PRELIMINARY REPORT ON STEEL BUILDING DAMAGE FROM THE DARFIELD EARTHQUAKE OF SEPTEMBER 4, 2010

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SUMMARY

This paper presents preliminary findings based on the performance of various steel structures during the Darfield earthquake of September 4, 2010, including concentrically braced frames, eccentrically braced frames, steel tanks, and steel houses. With a few exceptions, steel structures performed well during this earthquake, but much of this is attributed to the fact that seismic demands from the Darfield earthquake were generally lower than considered in their design.

INTRODUCTION

Many different interpretations of disaster resilience have been proposed in the literature. In one such concept, which has found broad acceptance, disaster resilience has been described as being a function of 4 R's, namely the *robustness* and *redundancy* of the infrastructure in its ability to limit damage, and *rapidity* and *resourcefulness* in returning the affected area to its pre-disaster condition [1]. If anything, the Darfield earthquake showed that robustness of the infrastructure is a cornerstone in achieving disaster resilience – that a high resilience level can be achieved by a society when the extent of structural damage is limited. The rate at which a community returns to its pre-disaster condition (i.e. the *Rapidity* dimension of resilience) was again demonstrated by this earthquake to be intrinsically coupled to the ability of the infrastructure to withstand the disaster without debilitating damage (i.e. the *Robustness* dimension of resilience). In the case of the Darfield earthquake, relative to expectations for a Magnitude 7+ earthquake, damage was moderate, making the community quite resilient. In fact, if not for the failures of unreinforced masonry buildings and damage consequent to soil liquefaction, the response and recovery requirements following this earthquake would have been minor.

However, in this case, preliminary investigation suggests that the predominantly good performance of modern engineered buildings of various construction materials and vintages in the affected region (including steel buildings), while partly attributed to the existence of effective seismic design requirements, is attributable to a significant degree on the fact that seismic demands from this earthquake were less than those corresponding to the design level, especially for structures in the Central Business District (CBD) of Christchurch with as-built first mode periods of under 1.5 seconds. See more on this below. Consistently, steel structures in the Canterbury area have suffered little damage, and this damage consisted of either slight evidence of plastic yielding, damage to concrete elements in lateral load resisting frames made of both steel and concrete, isolated instances of connector failures, and collapse of steel tanks and industrial steel storage ranks. The following provides an overview of this damage. Note that an all-inclusive survey of the performance of all steel buildings exposed to severe shaking has not been conducted. Confidential communications reporting evidence of minor inelastic behaviour and plastic hinging in buildings started to emerge about a week following the main shock, but specific details related to these cases have often not been disclosed in the public domain. These communications advise that none of the inelastic demand is sufficient to warrant repair or replacement of steel members; however some bolts in high strength structural bolted connections may be replaced as a precautionary measure where there is evidence of connection slip during the earthquake.

Also note that there are significantly fewer medium to high-rise steel buildings in the affected area than reinforced concrete ones, as a consequence of two factors:

- A strong tradition of seismic design using reinforced concrete and capacity design principles, as the legacy of Professors Park and Paulay who developed many of these concepts at the University of Canterbury in Christchurch in the 1970s and 1980s;
- A reticence to build multi-storey steel structures following labour disruptions by steel erectors that made steel construction crawl to a rest in the 1970s, and

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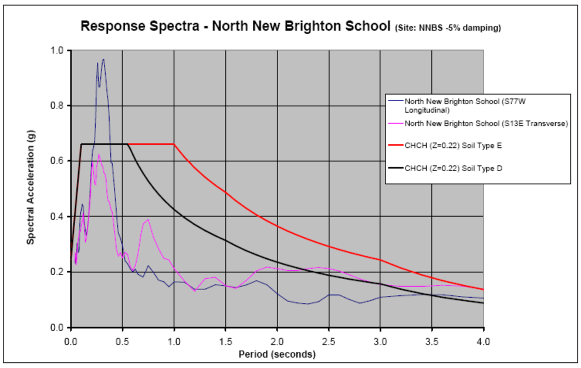
hampered the steel construction industry for the better part of the following two decades. However, that legacy disappeared during the 1990s, resulting in more multi-storey steel framed buildings built since around 2000.

