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Multistorey steel framed building damage from the Christchurch earthquake series of 2010/2011

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ABSTRACT: This paper presents preliminary field observations on the performance of selected steel structures in Christchurch during the earthquake series of 2010 to 2011. The Christchurch earthquake series comprised 6 damaging earthquakes, on 4 September and 26 December 2010, February 22, June 6 and two on June 13, 2011. Most notable of these was the 4 September event, at M7.1 and MM7 (MM as observed in the Christchurch CBD) and most intense was the 22 February event at M6.3 and MM9-10 within the CBD. The earthquakes impacted on a range of steel framed buildings, from single storey to the tallest building in Christchurch at 22 storeys. Many of the multi-storey buildings used eccentrically braced framed seismic-resisting systems (EBFs) and this earthquake series was the first time these systems have been pushed into the inelastic range. This paper gives an overview of the performance of selected buildings, with an emphasis on EBFs. Their performance in particular was very good and possible reasons for this are presented.

1 INTRODUCTION

The Christchurch earthquake series from September 4 2010 to 13 June 2011 comprised six 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) 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 strong shaking of each earthquake was short (around 10 to 15 seconds) the cumulative duration of strong shaking was over 60 seconds.

Caution was expressed following the September and February earthquakes that the short duration of strong shaking in each event meant that duration related damage might have been suppressed compared with what one could have seen from a single earthquake of longer duration. This caution is less warranted when considering the duration of the total earthquake series. Furthermore, there were reports of duration damage such as low cycle fatigue fracture of reinforcing bar and attachment details to cladding panels following the June 13 events. Metallurgically, the extended period of this earthquake series is likely to have been more severe than a single event of comparable duration, due to strain ageing of the steel from the most intense 22 February earthquake raising the yield strength and decreasing the ductility of yielded components before the second strongest event of 13 June. For these reasons, the performance of steel structures is instructive, providing a unique opportunity to gage the adequacy of the current New Zealand seismic design provisions for steel structures, as presented in NZS 3404 and HERA Report R4-76 (Feeney and Clifton, 1994/2001). This is the objective of the paper, with a focus on EBFs.

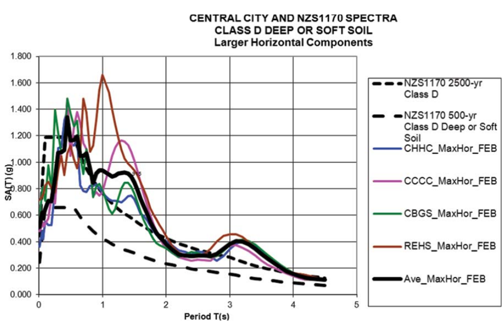
2 SEISMIC DEMAND

This section focuses on the February 11 event demand, which was significantly the most severe of the series. Figure 1 shows the CBD ultimate limit state (ULS) design spectrum and the 2500 year return period spectrum (which is the maximum considered event (MCE) for buildings of normal importance to NZS 1170.5, 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.

Looking at the average spectrum from the 4 recording stations in Figure 1, if this record were to be used in Numerical Integration Time History Anlaysis 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.

3 STEEL STRUCTURES IN THE CHRISTCHURCH AREA

The number of steel structures is relatively low in the Christchurch area. This is attributed to both the historical availability of cheap concrete aggregates deposited in riverbeds flooded by the seasonal melting in the mountain range and glaciers west of Christchurch (leaving the riverbed mostly dry



Notes:
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.

The short dotted black line is the Maximum Considered Event design spectrum for normal importance buildings for Class D soil in the CBD.

The solid thick black line is the average from the 4 recording stations all of which are within 1 km of the CBD and in similar ground conditions.

Figure 1. NZS 1170.5 spectra and largest horizontal direction recorded from the CBD Strong motion records.

Table 1. Multi-storey steel rramed buildings of significance in christchurch CBD and suburbs.

