STEEL BUILDING DAMAGE FROM THE CHRISTCHURCH EARTHQUAKE OF FEBRUARY 22, 2011, NZST

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SUMMARY

This paper presents preliminary field observations on the performance of selected steel structures in Christchurch during February 22, 2011, Magnitude 6.3 event. In the downtown area of Christchurch, this event was considerably more severe than that from the September 4, 2010, Darfield earthquake. 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, 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.

INTRODUCTION

Widespread failures of unreinforced masonry buildings and severe soil liquefaction across the city of Christchurch, along with the collapse of a few reinforced concrete buildings, contributed to make the February 22, 2011, earthquake a tragic national disaster of much more severe impact than the earlier, Sept. 4, 2010 Darfield event. The 5 km shallow depth of that earthquake's hypocenter, at an horizontal distance of roughly 10km from the city's Central Business District (CBD) resulted in ground excitations between 3 and 6 times higher than those recorded during the 2010 main shock. Preliminary estimates indicate that this event exceeded the ultimate limit state design level specified by the New Zealand seismic loading standard by as much as 100% over some period ranges. For that reason, the performance of steel structures, even without damage, is instructive, providing a unique opportunity to gage the adequacy of the current New Zealand seismic design provisions for steel structures. This is the objective of the paper.

However, in interpreting the results presented here, it is important to recognize that the strong shaking, while very intense when comparing its response spectra with the design spectra, lasted on the order of 10 seconds (as typically expected for an earthquake of Richter Magnitude M6.3). As such, only a couple of cycles of inelastic deformations would have been induced by this aftershock in flexible structures having periods greater than 6 seconds. Consequently, this seismic event did not provide an opportunity to observe performance under the progressively degrading structural system properties that are possible during longer duration earthquakes. Thus, caution is warranted in that conclusions pertaining

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to satisfactory seismic performance for any kind of structural system during this aftershock could be optimistic, and thus sweeping extrapolations must be avoided.

Finally, whether the February 2011 event was an aftershock of the September 2010 one, or not, remains the matter of debate at the time of this writing. Here, "earthquake" and "aftershock" are used interchangeably for convenience, without any intended implications. It is important to note that when engineers were assessing buildings following the 4 September event under the Christchurch City Council/Civil Defence Emergency Management regime, they were instructed to assess buildings on the basis of the largest expected aftershock, which was considered as one Richter Magnitude/Mercalli Magnitude lower than the main event. The main event was Mw 7.0, with MM VII in the Christchurch CBD, meaning assessments were expecting up to Mw 6 and MM VI. The 22 February event in the CBD was Mw 6.3 but MM IX [Hayes G et al, 2011] which was much more intense than the largest aftershock expected.

SEISMIC DEMAND

Contrary to the 2010 Darfield earthquake, seismic demands from this earthquake were substantially more than those corresponding to the design level in the Central Business District (CBD) of Christchurch. FIgure 1 shows the CBD ultimate limit state (ULS) design spectrum and maximum considered event (MCE) spectrum for buildings of normal importance, 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 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 on the order of 10 seconds.