COMPARISON OF EARTHQUAKE INTENSITY WITH DESIGN INTENSITY

It is possible to make comparison of the earthquake intensity with the design intensity through comparing the 5% damped spectra from strong ground motion stations throughout the region with the elastic design spectrum CZ from NZS 1170.5 [2].



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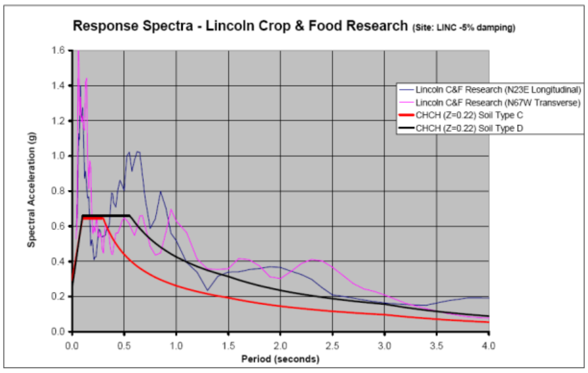

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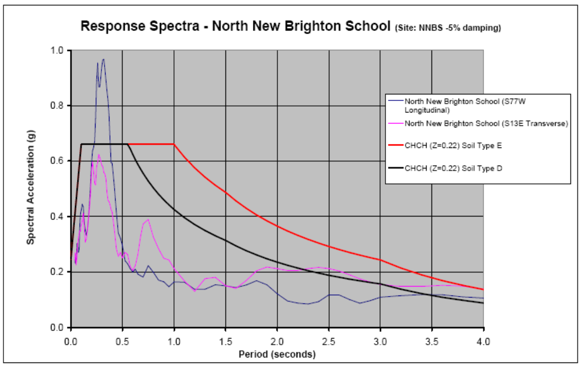

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Figure 1: *5% damped spectra from various locations with design ULS spectra for the relevant soil types added; (a) from Christchurch hospital - soil type D; (b) from Lincoln crop and Food research - soil type D; (c) from North New Brighton - North east of city, soil type D/E [Generated by C. Clifton based on data retrieved from Geonet].*

Preliminary results from this comparison as of 13 September show the following:

1. Intensity varies considerably throughout the region.
2. Modification from the underlying soils is significant.
3. In the Central Business District (CBD):
   - Ground conditions were stable;
   - The intensity was approximately 60-70% of design ultimate limit state (ULS) for periods of under 1.5 seconds and increased to 100% ULS for periods over 2 seconds, especially in the north-south direction where it exceeded the ULS design spectrum (see Figure 1a for a clear example of this);
   - The earthquake's north-south component was noticeably stronger.
4. To the northwest/west/southwest of the city (i.e. near the epicentral region) the intensity was up to 100% ULS (see example Figure 1b).
5. To the northeast and east of the City the intensity was under 50% ULS but there was very significant soil instability and ground movement in these regions (see example Figure 1c).

A significant aftershock on September 8, 2010 (4 days after the main shock), caused higher spectral accelerations in the city, for some periods, than did the main shock.

BEHAVIOUR OF ECCENTRICALLY BRACED FRAMES

A number of eccentrically braced frames (EBF) were recently constructed in Christchurch. Given the limited seismic demands during this earthquake on low to medium rise buildings, they generally performed well; this was also the case for the tallest such building. The 22-storey Pacific Residential Tower in Christchurch's CBD comprises one EBF frame in each perimeter wall with the EBFs at an unusually shallow angle [3]. The building was subjected to greater than 100% of ULS design earthquake loading in the north-south direction and also performed with no visible structural damage. This is partially attributed to the building being designed for a lower level of structural ductility demand than is typical for an EBF due to its height and plan dimensions. The behaviour of a few of these frames is discussed below.

