

Performance of Eccentrically Braced Framed Buildings In The Christchurch Earthquake Series of 2010/2011



G.C. Clifton, H. Nashid, G Ferguson, M Hodgson and C Seal
The University of Auckland, New Zealand

M Bruneau
University at Buffalo, USA

G.A. MacRae
University of Canterbury, New Zealand

S Gardiner
CPG New Zealand Ltd, New Zealand

SUMMARY:

This paper presents a summary of Eccentrically Braced Frame (EBF) building performance in the Christchurch earthquake series of 2010/2011. This earthquake series was the first event worldwide to push modern EBFs into the inelastic range and impacted on recently built EBFs from 3 to 22 storeys in height. Their response has been generally better than expected. This paper describes the range of buildings impacted, the seismic demand and key overall aspects of the behaviour. It presents the research work currently underway towards the development of a robust method of non-destructive determination of active link plastic demand and the principal case study building to which these provisions are to be applied.

Keywords: Steel, eccentrically braced frame, earthquake, repair

1. INTRODUCTION

The Christchurch earthquake series from September 4 2010 to 23 November 2011 comprised eight damaging earthquakes. Detailed analyses of the comprehensive set of strong motion data recorded shows that the 4 September event was approximately 0.7 times the Ultimate Limit State (ULS) 500 year return period design level specified by the New Zealand seismic loading standard over the period range of 0.5 to 4 seconds, the 22 February event was 1.5 to 2 times the ULS and the largest 13 June earthquake 0.9 times ULS. While the duration of short period strong shaking of each earthquake was short (around 10 to 15 seconds) the cumulative duration of strong shaking was over 60 seconds. The duration of long period strong shaking was longer.

Table 1.1 Multi-Storey Eccentrically Braced Framed Buildings of Significance in Christchurch CBD and Suburbs

No of Storeys	Seismic-Resisting System	Floor System	Year Completed
22	EBFs and MRFs	Composite Deck and Steel Beams	2010
12	EBFs and MRFs	Composite Deck and Steel Beams	2009
3+ ^{Note 2}	EBFs	Precast columns and hollow-core units with topping	2003
5 ^{Note 3}	EBFs	Composite Deck and Steel Beams	2008
5	EBFs	Precast columns and hollow-core units with topping	2010

Notes:

1. The buildings are listed in order of severity of shaking in the 22 February 2011 event, greatest at top
2. Currently 3 storeys; with provision for additional 1 storey
3. Structural system includes provision for two floor levels not currently built

Construction of modern steel buildings in Christchurch started to receive due consideration in the late 1980's. Hence, most of the steel buildings in the Christchurch area are recent and designed to the latest seismic provisions. In particular, a few notable buildings having steel frames opened less than three years prior to the February 2011 earthquake. In addition, a number of principally concrete framed buildings built in the last decade incorporate eccentrically braced framed (EBF) seismic-resisting systems. Table 1.1 lists some of the principal multi-storey eccentrically braced framed buildings in the CBD and some in the suburbs.

2. SEISMIC DEMAND

This section focuses on the February 11 event demand, which was significantly the most severe of the series. Fig. 1.1 shows the CBD ultimate limit state (ULS) design spectrum and 1.8 times this value (which is nominally the 2500 year return period spectrum and is the maximum considered event (MCE) for buildings of normal importance to NZS 1170.5:2004, to which the buildings described in this paper were designed), the larger horizontal components from the four strong motion recorders in the CBD and the average of these components. The average is above the MCE for periods of 0.3 seconds and above, except for the period range of 1.8 to 2.7 seconds, where it still above the ULS level. In 2011 the design seismicity of Christchurch was increased by 36% following a re-evaluation of the seismic model.

Looking at the average spectrum from the 4 recording stations, if this record were to be used in Numerical Integration Time History Analysis to NZS 1170.5, it would have a scale factor not greater than 1 against the 2500 year return period for Class D soils for most building period ranges of interest.

