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RECONSTRUCTING CHRISTCHURCH: A Seismic Shift in Building Structural Systems

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Reconstructing Christchurch: A Seismic Shift in Building Structural Systems

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Bruneau has received several national awards and recognitions for his work and has been inducted as a fellow of the Canadian Academy of Engineering. He is also an ASCE Fellow, and a member of various AISC and CSA committees developing design specifications for bridges and buildings. He has conducted numerous reconnaissance visits to disaster-stricken areas and has served as Director of MCEER. He has also participated in various expert peer-review panels, project-advisory committees, and special project design teams. A detailed outline of qualifications is available at www.eng.buffalo.edu/~bruneau.



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Gregory MacRae works at the University of Canterbury in Christchurch, New Zealand. He has lived through the earthquakes and the Christchurch rebuild. After working in a consulting company, he has spent 30 years conducting teaching and research related to structural and earthquake engineering in Japan, the USA, India and New Zealand. Key areas where he has initiated, or worked with others initiating, thought change that has been recognised in standards and practice include: steel structure design, structural earthquake stability and the residual displacement estimation, PREstressed Structural Seismic Systems (PRESSSS), displacement-based design, the continuous column concept, frame stability, and friction connections. In particular, his current work focuses on low-damage construction, decision support tools, steel structures and structural dynamics. Results from his studies, disseminated in publications and keynote talks in various countries, have influenced the construction of many structures in New Zealand, Japan and the USA.

He is currently head of the Structural Engineering Cluster at the University of Canterbury, the New Zealand representative to the International Association of Earthquake Engineering, a member of standards committees related to earthquake and structural steel design, and a Quake Centre board member. Among other activities, he has been an associate editor for the American Society of Civil Engineering (ASCE) Journal of Structural Engineering, the chair of the ASCE Seismic Effects Committee, the founder of the Earthquake Clearinghouse after the initial 2010 Canterbury Earthquake, and an author for the Royal Commission on the Canterbury Earthquakes. He has been a director of the board of the World Seismic Safety Initiative (WSSI) and a senior advisor to the board. Further information is available at <https://sites.google.com/site/gregoryamacrae/home>.

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Executive Summary

After the 2010–2011 Canterbury earthquakes, much of the Christchurch Central Business District (CBD) was demolished, and a new city has emerged in its place. This document describes a study conducted to (a) quantify the extent to which various types of structural system have been used in the new buildings constructed by early 2017, and (b) identify some of the drivers that have influenced decisions about the selection of structural material and specific structural systems used.

The study involved a series of interviews with the structural designers of more than 60% of the post-earthquake buildings constructed to date in Christchurch's CBD (i.e., 74 buildings), as well as with engineers from Wellington and Auckland, an architect, a project manager, and a developer. Data was also collected from various sources (including Christchurch's City Council database), and quantitative information on structural forms and decision drivers has been assembled for the 74 buildings considered.

Major findings are that:

- While before the earthquakes almost all buildings in the Christchurch CBD and Addington areas had reinforced concrete (RC) frames or walls as their structural systems, in the rebuilding of Christchurch that has taken place since 2011, the number of buildings with steel, RC, and timber lateral-force-resisting systems has been in the ratio of approximately 10:10:1. However, the floor-area ratios of the same buildings with steel, RC, and timber lateral-force-resisting systems is about 79:20:1, because the steel systems tend to have been used in larger structures. Furthermore, for the above RC buildings, the internal gravity frames have been found to be of structural steel three-quarters of the time.
- Concrete structures in the rebuild were nearly all structural wall systems. Exceptions encountered were (i) a base-isolated building where RC moment frames were used in one of the building's orthogonal directions, and (ii) a building where rocking RC walls were used in one direction.
- Steel buildings have been constructed using a variety of lateral-load-resisting systems. The most frequently used systems, by decreasing numbers of buildings in which they have been implemented, are: buckling restrained braced (BRB) frames, traditional moment-resisting frames (MRFs), MRFs with reduced beam sections, eccentrically braced frames (EBFs) with replaceable links, concentrically braced frames (CBFs), traditional EBFs, rocking steel frame systems, and MRFs with friction connections. Most new base-isolated buildings are supporting either steel MRFs or CBFs. When considering only non-base-isolated buildings, BRB frames have been used in buildings making up nearly 40% of the total new constructed floor area.