Figure 2: *Global elevation of shopping mall on Dilworth St and Clarence St, Christchurch (43°31'53"S-172°36'05"E) [Photo by M. Bruneau].*

Typical EBFs in the three level parking garage of a shopping mall are shown in Figure 2. Note that two-tiers of bracing were used at each level in this structure (Figure 3a), which required the addition of channels along the EBF mid-height beams to provide lateral bracing of the links (Figure 3b). None of the EBF links showed evidence of yielding. Unrelated to the performance of the EBF, steel plates tying precast panels suggested slight slippage at those ties locations, embedment plates that fastened into more than one precast panels were subject to failure as the panels slid relative to the embedment plate (Figure 3c), and a few shear failures were observed at the corners of precast units on steel supports (Figure 4). For perspective, although a few URM buildings suffered damage a block away, all other surrounding buildings were also free of visible structural damage.

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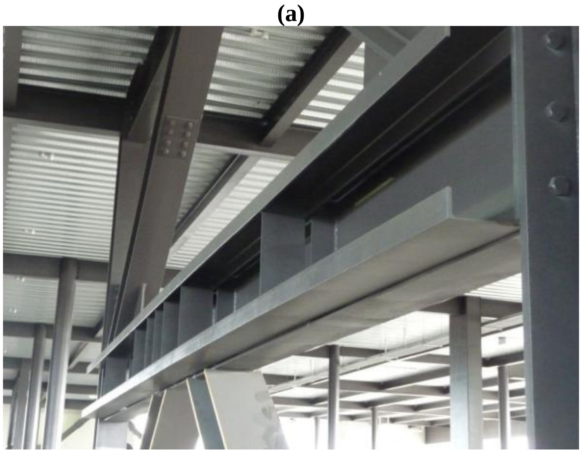

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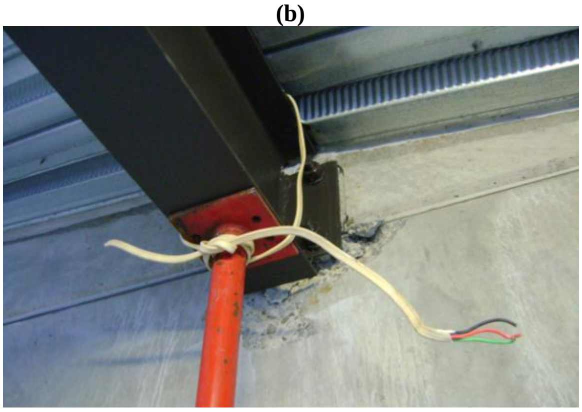

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Figure 3: ***Shopping mall on Dilworth St and Clarence St, Christchurch; (a) Storey elevation [Photo by M. Anagnostopoulou]; (b) Lateral bracing of EBF link using channels [Photo by M. Bruneau]; (c) Failed embedment plate into precast concrete wall panels [Photo by C. Clifton].***

Figure 4: ***Shear failure at support of precast panel, shopping mall on Dilworth St and Clarence St, Christchurch [Photo by M. Bruneau].***

The EBFs used in a hospital parking garage also performed well (Figure 5). These differ from the previous ones in that concrete columns are used together with the steel beams and braces. The frames throughout the garage did not have evidence of inelastic action, except in a ramp built at the top level to accommodate the future addition of storeys. The EBF only supported the east end of that ramp, and the reinforced concrete columns at the ramp expansion joint west of that EBF suffered shear failures as shown in Figure 6a. Slight flaking of the paint on that EBF link also suggests it underwent some limited yielding (Figure 6b). A few other links exhibited similar instances of flaked paint. Some of the steel plates used to laterally brace the links were observed to be bent, suggesting that the link started to laterally-torsionally buckle until restrained (Figure 6c and 6d). Note that these restraints will provide effective lateral restraint to the top flange of the EBF active link only.