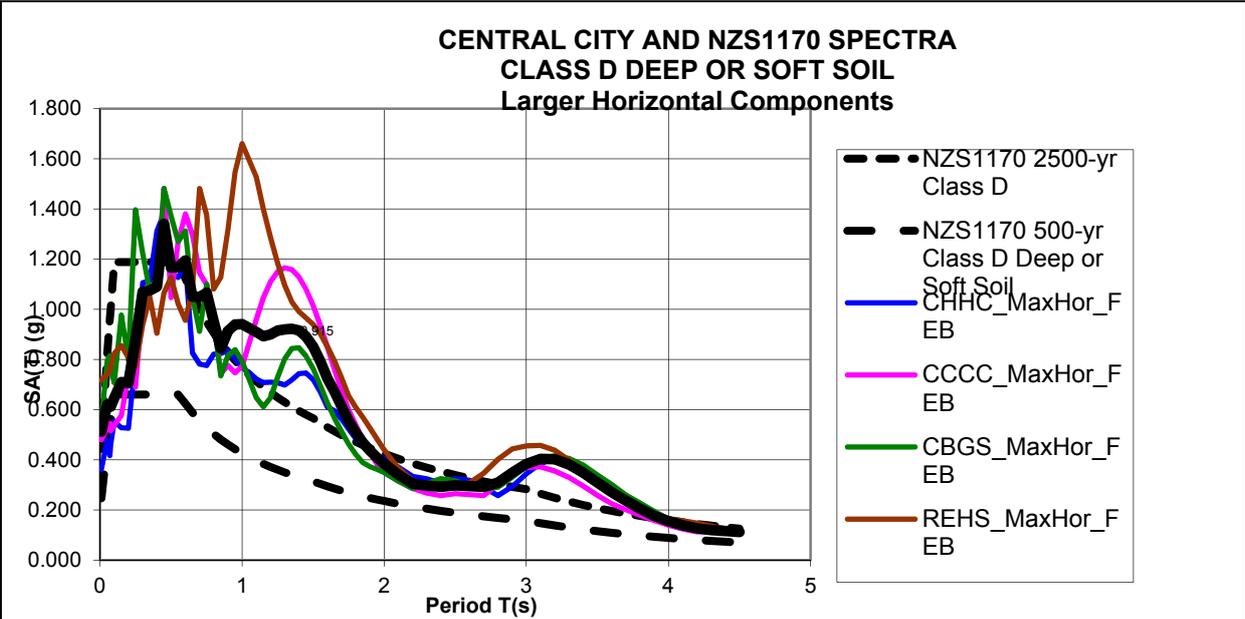


Figure 1.1 NZS 1170.5 Spectra and Largest Horizontal Direction Recorded from the CBD Strong Motion Records

Notes:

1. The long dotted black line is the ULS design spectrum for normal importance buildings for the soft soil type, Class D, generally considered in the CBD
2. The short dotted black line is the Maximum Considered Event design spectrum for normal importance buildings for Class D soil in the CBD
3. The solid thick black line is the average from the 4 recording stations all of which are within 1km of the CBD

3. OVERALL PERFORMANCE

3.1. Pattern of damage and damage threshold

The 12 storey and 22 storey buildings were the most strongly shaken of this building set. Almost all the active links in these buildings were strained into the inelastic range and exhibited paint flaking and Luder lines; an example of one of the most heavily strained active links in the 12 storey building is shown in Fig. 3.1 (c). Also the 3+ storey parking structure showed link fracture due to non-standard connection detailing (Clifton, Bruneau et al. 2011).



Figure 3.1 Club Tower [Photos by M. Bruneau and C Clifton]: (a) Global view; (b) cracking of partition in cantilevering portion of story; (c) paint flaking of EBF link in the ceiling space; (d) general view of office showing minor disruption

For the 12 storey building, estimates of the peak inelastic demand in the active link shown in Fig. 3.1(c) were made by two independent means. Firstly, through visible assessment of the active link yielded web metal and secondly through estimation of the peak interstorey building drift and conversion of this to a peak inelastic demand in the EBF active links. Both methods gave a peak shear strain of approximately 4%. The links were free of visible residual distortions. Assessment of damage accumulation in the steel at a peak shear strain of 4% over an estimated two complete cycles of loading using the damage criterion developed by Seal, (Seal, Hodgson et al. 2009) and change in transition temperature based on the work of Hyland, (Hyland and Ferguson 2006) showed that the yielded active links have used less than 20% of their total cumulative strain capacity and were therefore sufficiently robust to be left in place. Previously reported slab cracking (Bruneau, Anagnostopoulou et al. 2010) could not be detected as the concrete floor slab was covered by floor carpeting, except at the fixed end of one segment of the floor cantilevering on one side of the building (a feature present only over two stories for architectural effect). Crack widths in general monitored after each earthquake in the series did not increase after February 22, 2011, indicating that inelastic

demand in the 12 storey building occurred only in that event.

