Lessons Learned for Steel Seismic Design from the 2010/2011 Canterbury Earthquake Series

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Abstract

The Canterbury earthquake series of 2010/2011 has turned the city of Christchurch into a full scale natural laboratory testing the structural and non-structural response of buildings under moderate to very severe earthquake shaking. The lessons learned from this, which have come at great cost socially and economically, are extremely valuable in increasing our understanding of whole building performance in severe earthquakes.

Given current initiatives underway on both sides of the Tasman towards developing joint Australasian steel and composite steel/concrete design and construction standards that would span a very wide range of geological conditions and seismic zones, these lessons are relevant to both countries.

This paper focusses on the performance of steel framed buildings in Christchurch city, with greatest emphasis on multi-storey buildings, but also covering single storey steel framed buildings and light steel framed housing. It addresses such issues as the magnitude and structural impact of the earthquake series, importance of good detailing, lack of observed column base hinging, the excellent performance of composite floors and it will briefly cover research underway to quantify some of these effects for use in design.

Keywords: earthquake, steel, building, performance

1. INTRODUCTION

Development of seismic design procedures involves establishing desired regimes of behaviour, experimental testing of critical components to establish their performance, development of design methods to generate the desired behaviour and validation of these methods through numerical time history analysis of structural models under suitably scaled earthquake records. Due to resource limitations, it is not feasible to construct buildings in compliance with these design procedures and test them under actual earthquake conditions. Lessons on building performance from severe earthquakes from other countries provide more information, but they don’t provide direct evidence of the adequacy of New Zealand’s seismic design procedures as the buildings impacted are not necessarily compliant with these procedures.

That is why the 2010/2011 Christchurch earthquake series has been so important to the advancement of seismic design in New Zealand – it has severely tested modern buildings, built to New Zealand design procedures, in a large natural laboratory. The nature of the earthquakes, being of Maximum Considered Event level but delivered in instalments, has allowed us to investigate the performance of the structures at stages throughout the earthquake series. The advantages go deeper than that, however. The Christchurch CBD is well instrumented with free field strong motion recorders, which record two perpendicular components of horizontal ground motion and the vertical component. Good records of all the major earthquakes in the series have been obtained. Most buildings in the Christchurch CBD have their principal axes oriented in the same direction as the horizontal components of the free-field strong motion recordings, meaning that it is possible to determine, for a given building, the likely free field strong ground motions in the two principal directions that were experienced and then to compare the structural response of the model with that of the real building. This process, as described by Storie (Storie 2013), allows a reasonable determination of the free field strong ground motion to be determined for the base of the building, to which the structure can be shaken numerically and the response of key parameters compared with those of the actual building. For buildings that exhibited stable, predictable inelastic response, that work is expected to quantify the influence of the soil-foundation-structure interaction (SFSI) on the response of the superstructure. This is topic is returned to in section 4.1.

The earthquake series comprised a series of high intensity events which are geologically much less likely in Australia. However, Australian seismic design provisions are covered by the same or similar suite of loadings and design standards. Therefore of interest in both jurisdictions is how the buildings performed in relation to the models and to the level of response designed for. These aspects will be the focus of a latter section of this paper.

This paper presents an overview of the lessons learned. It commences with a general overview of the Christchurch earthquake series, then goes into an overview of general building performance. This is followed by the principal section, dealing with multi-storey steel buildings. Following that is more brief coverage of lessons learned from long span single storey buildings, pallet racking systems, light steel framed houses and fire following earthquake. Conclusions and the reassessment of research priorities round out the paper.

2. THE 2010/2011 CHRISTCHURCH EARTHQUAKE SERIES

The Christchurch earthquake series from 4 September 2010 to 23 November 2011 comprised eight damaging earthquakes. Analyses of the comprehensive set of strong motion data conducted recorded shows that the 4 September shaking in central Christchurch was approximately 0.7 times the Ultimate Limit State (ULS) 500 year return period design level for Christchurch specified by the New Zealand seismic loading standard (NZS1170.5 2004) over the period range of 0.5 to 4 seconds, the 22 February shaking was 1.5 to > 2 times the ULS and the largest 13 June shaking was 0.9 times ULS. While the duration of short period strong shaking of each earthquake was short (around 10 to 15 seconds) the cumulative duration of strong shaking was over 60 seconds. The duration of long period strong shaking was longer. The magnitude and intensity of the damaging earthquakes is as given in Table 1.