Figure 5: ***Typical bent in parking garage on St Asaph St and Antigua St, Christchurch (43^{0}32’10”S-172^{0}37’41”E) [Photo by M. Anagnostopoulou].***

(a)

(b)

(c)

(d)

Figure 6: ***Parking garage on St Asaph St and Antigua St, Christchurch; (a) Damaged reinforced concrete column [Photo by M. Anagnostopoulou; (b) Evidence of EBF link yielding [Photo by M. Bruneau];); (c) and (d) Bent lateral restraints [Photos by M. Anagnostopoulou].***

A 13–storey building (Figure 7), whose construction was completed less than a year before the earthquake, also relied on EBF as its lateral load resisting system. The EBFs were used on three sides of a stair/elevator core located on the west edge of the building. Beyond slight cracking of non-structural partitions, cracks were visible at the top of the concrete slabs parallel to the collector beams leading to the EBF (a composite metal deck slab with 80 mm deep trapezoidal profile, with approximate 150 mm overall slab thickness). These cracks, wider than hairline, suggested evidence of shear transfer in the concrete slab to the steel collector beams. The brittle intumescent paint coating used on the steel frames flaked in some of the EBF links, providing evidence of minor inelastic behaviour during the earthquake. The links were free of visible residual distortions. Interestingly, cracking patterns in the slab were observed 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); these cracks are likely to have been produced as a result of vibration modes excited by vertical ground motions.

INDUSTRIAL FACILITIES

Some warehouses in Rolleston, 3 miles from the eastern tip of the surface faulting, suffered limited damage. Although exposed to greater ground accelerations than all buildings in Christchurch, these industrial facilities have light roofs and are designed to resist high wind forces – with typical average design wind pressures (external and internal) of 0.8 to 1.6 kPa on exterior cladding of single storey buildings and snow loadings of at least 0.5 kPa.

Figure 7: ***General view of 13-storey building with EBFs on Worcester Blvd, Christchurch [Retrieved November 2010; http://www.skyscrapercity.com/showthread.php?t=474691&page=9].***

Most of these warehouses relay on concentrically braced frames built with slender steel rod braces for their lateral load resistance (Figure 8a). Braces are connected to the columns using one of various types of turn-buckle systems. One such system often used in these warehouses is a proprietary New Zealand system, which is sold as a kit and used a particular banana end fitting, as shown in Figure 8b. These are rated for earthquake loading following testing by the manufacturer, with a requirement of the test that failure occurs outside the connection region. Brace bars are typically pre-tensioned to 25% of their ultimate load using this system. A number of these braces were found to be sagging after the earthquake (by as much as 200 mm according to some technicians involved in the post-earthquake inspection and retrofitting work), both in the vertical braced bays and roof diaphragm braced bays. The loose braces were simply re-straightened by re-torquing the nuts at the end fitting after the earthquake. When tightening nuts remained in place throughout the earthquake, the presence of sag was indicative of effective axial plastic elongation of the braces.

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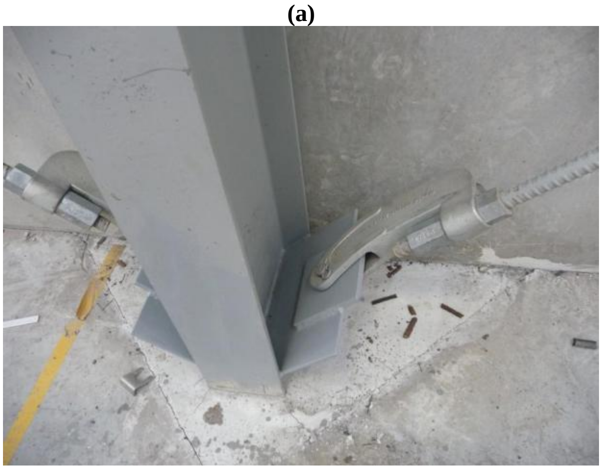

(b)

Figure 8: ***Warehouses in Rolleston; (a) Elevation of concentrically braced bay; (b) Banana end of proprietary brace connector [Photo by M. Anagnostopoulou].***