3.2 Contribution of the floor slabs

When the EBF system deforms inelastically it pushes the floor slab out of plane, as shown in Fig. 3.2. A composite floor system comprising concrete slab on steel deck on composite steel beams has a high out of plane resistance to this movement. This has been quantified through research into this system's performance in fire (Clifton, Gillies et al. 2010). A comparison of measured inelastic response of the 12 storey open plan building (based on sliding marks on the stairs) with the predicted response of the design models showed the building to be between 2 and 2.5 times stiffer than modelled and that the building has a post-earthquake residual drift of only 0.14%. This led to consideration that the out of plane resistance of the floor slab might be a significant source of this stiffness and unexpected ability to self-centre. An undergraduate study in 2011 (Mathieson 2011; Volynkin 2011) provided an initial quantification of these effects based on simple yieldline theory and showed that the floor slab decreased the peak lateral deflection of a hypothetical, 10 storey V-braced EBF designed to current New Zealand design practice (NZS3404 1997/2001/2007; Feeney and Clifton 2001) under a range of 10 representative earthquake records scaled to the ULS level by between 10 and 50% and the peak residual drift to less than 1/3 of that without the slab effect.

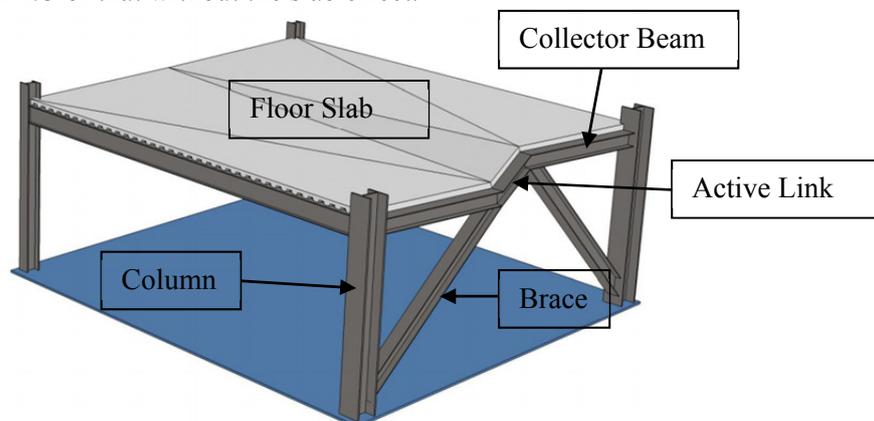


Figure 3.2 Floor slab contribution to EBF strength and stiffness

The significance of the floor slab to the strength and stiffness of EBFs is greater than that found from previous researchers, such as (Ricles and Popov 1987), who concluded the contribution of the slab was an increase in the shear resistance of 8-12% of the shear capacity of the active link alone. However, the contribution of the slab may degrade over successive cycles of inelastic loading and this is a key aspect for further research.