As a result of the earthquake series, the seismic zone factor, Z, for Christchurch has been increased from 0.22 to 0.3; the comparisons above and in Table 1 relate to Z = 0.22. To this author, the rationale for raising the Z factor is not clear.

Table 1 Magnitude and intensity of the Christchurch 2010/2011 earthquake series

Event Date	Richter Magnitude	MM Magnitude[1]	≈ Fraction of DLE[2]
4 Sept 2010	7.1	7	0.6 to 0.7
26 Dec 2010	5.5	7 to 8	0.6
22 Feb 2011	6.3	9 to 10	1.8 to 2.5
6 June 2011	5.3	7 to 8	0.6
13 June 2011	5.4	7 to 8	0.6
13 June 2011	6.3	8 to 9	0.9
23 Dec 2011	5.5	6 to 7	0.6
Note 1: MM magnitude in the Christchurch CBD			
Note 2: DLE ≡ Ultimate limit state event to NZS 1170.5 with Z = 0.22 (the 2010 design value)			

3. OVERVIEW OF BUILDING PERFORMANCE

With the cumulative intensity being at maximum considered event level, the requirement of the New Zealand Building Code (NZBC 1992) and Earthquake Loadings Standard (NZS1170.5 2004) is that the buildings should remain standing, severe structural and non structural damage is expected in conventional ductile buildings and the building will probably require replacement. Almost all modern buildings met the robustness requirement and some were able to be rapidly returned to service.

The full spectrum of damage was observed, as illustrated in Figure 1

(a) Structural and claddings damage [J Ingham]

(b) Collapsed ceilings and contents [G Banks]

(c) Ground instability [M Pender]

(d) Landslides /slope instability [M Pender]

Figure 1 Illustrations of damage from the 22nd February 2011 earthquake

In general:

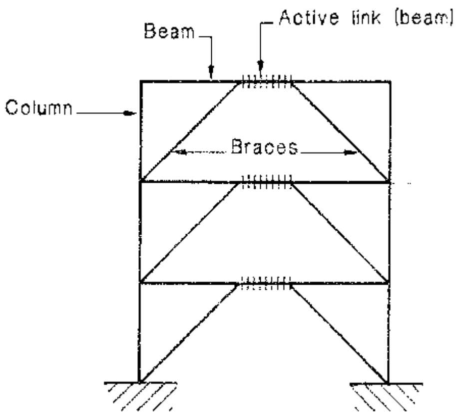
- Houses performed well for life safety, with light steel framed houses exhibiting the best performance in terms of cracking of internal wall linings and retention of brick veneer and roofing.
- Multi-storey steel framed buildings did not collapse; almost all self centered and were able to be returned to service with no to minimal structural repair and more extensive non-structural repair of cracks within stairwells and the like.
- Old buildings did not kill occupants but rather those outside
- Newer buildings that did collapse killed the most people
- Fire suppression systems worked extremely well

4. LESSONS LEARNED FROM MULTI-STOREY STEEL FRAMED BUILDINGS

4.1 Strength and stiffness; actual versus predicted

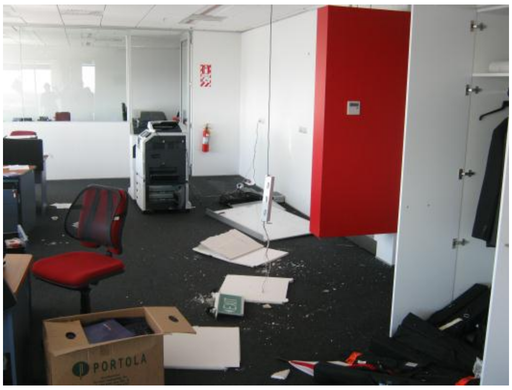
Modern, multi-storey steel framed buildings were designed to the requirements of capacity design, specified in (Feeney and Clifton 2001; NZS3404 1997/2001/2007). For example, eccentrically braced frame (EBF) systems are designed to concentrate damage into the active links (see Figure 2(b)) with inelastic demand suppressed in the remaining components. The 22 February 2011 earthquake was the first worldwide to push EBF systems into the inelastic range and their performance was as expected, with inelastic demand only in the active links and with only minor non-structural damage. An example is the 12 storey HSBC Tower, built in 2009, indicative photos of which are shown in Figure 2 (a, c, d). This building self centered to a maximum residual drift of 0.14% following the 22 February 2011 earthquake and was returned to service in July, 2011. Its post earthquake capacity was assessed in detail in 2012 in respect to the increased design seismic factor for Christchurch and the building was deemed to be capable of 100% New Building Strength, due principally to the demonstrated whole building strength and stiffness exhibited.