In one instance, fractures of banana bars near their pin end was observed in a roof braced bay (Figure 9a); it was alleged that this fracture occurred because the banana ends were oriented in a way that induced bending in their plates during vertical sagging of the bars, and that this would not have been the case if they had been oriented to allow rotation about their pin under gravity loads (i.e. by orienting the connector at 90 degree from how it had been installed). It was also suspected that some of these connectors were installed without pretensioning the nuts on both sides of the bar's fitting lug on the banana end, which could also explain the observed unsatisfactory behaviour; the absence of pretension creates impacts of the nuts to the connector under the reversed cycling loading, which can push the nuts away from the connector upon repeated impacts, and result in loss of bar pre-tension, followed by loss of bracing action (some nuts were reportedly found to have been displaced by as much as 200 mm from their original position). Finally, in one case, the grooves of the special purpose threaded bars used in this system were stripped within the connection itself, releasing the brace from its anchorage (Figure 9b), and in a few cases, the gusset plates to which the braces were connected suffered bearing failures.

None of the warehouses suffered damage beyond cosmetic cracking as a result of these behaviours.

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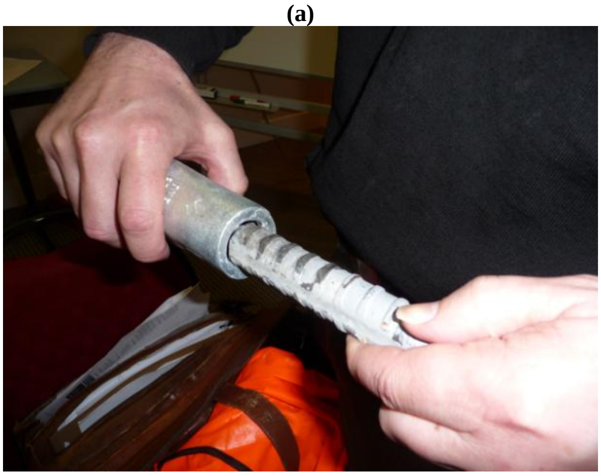

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(c)

Figure 9: ***Warehouses in Rolleston; (a) Bracing with fractured connection [Photo by A. Fussell]; (b) Stripped threaded bar [Photo by M. Bruneau]; (c) Damaged garage door [Photo by M. Bruneau].***

Non-structural damage in these structures was substantially more significant. For example, many warehouse doors became unhinged, moving substantial distances out-of-plane, i.e. yellow paint marks on the door shown in Figure 9c provided evidence that the door hit the yellow post during the earthquake. This was costly damage given that some of these doors cost up to $75,000, and that some individual warehouses suffered up to a dozen such door failures. There was also failure of cold formed steel storage systems inside some of these buildings, as shown in Figure 10a and 10b. Performance of these racks is covered in a separate paper.

(a)

(b)

Figure 10: ***Damaged industrial storage racks in Rolleston; (a) Global view; (b) Close –up view [Photos by M. Anagnostopoulou].***

DAMAGE TO HERITAGE STRUCTURE (STEEL AND URM)

The tallest unreinforced masonry (URM) building in Christchurch is shown in Figure 11. Built in 1905-06 [4] construction was as follows:

1. Bottom two storeys of reinforced concrete with encased structural steel members;
2. Top storeys comprise timber floors supported on an internal steel gravity frame;
3. The perimeter comprises lateral load resisting piers of URM tied at each storey level with a reinforced concrete bond beam encasing a rolled steel joist (RSJ);
4. The internal structure from level 3 upwards consists of a steel frame supporting timber floors with the beams from this internal frame sitting into pockets in the ring beam above the perimeter unreinforced masonry (URM) piers;
5. There was no connection between the beams of the internal steel frame and the perimeter walls to prevent these two systems from pulling apart.

The building remained stable under the regime of aftershocks up to end October. Because it is the first high-rise building built in Christchurch, it is considered by many to be of great historical value.

