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LESSONS FROM STEEL STRUCTURES IN CHRISTCHURCH EARTHQUAKES

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Abstract. *Lessons learned from the 2010/2011 Christchurch earthquake sequence about the behaviour of steel structures are described. Firstly, observed performance of steel structures is summarised. It is shown that many steel structures had very little damage. However, some structures suffered damage as a result of large foundation settlements. Yielding, buckling and fractures were observed in steel bridges and buildings. Reasons for observed damage are then described in the light of recent studies. It is shown that because of the lesser damage to steel structures and greater uncertainty over repair of reinforced concrete structures, that steel structures have become popular in the Christchurch rebuild. A number of these use low-damage systems.*

1 INTRODUCTION

Christchurch, a city of approximately 400,000 people and New Zealand's second largest city, is regarded as being in a zone of moderate seismicity. Prior to the 2010/2011 earthquake series, it had a seismic zone coefficient of 0.22, representing the expected peak ground acceleration in a 500 year return period from a uniform risk seismic model of New Zealand, compared to 0.40 for the capital city, Wellington. In 2010/2011 it was struck by a series of 6 damaging earthquakes, including one generating the strongest recorded peak ground accelerations worldwide. The first shaking event in the sequence was a magnitude 7.1 surface rupture with an epicentre approximately 40km west of Christchurch occurring on 4 September 2010. It generated PGA of between 0.12g and 0.18g in the Christchurch CBD and caused predominantly non-structural damage, damage to unreinforced brick structures, and no-one was killed. The third and most intense major event affecting Christchurch was on 22 February 2011. While it was a smaller magnitude 6.3 event, it was 5km from Christchurch, 5km deep and the energy of this blind thrust fault event was directed toward Christchurch city. It caused peak ground accelerations significantly greater than those in the M7.1 event, in the CBD as shown in Figure 1. Measured peak ground accelerations within central Christchurch were high as 0.6g, and significantly greater values were obtained close to the epicentre (MacRae, 2013) [1]). In addition to these two events, the city has been rocked by many other aftershocks, as shown in Figure 2. Response spectra from the September and February quakes at a site in central Christchurch are shown in Figure 3.

Ductile structures of normal importance in NZ are currently explicitly designed for 0.70 times the 500 year earthquake. This corresponds approximately to the 150 year earthquake and such design levels have been used since about 1982. Most significant steel structures in Christchurch were been built after this date. The level of shaking experienced was over 2 times the 500 year event, or 2.8 times the level of shaking they were explicitly designed for (0.7 times the 500 year event).

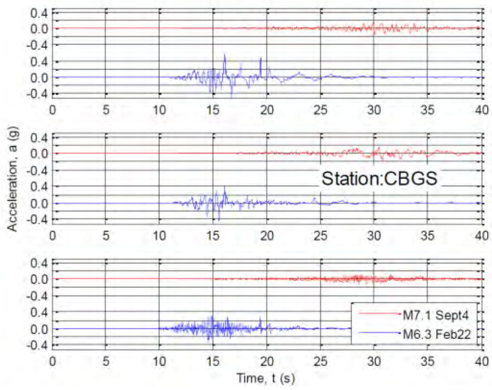


Figure 1. Christchurch Botanical Gardens records (NS, WE and Vertical components) (Geonet, 2012 [2])

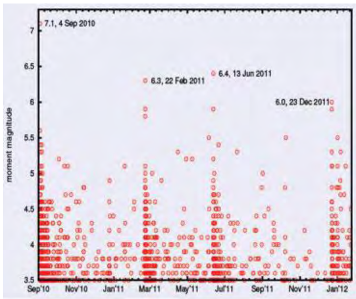
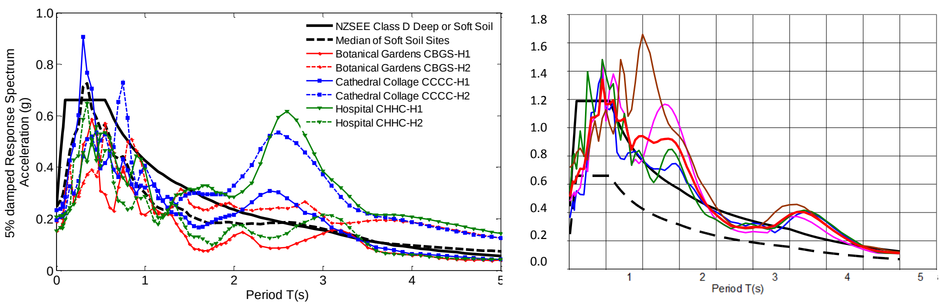