(a) Overall view of tower from North West Corner [M Bruneau]



(b) Member terminology for a V braced EBF [NZS 3404]

(c) Inelastic demand in EBF active link [C Clifton]



(d) Typical interior of office following 22 Feb 2011 earthquake [C Clifton]

Figure 2 HSBC Tower following the 22nd February 2011 earthquake

Because the pattern of inelastic demand in this building was as predicted and the peak inelastic demand during the earthquake in the north-south direction (the direction parallel to the external concrete wall shown in Figure 2 (a)) could be determined by scuff marks on the stairs, the ratio of actual building stiffness to predicted building stiffness could be established with reasonable accuracy. The predicted inelastic interstorey drift in accordance with (NZS1170.5 2004) under the design level event was 1.3%; the measured drift was ≈ 1% under 22 February earthquake, which was ≈ 1.8DLE as averaged from the 4 closest strong ground motion recording stations. This gives a ratio of actual building stiffness to model stiffness of ≈ 2.3.

The peak plastic strain in the EBF active links was approx. 7%, in the 5th level link in the East-West direction, shown in Figure 2 (c).This is less than 25% of the monotonic strain elongation capability of the steel.

Other steel framed buildings, such as the 22 storey Pacific Tower, showed similar ratios of actual building to model building strength.

4.2 Damage and disruption to non-structural components and to contents.

Following the 22nd February 2011 event, the author undertook inspections of multi-storey buildings with steel framing, concrete framing and with typically either moment-resisting or braced framed seismic-resisting systems. One item of particular interest was correlating the observed damage and disruption to non-structural components and contents to the flexibility of the building. The interest arises from questions as to the principal drives of such damage; is it floor accelerations which should be greater in a laterally stiff building, or lateral deformations, which will be greater in a flexible building. The author's conclusion, from observing the interiors of a range of buildings of similar height and materials of construction is that the extent of lateral movement is the largest driver of damage and disruption to non-structural components and contents. For example, Figure 1 (b) shows damage to a level 8 office in a flexible, perimeter moment framed building that underwent significant plastic hinging with an interstorey drift of ≈ 2.5%, while Figure 2 (d) shows damage to a level 8 office in HSBC tower, with measured interstorey drift of ≈ 1%. There is much less non-structural damage and contents disruption to the stiffer building.

4.3 Influence of composite floor slabs

When the EBF system deforms inelastically it pushes the floor slab out of plane, as shown in Figure 3. A composite floor system comprising concrete slab on steel deck on composite steel beams has a high out of plane resistance to this movement. This has been quantified through research into this system's performance in fire (Clifton et al. 2010). Not only was the HSBC building stiffer than expected (see section 4.1) but it had a post-earthquake residual drift of only 0.14%. This led to consideration that the out of plane resistance of the floor slab might be a significant source of this stiffness and unexpected ability to self-centre. An undergraduate study in 2011 (Mathieson 2011; Volynkin 2011) provided an initial quantification of these effects, based on simple yieldline theory, and showed that the floor slab decreased the peak lateral deflection of a hypothetical, 10 storey V-braced EBF designed to current New Zealand design practice (Feeney and Clifton 2001; NZS3404 1997/2001/2007) under a range of 10 representative earthquake records scaled to the ULS level by between 10 and 50% and the peak residual drift to less than 33% of that without the slab effect.