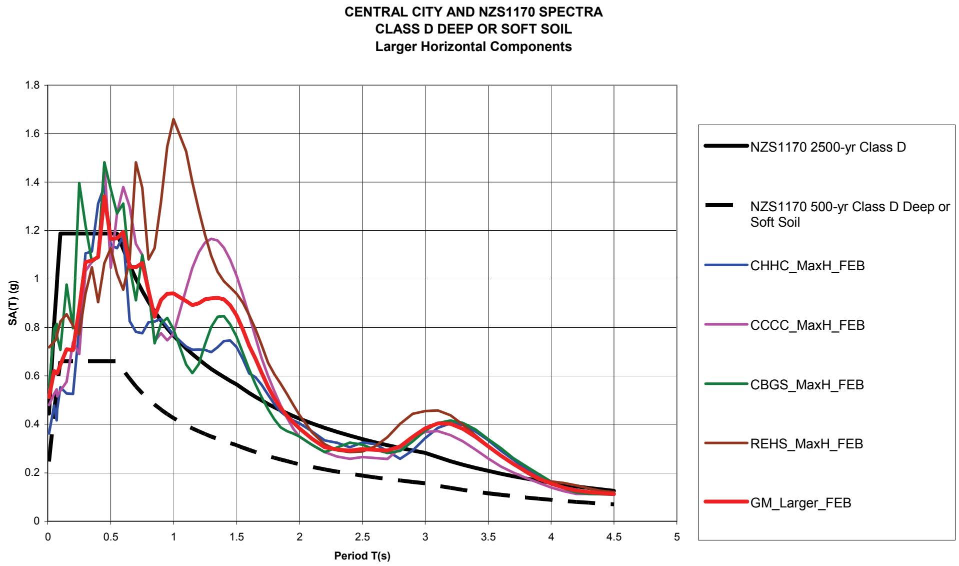

Figure 1:
NZS 1170.5 Spectra and Largest Horizontal Direction Recorded from the CBD Strong Motion Records

Notes to Figure 1:

1. The dotted 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

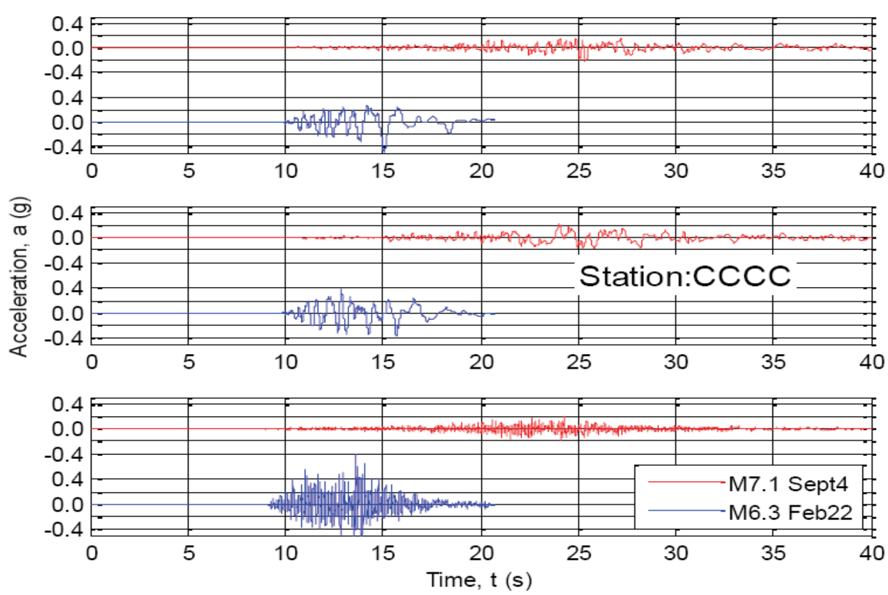

Figure 2 Horizontal and Vertical Spectra from the Canterbury College Strong Motion Recorder
Note CCCC = Christchurch Cathedral College

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 and accessible the rest of the year), and labor 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, with concrete still remaining the preferred construction material for engineered buildings. Hence, most of the steel buildings in the Christchurch area are of recent vintage, designed to the latest seismic provisions. The market share for steel framed structures 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 two 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, most of which 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	1993?
3	MRFs	Composite Deck and Steel Beams	2010
5	EBFs	Composite Deck and Steel Beams	2008
2+[Note 1]	EBFs	Precast columns and hollowcore units with topping	2008??
5	EBFs	Precast columns and hollowcore units with topping	2010

Notes:

1. Currently 2 storeys; with provision for additional 2 storeys

SEISMIC PERFORMANCE OF MULTISTORY ECCENTRICALLY BRACED FRAMES

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 Club Tower building, completed in 2009. Both were green-tagged following the earthquake, indicating that they were safe to occupy but would require some minor repairs of non-structural components.