Figure 2. Canterbury NZ Aftershock Sequence - Sept 2010 – January 2012 (Gavin, 2012 [3])



(a) September 2010 (b) February 2011

Figure 3. Central Christchurch 5% damping response spectra
(from Brendon Bradley, U. Canterbury, New Zealand [4])

The multi-storey steel structures that experienced this earthquake series included moment resisting frames (MRFs), eccentrically braced frames (EBFs) and concentrically braced frames (CBFs) ranging in height from 2 to 22 storeys. Many were able to be reused shortly after the shaking, but a few notable structures sustained damage as a result of foundation settlement, poor detailing, or other reasons. In addition there were some 50 modern light steel framed houses from 1 to 3 storeys in the strongly shaken regions, most of which suffered no to little damage.

Following these earthquakes, detailed studies were initiated both in New Zealand and the USA to (i) understand the reasons for damage, (ii) quantify the damage extent, and (iii) develop repair methods.

The lack of obvious damage in many buildings is likely due to both the high strength/stiffness ratio of steel, soil-structure effects, and structures possessing extra stiffness and strength than that considered explicitly in design. The extra stiffness/strength is predominantly as a result of concrete slab effects and the presence of vertical non-structural elements, such as interior walls and cladding. They can contribute as much as 50% of the total design force [5].

Steel yielding occurred in at least two of the events and possibly in up to six of them. Some significant fracture damage in some buildings was discovered seven months after the February 2011 shaking event. Recent studies at the Universities of Auckland, University of Canterbury, Holmes Solutions, University at Buffalo, and at the University of California Davis, have shown that both poor detailing and materials issues (e.g. low Charpy values) contributed to the fractures.

Building stakeholders are asking whether damaged building elements should be (a) left as is, (b) repaired somehow, or (c) replaced. To aid this decision, "remaining earthquake life" is being assessed. Non-destructive techniques, mainly based on change in material hardness, have been evaluated and compared with results of destructive testing. While there may be significant scatter in the results obtained, the technique is promising. In some cases, steel EBF link elements showing yield lines, but no significant buckling, were found to have used up more than 50% of their ultimate strain capacity as a result of both the construction process and the shaking events. A detailed procedure has been developed for assessment of the yielded shear links in EBFs and applied to the post-earthquake assessment of a 12 storey EBF framed building.

This paper provides a detailed description of the above issues from the international research/design/ construction team evaluating these steel structures. In addition, repairs implemented are described.

2 DAMAGE OBSERVED

A number of reports have been written regarding damage to steel structures (e.g. Bruneau et al. [6], (2010), Clifton et al. (2011) [7], Clifton et al. (2012) [8], Clifton et al. (2013) [9], Clifton (2013) [10], Gardiner et al. (2012) [11], Gardiner et al. (2013) [12]). The information is briefly summarized below:

(a) The shear component of the drift demands estimated using scuff marks on the stairs was generally significantly smaller than expected. For example, in HSBC tower, the peak shear drift was about 43% of that estimated from a model considering the average of the recorded ground motions in the area. This is likely to have been due to foundation effects, and increased stiffness due to the presence of slabs and non-structural elements not fully considered in the design. The slabs and non-structural elements are also likely to have contributed significantly to the strength.

(b) Buildings not subject to significant foundation deformations generally self-centred to within the initial construction tolerances of 0.2%.

(c) Tall steel buildings subject to aftershocks did not have residual displacements increase. For example, residual roof displacements of Pacific Tower of 60mm after the initial September shake reduced to 30mm after the February aftershock.

