

# STEEL STRUCTURES DAMAGE FROM THE CHRISTCHURCH EARTHQUAKE SERIES OF 2010 and 2011

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## SUMMARY

This paper presents preliminary field observations on the performance of selected steel structures in Christchurch during the earthquake series of 2010 to 2011. This comprises 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 Ms7.1 and MM7 (MM as observed in the Christchurch CBD) and most intense was the 22 February event at Ms6.3 and MM9-10 within the CBD. Focus is on performance of concentrically braced frames, eccentrically braced frames, moment resisting frames and industrial storage racks. With a few notable exceptions, steel structures performed well during this earthquake series, to the extent that inelastic deformations were less than what would have been expected given the severity of the recorded strong motions. Some hypotheses are formulated to explain this satisfactory performance.

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## INTRODUCTION

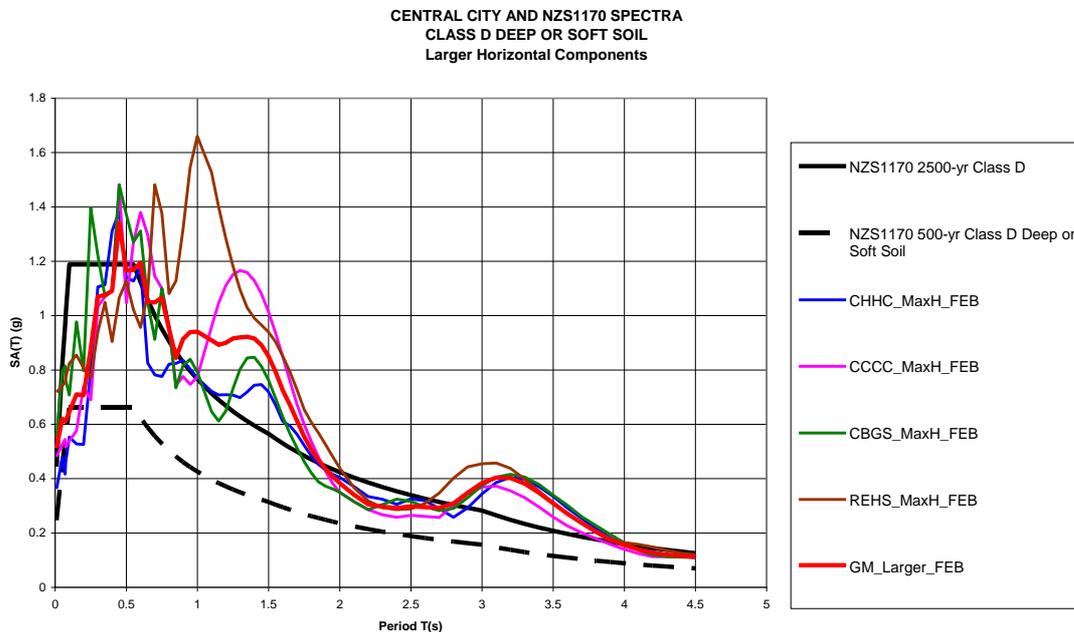
Widespread failures of unreinforced masonry buildings, the collapse of a few reinforced concrete buildings, structural damage to almost all multi-storey buildings and severe soil liquefaction across the city of Christchurch contributed to make the February 22, 2011, earthquake a tragic national disaster. The scale of human casualties and property damage from the February 22 event is in sharp contrast to the 4 September event and other earthquakes in the series, which did not cause loss of life. The 5 km shallow depth of that earthquake's hypocentre, at an horizontal distance of roughly 10 km from the city's Central Business District (CBD) resulted in ground excitations between 3 and 6 times higher than those recorded during the 4 September first event in the series. Detailed analyses of the comprehensive set of strong motion data recorded shows that the average of the recorded spectra within the CBD from the 4 September event was approximately 0.7 times the Ultimate Limit State (ULS) design spectrum 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 design spectrum and from the largest 13 June earthquake 0.9 times ULS design spectrum.

While the duration of high frequency 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. However, 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 bars 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 gauge the adequacy of the current New Zealand seismic design provisions for steel structures. This is the objective of the paper.