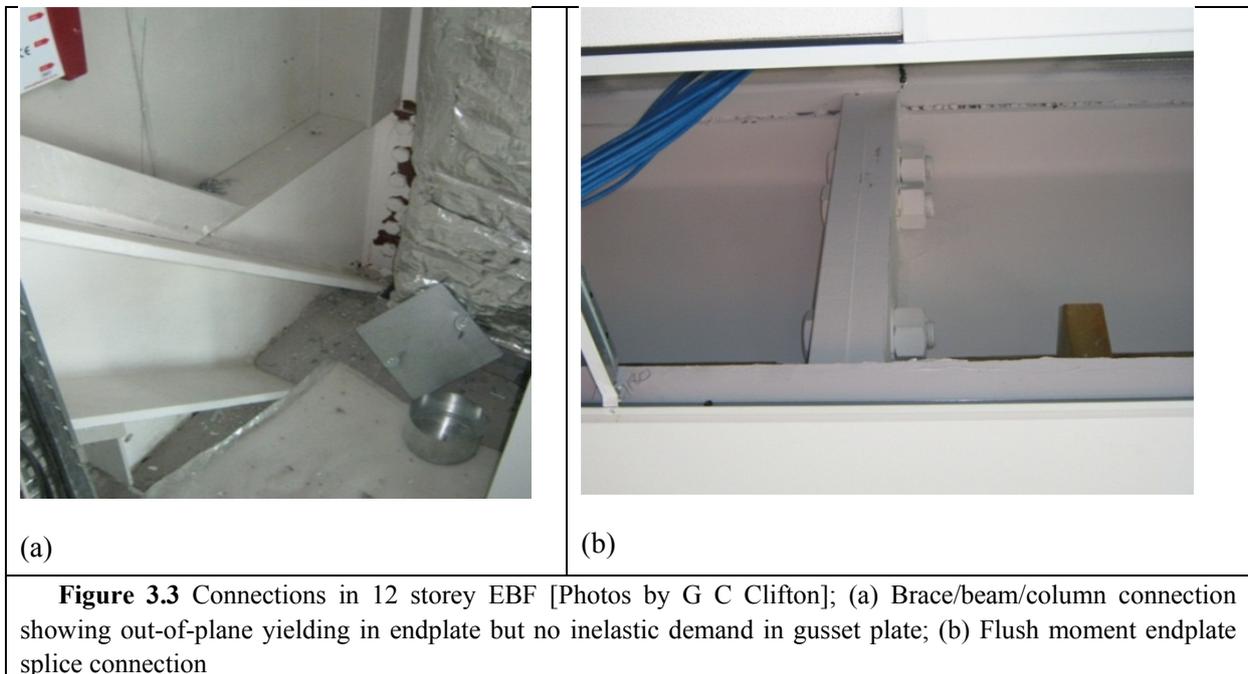
3.3 Performance of connections

Connections in these EBFs performed very well and as expected; similarly with all well designed and detailed connections from multi-storey buildings in the Christchurch CBD. Two examples from the 12 storey building are shown in Fig. 3.3. Fig. 3.3 (a) shows a brace/beam/column connection in which the gusset plate is welded to the beam and bolted to the column with a flexible end-plate connection, which is designed and detailed to be rigid for vertical load transfer and flexible in the horizontal direction, to accommodate change in the angle between beam and column during the earthquake. This flexible end-plate has undergone limited out-of-plane yielding, protecting the gusset plate from inelastic demand as required by NZS 3404 for gusset plates. Fig. 3.3 (b) shows a flush endplate splice in a MRF beam that has performed well. Bolted splices in seismic-resisting systems inspected showed no signs of slip, consistent with expectations in NZS 3404 and HERA Report R4-76.

3.4 Adequacy of the capacity design procedure

The capacity design procedure specified in NZS 3404 Clause 12.11.7 requires inelastic demand to be

concentrated into the active link, with inelastic demand in the braces, collector beams and columns suppressed by the capacity design procedure. That procedure is presented in (Feeney and Clifton 2001). In all EBFs responding inelastically, this requirement was observed to be met, with no inelastic demand in members outside of the active link.



4. DEVELOPMENT OF A FIELD BASED METHOD FOR DETERMINATION OF ACTIVE LINK INELASTIC DEMAND

The inelastic response of the active links in these frames means that their post-earthquake condition needs to be determined in order to decide whether they can be left in place or need repair. There are no criteria established nationally or neither internationally to provide guidance for checking when this threshold has been reached. This section briefly highlights an on-going experimental study to establish a non-destructive field based method to develop these criteria.

4.1 Outline of the proposed method

Hardness testing has been established as a commonly used practical method to predict tensile strength, yield strength, strength limits and other mechanical properties of steel. However, research on establishing a correlation between field determined hardness and plastic deformation in cyclically deformed structural steel elements is limited and of uncertain accuracy. The proposed study involves the following steps:

- Step1. Establish the relationship between monotonic plastic tensile strain and lab based hardness
- Step2. Identify the effects of surface treatment on hardness measurement
- Step3. Compare the accuracy of lab based and field based (Leeb) hardness measurements
- Step4. Establish link between hardness and monotonic plastic strain
- Step5. Incorporate the effects of strain ageing into this consideration
- Step6. Determine the link between Leeb hardness and magnitude of peak cyclic inelastic shear strain in a purpose made EBF active link test rig
- Step7. Determine the effect of the number of cycles of loading and the pattern of cyclic loading on the Leeb hardness