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 story, and foundations consist of a 1.6m 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). As observed following the September 2010 (Bruneau et al. 2010), evidence of inelastic deformation was limited to flaking of the brittle intumescent paint on the EBF links at some levels (Figure 3c), however later investigations have shown slightly more yielding in the active links of EBFs in the East-West direction. The links were free of visible residual distortions. 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 appeared similar to what had been previously observed. Substantial shear cracking of the sheetrock finish on the exterior wall of that cantilevering part of the floor was also observed (Figure 3b); only hairline cracking of sheetrock finishes was observed elsewhere throughout the building. One non-structural masonry block 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_μ in US practice) of up to 4, a level of link deformations that would correspond to significant shear distortions of the links. Yet, only minimal flaking of the paint in the EBF links was observed. Beyond the usual factors contributing to overstrength in steel frames (e.g. expected yield strength exceeding nominal values, modeling assumptions, etc.), a number of additional factors can explain behavior in this particular case, including strength of the composite floor slab action (neglected in design), mobilization of the solid non-structural wall concrete cladding adjacent to the staircase 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 ductility factor (μ) was low at 1.25.

Overall, the building was designed for a 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.

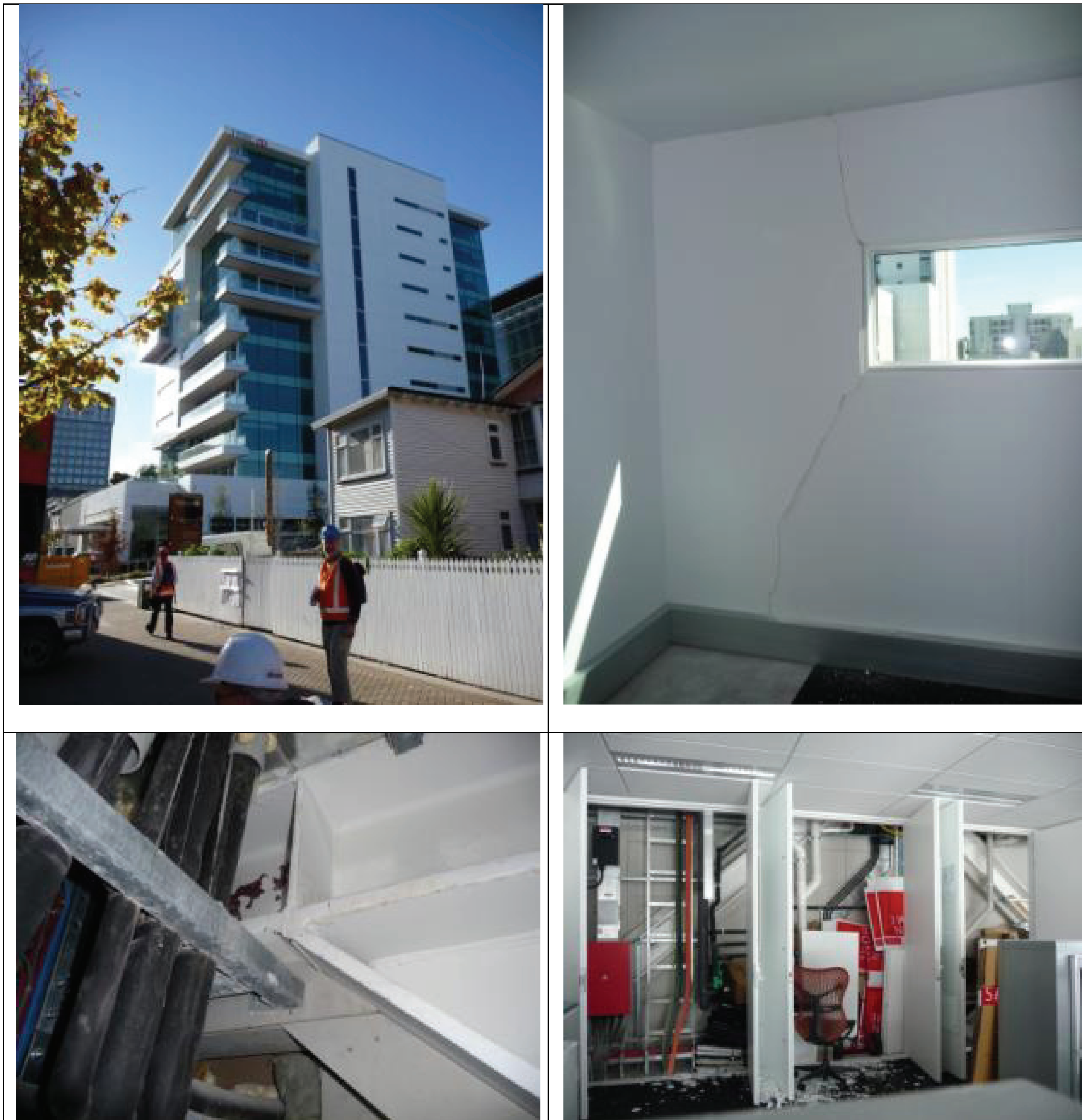

Figure 3: Club Tower [Photos by M. Bruneau]: (a) Global view; (b) cracking of partition in cantilevering portion of story; (c) paint flaking of partially hidden EBF link; (d) global view of EBF braces obstructed by various utility runs.

As a new landmark and the tallest building on the Christchurch skyline[6], the 22-storey Pacific Residential Tower consists of perimeter EBFs up to the six floor (one on each building face), shifting to EBFs around the elevator core above that level, with a transfer slab designed to horizontally distribute the seismic loads at that transition point. EBFs at levels below the transfer slab were visible, 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 were not integral with the floor slab and so did not benefit from the strength increase provided at lower stories. 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, 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. The EBF at all other levels were hidden in architectural finishes, and absence of damage to those finishes suggested limited inelastic deformation, except at level six where a few of the hotel room doors along the corridor could not be closed, which suggests greater residual deformations at that level. More detailed investigation of that level is a priority for detailed evaluation.