## SEISMIC DEMAND

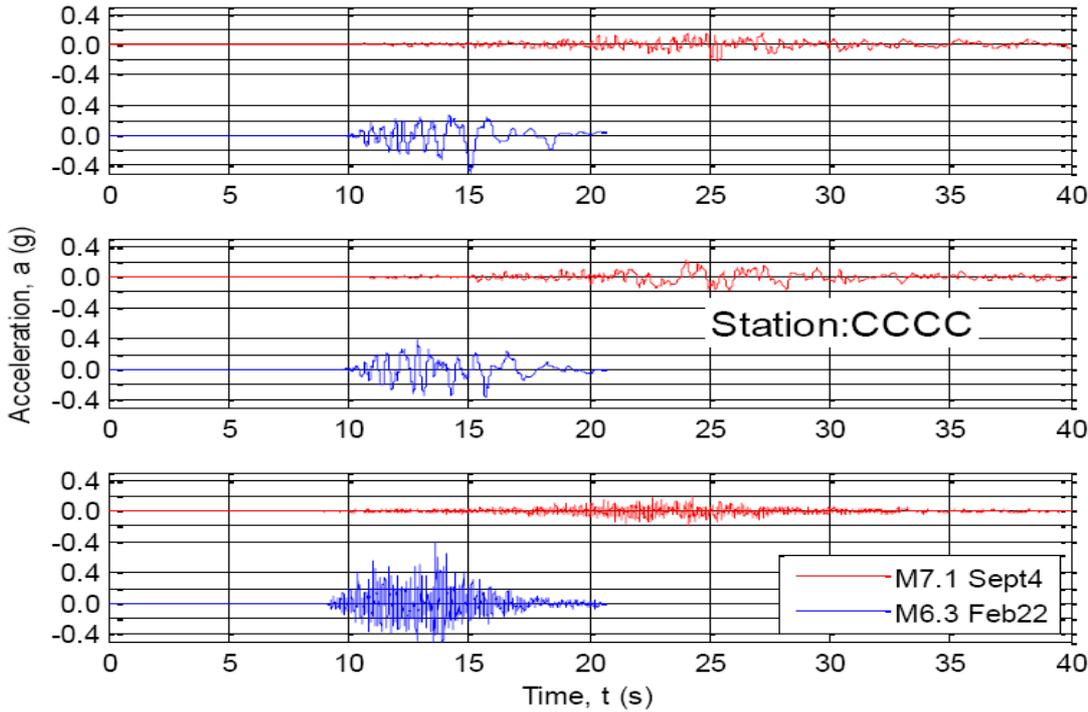
This section focuses on the February 11 event demand, which was the most severe of the series. Figure 1 shows the CBD ultimate limit state (ULS) design spectrum and maximum considered event (MCE) spectrum for buildings of normal importance (based on a 2500 year return period), the larger horizontal components from the four strong motion recorders in the CBD and the average of these components. The average is above the 1.8ULS for periods of 0.3 seconds and above, except for the period range of 1.8 to 2.7 seconds, where it still remains substantially above the ULS level. The corresponding earthquake excitations from one of the strong motion recording stations in the CBD, given in Figure 2, show substantially greater accelerations recorded during the aftershock compared to the main shock, and also highlight the relatively short duration of strong motion, typically in the order of 10 seconds.



**Figure 1: NZS 1170.5 Spectra and Largest Horizontal Direction Recorded from the CBD Strong Motion Records.**

Notes to Figure 1:

1. The dashed line is the ULS design spectrum for normal importance buildings for the soft soil type, Class D, generally considered in the CBD
2. The solid black line is the Maximum Considered Event design spectrum for normal importance buildings for Class D soil in the CBD
3. The solid red line is the average from the 5 recording stations



**Figure 2: Horizontal and Vertical Spectra from the Canterbury College Strong Motion Recorder.**  
 Note: CCCC = Christchurch Cathedral College (within the CBD).

**MULTI-STOREY STEEL STRUCTURES IN THE CHRISTCHURCH AREA**

The number of multi-storey 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 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 following the end of the early-1990s recession. 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.

**Table 1. Multi-storey Steel 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
7	Shear Walls and CBFs	Composite Deck and Steel Beams	1985
7	Perimeter MRFs	Composite Deck and Steel Beams	1989
3	MRFs	Composite Deck and Steel Beams	2010
5	EBFs	Composite Deck and Steel Beams	2008
3+ <sup>Note 1</sup>	EBFs	Precast columns and hollowcore units with topping	2003
5	EBFs	Precast columns and hollowcore units with topping	2010

Notes: 1. Currently 3 storeys; with provision for additional 1 storey.

### SEISMIC PERFORMANCE OF MULTI-STOREY ECCENTRICALLY BRACED FRAMES

Two recently designed and built multi-storey buildings in the CBD had eccentrically braced frames as part of their lateral load resisting system, these being the 22-storey Pacific Residential Tower in Christchurch's CBD, completed in 2010, and the Club Tower building, completed in 2009. Both were green-tagged following the earthquake, indicating that they were safe to occupy even if they will require some minor repairs. In the latter case this includes at least one active link replacement as is described below