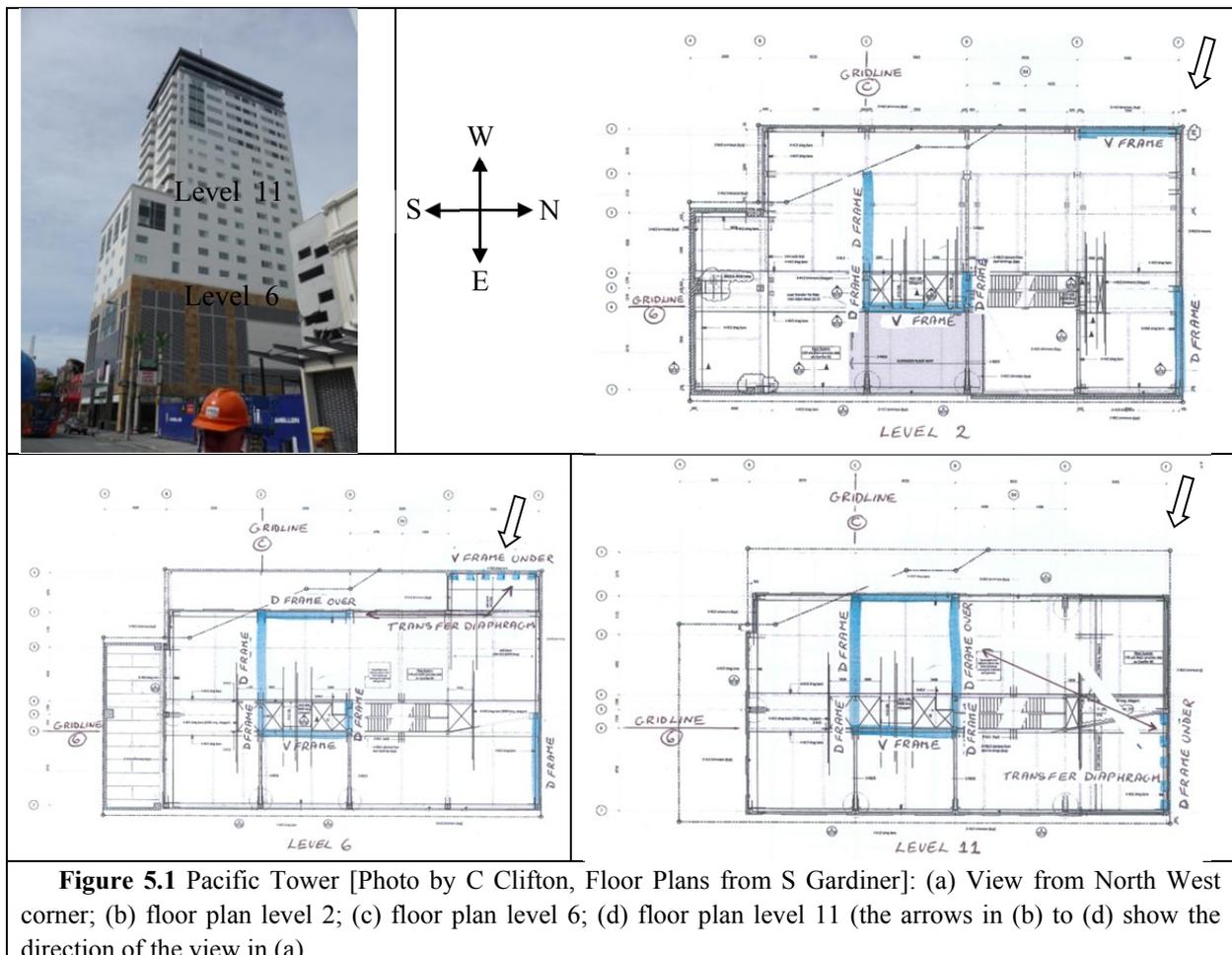
- Step8. Using the damage model from (Seal, Hodgson et al. 2009) determine the damage and hence the remaining life from the samples in step 7
- Step9. Establish an appropriate inelastic loading regime for normal directivity and for near fault earthquakes which is a function of the length of strong ground motion shaking
- Step10. Combine the results of steps 6 to 9 into a method for firstly determining the expected loading regime as a function of the strong ground motion shaking, with the peak strain to be determined from the in-situ Leeb hardness testing of the damaged active links and secondly determining the extent of damage and hence decision on whether the active link can be left in place or needs replacement

The second author is undertaking steps 1 to 8, which will allow the link between magnitude of cyclic loading and extent of damage to steel for an active link to be determined. In addition, the second author is leading the testing of yielded components from the case study building described in the next section, from which suitably robust guidance on the determination of whether to leave or to replace yielded components in that building can be made. It is expected that the last two steps will require another research project of PhD length and scope.

5. CASE STUDY: 22 STOREY EBF BUILDING PERFORMANCE

5.1 Building characteristics and extent of earthquake demand

Fig. 5.1 shows a general view of the building and three floor plans, at levels 2, 6 and 11.



The building layout involves step backs to the floor plan at levels 6 and 11, both of which require changes to the position of the seismic resisting system in plan and hence for the floor slab to act as a horizontal transfer diaphragm at levels 6 and 11 and also, to a lesser extent, at level 18. Furthermore between level 2 (the first suspended level) and the underside of level 6 there are 4 storey voids housing a car stacking system which means no floor slab over 4 levels in the North-West corner

The first three translational periods for the building in each direction obtained from the design model and the participating masses are given in Table 5.1. The torsional modes were at 2.3, 0.9 and 0.5 seconds for the fundamental, second and third torsional modes respectively.

Table 5.1: Pacific Tower Translational Periods in Each Principal Direction (from building model)

Translational Period Mode	East-West Direction (seconds)	North-South Direction (seconds)
First	3.3 (72% mass participating)	4.0 (65% mass)
Second	1.1 (13% mass)	1.3 (13% mass)
Third	0.5 (4% mass)	0.7 (2.5% mass)

When these modes are compared with the earthquake spectra (see Fig. 1.1) they show significant excitation above design levels in the first and second modes. The pattern of inelastic response is consistent with second mode participation but lack of visible damage above level 16 is not consistent with an expected strong first mode response.