Storeys	S-R S*	Floor system	Completed
22	EBF & MRF	Composite Deck and Steel Beams	2010
12	EBF & MRF	Composite Deck and Steel Beams	2009
7	Shear Wall & CBF	Composite Deck and Steel Beams	1985
7	Perimeter MRF	Composite Deck and Steel Beams	1989
3	MRF	Composite Deck and Steel Beams	2010
5	EBF	Composite Deck and Steel Beams	2008
3+**	EBF	PC columns and hollowcore units with topping	2003
5	EBF	PC columns and hollowcore units with topping	2010

* Seismic-Resisting System.
** 1. Currently 3 storeys; with provision for additional 1 storey.

and accessible the rest of the year), and labour disputes in the 1970s that crippled the steel industry in New Zealand until the 1990s. 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. The market share for steel framed structures nationally has increased considerably in the last few years to be close to that of reinforced/precast concrete structures. In particular, a few notable buildings having steel frames opened less than three years prior to the February 2011 earthquake. Table 1 provides a listing of the multi-storey steel framed buildings in the CBD and some in the suburbs. There are a similar number of lower rise modern steel framed buildings in the suburbs that are not listed in this table. In addition, a number of principally concrete framed buildings built in the last decade include part gravity steel frames and/or part seismic-resisting systems. Most of these later structures are not listed in this table.

4 SEISMIC PERFORMANCE OF HIGH-RISE ECCENTRICALLY BRACED FRAMED BUILDINGS

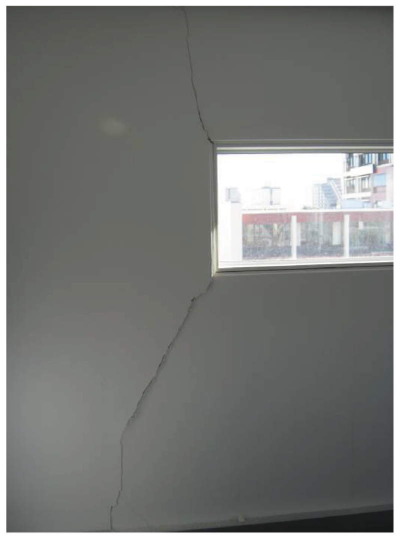
4.1 *General*

Two recently designed and built multistory buildings in the CBD had eccentrically braced frames as part of their lateral load resisting system. The 22-storey Pacific Residential Tower in Christchurch's CBD, completed in 2010, and the 12 storey Club Tower building, completed in 2009. Both were green-tagged following the earthquake, indicating that they were safe to occupy but could require some minor repairs. This section focuses on the behaviour of these two structures.

4.2 *12 Storey building frame*

The Club Tower Building (Figure 2a), also known as the HSBC building, has eccentrically braced frames located on three sides of an elevator core eccentrically located closer to the west side of the building, and a ductile moment resisting frame (DMRF) along the east façade. The steel frame is supported on a concrete pedestal from the basement to the 1st story, and foundations consist of a 1.6 m thick raft slab. Initially, only the EBFs on the east side of that core could be visually inspected without removal of the architectural finishes (Figure 2d), however, more detailed investigation was made of the South side active links through removal of ceiling tiles to ascertain the most significantly yielded braces. Figure 2c shows a link at level 3 on the South side which has the greatest observed inelastic demand. Estimates of the peak inelastic demand in that brace were made by two independent means. First was

(a) Global view



(b) cracking of partition in cantilevering portion of story

(c) paint flaking of EBF link in the ceiling space

(d) global view of EBF braces obstructed by various utility runs

Figure 2. Club Tower [Photos by M. Bruneau and C. Clifton].

through assessment of the visible state of the metal in the yielded web of the active link and secondly through estimate of the peak interstorey drift. Both methods gave a peak shear strain of between 3% and 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, (2009) and change in transition temperature based on the work of Hyland, (2006) showed that the yielded active links have sufficiently robust metallurgical properties to be left in place. Previously reported slab cracking (Bruneau et al., 2010) could not be detected as the concrete floor slab was covered by floor carpeting, except at one location at the fixed end of a segment of the floor cantilevering on one side of the building (a feature present only over two stories for architectural effect). Crack widths after February 22, 2011 appeared similar to what had been observed after September 4, 2010, being localised only. Substantial shear cracking of the gypsum plaster board (sheetrock) finish on the exterior wall of that cantilevering part of the floor was also observed (Figure 2b); only hairline cracking of gypsum plaster board finishes was observed elsewhere throughout the building, supporting post-earthquake survey measurements showing that the building has a post-earthquake residual drift of only 0.1%. One non-structural masonry block wall installed for sound proofing purposes adjacent to mechanical units on the pedestal roof suffered minor shear cracking, where it had been placed hard against a cantilevering floor beam.