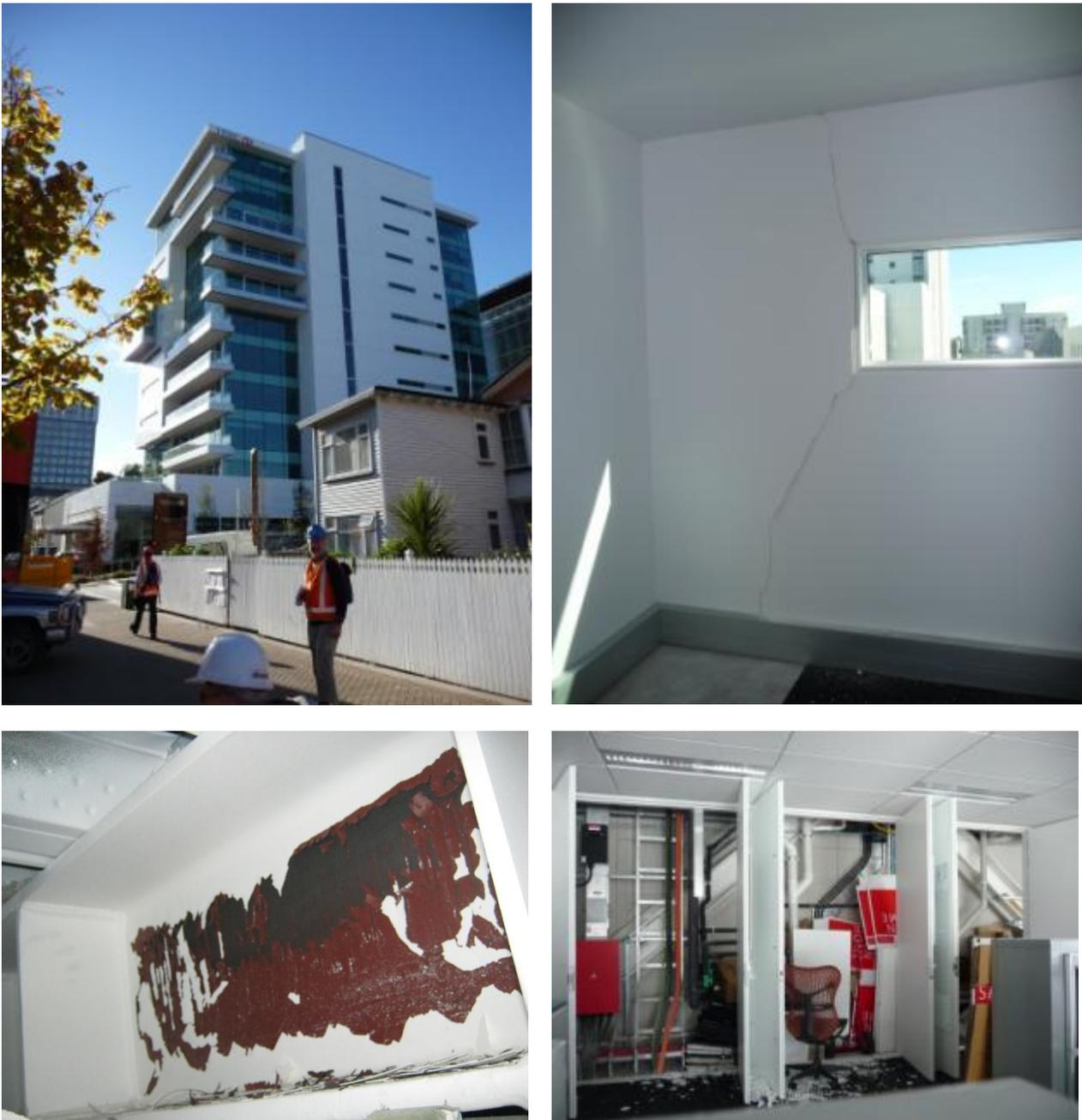
The Club Tower Building (Figure 3a) has eccentrically braced frames located on three sides of an elevator core eccentrically located closer to the west side of the building and a ductile moment resisting frame (DMRF) along the east façade. The steel frame is supported on a concrete pedestal from the basement to the 1st storey, and foundations consist of a 1.6 m thick raft slab. Only the EBFs on the east side of that core could be visually inspected without removal of the architectural finishes (Figure 3d), 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 3c 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 through assessment of the visible state of the metal in the yielded web of the active link and secondly through an 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 the change in transition temperature based on the work of Hyland *et al* (2004) and Hyland and Fergusson (2006) showed that the yielded active links have sufficiently robust metallurgical properties to be left in place. This had to be made by assessment as material could not be taken from the yielded links for direct testing, which would have then required active link replacement. 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 storeys 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 3b); 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 up 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 ( $\mu$ , equivalent to  $R_u$  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 the behaviour in this particular case, including the strength of the composite floor slab action (neglected in design), mobilization of the solid non-structural wall concrete cladding adjacent to the staircase, elastic stiffness of the gravity frame especially the columns and the relatively short duration of the February 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 ductility factor ( $\mu$ ) was low at 1.25.

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 it 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.



**Figure 3:** *Club Tower: From top down and left to right: (a) Global view; (b) cracking of partition in cantilevering portion of story; (c) paint flaking of partially hidden EBF link; (d) global view of EBF braces obstructed by various utility runs. [Photos by M. Bruneau and C Clifton].*

As a new landmark and the tallest building on the Christchurch skyline<sup>6</sup>, the 22-storey Pacific 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 around the elevator core above that level, 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

<sup>6</sup> *The Grand Chancellor Hotel is 85 metres, the Price Waterhouse Cooper building is 76.3 metres, and the C1 Building (a.k.a. the Pacific Tower) stands at 73 metres, is topped by a 13 metres spire, for a total of 86 metres.*

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 ( $\mu$ ) 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 and a more typical design ductility factor range for such buildings is 2 to 3. 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 4e) in the top level (underside of Level 6) of the EBF system at the North-Western corner of the building. 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 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 site bolted endplate active link. The replacement was 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 the frequency of inspection reduced as no further examples requiring replacement were