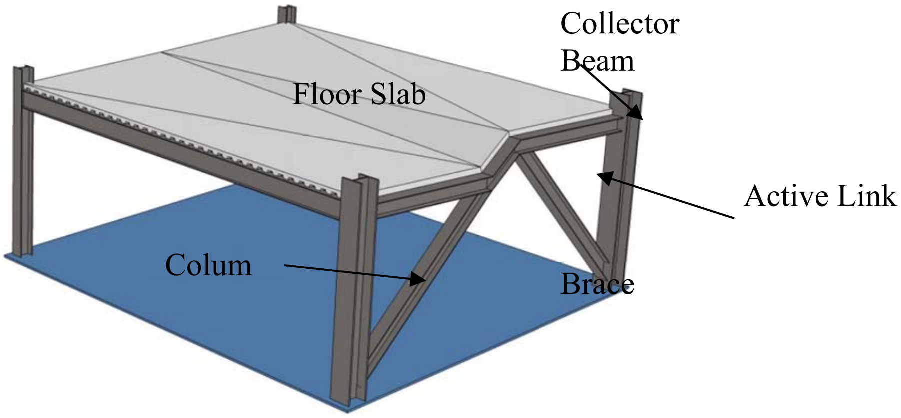


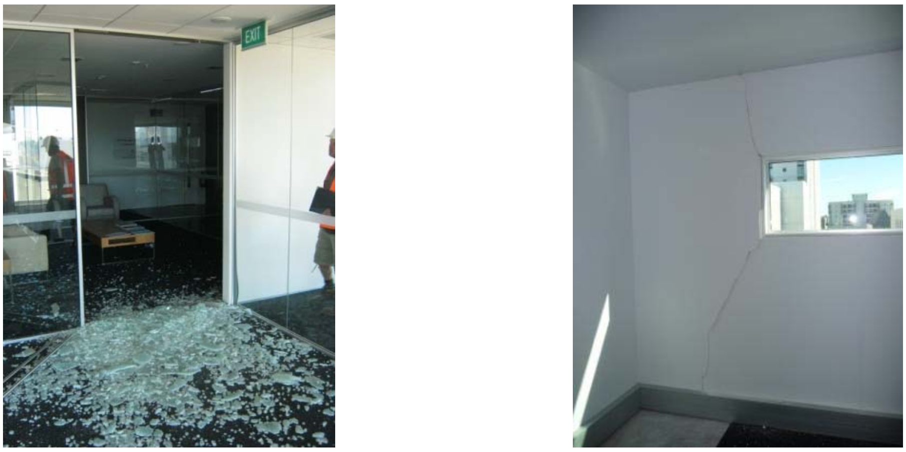
Figure 3 Floor slab contribution to EBF strength and stiffness

The significance of the floor slab to the strength and stiffness of EBFs is greater that found from previous researchers, such as (Riccles and Popov 1987), who concluded the contribution of the slab was an increase in the shear resistance of 8-12% of the shear capacity of the active link alone. However, the contribution of the slab may degrade over successive cycles of inelastic loading if pushed into the inelastic range and this is a key aspect for further research. In a study on the effect of a floor slab on the strength and stiffness of a stepping base concentrically braced frame (CBF) system, Wijanto (Wijanto 2010) determined that the inclusion of the floor slab out of plane properties increased the stiffness by a factor of 2 compared with modelling just as an in-plane diaphragm.

With regard to in plane strength, stiffness and diaphragm action, composite floor slabs performed very well. This is especially evident from the detailed floor slab survey of Pacific Tower, reported in (Clifton et al. 2012) which showed only minor cracking on any of the 22 floor levels, including at the two major transfer diaphragm levels and around the inelastically responding active links. The largest crack with was 1.5mm and approx. 30m length of cracking over 0.5mm width was repaired by epoxy grouting.

4.4 Effects of vertical acceleration

PGV levels in the February 22nd 2011 earthquake exceeded 1g within the CBD, which exceeded the PGH levels in that region. This very high level had only minor influence on steel framed buildings; in HSBC Tower, for example, it dislodged glass doors from their hinge supports (see Figure 4 (a)) and enhanced non-structural internal wall lining cracking in cantilevered window boxes out the building's north face (see Figure 2 (a) for location and Figure 4 (b) for details).