Given the magnitude of the earthquake excitations, with demands above the ULS design level, substantial yielding of the EBF links would have been expected. EBFs designed in compliance with the NZS 3404 (SNZ, 1997/2001/2007) provisions are typically sized considering a ductility factor (μ, equivalent to R in US practice) of up to 4, corresponding to a level of link deformations that would correspond to significant shear distortions of the links. Yet, yielding was below that determined necessary in subsequent detailed assessment to require structural replacement of the EBF active links. Beyond the usual factors contributing to overstrength in steel frames (e.g., expected yield strength exceeding nominal values, modelling assumptions, etc.), a number of additional factors can explain behaviour in this particular case, including strength of the composite floor slab action (neglected in design), moblization of the solid non-structural wall concrete cladding adjacent to the staircase, elastic stiffness ofthe gravity frame especially the columns and the relatively short duration of earthquake excitation.

The ductile MRF along the east wall did not show any evidence of yielding. Its design had been governed by the need to limit drift, particularly under torsional response due to the eccentricity of the core, and its corresponding effective design ductility factor (μ) was low at 1.25. Because the building strength and stiffness was over 2 times that designed for (based on observed lateral drift versus design lateral drift), this frame did not visibly yield.

Overall, the building was designed for a slightly lower level of structural ductility demand than is typical for an EBF, due to its height and plan dimensions, and performed well during the earthquake. No structural repairs were required; non-structural remedial work consisted of minor dry wall crack repair and realignment of the lift guide rails. The building was open and fully reoccupied in July 2011, becoming the first normal importance high rise building in Christchurch to be returned to use following the earthquake series.

4.3 *Connections*

Connections in Club Tower, as in other modern steel framed buildings, performed very well and as expected. Two examples from Club Tower are shown. Figure 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. Figure 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.

Figure 8a shows a welded beam to column connection in one of the first steel framed buildings of the modern era. It is a 7 storey perimeter moment resisting frame (PMRSF) building, located in a region of unstable ground. The performance is covered in section 6.

4.4 *22 storey tower*

As a new landmark and one of the tallest buildings on the Christchurch skyline, the 22-storey Pacific Tower, also known as C1 Tower, consists of perimeter EBFs up to the sixth floor on the western side and up to the eleventh floor on the north side of the building, shifting to join the other EBFs

(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 3. Connections in Club Tower Building, Christchurch [Photos by G.C. Clifton].

around the elevator core above those levels., with transfer slabs designed to horizontally distribute the seismic loads at those transition points. Several sections of the EBFs at levels below the level 6 transfer slab were visible, apart from at the top of the perimeter system, as these levels housed a mechanical multilevel parking elevator system. The separate bracing system of that mechanical device consisted of flat plates connected with turnbuckles and hooks. Some of those details failed as the bars un-hooked when returning into compression after tension yielding excursions that elongated the braces. The EBFs at intermediate locations (on the NW frame) were not integral with the floor slab and so did not benefit from the strength increase provided by that integral action throughout the rest of the building. A range of views for this structure are given in Figure 4.

Paint flaking and residual link shear deformations were observed in the EBF links at those levels. Design of the EBFs in that building was governed by the need to limit drift, with a corresponding resulting design ductility factor (μ) of 1.5 (even though up to 4.0 is permitted for EBF systems, as mentioned earlier). This is typical of EBFs in tall buildings in New Zealand's moderate to low seismic zones; Christchurch is moderate in accordance with the earthquake loadings standard, NZS 1170.5. When the initial internal inspections were undertaken, there was an absence of significant damage to architectural and other non-structural finishings except at level 6 where a few of the hotel room doors along the corridor could not be closed, suggesting greater residual deformations at that level. This level was the first in which a detailed evaluation was undertaken. One fractured EBF active link was discovered (Figure 4d) in the top level (underside of Level 6) of the EBF system at the North-Western corner of the building in August 2011, 6 months after the February event. When the fracture was caused is not known, however slightly concentrated non-structural damage at that level following the February event may indicate that it happened then. The frame sits behind the louvre system nearest the camera in Figure 4a. This link had undergone at least one full cycle of web panel yielding, as evidenced by the diagonal pattern of Luders' lines in the web panel, prior to a fracture propagating from one top corner across the active link region and resulting in significant residual deformation. Temporary strap cross-bracing was welded to the active link frame to provide lateral load resistance while a repair strategy was implemented, which comprised cutting out the damaged link, welding on an endplate system to each collector beam/brace face and replacing with a replaceable site bolted endplate active link. The replacement is scheduled for early October 2011 and is the only repair to the structural frame required for this building. A detailed evaluation was undertaken of all active links in the adjacent storeys and throughout the building, with no further links requiring replacement being found and discussion ongoing about replacing the link shown in Figure 4b . This inspection required removal of architectural finishes.