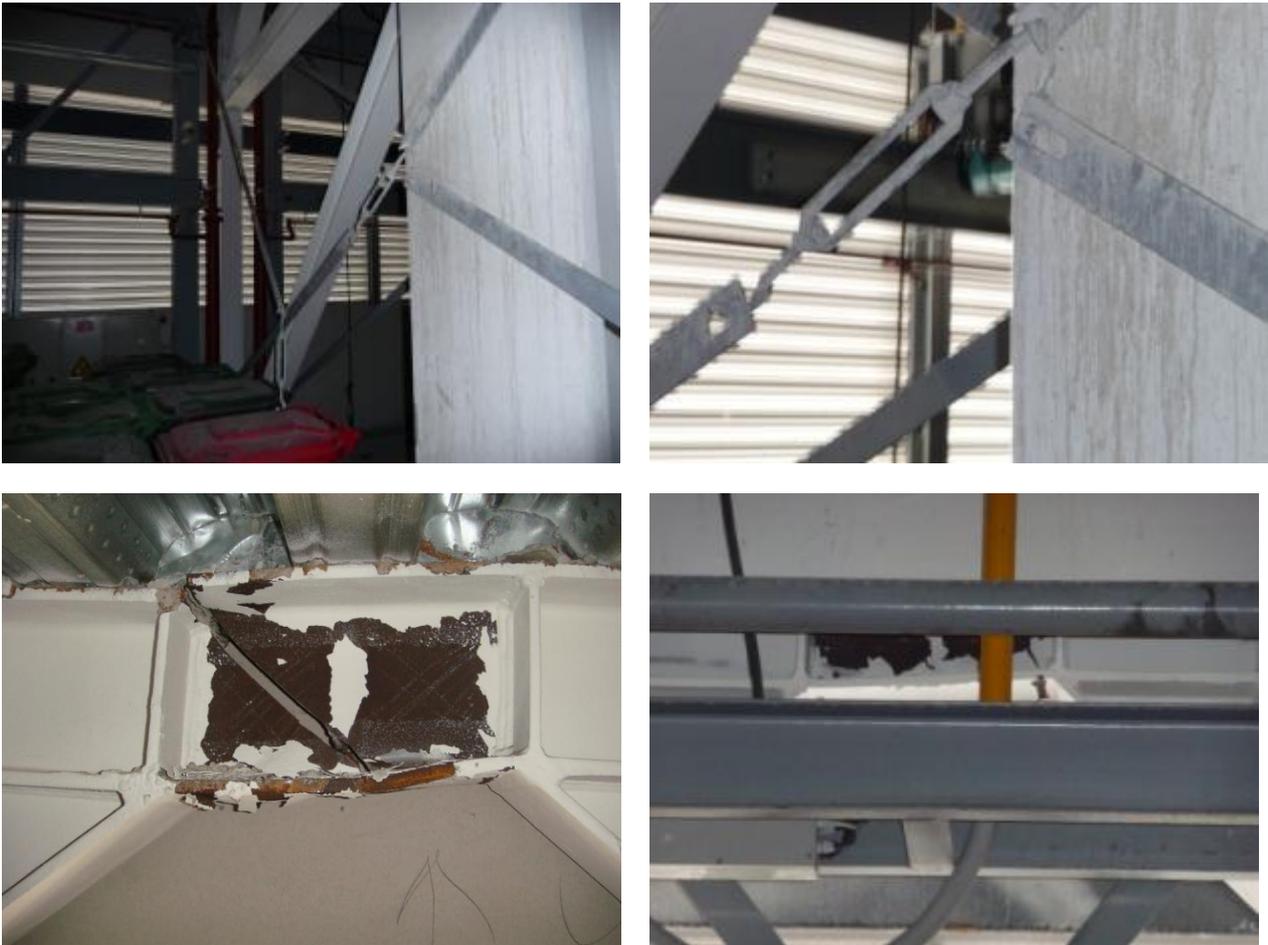
found. This inspection required removal of architectural finishes.

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 the 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 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 is certainly the one requiring least structural and non-structural repair.

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 interstorey 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.





**Figure 4:** Pacific Tower From top down and left to right: (a) (previous page) Global view; (b) (previous page) Flaked paint on EBF active link; (c) and (d) Multi-story mechanical garage stacker failed braces; (e) Fractured EBF active link in top level of EBF system in front face of atrium; (f) Residual shear deformations of EBF link in car stacker tower. [Photos by M. Bruneau and C Clifton].

#### SEISMIC PERFORMANCE OF ECCENTRICALLY BRACED FRAMES IN PARKING GARAGES

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), however there was evidence of very minor movement of the bolted splice connections in the braces. 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 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 is reported from the three June earthquakes.



**Figure 5:** Shopping mall on Dilworth St and Clarence St, Christchurch From left to right; (a) View from the East, (b) Fracture of Precast Spandrel Beams on South side. [Photos by G. MacRae].

The EBFs in a hospital parking garage closer to the epicentre (Bruneau et al. 2010) also performed well, although some link fractures were observed in two braced bays (Figure 6). Note that 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.

Note that this parking structure was also designed to accommodate two additional floors. Yet, some of the links at the first storey showed paint flaking as evidence of inelastic deformations. Evidence of 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 ground acceleration experienced by the structures with 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, in spite of being only 1.5 kms away from station CCCC in Figure 2. 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 6(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 6(c), 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 spread 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(e), there was no damage to this panel zone region.
- 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 storeys, 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 joint to meet the design requirement for support length of the 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(d) and 6(e). No lateral movement or twisting of the ends of the active links was observed, indicating 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.