(d) Steel buildings were the first multi-storey buildings to be reinstated after the Canterbury earthquakes. For example, the 2009 12 storey HSBC Tower self-centred to a maximum residual drift of 0.14% following the 22 February 2011 earthquake and was returned to service in July, 2011. It had peak link plastic shear strain demands of about 5%. This was the first multi-storey building to be reused in the central building district.

(e) Column base hinging was not seen in any modern steel structures; in some buildings on unstable ground there was apparent yielding in the foundation system.
(f) Buildings with stiff lateral force resisting elements in the direction considered generally had less non-structural damage and contents disruption than more flexible buildings, or loading directions in which the building was more flexible.
(g) Proprietary connections to CBF braces generally behaved well but there were some fractures.
(h) The 22 February 2011 earthquake was the second worldwide, after the Nisqually earthquake, to push EBF systems into the inelastic range. The extent of this inelastic demand was greater in the Christchurch event including development of the full plastic mechanism associated with capacity design.
(i) EBF beam yielding was confined to the active link zone in properly detailed frames as expected.
(j) Composite floor slabs showed only minor cracking above EBF links. For example, over the 22 floor levels of Pacific Tower, the largest crack width was 1.5mm.
(k) In multi-storey frames (Clifton et al. 2011 [7]) fractures occurred in
   - an active link in Pacific Tower
   - welded I-section brace-to-beam connections not connecting directly (i.e. misfitting). Here the beam flanges did not connect directly to the beam at the web tension/compression stiffener locations in a parking structure
   - steel columns with inadequate anchorage into the floor system, and
   - brace-to-column connections in some concentrically braced framed systems
(l) In long span portal frame structures, portal frames and baseplates performed very well, typically with no structural damage (Clifton et al. 2011 [7]). The greatest cause of building damage was from ground instability, which led to subsequent bracing system failures in some instances and concrete external wall failures. Isolated out of plane failures of external wall panels occurred due to failures of the connections into the steel frames. Isolated examples of proprietary roof bracing system failure through fracture occurred, typically where the rods going into the holding unit were not bolted both sides and so were subject to severe impact loading during the earthquake as the braces slid back and forth in their holding units.
(m) The only severe fire in a multi-storey building occurred in a building that had suffered a complete structural collapse. This showed the effectiveness of modern detection and shut-off devices for gas and electricity.

3 FINDINGS FROM SUBSEQUENT STUDIES

(a) Foundation flexibility, from the connection, footing/foundation pad, and soil below may well have contributed to column base flexibility (Borzouie et al., 2013 [13]).
(b) The rotational flexibility of actual connections have a stiffness of around 70% to 80% of the NZS 3404 Clause 4.8.3.4.1 “fixed base” values of $1.67(EI/L)_{column}$ or around 90 to 140 kNm/mrad for typical columns (Clifton 2013) [10].
(c) Pile head rotational stiffness is typically 350 to 450 kNm/mrad (Pender 2012 [16]) resulting in a likely increase of rotation of at least 10 mrad elastic rotational flexibility at yield.
(d) Kanvinde (2012) [14] found that heavy baseplate base connections had a rotational stiffness of approximately $1.5(EI/L)_{column}$ and an elastic rotational limit of over 0.017rad.
(e) The post-earthquake capacity of some frames which had been subject to earthquake shaking has been found to be sufficient to meet close to 100% of the requirements of the new building standard without requiring significant further remediation, when considering the increased stiffness and strength from the earthquake induced yielding of the seismic resisting system. This is despite the new building standard for Christchurch having a design acceleration coefficient of 0.30, which had been increased from 0.22. [9]. An example is HSBC Tower, which has an overall evaluation of 87% NBS based on a detailed assessment [15], compared with 94% NBS evaluation for the pre-earthquake condition of the building against the current increased design

requirements. If one included the apparent increase in strength and stiffness from non structural components and soil foundation structure interaction this would further increase.

(f) Momtahan and Clifton (2015) [17], using a model for the out of plane strength and stiffness of floor slabs in EBFs, indicate that floor slab effects decreased peak and residual drifts of an EBF subject to a series of time history analyses to about 80% and 30% of that of the building without slabs respectively; especially when considered in conjunction with the benefits of soil foundation structure interaction.