The seismic resisting system comprises principally a mix of D braced and V braced EBFs, one of which terminates at level 6 and the other at level 11, with the frames around the central core mostly continuous up the height of the building. The design procedures for EBFs prior to the Christchurch earthquake series allowed for a design ductility factor (μ) of up to 4.0 for the EBF system, although building lateral drift limits or limits on the active link plastic rotation capacity often prevent this extent of ductility from being used. For Pacific Tower, due to drift limitations, a component of standardisation of active link section sizes across a number of levels and the D-braced active links designed not to yield until both the gravity shear and design earthquake shear components had been overcome, the actual design ductility factor was approximately 1.5 for each principal direction of the structure. (This corresponds approximately to a North American value of $R_w = 2.1$).

5.2 Structural damage and planned repair work

The devastation brought to the Christchurch CBD by the earthquake series has meant that public access to this building will not be attained until earliest mid-2012 and that resources for investigation of the few lesser damaged buildings such as Pacific Tower has been limited. For these reasons, the detailed structural review of Pacific Tower is 75% complete and replacement of active links in one frame has not commenced. The remainder of section 5 gives a brief overview of the earthquake damage and planned repair work and some preliminary results from field based hardness testing of the active link components.

Structural damage requiring repair before building reopening is confined to:

1. Cracking of the concrete slab, principally in the corners of the floor slabs especially at levels 6, 11 to 13. This appears to be due to torsional effects, however there are no indications of diaphragm failure, detachment of slab from beams or slab reinforcement fracture. This is especially encouraging given the three levels of transfer diaphragm required in the building and the widespread failure of diaphragms, especially transfer diaphragms, in other multi-storey buildings in Christchurch (Kam, Pampanin et al. 2011)
2. Of the total lengths of crack width measured, 1% are over 1mm in width, 10% between 0.5mm and 1mm in width, 40 % between 0.5 and 0.3mm in width and the rest under 0.3mm in width. No reinforcement crossing the cracks is reported to be fractured. . (The slab was reinforced with a grid of high strength deformed bars as well as supplementary drag bars all earthquake ductility class to the reinforcing bar standard; cold-drawn mesh was not used, and likely contributed to the

satisfactory diaphragm performance).

- The EBF frame on the west side near the north west corner that runs from level 2 to level 6 and comprises a V braced configuration with links at 400mm clear length is having all active links replaced, as the top link has fractured, as shown in Fig. 5.2(a). The clear length of all other active links in the building are around 800mm and so inelastic demand in these links is approximately double that of the others.

Crack widths of up to 0.3mm comprise strong crack control to NZS 3404, up to 0.5mm width moderate crack control and over 0.5mm in width minor crack control. Only the latter is considered unacceptable in internal slabs. The repairs to the slab cracking will comprise epoxy grouting of all cracks greater than 0.5mm in width.



Figure 5.2: Fractured EBF in Pacific Tower, Left and the Proposed Replacement Detail, Right. Courtesy of Sean Gardiner, CPG NZ Ltd.

Replacement of the active links will be achieved by cutting out the active link and adjacent braces and collector beam (approximately 300mm outside the active link and panel zones) and replacing these with the shop fabricated detail shown in Fig. 5.2(b) which is site welded to the existing stub collector beams and braces. Replacement will take place in early May 2012.

5.3 Extent of inelastic demand as indicated by hardness testing

Field based hardness testing using a Leeb Hardness tester (ASTM 2008) has been undertaken on the active links and adjacent braces, collector beams and collector beam panel zones by a third party testing organisation. This data is presented in Fig. 5.3 for two of the active links that run the full 22 storeys of the building, the locations of which are shown in Fig. 5.1.

The measured hardness of the steel is approximately proportional to the strength of the steel, so in measuring the hardness values in the panel zone webs, which underwent visible yielding on some levels and comparing those values with the hardness values from the adjacent panel zones, which did not undergo visible yielding, changes in the strength and hence post-earthquake properties resulting from plastic deformation of the active links can be indicated. As noted in section 4, there is much work to be done to robustly take account of the influence of sampling conditions, surface roughness, influence of multiple cycles, so these results are indicative only. The results are that:

- For the grade of steel, all results are consistently lower than the expected HLD value of 420 (ref) by around HLD 40. At the time of writing this paper the reason for the low values is not known; it will be reported on at the conference. However, clear trends are apparent from the data. The baseline for the hardness from the panel zones can be taken as 380 for both frames, as the differences are very minor.
- From work undertaken by the second author in his PhD research, HLDs of 420 and 440 correspond to monotonic strains of around 7% and 20% respectively.