**Figure 6:** *Parking garage on St Asaph St and Antigua St, Christchurch From top down and left to right: ; (a) Redundancy provided by multiple EBF bays; (b) Evidence of EBF link yielding; (c) Fractured link at lower level EBF; (d) Evidence of inelastic deformations at top level EBF; (e) Close-up view of same; (f) Displacement at expansion joint, top ramp. [Photos by M. Bruneau and C Clifton].*

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.

## CONNECTIONS

Connections in modern steel frames performed very well and as expected. Figure 7 (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. Figure 7 (b) shows a flush end-plate splice in a MRF beam that has performed well.

In a moment end-plate connection in a portal frame building in a strongly shaken region on soft ground near the fault, tensile failure of a row of bolts was observed. The connection had not opened up during the earthquake and was rapidly repaired.

No damage was evident from a brief internal inspection to the welded beam to column connections in the 7 storey perimeter moment frame building mentioned below and shown in Figure 10. These connections will require detailed evaluation as part of the assessment of this building as any local fractures, e.g. between the beam flange and column flange in a similar manner as occurred in the 1994 Northridge earthquake, would be hidden by the passive fire protection and only three joints were looked at in detail during the visit in which those pictures were taken.

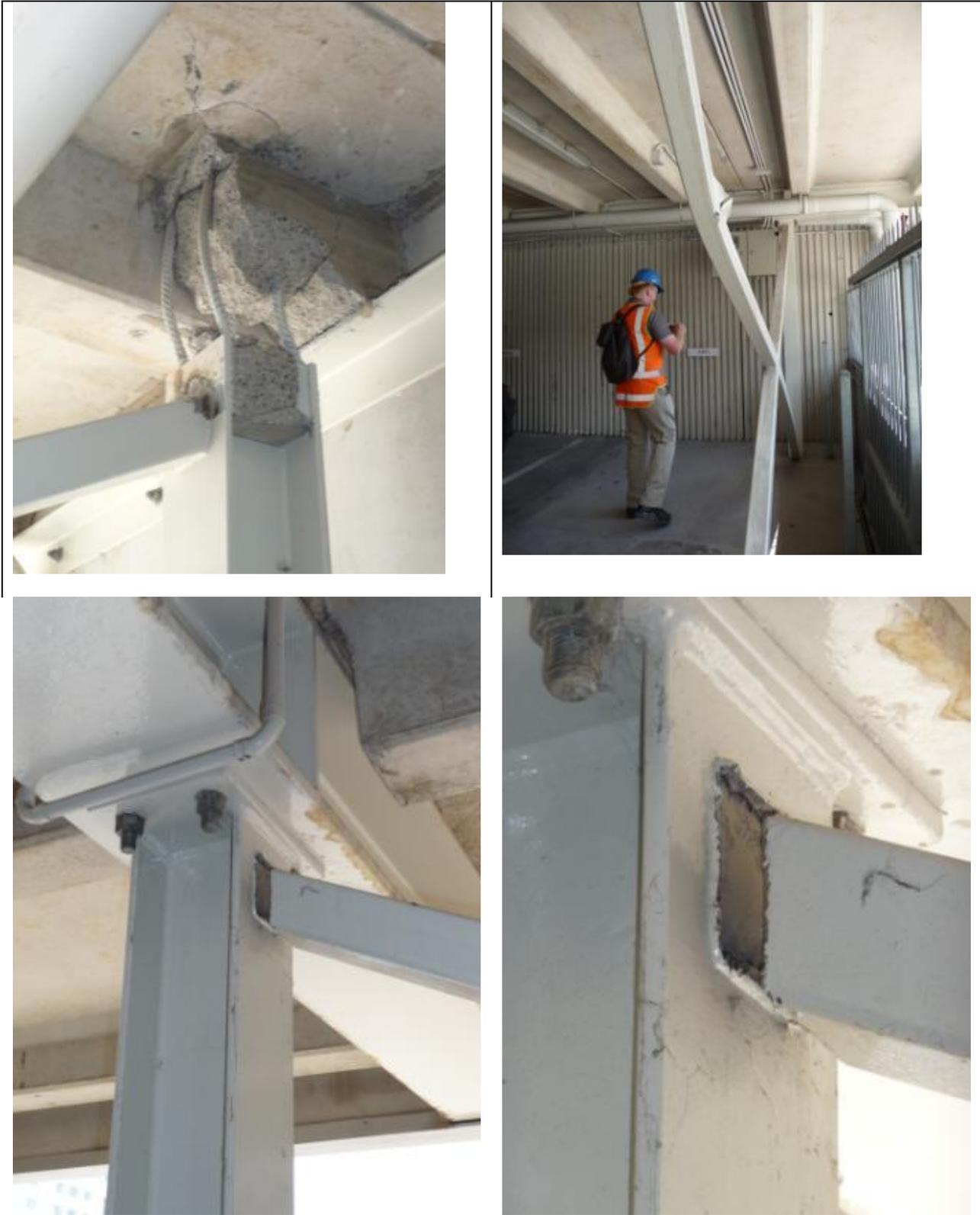


**Figure 7:** Connections in Club Tower Building, Christchurch From left to right: ; (a) Brace/beam/column connection showing out-of-plane yielding in endplate but no inelastic demand in gusset plate; (b) Flush moment endplate splice connection. [Photos by G C Clifton].