(g) Imani and Bruneau (2013) [18] have shown that FEM could be used to estimate the initiation of link fracture in the EBF parking structure mentioned in item 2k above which had a misaligned brace to active link panel zone connection.

(h) Kanvinde et al. (2014) [19] through time history analyses to estimate the demands in the EBF parking structure, and fracture analyses to capture the capacity, indicate that under design level shaking, fracture would not have been expected even with the offset if the brace flange, while it was expected under the level of shaking experienced. They also state that even if the flange offset did not exist, the high level of shaking may have caused fracture in some connections even if the misalignment had not occurred.

(i) The Canterbury Earthquakes Royal Commission (CERC) required a potential of lack of redundancy in EBF systems to be addressed. Since most steel EBF systems comprise only two braced bays, separated in plan, in each principal direction there is less redundancy than in a multi-bay moment resisting system or an EBF with more braced bays in each principal direction.

   i. This concern has been simply addressed by mobilizing the gravity load carrying system contribution by requiring the columns of this system to be effectively continuous (MacRae, 2010 [20]) and ensuring they are all tied into the floor slab (Clifton and Cowie [21].

   ii. Evidence from Pacific Tower indicates that the lack of redundancy is not necessarily critical, were fracture occurred in one active link at the top of a 6 storey EBF which transitioned across to a main frame EBF running from level 6 to level 22. The influence of the fracture of a key link in this load path on the overall building behaviour was minimal, so much so that the fracture was not discovered until 6 months after the likely occurrence.. This also meant that there were only two EBF systems up the full height of the building in the North-South direction during the June 13 event; the second most intense of the earthquake series.

(j) Paton-Cole et al. (2011) [22] in shaking table tests on cold-formed steel framing houses with brick veneer representative of that of Wellington houses indicate no damage under 25% of the 500 year shaking, hairline cracking under 500 year shaking, no brick loss under 1.8 times the 500 year shaking, and minor brick loss under 2.9 times the 500 year shaking. Performance in the Christchurch earthquake sequence was consistent with this, where there was at worst minimal hairline cracking of plasterboard linings for houses on good ground.

(k) Steel framed buildings can be repaired by cutting out and replacing damaged components, even when the structural system was not designed or detailed with a repair procedure in mind. The replacement of 42 active links in Pacific Tower [12] is a good example of this

3 ASSESSMENT OF REMAINING LIFE OF DAMAGED STRUCTURES

For steel structures, success has been obtained in determining the level of damage in eccentrically braced frame active links by Nashid et al. (2013) [23, 24] using the field based hardness. This method takes into account the considerable variation of hardness readings through requirements for surface preparation of the surface to be hardness tested, where and how to take the measurements, how to establish a baseline for the unyielded material, the assessed loading regime etc. It has been applied to the assessment of a 2010 built 12 storey EBF structure in Christchurch to determine what yielded active links could be left in place [15]. It is only applicable to the webs of shear links that have yielded in a principally shear mode.

4 SEISMIC VULNERABILITY OF EXISTING STRUCTURES

In NZ building structures are considered to be "earthquake prone" if they are assessed to not be able to resist a shaking intensity of 33% of the new building standard considering likely properties. Local jurisdictions are required to develop bylaws about how these should be treated. Usually they are permitted to be used for only a limited amount of time. If they are not retrofitted to the minimum level, they must be vacated. The first step in this process is the assessment of the buildings as to whether or not they are earthquake prone. The result of the assessment has large economic and societal consequences and different groups have different ideas about this (MacRae, 2015 [25]). It is therefore important to get this assessment as right as possible.

An example of one structural form where there is significant discrepancy between standard calculations and observed performance is low-rise industrial steel portal frame structures. Some of these have been assessed to have strengths approximately 25% of that to the new building standard (nbs). However, of the many frames that went through the Christchurch earthquakes, where there was up to 200% of the NBS shaking, very few came near collapse and many suffered no damage at all! A number or reasons have been cited for this more than 800% difference between observed and computed capacity. These include the influence of soil structure interaction, "non-structural" elements contributing to the response, different end fixities, the lack of consideration of nonlinear elastic buckling response of the members, and conservative assumptions of engineers undertaking the studies. In a recent undergraduate study (Makwana and Nanayakkara, 2014 [26]) used the computer software ABAQUS to capture elastic and inelastic out-of-plane buckling deformations and considered likely rotational stiffness and explained the observed behaviour. Practitioners generally use simple frame analysis software for their assessments so cannot capture many important effects. This lack of consideration of the likely response results in possible unnecessary retrofit of many structures.