(a) Dislodged glass door [C Clifton}

(b) Cracking to internal linings of window-box [C Clifton}

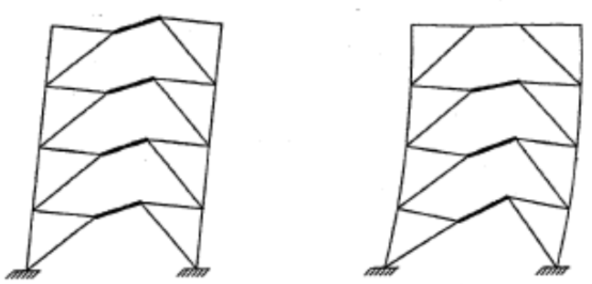
Figure 4 Effects of vertical acceleration on HSBC Tower

The long span floors in HSBC Tower also had a measured midspan deflection after the earthquake up to 4mm greater than before; which is considered due to a trampoline effect from the vertical acceleration causing minor negative rotation of the simple end connections with a residual permanent component at the end of shaking. Mapping of cracks in the Pacific Tower composite slabs (reported in (Clifton et al. 2012)) showed minor cracking over the supporting secondary beams that could also have been due to vertical movements (or could have been pre-existing shrinkage and creep cracking).

Vertical ground accelerations were more serious in buildings with reinforced concrete transfer beams, leading to shear failures near the supports and bearing failures over the supports. They also are considered a contributing factor to compression failure of some reinforced concrete shear walls (Kam et al. 2011) , with detailed investigations just getting underway to quantify this influence further.

4.5 Adequacy of the capacity design procedure

The capacity design procedure for steel seismic-resisting systems is based on the structure being displaced laterally so that yielding hinges form in all the primary seismic resisting system elements to give a yielding mechanism ((Feeney and Clifton 2001; NZS3404 1997/2001/2007). For eccentrically braced frames, the yielding mechanism means the EBF forms a plastic collapse mechanism, with yielding in each active link and in theory at the column bases, if these are sufficiently rigid (more on this in section 4.6). The assumed plastic collapse mechanism is shown in Figure 5



(a) Assumed rigid plastic shape

(b) Actual inelastic response under extreme earthquake

Figure 5 EBF plastic collapse mechanisms

This plastic collapse mechanism assumes uniform inelastic demand in each active link, meaning that the collector beams at a given level and the braces framing up into that level are designed for the overstrength actions from the brace. The columns are designed for the cumulative overstrength actions at and above the storey under consideration.

The capacity design derived actions based on overstrength can therefore become very large, especially on the columns, and so upper limit design actions are specified by NZS 3404 on the secondary elements of the seismic resisting system (the braces, collector beams and columns). Considerable debate at Standards Committee level went into the determination of the upper limit actions in NZS 3404 (Clause 12.3.3.4) in each edition of the Standard. The latest provisions, introduced in the 2007 amendment, base the upper limit actions on those from analysis for elastic response ($\mu = 1.0$) for the actual displacement ductility factor, $\mu_{act} \leq 1.8$ and for nominally ductile response ($\mu = 1.25$) for the actual displacement ductility factor, $\mu_{act} > 1.8$. In the case of the HSBC Tower, $\mu_{act} \approx 2.2$, the upper limit actions were typically greater than the overstrength derived actions and so the design actions were based on the overstrength actions. In the case of the 22 storey Pacific Tower, $\mu_{act} \approx 1.5$ and the upper limit actions governed the design of the columns and some of the braces and collector beams.

In the 22 February 2011 earthquake, with no exceptions, inelastic action was confined to the active links in those two buildings. In HSBC Tower, the demand was similar up all levels, meaning the inelastic shape approximated that in Figure 5 (a). In Pacific Tower, inelastic demand concentrated into the lower 8 storeys of the building, due significantly to the non-structural contribution of the numerous full height fire and acoustic rated walls in the upper 14 storeys, which comprise hotel rooms and apartments. In all the other steel framed buildings surveyed in the Christchurch CBD, inelastic demand was limited to the primary seismic-resisting system elements, even in one 7 storey perimeter moment-resisting steel framed (PMRSF) building that was severely impacted by differential ground instability, with differential settlement of over 100mm between the central gravity system and the external PMRSFs. Given the range of buildings impacted comprised MRFs, EBFs and CBFs, ranging from 3 to 22 storeys in height and with the capacity design derived design actions governed by overstrength in some cases and upper limit actions in others, this outcome supports the adequacy of the current capacity design procedure.

4.6 Column base fixity

Figure 5 shows the expected inelastic shapes generated by a severe earthquake; (a) the ideal shape and (b) the actual shape under extreme response. As described in section 4.5, both examples were generated in the 22 February 2011 earthquake. In theory, if the column bases are fixed, this requires column base hinging. However, in practice, none was observed in any of the steel framed buildings investigated in Christchurch and none has been reported in any multi-storey steel framed building sited on stable ground.