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. 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 design to provide easy inspection of key structural elements following earthquakes.

[6] 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.

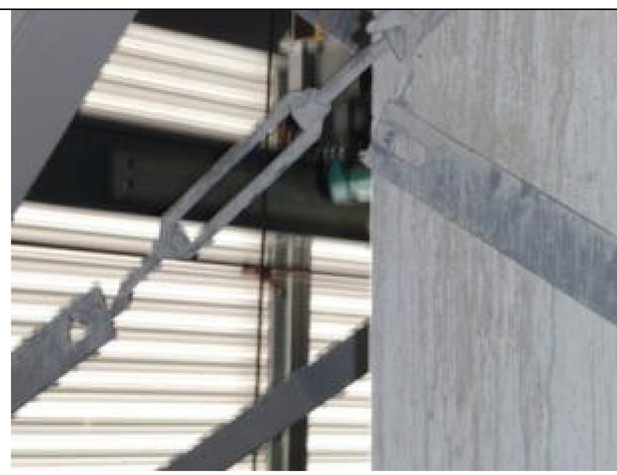

Figure 4 Pacific Tower [Photos by M. Bruneau]: (a) Global view; (b) and (c) multistory mechanical garage failed braces; (d) Flaked pain on EBF link; (e) Residual shear deformations of EBF link.

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 again inspected following the aftershock.

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 some evidence of 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.

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

The EBFs in a hospital parking garage closer to the epicenter (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 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 story 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 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, 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 closeup in Figure 6(c) were of particular concern as these were the first fractures recorded in EBFs worldwide. 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 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(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 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(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 despite only being applied to the top flange and for EBFs not integral with the slab above.

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

The fact that some inelastic demand has been observed in EBF active links (but less than expected given the severity of the earthquake) along with two fractures raises the issue of establishing criteria for repair or replacement. These are being worked on as a matter of urgency and will be the topic of further publications.

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 endplate 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, one example of a row of bolts in a moment end-plate (MEP) connection in a portal frame suffering tension failure has been observed. The connection had not opened up during the earthquake and was rapidly repaired.