5 NEW BUILDINGS IN CHRISTCHURCH

Before the recent earthquakes, the majority of buildings in Christchurch have been of reinforced concrete construction. This is a result of the strong influence of Park and Paulay from the nearby University of Canterbury, the early development of standards for life safety for RC structures, the high quality and abundent aggregates available from nearby rivers and the resulting well developed construction infrastructure. While such structures generally provided life safety performance under levels of shaking more than twice that explicitly designed for, the damage incurred was such that the level of confidence that they could sustain similar design level events was low, and repair was difficult and expensive. For example, in some buildings only a single crack or a few large cracks occurred in the beams beside the columns in strong column weak beam MRFs. This was different from that observed in many experimental tests where there were a larger number of smaller cracks. The reason for this behaviour has been attributed to the presence of the slab, higher strength concrete than that assumed in design at 28 days, and dynamic monotonic (pulse type) deformation. While the remaining earthquake life of beams which had been subject to such deformations is difficult to know, was considered that in many cases the damage incurred may significantly compromise the building's performance in later events. With these large cracks, there was strain penetration of the main reinforcing bar into the nearby column. Repair of such joints was often regarded as being difficult. Furthermore, many buildings were insured and the insurance was to restore the structures to pre-quake levels. Other similar issues were found with wall structures. For all these reasons, many reinforced concrete systems in Christchurch have been demolished. Also, they are regarded as being difficult to repair and reinstate. For this reason steel structures have become popular in the Christchurch rebuild.

After the earthquakes, a wide variety of steel structures have been built and are continuing to be built. These include the traditional moment frame and eccentrically braced frame (EBF) structures. However, a number of new buildings are being used, some with low-damage, or easy reparability issues. These include buildings with buckling restrained braces, and EBFs with replaceable links. Rocking steel frames have also been used for a hospital, and friction dissipation is being used instead of yielding, especially in moment frame structures. Also, to obtain more open spaces, two way frames using concrete filled tubular columns and external diaphragms, are being used.

For some of these newer systems full design guidance is not yet available, so there is a lot of discussion between industry and the universities about the best way to do things. The rise in construction of these newer systems has also helped identify inadequacies in existing construction which also need to be addressed (MacRae, 2015 [25]). Collaboration with the technical societies, the standards body, universities, and the Ministry of Business Industry and Enterprise is underway to define and address these issues.

5 CONCLUSIONS

During the 2010-2011 Canterbury earthquake sequence buildings within Christchurch were subjected to very strong earthquake shaking. A number of lessons relating to the steel structures are:

1) Immediate reconnaissance after the Canterbury earthquake indicated very little damage, however various types of buckling and fracture occurred. The lack of damage may be a result of drift controlling the member sizes resulting in low expected ductility demands, and foundation/slab/non-structural element effects which meant the structure was twice as much as expected.
2) Subsequent studies indicate that:
   (i) soft-soil and foundation flexibility effects may have minimized the distress at the foundation,
   (ii) slab and non-structural element effects may have significantly decreased the response, and together with the likely member strengths mean that structures may be able to resist significantly greater levels of earthquake shaking than those for which they have been designed,
   (iii) hardness has the possibility of being used to estimate remaining earthquake life of a damaged member, and
   (iv) lack of redundancy may be able to be addressed using continuous columns, but a lack of redundancy does not always lead to collapse
3) Assessments of the behaviour of steel structures indicate that hardness has the possibility to be used to estimate the remaining earthquake life of some yielded elements.
4) Steel structures and steel-concrete composite structures have become systems of choice in the Christchurch rebuild. This is because of their reparability compared to reinforced concrete structures.

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