3. For these two frames, any average reading over HLD 420 implies that the active link may need replacement once a more robust evaluation is performed.
4. For the two frames any average readings of around HLD 440 may mean immediate replacement is recommended. One link meets this criterion, being that at level 2 on Grid 6 and it has been recommended that this link is replaced before the building is reopened.

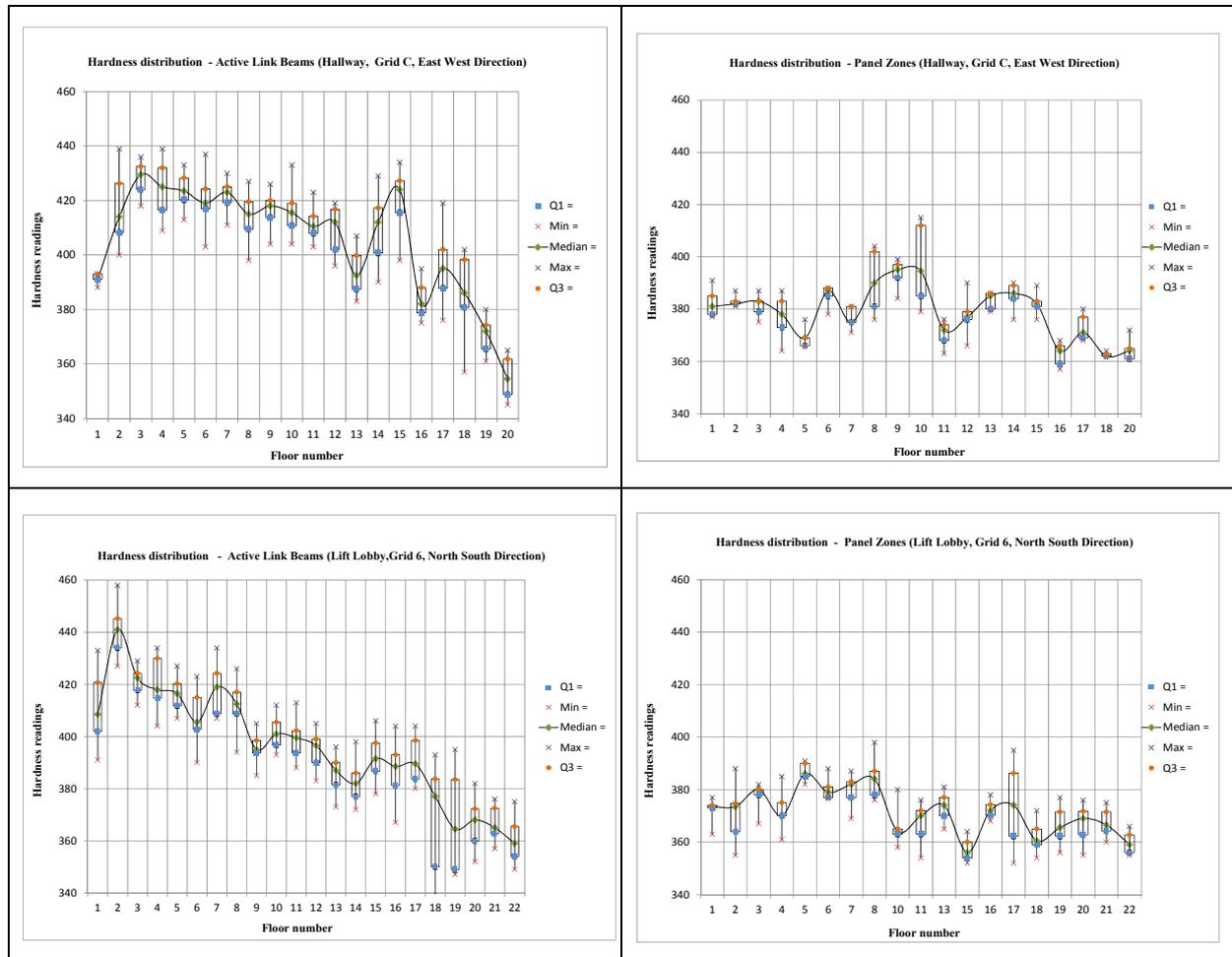


Figure 5.3: Leeb hardness values for selected components for two of the full height EBF systems in Pacific Tower. Clockwise from top left: (a) Hardness values for the active links in the D braced frame in the stairwell on gridline C, oriented East-West; (b) Hardness values for the panel zone in the collector beam adjacent to (a); (c) Hardness values for the active links in the V braced frame on gridline 6, oriented North-South; (d) Hardness values for the panel zone in the collector beam adjacent to (c).