After considerable (and sometimes heated) debate, this building is being demolished. While it was technically possible to stabilise and retrofit the building, the URM would have had to be first stabilized and then strengthened to be able to survive undamaged an earthquake matching the design level (the September 2010 earthquake was about 60% to 70% of the design ultimate limit state earthquake for Christchurch, which, incidentally, is in some cases the mandated level of strengthening required in a retrofit). The internal steel frame would have had to be robustly tied into the perimeter walls. The URM piers on the North and West sides would have had to be restored to undamaged appearance, as it was their visual appearance to a large degree that gave the building its status (Figure 12). It was finally judged not to be economically viable to do this on the damaged building and within an acceptable timeframe.

This building highlights the issue facing private owners of buildings of national historical significance, who have limited resources and commercial interest, and have to make major decisions in a short time-frame when their buildings are damaged by a severe earthquake.

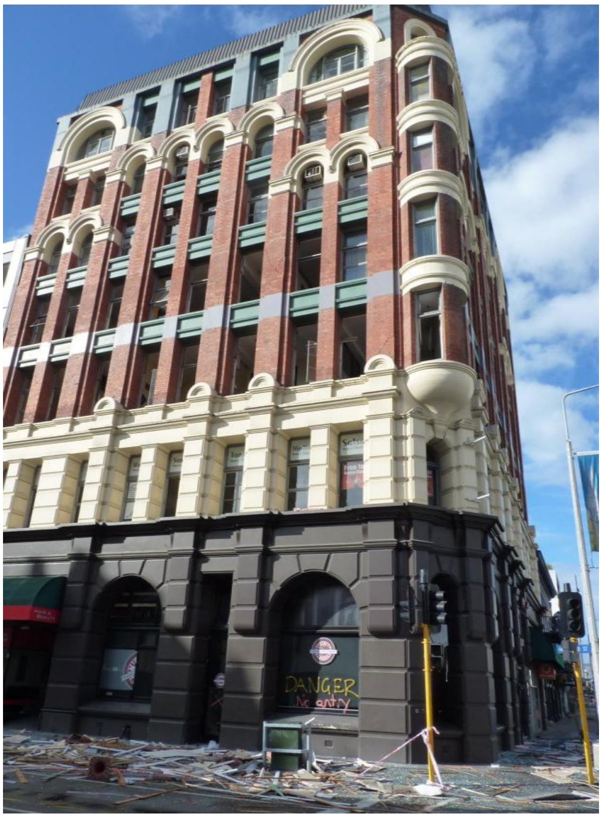

Figure 11: ***Elevation of heritage building on Manchester St and Hereford St, Christchurch (43^{0}31'56"S-172^{0}38'23"E) [Photo by M. Anagnostopoulou].***

Figure 12: ***Damaged front piers of heritage building on Manchester St and Hereford St, Christchurch [Photo by M. Anagnostopoulou].***

COLLAPSE OF STEEL TANKS

Many steel tanks collapsed during this earthquake. The failures are similar to that observed in the 1987 Edgecumbe earthquake in which the most significant damage was to thin walled tanks and silos. In most instances the failures are due to one or more of the following:

- Rotational or bearing failure of short columns supporting rigid tanks due to soft storey action;
- Failure of bracing units supporting the tanks;
- Tearing failure at the attachment of the supporting structural steel frame into the thin walled tank itself;
- Foundation instability due to each supporting column being on an individual pad footing with no interconnection between the plates.

A design document [5] produced by the New Zealand Earthquake Commission (1990) following that 1987 earthquake recommended proper base support and details for tanks and silos. Figure 13c shows a well performing silo with what appears to be a detail designed to that publication.

One interesting silo designed specifically for earthquake loading is Holcim's 2,000 tonne silo at Lyttelton. It is a steel silo on an 8 sided steel base. It is 9.8 metres in diameter and stands over 25 metres high. The silo was originally built in 1969, and was located at Port Otago. It was relocated from Dunedin to Lyttelton in 1990. At the time design earthquake loads in Christchurch were 25% higher than those in Dunedin, so the base had to be modified to resist the higher loads. This was done by cutting away parts of the existing diagonal braces to create a yielding tension brace system as shown in Figure 15. The remaining elements of the silo base, the ring beam and the bottom part of the silo barrel were strengthened to resist the over-strength capacity of the yielding tension braces. When the earthquake struck, the silo was completely full of cement. All of the tension braces yielded at their necked-down sections as intended, but there was no other damage to the silo or its base. The yielded tension braces are now in the process of being cut out and replaced. Information on this has been provided by Mike Fletcher with kind permission by Holcim.