This type of failure has not been reported in either EBFs tested in the laboratory or from damage reports from other earthquakes; the reasons for this link fracture are not currently clear and it is to be the subject of a detailed metallurgical and structural evaluation once removed.

As with Club Tower, some repair of dry-wall cracking and realignment of lift shaft guide rails is the only other work required and the intention is

(a) Global view

(b) Flaked paint on EBF active link with residual deformation

(c) Multi-story mechanical garage stacker failed braces

(d) Fractured EBF active link in top level of EBF system in front face of atrium

Figure 4. Pacific Tower [Photos by M. Bruneau and C. Clifton].

to have this completed in time for the building to be fully opened when public access is restored into this area. It is also worth noting that this may be the only one of the six high-rise buildings in Christchurch that will be returned to service and requires much less repair than the others.

It is noted that having the lateral load resisting system hidden by architectural elements is a hindrance to post-earthquake inspection, making it often only possible to infer the presence of structural damage from the cracking of non-structural finishes and other evidence of large inter-story drifts until the linings are removed. While this may work well in many cases, experience following the Northridge earthquake suggests that major fractures of structural elements may remain hidden for years if only non-structural damage is relied upon as an indicator of possible problems with the lateral load resisting structure. Future building code committees may consider the merit of requiring that buildings be designed to provide easy inspection of key structural elements and critical non-structural elements following severe earthquakes.

5 SEISMIC PERFORMANCE OF LOW TO MEDIUM RISE ECCENTRICALLY BRACED FRAMED CAR PARKING BUILDINGS

The two low-rise parking garages having eccentrically braced frames described in Bruneau et al., (2010) were inspected following the February 22 event.

The EBFs in a three level parking garage of a shopping mall west of the CBD did not exhibit inelastic deformations (Figure 5a). The basically elastic response of the EBFs is not surprising in this case, given that these frames had been designed to accommodate three additional parking levels to be added at a later time and the intensity of shaking

(a) View from the East

(b) Fracture of Precast Spandrel Beams on

Figure 5. Shopping mall on dilworth St and clarence St, christchurch [Photos by G. MacRae].

was lower than in the CBD. Live load present at the time of the earthquake may also have been less than considered in design, although it was higher than in the September earthquake when the shopping mall was not occupied. Movement of precast units previously reported was observed to have intensified. This resulted in fracture of the spandrel panels beside the epoxy mastic connection between panels, presumably indicating that the epoxy mastic was stronger than the precast panels in tension (Figure 5b). These fractures occurred in all panels over the height of the structure. These spandrel panels were also designed to carry gravity loads in the parking structure so their fracture compromised the serviceability of the building. No further damage has been reported from the three June earthquakes.

An EBF braced hospital parking garage closer to the epicentre (Bruneau et al., 2010) also performed well, with limited damage to the links and gravity load carrying system and no visible residual drift, although some link fractures were observed in two braced bays (Figures 6b and 6d). At least six EBF frames were used at each level in each of the buildings' principal directions, and that this significant redundancy contributed to maintain satisfactory seismic performance of the building in spite of those significant failures. Residual drifts of the parking structure or damage to the gravity load carrying system were not visually noticeable, which suggests that these fractures would have not have been discovered if hidden by non-structural finishes.

This parking structure was also designed to accommodate two additional floors. Yet, some of the links at the first story showed paint flaking as evidence of inelastic deformations. Evidence of minor soil liquefaction was also observed over parts of the slab on grade. Depending on the foundation type, liquefied soils can act as a sort of base isolation or as a method to lengthen the period. This generally results in a lower yield acceleration and lower structural demands. As such, it is possible that this parking garage was not subjected to ground motions as severe as those shown in Figure 1. However, because these EBFs were not drift dominated, they were designed for the maximum $\mu = 4$ ductility demand. Also these active links were added as finished components into the largely precast concrete structure and so were not tied into the floor slab with shear studs as they were for the taller buildings previously discussed. This meant that they did not have the same strength enhancement due to resistance to out of plane deformation of the floor slab as the taller buildings had.

The fractures, as shown in close-up in Figure 6b and c, were of particular concern as these were the first fractures recorded in EBFs worldwide (the Pacific Tower fracture as mentioned above was discovered later). Further puzzlement was added by the fact that the fracture plane, shown in Figure 6c, indicated a ductile overload failure rather than a brittle fracture. However, the likely explanation lies in the offset of the brace flange from the stiffener. This offset is shown in Figure 6(c) and means that, when the brace was loaded in tension, the axial tension force in the brace fed into the active link/collector beam panel zone through a flexible beam flange rather than directly into the stiffener. This meant that the junction between the unstiffened beam flange and the beam web was severely overloaded, leading to fracture between these two surfaces and this fracture spreading across the beam flange and through the web. Evidence in support of this is from the following:

- where the flanges of the brace line up with the stiffeners, as in the right hand side of the active link shown in Figure 6(b) or the panel zone shown in Figure 6(d), there was no damage to this panel zone region.