This raises the question as to why not? The answer to this must lie in the elastic rotational flexibility of the nominally fixed base details.

NZS 3404 Clause 4.8.3.4.1 requires a "fixed base" to have an upper limit rotational stiffness of $1.67(EI/L)_{column}$ which translates to around 90 to 140 kNm/mrad for typical column sizes. Studies of the rotational flexibility of actual connections, being undertaken for SCNZ, show that the rotational stiffness of the recommended moment resisting column base detail into a concrete pad has a stiffness of around 70% to 80% of this value.

Experimental testing on heavy baseplates shows they have an elastic rotational limit of over 17 mrad and a rotational stiffness of approx $1.5(EI/L)_{column}$ (Kanvinde 2012).

First principles considerations of moment resisting column baseplate flexibility onto a rigid concrete base undertaken for SCNZ show (Clifton 2013) the potential to develop up to 20 mrad of elastic rotation in the column base system through elastic squashing of the concrete on the compression side, elongation of the hold down bars on the tension side and a small contribution from baseplate flexibility. It also points to a design approach around commencing with a column base rotational stiffness of $1.0(\mathrm{EI/L})_{\mathrm{column}}$ for analysis, then designing the baseplate and adjusting the length of the hold down bolts to achieve this target stiffness. The moment capacity at a target drift of 1.2 to 1.5% would then be determined to ensure it is less than the column base moment capacity reduced as required by axial load. This procedure has been included in the University of Auckland course Civil 714: Multi-storey Building Design and is being used on a 4 storey moment-resisting steel framed building under final design at the time of writing this paper.

Columns rigidly connected into piles also showed no evidence of column base hinging in the earthquake series. Determination of pile head rotational stiffness using (Pender 2012) show typical values of 350 to 450 kNm/mrad. This would impart between 5 and 10 mrad of elastic rotation to a column connected to a pile, with more effective lateral flexibility due to differential vertical movement. Thus it is likely that a pile based foundation system for an EBF or CBF would develop at least 10 mrad elastic rotational flexibility, allowing the observed inelastic displacements in the superstructure to develop without requiring column base hinging. One way of making a targeted foundation rotational stiffness practicable is through the use of Belleville Springs (eg (Solon_Manufacturing_Co 2013) which allow the flexibility on the tension side of the column baseplate system to be adjusted to meet the target rotational stiffness.

4.7 Attention to load path

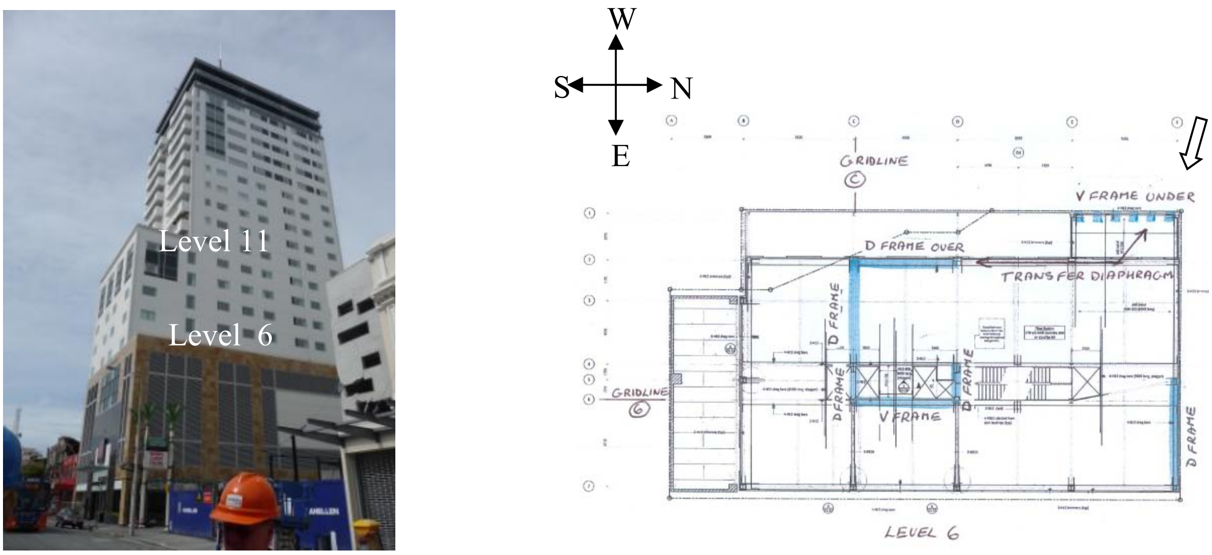
While most steel structures performed very well, there were some failures of components. With one exception, being an active link fracture in Pacific Tower reported in previous papers (Clifton et al. 2011; Clifton et al. 2012), observed failures were due to lack of adequate load path, due to one or more of:

- rigid welded I section connections misaligning to the web tension/compression stiffeners
- inadequate anchorage of steel columns into the floor system and of tension braces to columns in some concentrically braced framed systems

Most of these observed failures are described in (Clifton et al. 2011).

4.8 Redundancy

Most steel EBF systems comprise only two braced bays, separated in plan, in each principal direction. The rest of the structure is designed to directly support gravity loading only and provide the required flexibility to sustain the earthquake induced displacements. This provides less redundancy than a multi-bay moment resisting system or an EBF with three or more braced bays in each principal direction. The potential issues arising from this lack of redundancy in EBF systems was specifically mentioned by the Canterbury Earthquakes Royal Commission (CERC) as an issue to be addressed. This has been done very simply by mobilizing the contribution of the gravity load carrying system through requiring the columns of this system to be effectively continuous and ensuring they are all tied into the floor slab.



(a) View from North West corner (b) floor plan level 6

Figure 6 Pacific Tower [Photo by C Clifton, Floor Plans from S Gardiner] (the arrows in (b) show the direction of the view in (a)

However, one critical piece of evidence from Christchurch suggests that, at least in steel framed buildings with composite floors, this lack of redundancy is not critical. Figure 6 shows details of Pacific Tower. In the North-South direction, there are only two EBF systems up the full height of the building. On level 6, shown in Figure 6(b), the V frame under the NW corner stops and the EBF transfers to the D frame half way along the building for levels 7 to 22. The top link in the NW corner EBF fractured, either in the February 22nd 2011 earthquake or in the most intense earthquake of 13 June, partly due to use of steel with less than the specified Charpy Impact energy. In principle, this broke the continuity of the seismic-resisting system on the west side of the building and should have generated high torsional actions especially around the 6th level. Evidence from non-structural linings cracking showed a very slight increase in inelastic demand in the 6th and 7th storeys, however there was no evidence of enhanced torsional movement. The floor slab on level 6 functioned as an effective transfer diaphragm. This building was repaired and returned to service in May 2013.

5. LONG SPAN STEEL PORTAL FRAME BUILDINGS

These are extensively reported in (Clifton et al. 2011) and due to lack of space details are not presented herein. The key points in regard to these buildings were that:

1. The portal frames and baseplates performed very well, typically with no structural damage
2. 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

3. Isolated out of plane failures of external wall panels occurred due to failures of the connections into the steel frames
4. 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.

6. LIGHT STEEL FRAMED HOUSES

There were around 50 light steel framed houses in the strongly shaken areas. All were new construction, having been built within the last 10 to 15 years.

Typically, they comprised light framed systems of one or two storeys on concrete slab on grade, with particle board or ply second storey floor and with long run steel or pressed tile roofing and brick veneer.

The seismic performance of brick veneer onto steel framing was extensively tested at the University of Melbourne in 2009 (Paton-Cole et al. 2011), through performance of a representative system designed for Wellington seismicity on shaking table. Earthquake intensities ranged from serviceability event (SLS1) level to 1.6xmaximum considered event (MCE) level, applied through the scaled 1940 El Centro record. System performance was excellent; no damage under SLS, hairline cracking under DLE, no brick loss under MCE and finally minor brick loss at 1.6xMCE (PGA_H = 0.95g).