(a)

(b)

(c)

Figure 13: ***Farm tanks in Darfield; (a) Example collapse [Photo by M. Anagnostopoulou]; (b) Close-up view [Photo by M. Bruneau]; (c) Tank with continuous strip footing [Photo by M. Anagnostopoulou].***

(a)

(b)

Figure 14: ***Farm tanks in Darfield; (a) Tanks with brace legs; (b) Close-up view of collapse [Photos by M. Anagnostopoulou].***

LIGHT STEEL FRAMED HOUSING

Timber framing has been traditionally used for housing in New Zealand. However, the use of light steel frames for housing is a growing industry. Most are typically clad with brick veneer, consisting of 70 mm thick bricks supported laterally by the steel frame.

There were approximately 40 houses built with light steel frames in the strongly shaken region. They all performed well where the underlying ground remained stable, with the worst reported damage in these cases being hairline cracks in the gypsum board lining of some internal walls. In one instance, a brick of the external cladding became loose, as shown in Figure 16.

A few houses situated where the ground slumped and spread underneath the house suffered greater damage.

CONCLUSIONS

Steel structures generally performed well during the Darfield earthquake of September 4, 2010, with some minor exceptions. A few slender bars in concentrically braced frames fractured, some links in eccentrically braced frames exhibited slight yielded (as expected), a few steel tanks of older vintage collapsed (others performed well), and steel houses performed comparably to other residential dwellings. However, given that much of this satisfactory performance is attributable to the fact that seismic demands from this earthquake were generally lower than considered in their design, caution is warranted against overconfidence, as future earthquakes pushing structures to their design level will better test contemporary seismic design requirements for steel structures of all types.

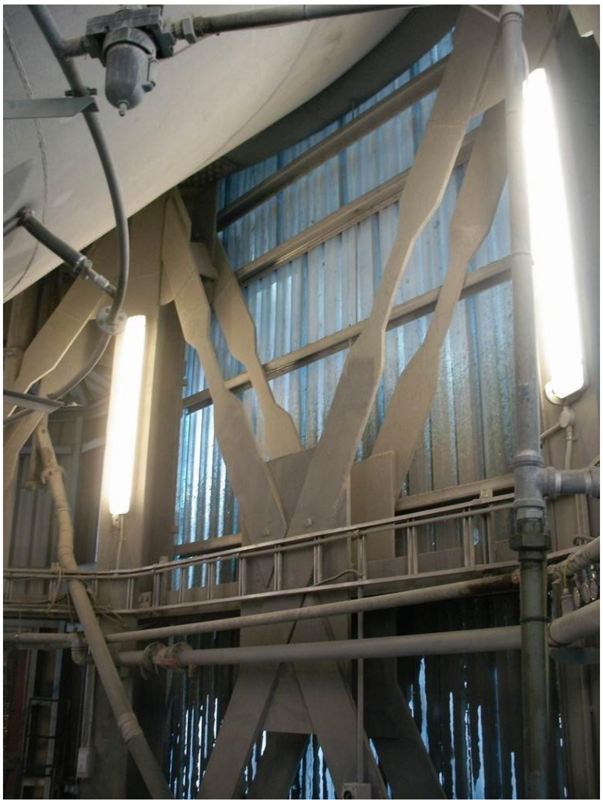

Figure 15: ***2,000 tonne capacity cement Silo at Holcim Depot, Lyttleton, with deformed dogbone braces [Photo by Buchanan and Fletcher].***

Figure 16: ***Light steel frame house from epicentral region [Photo Courtesy of Graham Rundle].***

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