(a) Evidence of EBF link yielding

(b) One of the two active links with fractures

(c) Close-up view of most severely fractured active link

(d) Active link at top storey supporting partially completed ramp to future new level

Figure 6. Parking garage on St Asaph St and Antigua St, Christchurch [Photos by M. Bruneau].

- the damage to the panel zone region is directly proportional to the eccentricity between the brace flange and the active link end stiffener.

This shows that load path through the as-constructed detail is particularly important when inelastic demand is required from the system.

Also, the ramp at the top level, built in anticipation of future additional stories, suffered damage as the only EBF on the upper segment of the ramp was located at the east end of that ramp, inducing torsional response and shear failure of the columns in moment-frame action at the west end of the ramp—these shear failures had not been repaired by the time of the aftershock and exhibited more significant damage (temporary lateral bracing were installed to prevent further sway motions). Steel angles, originally added at the expansion join to meet the design requirement for support length of hollow-core slab prevented unseating of the ramp. The EBF link at the ramp level itself exhibited substantial inelastic distortions.

The lateral bracing of the active links in the building shown in Figure 6 was only in the form of a confining angle each side of the top flange, as shown in Figure 6(b) and 6(d). No lateral movement or twisting of the ends of the active links was observed,

showing that the lateral restraint provisions had been adequate in practice, despite only being applied to the top flange and for EBFs not integral with the slab above also being non-compliant with NZS 3404.

As of mid-2011, the fractured active links have been cut out and are being replaced to bring the building back into service in advance of when public access is restored to this area. This includes welding on end plates to the damaged active links and fitting a specific measured active link to the gap.

6 MULTI-STOREY MRF BUILDINGS

A new parking garage (construction completed after the September 2010 earthquake) appeared to have performed very well, with no visible sign of inelastic deformation at the beam-to-column connections (Figure 7) or in any other part of the structure. However, this assessment could only be done from the ground below as a collapsed concrete car parking building next door precluded access into the building.

A low rise MRF/CBF building in the CBD, which housed a gymnasium, was inspected in detail internally and externally and had no structural damage.

Finally, a 7 storey building, located in a region of the CBD that exhibited significant ground instability was inspected inside and out. The structure consists of a perimeter moment resisting frame along all 4 sides, with a non-structural stair and services core and composite floor. Inspection of the steel frame and floor showed no visible damage (e.g., Figure 8a), however the perimeter frame had sunk a noticeable amount in relation to the core and had acted as pinned base, causing significant interstorey drift which has subsequently

(a) Global view

(b) Typical moment resisting beam to column connections.

Figure 7. Low-rise MRF parking garage [Photos by M. Bruneau].

(a) Typical beam to column connection with fire protection partially removed for inspection

(b) Gap opening in stair at landing due to lateral movement (stairs had independent structural support).

Figure 8. Seven storey PMRF building [Photos by C. Clifton].

significantly damaged stairs (Figure 8b) and non-structural components. The extent of ground movement around the building was considerable and it is likely that significant foundation movement has occurred. The question of what to do with this building will rest on what has happened below ground.

7 CONCLUSIONS

Steel structures generally performed well during the Christchurch earthquake series, comprising 6 damaging events from 4 September 2010 to 13 June 2011, with intensity up to 2x ULS design level and cumulative duration of strong ground shaking in excess of 60 seconds However, a few eccentrically braced frames developed link fractures, CBF brace fractures were observed in connections unable to develop the brace gross-section yield strength in violation of capacity design principles (Bruneau et al., 2011), and multiple industrial steel storage racks collapsed (damage to these CBF and racks has not been presented in this paper, but will be part of a forthcoming paper to be published in the Bulletin of the New Zealand Society of Earthquake Engineering).

The discovery of a fractured active link in a 22 storey building, in which all other links performed well, is unexplained at the time of writing this paper, and it will be a priority to determine the cause of that fracture when the damaged link is removed and accessible for close inspection.

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The authors are also grateful to the engineers/owners of specific building to allow identification of these buildings in this paper. This is especially appreciated in view of the ongoing aftershock sequence which is causing increasing 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.

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