The hardness evaluations have been undertaken on all active links shown in Fig. 5.1; the above are two typical examples. The results show that maximum inelastic demands have occurred on levels 2 and 6 and that, above level 16, inelastic demand has been negligible.

Highest demand has been from the very short active links in the North-West V braced frame, which is the frame shown in the top right corner of the floor plans at levels 2 and 6 in Fig. 5.1. Average HLD readings of 480 have been recorded in the fractured link. These links are being replaced and comprehensive testing on the extracted samples is scheduled for May 2012.

6. CONCLUSIONS

The February 22, 2011 earthquake was of short duration but high severity, corresponding to near maximum considered event level of intensity for the Christchurch CBD. The earthquake impacted on a

range of modern, multi-storey steel framed buildings with eccentrically braced framed seismic-resisting systems ranging from 3 to 22 storeys in height. It was the first earthquake worldwide to push this type of structure into the inelastic range, illustrating the behaviour of complete structures incorporating this common form of seismic-resisting system.

This paper has described key aspects of the buildings' behaviour and a short case study on the repairs being undertaken to the tallest and most heavily impacted building. Due to delays in post-earthquake assessment and repair the work is still underway at the time of completing this paper, therefore it comprises a work in progress rather than reporting complete details. It is expected that the repairs will be completed by mid-2012 enabling them to be reported in the oral presentation to the conference.

ACKNOWLEDGEMENTS

This work was funded in part by the Ministry of Science and Innovation through the Engineering theme of the Natural Hazards Research Platform of New Zealand, the University of Auckland, and the University of Canterbury. Participation of Michel Bruneau to the earthquake reconnaissance study in 2011 was funded by MCEER (University at Buffalo).

The authors are also grateful to the engineers/owners of specific buildings to allow identification of these buildings in this paper. This is especially appreciated in view of the extended duration of this earthquake series which has caused uncertainty and loss of confidence by the public in multi-storey building performance in severe earthquakes.

However, any opinions, findings, conclusions and recommendations presented in this paper are those of the writers and do not necessarily reflect the views of the sponsors.

REFERENCES

- ASTM (2008). ASTM:A956-06 Standard test methods for Leeb testing of steel products. Pennsylvania, USA, ASTM International.
- Bruneau, M., M. Anagnostopoulou, et al. (2010). Preliminary report on steel building damage from the darfield earthquake of september 4, 2010. *Bulletin of the New Zealand Society for Earthquake Engineering* **43(4)**: 351-359.
- Clifton, G. C., M. Bruneau, et al. (2011). Steel Structures Damage 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., A. G. Gillies, et al. (2010). The slab panel method: Design of composite floor systems for dependable inelastic response to severe fires. *Structures in Fire - Proceedings of the Sixth International Conference, SiF'10*: 492-499.
- Feeney, M. J. and G. C. Clifton (2001). Seismic Design Procedures for Steel Structures including Tips on Seismic Design of Steel Structures. Manukau City, New Zealand, New Zealand Heavy Engineering Research Association.
- Hyland, C. and G. Ferguson (2006). A fracture mechanics based approach to the assessment of steel seismic-resisting systems. *Key Engineering Materials* **312**: 89-94.
- Kam, W. Y., S. Pampanin, et al. (2011). Seismic Performance of Reinforced Concrete Buildings in the 22 February Christchurch (Lyttleton) Earthquake. *Bulletin of the New Zealand National Society for Earthquake Engineering* **44(4)**: 239-278.
- Mathieson, C. D. (2011). Effects of Floor Slab Resistance on the Inelastic Behaviour of V-Braced Eccentrically Braced Frames. *UofA CEE Part 4 Project Reports 2011*, Auckland, New Zealand.
- NZS3404 (1997/2001/2007). Steel structures standard, incorporating Amendments 1 and 2, Wellington New Zealand : Standards New Zealand.
- Riccles, J. M. and E. P. Popov (1987). Experiments on eccentrically braced frames with composite floors. Berkeley, California, Earthquake Engineering Research Center, University of California: 324 pp.
- Seal, C. K., M. A. Hodgson, et al. (2009). A novel method for predicting damage accumulation in seismically deformed steel. *Journal of Constructional Steel Research* **65(12)**: 2157-2166.
- Volynkin, D. (2011). Effect of the Floor Slab Resistance on the Inelastic Behaviour of V Braced Eccentrically Braced Frames. *UofA CEE Part 4 Project Reports 2011*, Auckland, New Zealand.