Figure 7: Connections in Club Tower Building, Christchurch [Photos by G C Clifton]; (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

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 NZS3404 . The westernmost CBF performed better, without fractures, with visible post-earthquake residual buckling as a consequence of brace elongation.

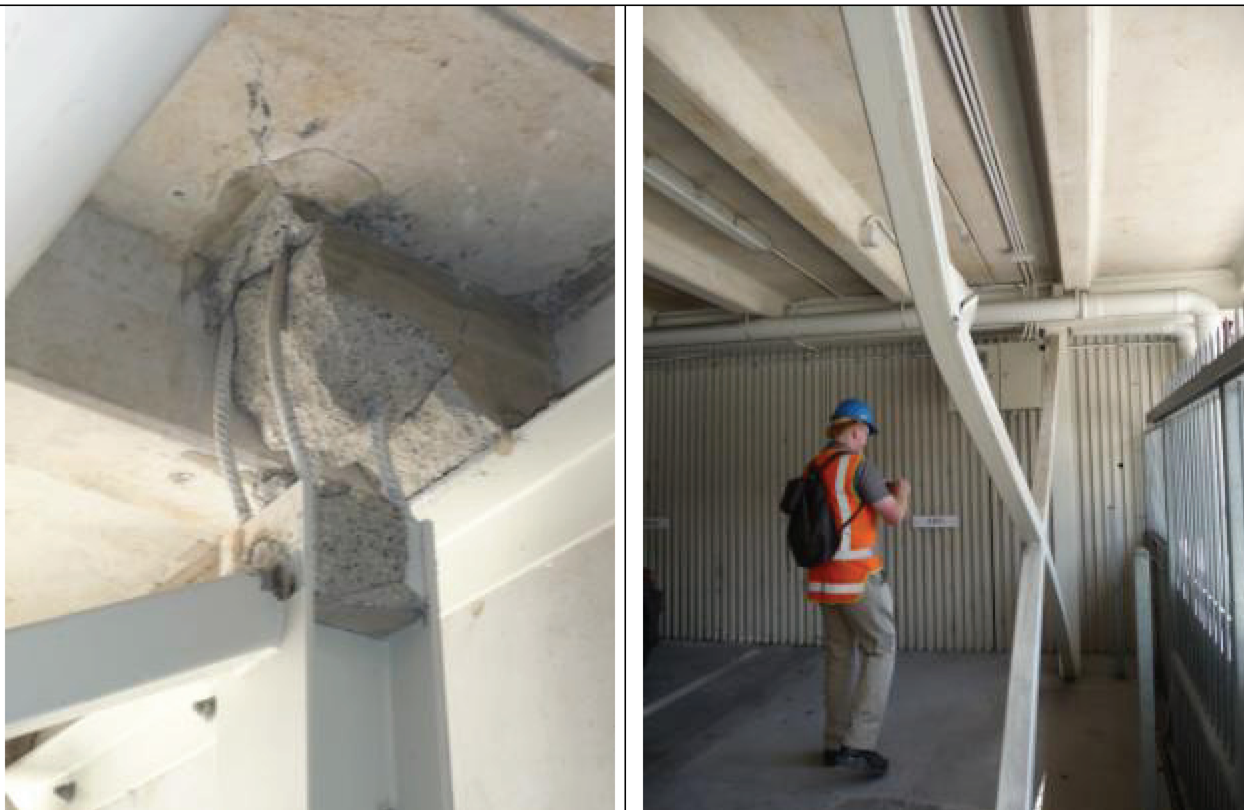

Figure 8: Low-rise CBF parking garage [Photos by M. Bruneau]: (a) Poor column connection detail; (b) Buckled brace; (c) and (d) Fractured non-ductile brace-to-column connection.

A seven storey steel framed hotel building with 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 due to fatal damage in lower level shear walls of its concrete frame system. It is hoped to visit this building, if it is still intact, once the Grand Chancellor has been demolished.

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 a distance as a pre-existing post-tensioned concrete section of that garage (together with the spans between the older and newer part of that parking garage) collapsed onto its access ramp.