Performance in the Christchurch earthquake series was consistent with this; no damage to minimal hairline cracking of plasterboard linings for houses on good ground. The most outstanding example of LSF performance was a two storey light steel framed house with Oamaru stone cladding, situated very close to the epicenter of the 22 February 2011 earthquake and to a strong motion station that recorded PGA_H and PGA_V = 1.8g in that event (a higher PGA_V was recorded at another station, but that figure is suspect to falling material in the vicinity of the recorder). Figure 7 shows two views of this house, showing the movement of the Oamaru stone on its bedding planes. There was also minor cracking to some internal wall linings. Both were

Figure 7 Two storey light steel frame house with Oamaru stone cladding [T Just]

readily repaired. With the Oamaru stone units being 4 times the weight of a standard clay brick and the acceleration demand being double that of the shaking table experiments, this was a much more severe test of the light steel framed house system and the most demanding example studied from Christchurch.

7. FIRE FOLLOWING EARTHQUAKE

Fire following earthquake is a well documented event. For example, the Kobe earthquake of 1995 killed some 5,500 people and fire razed over 10 hectares of the city. Observations of the cause of fire start and fire spread following earthquake have led to the following recommendations (Spearpoint 2008) on how to reduce the probability of major loss in fire following earthquake:

- Providing robust and reliable earthquake shut-off systems for electricity and gas and ensuring they are well maintained
- Provision of adequate earthquake resistance and adequate fire protection especially fire separations for all buildings
- Active and passive systems to be provided with earthquake resistance
- Building earthquake resistance into water supplies within cities and buildings
- Seismic restraint of potential ignition items and liquid fuels
- Reliability of stairs and escape routes for both earthquake loading and fire safety
- Earthquake resistant fire stations and communications facilities
- Co-ordinated local government and Fire Service planning for hazard assessment of essential lifeline and emergency response
- Avoiding electrical fires by ensuring that water supplies are restored before electricity is turned back on

In the Christchurch earthquake series of 2010/2011 there was only one example of severe fire in a multi-storey building following the earthquakes and that was in a building that had suffered a complete structural collapse. The most severe of the earthquakes, on 22nd February 2011, occurred at 12.51pm, ie at peak lunch time and the most vulnerable time for fire following earthquake to occur in a New Zealand city centre. The difference between Christchurch and Napier following the 1931 earthquake is especially pronounced. In both instances the earthquake struck in the middle of the working day and more late 19th century and early 20th century buildings collapsed in the 22nd February earthquake in Christchurch then in Napier in February 1931. While the Napier CBD was devastated by fire, the Christchurch CBD was not. This showed the effectiveness of the modern detection and shut-off devices for gas and electricity and also potentially the benefits of improved health and safety working practices in business prone to fire following earthquake.

8. CONCLUSIONS

The key conclusions are:

1. The Christchurch earthquake series was maximum considered event level for the CBD/City, due to the peak intensity of shaking in the strongest earthquake and the cumulative duration of strong shaking from the 7 damaging earthquakes
2. Well designed and detailed buildings performed well and were typically over 2 times stiffer and stronger than predicted
3. The capacity design procedure for steel framed seismic-resisting systems worked well in directing inelastic demand into specified parts of the structure and suppressing it in other parts
4. The current design and detailing provisions require no major changes for buildings designed and detailed as they were in the earthquake affected region;

column base stiffness should be more realistically modelled and design based on these remaining elastic under the design level ultimate limit state event
5. Composite slabs delivered high in-plane strength and ductility and delivered out-of-plane stiffness to contribute to increased strength and self centering capability
6. Fire suppression systems worked very well in buildings that did not collapse
7. 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

9. REASSESSING RESEARCH NEEDS AND PRIORITIES FOR STEEL STRUCTURES IN LIGHT OF THE CHRISTCHURCH EARTHQUAKE SERIES

There is a saying amongst earthquake engineers that the real test of design procedures and systems is in the field. The Christchurch earthquake series has provided a severe test of these procedures and systems and offered an unprecedented opportunity for seismic researchers to advance their understanding of whole building behaviour under severe earthquakes. It has also shifted the focus for new buildings from ductile solutions to low damage solutions, with all the opportunities and challenges of this new and demanding area.

In the reinforced concrete area, research into the performance of shear walls and moment frames is commencing to determine why the yielding regions showed only a few large cracks instead of a distributed network of fine cracks. Initial indications are that this has caused high localised strain demand in the reinforcement, reducing the threshold for repair of damaged reinforced concrete structures. Quantifying the strain demand and developing a dependable repair mechanism are high priority research topics.

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