### CBF BUILDINGS

A single suspended level parking garage with concentrically braced frame (CBF) was found to have performed poorly (Figure 8). The garage had solid pre-cast panel walls on three sides, and two individual CBF bays along the fourth side (one bay on each side of the garage door). While the columns of the westernmost CBF tied to a steel beam at their top, the easternmost CBF was not similarly aligned with a steel beam. A non-ductile reinforced concrete extension

framing into a concrete beam at the top performed poorly. The other brace of that frame failed at the welds under tension loads; these welds did not appear to be designed to develop the tension capacity of the brace according to the capacity design principles of NZS 3404. The westernmost CBF performed better, without fractures and with proper attachment to the supported floor at both columns, with visible post-earthquake residual buckling as a consequence of brace elongation.



**Figure 8:** *Low-rise CBF parking garage From top down and left to right: : (a) Poor column connection detail; (b) Buckled brace; (c) and (d) Fractured non-ductile brace-to-column connection. [Photos by M. Bruneau].*

A seven storey steel framed hotel building with a combination shear walls in one direction and CBFs in the other direction, could not be inspected because of its immediate proximity to the 22 storey Grand Chancellor Hotel which was considered to

be in a state of imminent collapse following the 22 February earthquake. It is hoped to visit this building, if it is still intact, once the Grand Chancellor has been demolished. There is no indication of damage from the street.

### 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 9) 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.

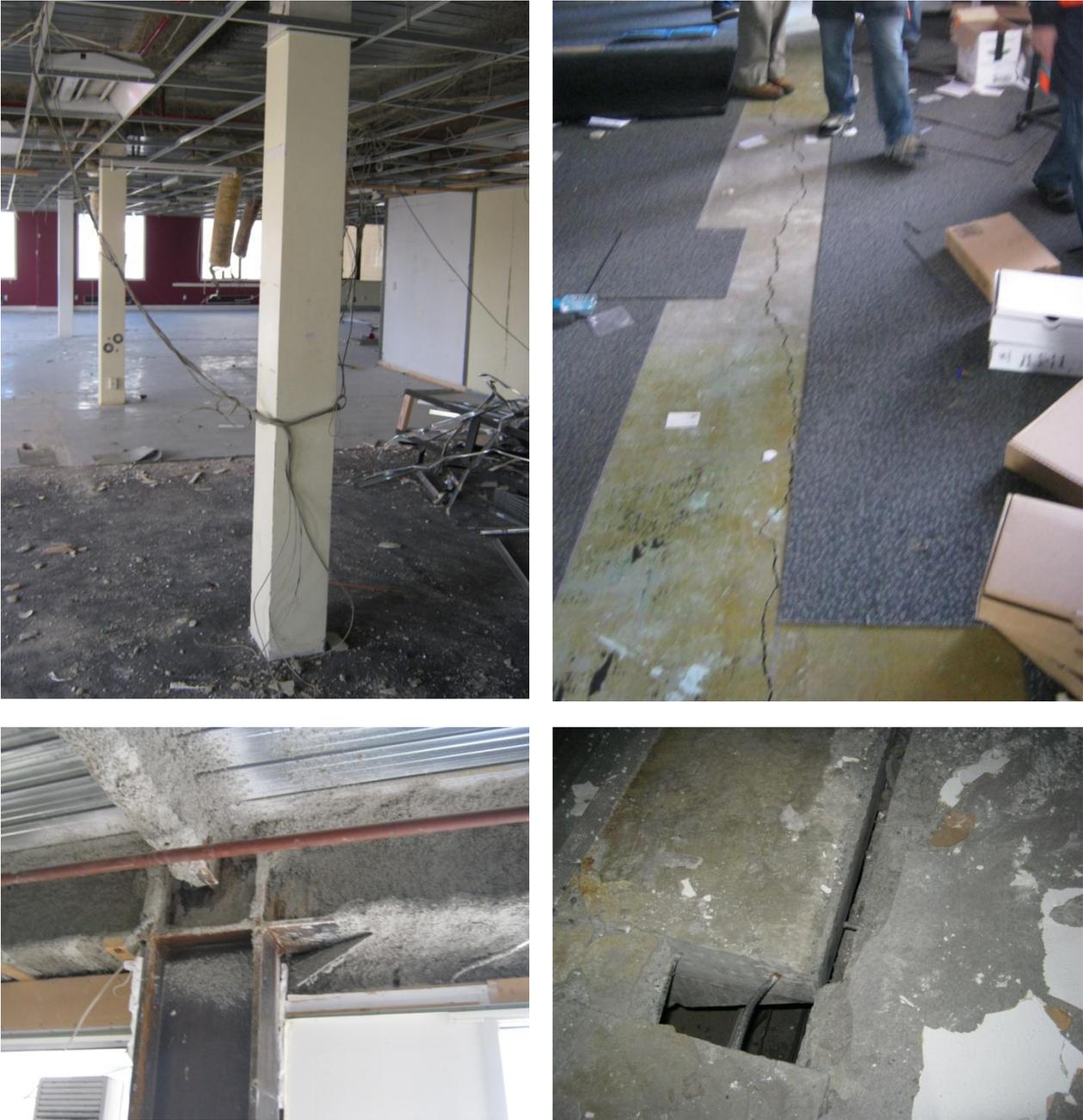
A low rise MRF building in the CBD, which housed a gymnasium, was inspected in detail internally and externally and had no structural damage.