A low-rise MRF building also could not be inspected being located too close to the Grand Chancellor hotel.

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

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 10). 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 10: Braced dome at top of Regent on Worchester Building [Photos by M. Bruneau]: (a) Global view; (b) Close-up view.

Steel braced frames were sometimes used to retrofit unreinforced masonry structures (e.g. Figure 11). 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 11: Braced frame as a retrofit to unreinforced masonry building [Photos by M. Bruneau]: (a) Close-up view; (b) Global view.

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 12 shows one group of three buildings, with (a) showing these following the September 2010 event and (b) showing (from a different vantage point) the three following the February 2010 event. Note especially the strengthened building on the corner has collapsed

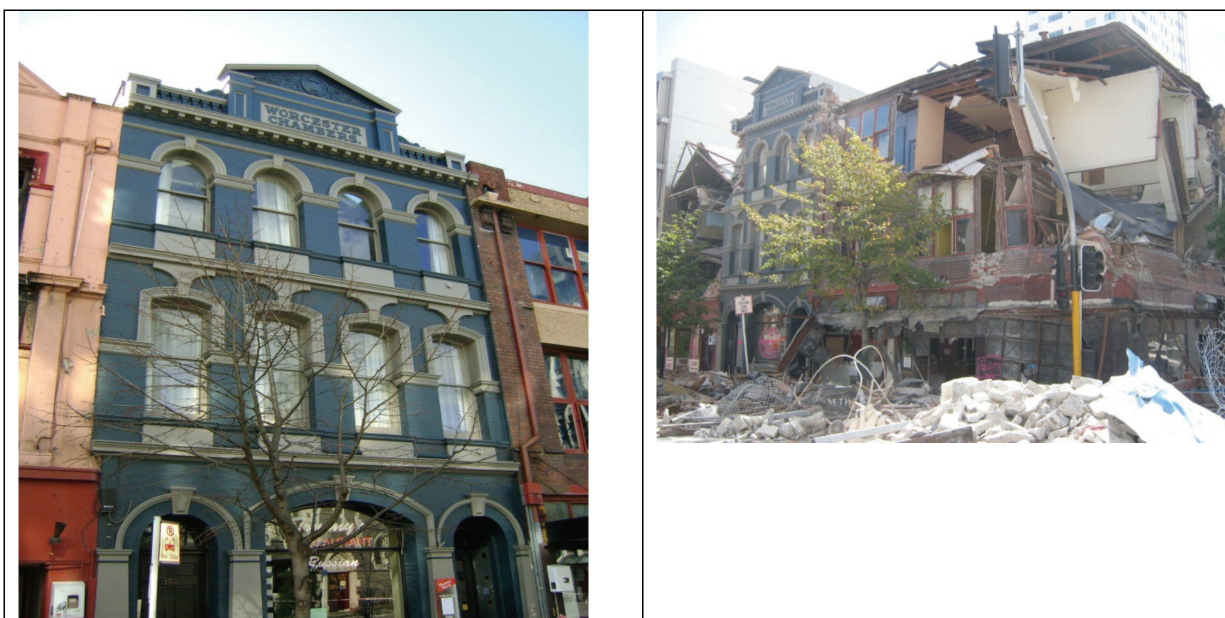

Figure 12: Strengthened URM buildings: (a) is following the September 2010 event and (b) following the February 2011 event, taken from a slightly different view-point.

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 epicenter 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 13). 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 potential brittleness and believe that their performance 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 13: Example of fractured banana end of proprietary brace connector in the roof plane of a long span steel portal frame building [Photos by M. Bruneau]: (a) Global view; (b) Close-up view.

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 14. 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.

	
	

Figure 14: 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 15). 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; although the performance has to be considered in light of the very high intensity of shaking.

Figure 15: Industrial storage racks that survived, with evidence of soil liquefaction [Photos by M. Bruneau]; (a) Global view; (b) Close –up of buckled brace.

