. Two common types of EBFs with member terminology (from Nashid 2015).

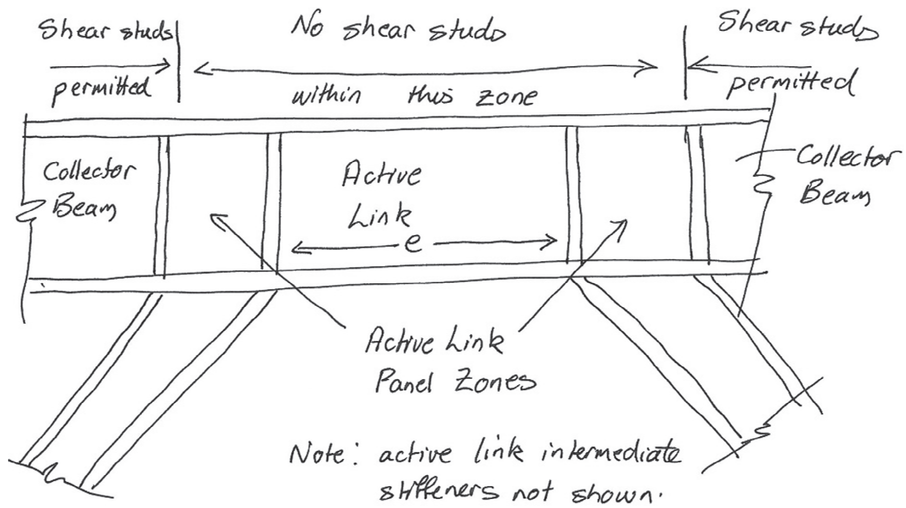


Figure 4. Terminology for EBF active link components.

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. The design and detailing requirements for this are covered in (Clifton and Cowie 2013).

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 procedure presented in (Clifton and Ferguson 2015) has been developed to provide that guidance, to the following post-earthquake scenario:

(1) There has been a severe earthquake, sufficient to push buildings with EBF systems into the inelastic range.

(c) 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.

This paper presents a summary of the current procedure and key items of future research required to complete the procedure.

Determination of Post-Earthquake Capacity of the Yielded EBF System

This is presented in a sequential step by step basis.

Step 1: Initial post-earthquake evaluation of the building and yielded links. This covers the following:

1. Determine whether the building has self centered to within acceptable limits. From field experience in Christchurch this requires a residual drift of not more than 0.15%. If the residual drift is beyond that, up to around 1%, and the building meets all the other criteria below, then straightening of the building is required and is technically feasible. A scheme is being developed at the University of Canterbury to use the aftershocks, in conjunction with a one way acting tension braced system installed into the displaced building, to achieve that.
2. Survey the building services to make sure all are operational or repaired prior to returning the building to service. Field experience from Christchurch showed that in well designed and detailed modern steel buildings, which experienced maximum interstorey deflections below 1.5%, services damage was minimal.
3. Check the structural system for all visible evidence of inelastic demand and document location. Check especially that inelastic demand has occurred only in elements designed and detailed to dependably accommodate it. This includes in connections and column splices. Examples of the type of damage expected are given in (Clifton and Ferguson 2015)
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. Check for instability in the foundation system or supporting ground
5. Undertake a Visual Examination, in accordance with the appropriate structural welding standard (in New Zealand this is AS/NZS 1554.1 (AS/NZS_1554.1 2004)) of all critical locations within the EBF frame for evidence of material cracking or fracture and also check the extent of visible out of plane inelastic buckling. These critical locations are defined in NZS 3404.1: 2009 (NZS3404.1 2009) or an equivalent document for overseas locations. . Document any instances for subsequent replacement of that component; any component showing indications of material cracking, fracture or visible out of plane inelastic buckling should be replaced.
6. Undertake a visual inspection of visible parts of the floor slab to determine the extent of seismic-induced cracking greater than 1.5mm (1/16 inch) and between 0.75mm and 1.5mm (1/32 and 1/16

inch) in width. This should initially be a rapid inspection of any visible concrete surfaces, concentrating on regions around the seismic resisting systems. Cracks greater than 0.75mm in width will typically form in lightly reinforced concrete slabs in service and these need to be distinguished from seismic induced cracks. Guidance on distinguishing between these two forms of cracking is given in (Clifton and Ferguson 2015) as well as the rationale behind these two limits. The larger limit is associated with potential fracture of cold drawn mesh; the smaller limit with loss of aggregate interlock along the crack.

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 when applied to welded three plate section active links.

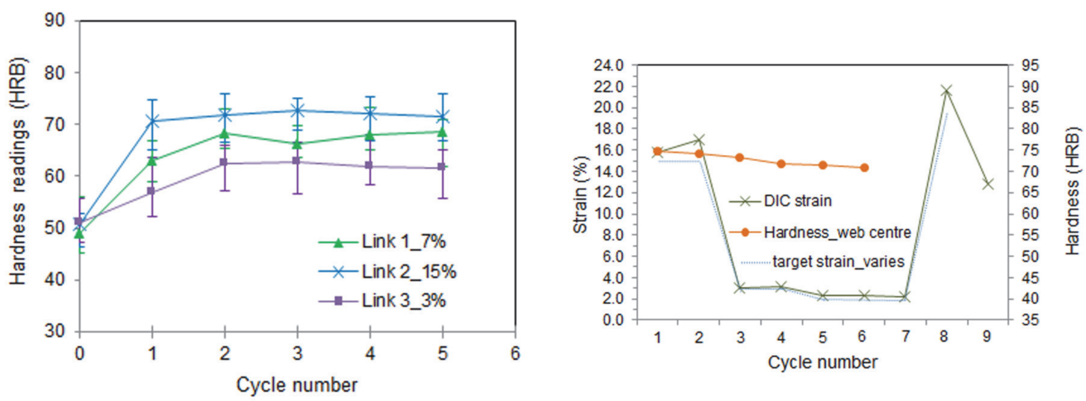
Step 2: Undertake Hardness Testing of the EBF Active Link Yielded Webs and of Control Surfaces. This key step is required in order to determine the peak plastic shear strain that the yielded EBF active link has sustained. The procedures used are those developed by (Nashid 2015), based on the use of the portable, battery powered Leeb Hardness Tester TH170, as shown in Figure 5 . Length limitations in this paper prevents inclusion of the detail; summary of key points is as follows:

1. The testing needs to be done on a sufficient number of storeys to establish a profile of inelastic demand up the EBF frame
2. Hardness testing must be undertaken within specific regions of the web of the yielded active links and of control surfaces. At least 5 sites must be chosen within this region to do the readings from. 7 hardness readings per site must be used.
3. The surface to be tested must be cleaned to bare metal with an average surface roughness Ra not exceeding 0.4μm.
4. Seven hardness readings per site must be used with the lowest and the highest readings discounted. This gives 25 readings per surface, used to determine the mean and standard deviation.
5. Baseline readings from an area of the EBF that has not undergone any inelastic action from fabrication or from the earthquake must be taken. Guidance on suitable locations are given in (Nashid 2015) or (Clifton and Ferguson 2015).
6. The change in hardness for each yielded active link is the difference between the yielded region and the baseline value.

Figure 5. Integrated portable leeb hardness tester TH170.

Step 3: Determine the Change in Mechanical Properties of the Active Links. The yielding will increase the yield stress, fy, and tensile strength, fu and will decrease the ultimate fracture strain. Equations for this are presented in (Clifton and Ferguson 2015).

Step 4: Determine the Peak Plastic Shear Strain Based on the Change in Hardness and the Estimated Loading History. This step uses first the linkage between change in hardness and peak plastic shear strain determined by (Nashid 2015) and also uses the finding that the hardness corresponds to the peak plastic shear strain reached in the cyclic loading regime, irrespective of where in the loading regime this peak cycle occurs. Nashid also found that for cycles involving plastic strains of less than 15%, it takes until the second cycle for the full hardness to be achieved, whereupon the hardness stays constant for subsequent cycles. These two findings are illustrated in Figure 6.



(a) Hardness comparison at the centre of the webs using Leeb TH170 tester for uniform loading

(b) Target strain compared with DIC strain and hardness measured by TH170 tester

Figure 6. Relationship between Hardness and Plastic Strain; Key Findings from (Nashid 2015).

Secondly is the relationship between the number of cycles of inelastic action as a proportion of the maximum cycle, from (Choi 2013) and determined from inelastic time history analysis (ITHA) of a range of EBF systems under 7 scaled earthquake records. This relationship is given in Table 1.

With these two lots of information, the average peak plastic strain can be determined as specified in step 4, section 2 of (Clifton and Ferguson 2015). This procedure allows for the peak plastic strain taking two cycles to plateau for lower plastic strains (see Figure 6(a)).

Table 1 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

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 1.

The cumulative plastic shear strain demand (CPD%) is then determined from $\sum(S, N)$.

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 (CVN) 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).

However, observations of active links in one EBF frame from Christchurch (Clifton, Bruneau et al. 2011, Clifton and Ferguson 2015) showed that to get brittle fracture, both low CVN and a crack initiator in an adverse location are required.

These factors were into account in an expert fracture analysis of that building in setting the criteria for determining the post-earthquake susceptibility of a yielded active link to brittle fracture in a subsequent severe earthquake event.

Step 7: Determine Whether the Active Links can be Left in Place or Require Reinstatement.

1. With regard to plastic shear capacity, if the CPD from step 5 ≤ 50% of the CPD associated with commencement of fracture in the tests by (Nashid 2015), the active link can be left in place.
2. With regard CVN values, if the S0 seismic condition is met for the yielded steel component, the active link can be left in place.
3. With regard to CVN values, if the S0 seismic condition is not met for the yielded steel component, then expert fracture assessment is required at this point in time

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 and capacity design application (Clifton and Cowie 2013), in order to distribute inelastic demand over the height of the frame. When active links have yielded, they will have increased in strength and reduced in ductility. If some yielded links are replaced, it means that their yield and tensile strengths would be weaker than the yielded links remaining in place. This may require use of a higher grade of steel in the replacement links to maintain an acceptable distribution of strength up the height of the frame and Step 8, section 2 of (Clifton and Ferguson 2015) gives limits to achieve this.

Step 9: Determine the %NBS for the EBF Frame and for the Building. Classification of earthquake prone buildings or determination of the post-earthquake capacity of modern buildings uses a %New Building Strength (%NBS) system. Step 9, Section 2 of (Clifton and Ferguson 2015) details how to apply this to the EBF frame and the overall building.

It also presents a detailed design example, in section 4 of (Clifton and Ferguson 2015). Section 3 contains guidelines on how to undertake the replacement of damaged active links, where required.

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 2015) 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? The effect of strain ageing from inelastic shear deformation on the tensile performance is known and applied into the change in mechanical properties, but its influence on the cyclic shear performance is not known. 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 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. 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 of a well designed and detailed active link?

A Master of Engineering project is underway in 2016 to address the first two questions; the third will be addressed in a follow on Master of Engineering project in 2017.

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