Finally a 7 storey building located in the region of the CBD that exhibited significant ground instability was inspected

inside and out. The structure comprises 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, however the perimeter frame had sunk a noticeable amount in relation to the core (Figure 10b) and had acted as pinned base, causing significant interstorey drift which has subsequently significantly damaged stairs (Figure 10d) 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.



**Figure 9:** Low-rise MRF parking garage From top down and left to right:: (a) Global view; (b) and (c) Typical moment connections. [Photos by M. Bruneau].



**Figure 10:** Seven storey PMRF building From top down and left to right:: (a) Internal view of partially stripped floor; (b) Crack in ground floor slab where perimeter (to the right of the crack) has moved downwards relative to the core; (c) Typical beam to column connection with fire protection partially removed for inspection; (d) Gap opening in stair at landing due to lateral movement (stairs had independent structural support). [Photos by C Clifton].

### HISTORICAL BUILDINGS

Partial out-of-plane failure around the dome at the top of the Regent Theatre Building revealed that a braced steel frame had been used there (Figure 11). Although subsequent inspection will be required to verify the integrity of the connections, it appeared to be in good condition from a distance. The building was built before 1910 and the scene

was reminiscent of pictures of similar buildings following the 1906 San Francisco earthquake. However, the CBFs appeared to be welded construction (to be verified) which means they are likely to be newer than the rest of the building and had been added in a subsequent retrofit.



**Figure 11: Braced dome at top of Regent on Worchester Building From left to right : (a) Global view; (b) Close-up view. [Photos by M. Bruneau].**

Steel braced frames were sometimes used to retrofit unreinforced masonry structures (e.g. Figure 12). Drift limits to prevent failure of the unreinforced masonry typically

govern design in those instances, which explain the significant member sizes of these frames proportional to the reactive mass, and their elastic response.



**Figure 12 : Braced frame as a retrofit to unreinforced masonry building From left to right : (a) Close-up view; (b) Global view. [Photos by M. Bruneau and C Clifton].**

Buildings in the CBD that had been strengthened prior to the September 2010 earthquake typically suffered minimal to no damage in that event. They were not so fortunate in the much stronger February 2011 event. Figure 13 shows one group of

three buildings, (a) following the September 2010 event and (b) following the February 2010 event (from a different vantage point). Note especially the strengthened building on the corner has collapsed.



**Figure 13:** Strengthened URM buildings From left to right: (a) is following the September 2010 event and (b) following the February 2011 event, taken from a slightly different view-point. [Photos by C Clifton].

Finally, note that the heritage structure described in Bruneau *et al.* (2010) at the corner of Manchester and Hereford streets, severely damaged by the September 2010 earthquake, had been demolished by its owner prior to the February aftershock.

#### INDUSTRIAL and EDUCATIONAL FACILITIES

Many warehouses close to the epicentre suffered limited damage. These industrial facilities typically have light roofs and are designed to resist high wind forces; light rod braces are typically used for this purpose. Following the earthquakes, steel fabricators inspected multiple warehouses, and retightened sagging braces that had stretched due to yielding during the earthquake.

As was the case following the September 2010 Darfield earthquake, a proprietary system often used in these warehouses (sold as a kit) which used a particular banana end fitting, suffered some brittle failures of the cast-steel connectors (as shown in Figure 14). These occurred in a new warehouse when the fitting fell to the ground following the shearing of the pin retaining clip. Given that these connectors are rated for earthquake loading based on static tests conducted by the manufacturer, in light of the few fractures reported following the two earthquakes, some engineers have expressed concerns regarding their strength and potential brittleness. Performance of this and similar systems needs to be validated under a dynamic test regime more representative of their expected seismic demands, particularly simulating the impact forces applied when previously-buckled braces re-tighten during earthquake excitations.



**Figure 14:** Example of fractured banana end of proprietary brace connector in the roof plane of a long span steel portal frame building From left to right: (a) Global view; (b) Close-up view. [Photos by M. Bruneau].

Extensive failure of steel storage racks was observed in industrial facilities, in some cases in spite of additional measures taken following the September earthquake. For example, one facility owner who had racks stacked 6 pallet-levels high that collapsed during the September 2010 earthquake, purchased new racks “designed to resist Magnitude 7 earthquakes of the type expected in [the most active seismic zone of] Wellington” and re-structured his operations to limit stacking to three levels. In spite of those measures, all racks experienced total collapse, as shown in



Figure 15. While racks that failed in the transverse direction could have been pushed due to “spilling” of the pallets and piling up of the products into the aisles, this was not a factor in the longitudinal rack failures that exhibited a combination of overloaded and fractured beam to column connections, and column local buckling. It appeared that the semi-rigid beam to column connections in the longitudinal direction were too weak for the intensity of shaking and design gravity loads. See also (Uma and Beattie, 2011).



*Figure 15: Example of collapsed industrial storage racks.*



*Figure 15 (continued): Example of collapsed industrial storage racks. [Photos by M. Bruneau and G. C. Clifton].*

Anecdotally, in another facility, existing racks had been retrofitted by coupling two racks back-to-back with flat bar braces (Figure 16). These bars showed evidence of elongation and residual buckling, but did not collapse, in spite of floor movements due to liquefaction, whereas the only rack that was not retrofitted (for it was not adjacent to a second rack to which it could have been tied) collapsed. The racks has also

been allegedly tied to the rafters to prevent longitudinal failures, but such ties could not be identified.