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 deformations. One such example is shown in Figure 16, showing a fractured brace and its counterpart buckled brace.

Figure 16: Industrial facility roof bracing [Photos by M. Bruneau]; (a) Global view, showing buckled brace and fractured brace; (b) Close –up view of fractures weld of tension brace.

Anchorage of tilt-up walls to steel structures also failed in a few instances. Figure 17 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 17 : Failure of tilt-up panel connections [Photos by M. Bruneau]; (a) Global view; (b) Close –up view of fractures connection; (c) Global view of buckled beams; (d) Local view of one such beam.

Figure 18 shows the steel structure standing when the roofing has collapsed. This shows remarkable performance of the steel members, but poor performance of the roofing/connections.

Figure 18 : Failure of roof and walls in older industrial facility on Salisbury Street [Photos by 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 19a . 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 19b.

(a) Overall View from the South (b) Baseplate bolt at SE corner of the building

Figure 19. Heathcote Valley Primary School Steel Moment Frame building (Photos: MacRae)

A steel framed wall with a brick façade was erected in a small park as shown in Figure 20, in a part of town that where significant overal 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 this earthquake.

(a) Overall View of Wall

(b) **Back View of Wall**

Figure 20. September 2010 Rebuilding Stand Consisting of Bricks Supported by Steel Frame (Photos: MacRae)

LIGHT STEEL FRAMED HOUSES

There are 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.

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 21).

Figure 21 – Undamaged older steel pedestrian bridges on the Avon Rover near the CBD (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 22). The old rail bridge over the Waimakariri river behaved well even though it was clear that the pier had moved over 100mm toward the river and back during this shake (Figure 23a). The old road bridge suffered some longitudinal buckling of the lower flange of one beam (Figure 23b) as well as some spalling of concrete on the west side of the abutment. The only major modern steel bridge at the Port of Lyttleton, a three-span continuous plate girder, had only minor damage at the abutment (Figure 24).

Figure 22 – 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. (Leon)

Figure 23 – Waimakariri Bridges, South end (a) Old Rail Bridge, (b) Old road bridge (MacRae)

Figure 24 – Lyttleton Port Bridge (a) Plate Girder (b) Abutment Spalling (MacRae)

The following footbridges were damaged in the September 2010 earthquake and had not been repaired. Due to further lateral spreading and slumping of abutments, they were even more damaged in this shaking.

Figure 25 – Footbridges (a) Truss bridge over Avon River, (b) Suspension bridge with timber deck Over Kaiapoi river, (c) Suspension bridge at Groynes (MacRae)

CONCLUSIONS

Steel structures generally performed well during the February 21, 2011, event following the September 4, 2010, Darfield earthquake. 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.

While the satisfactory performance is positive in light of the seismic demands largely exceeding the code-specified design spectra, caution in warranted before making generalization as the number of inelastic excursions remained small for structures having a high fundamental period of vibration (such as tall buildings), which only exhibited at most 3 cycles of inelastic demand albeit to a high level of acceleration.

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REFERENCES

Bruneau, M., Anagnostopoulou, M., MacRae, G., Clifton, C., Fussell, A., “PRELIMINARY REPORT ON STEEL BUILDING DAMAGE FROM THE DARFIELD EARTHQUAKE OF SEPTEMBER 4, 2010”, Bulletin of the New Zealand Society for Earthquake Engineering, Vol.43, No.4, pp.351-359.

Hayes G et al, “THE 09/03/2010 DARFIELD EARTHQUAKE AND ITS AFTERSHOCKS, INCLUDING THE 02/21/2011 CHRISTCHURCH EVENT”, Educational Slides, US Geological Survey, National Earthquake Information Center, 2011.

Standards New Zealand, NZS 1170 Part 5: 2004 'Earthquake actions - New Zealand' part of the Joint Australasian Loadings Standard set AS/NZS 1170 'Structural Design Actions'.

Standards New Zealand, NZS 3404: 1997 incorporating Amendment No 1:2001 and Amendment No 2: 2007, Standards New Zealand, Wellington, New Zealand

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