These above selected examples highlight the fact that performance of industrial storage racks is a major issue that remains to be satisfactorily addressed; however their performance has to be considered in light of the very high intensity of shaking.



*Figure 16: Industrial storage racks that survived, with evidence of soil liquefaction From left to right; (a) Global view; (b) Close –up of buckled brace. [Photos by M. Bruneau].*

Multiple examples of tilt-up panel movements due to ground liquefaction were observed, sometimes leading to fracture of non-ductile braces unable to accommodate the imposed



**Figure 17: Industrial facility roof bracing** From left to right ; (a) Global view, showing buckled brace and fractured brace; (b) Close –up view of fractures weld of tension brace. [Photos by M. Bruneau].

Anchorage of tilt-up walls to steel structures also failed in a few instances. Figure 18 shows roof beams buckled in compression by the inward movement of the tilt-up panels, and failure of the anchors due to their outward movement (i.e. away from the building). Given that this happened in modern construction, and because tilt-up walls of greater slenderness have progressively been implemented in New Zealand, a

careful re-assessment of their seismic design provisions may be desirable.

Figure 19 shows the steel structure standing when the roofing has collapsed. This shows very good performance of the steel members, but poor performance of the roofing/connections.



**Figure 18 : Failure of tilt-up panel connections** From top down and left to right: (a) Global view; (b) Close –up view of fractures connection; (c) Global view of buckled beams; (d) Local view of one such beam. [Photos by M. Bruneau].



**Figure 19: Failure of roof and walls in older industrial facility on Salisbury Street. [Photo by G MacRae].**

At Heathcote Valley Primary School some of the most extreme shaking during the event was recorded. There was one new single storey building with a steel moment frame and block walls as shown in Figure 20a . After the earthquake the

wall was leaning to the east at the southern end, and to the west at the northern end. The concrete baseplate was blown out on the southeast side of the building as shown in Figure 20b.



**(a) Overall View from the South.**



**(b) Baseplate bolt at SE corner of the building.**

**Figure 20: Heathcote Valley Primary School Steel Moment Frame building [Photos by G MacRae].**

A steel framed wall with a brick façade was erected in a small park as shown in Figure 21, in a part of town that where significant overall structural damage occurred. The wall was placed there after the September 2010 earthquake as states

“Rebuild, Brick by Brick”. The wall suffered no damage during the subsequent earthquakes and the brick ties between the steel framing and the bricks showed no signs of distress.



**Figure 21: September 2010 Rebuilding Stand Consisting of Bricks Supported by Steel Frame. [Photos by G MacRae].**  
 (a) Overall View of Wall  
 (b) Back View of Wall

### LIGHT STEEL FRAMED HOUSES

There is a small number of light steel framed houses in the affected area. Preliminary reports are that damage to framing, brickwork and linings was less than from the September earthquake, discounting damage resulting from soil liquefaction and lateral spreading.

In one house with brick veneer, a few bricks on the top course and adjacent to window openings had been loosened, but not dislodged.



**Figure 22: Undamaged older steel pedestrian bridges on the Avon Rover near the CBD. [Photos by R: Leon].**

Although substantial liquefaction occurred along the Avon River near the CBD, the only older steel bridge in this area only showed spectacular buckling of its fascia arches; the actual bridge, supported on straight riveted girders appeared undamaged even though large settlements had occurred at the abutments (Figure 23). The old rail bridge over the Waimakariri river behaved well even though it was clear that

This behaviour is consistent with the very good performance of brick veneer on steel framing in a series of shaking table tests at the University of Melbourne in 2009 (Paton-Cole et al, 2009).

### BRIDGES

There are relatively few steel bridges in the Christchurch area. A pedestrian arch bridge at the Antigua Boatsheds and one at Victoria Square showed no visible damage (Figure 22).

the pier had moved over 100 mm toward the river and back during this shake (Figure 24a). The old road bridge suffered some longitudinal buckling of the lower flange of one beam (Figure 24b) as well as some spalling of concrete on the west side of the abutment. The only major modern steel bridge at the Port of Lyttelton, a three-span continuous plate girder, had only minor damage at the abutment (Figure 25).



Figure 23: From top down and left to right: Colombo Street bridge (a) Slumping of riverbank close to bridge; (b) Buckling of fascia arches; (c) Slumping of abutments at end of bridge; (d) Undamaged straight riveted girders. [Photos by R Leon].



Figure 24: Waimakariri Bridges, South end (a) Old Rail Bridge, (b) Old road bridge. [Photos by G MacRae].



Figure 25: Lyttelton Port Bridge (a) Plate Girder (b) Abutment Spalling. [Photos by G MacRae].

The footbridges shown in Figure 26 were damaged in the September 2010 earthquake and had not been repaired at the time of the February 22 event. Due to further lateral spreading

and slumping of abutments, they were even more damaged in this shaking.



**Figure 26: Footbridges; From top down and left to right: (a) Truss bridge over Avon River, (b) Suspension bridge with timber deck Over Kaiapoi river, (c) Suspension bridge at Groynes. [Photos by G MacRae].**

### 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, and multiple industrial steel storage racks collapsed.

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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