- The most common timber frames consisted of laminated veneer lumber, used in approximately 3% of the buildings.
- Of the 74 buildings considered, 9% of buildings used hybrid systems, 14% were base isolated, and 3% used viscous dampers.
- Beyond increased demands for both serviceability and design level earthquakes implemented in design standards following the earthquakes, there has been no mandatory requirement to use more resilient structural systems as part of the Christchurch rebuild. Nonetheless, many engineers and owners were aware of the benefits of more resilient systems and this has generally been part of the discussions in deciding the structural forms. Many engineers stated that they also designed their buildings to have design level earthquake drifts much less than the maximums permitted in the standards. This was considered to limit structural and non-structural damage.
- The decision about which structural system to use for each specific building depends on many factors, including the person making the decision. From the survey conducted, it was found that the engineer chose the structural systems in the majority of the cases. This was followed by the owner requesting lowest cost, the owner selecting a “low damage” solution, and the owner requesting an IL3 building. While the structural engineer has a significant say, it became clear from the interviews that the decision about the system is made as part of a group that includes the client, the architect, and other parties (i.e., project manager, quantity surveyor, etc.), and that considerations of cost, construction speed, perceptions of structural performance and building post-event operation, tenants’ desires, engineering culture, time since the last nearby earthquake, cash flow of the client, and other factors are also significant. The decision varies by location throughout the country and is also affected by the local availability of construction skills, access to resources, and the strengths of relationships. As such, while some structural forms are more common than others, there is no single dominant form throughout the country. It is also worth noting that while Christchurch’s widespread insurance coverage has permitted the rebuild, with over NZ \$40 billion being reinvested in Canterbury, the insurance industry does not seem to be having a significant influence on the types of structural systems used in the rebuild.
- The shift towards steel structures was attributed to a combination of factors. These include the NZ legislative framework, which allows new systems; the perceptions of low damage and reparability of steel structures after the Canterbury earthquake sequence; the low price of steel compared to several years before; the fast erection speed of structural steel; the availability of economical flooring systems that performed well and are compatible with steel buildings; the advent of

economical methods to design for fire; the poor soil conditions in Christchurch, giving an economic advantage to light structures; the advent of “low-damage technologies” (defined below), which are easy to connect to steel structures; the availability of some systems with known strength (e.g. BRB systems) and therefore little section overstrength allowing economical design; and perceptions about performance and procurement issues with structural systems using some other materials.

Furthermore, on the basis of the above findings and discussions with those interviewed, the following key points can be drawn:

It is becoming a more widely held belief that preventing loss of life as a seismic performance objective is simply not sufficient for a good modern structure

While all structural systems designed according to modern New Zealand standards are believed by engineers, architects, clients, and other stakeholders in the construction industry to meet their design target of preventing loss of life during an earthquake, the industry (without governmental intervention) has generally moved away from code-compliant systems that will undergo high ductility demand, develop high displacement/drift (creating significant damage to the structure and non-structural elements), exhibit damage mechanisms not considered directly in most standard frame analyses (such as beam elongation effects that produce floor damage), and are difficult to inspect/repair/reinstate after a major event. For that reason, RC moment frames as lateral-force-resisting systems, which were ubiquitous before the earthquakes, are practically non-existent in the CBD rebuild. Note that the only RC moment frame encountered in this study of the rebuild was supported on a base-isolated structure and was expected to sustain only low ductility demands. It will be interesting to track whether this practice will last as the effects of the earthquakes progressively become less vivid in the collective memory.

Structural engineers’ professional opinions impact the adoption of low-damage systems

In New Zealand, structural systems that are specifically designed to limit seismic damage in structures and that do not need to be fully replaced immediately after a major event have been termed “low-damage technologies/structures”. Not all low-damage systems are equal in terms of construction cost, expected performance (structural and non-structural), post-event inspection requirements/costs, or post-event reinstatement requirements/costs. These performance and cost issues relate to the whole building (including structural and non-structural effects) for continued occupancy and use. There are differences in professional opinions regarding how some of these systems will perform under 3-D earthquake shaking or whether the total costs for these systems will be as low as stated by their

promoters/advocates. For this reason, a number of engineers mentioned considering only “established” low-damage solutions.

While some of the systems require high technology, other ways mentioned to control building damage simply involved using some of the traditional systems while limiting drifts and ductilities.

Tenant expectations strongly impact choice of structural systems for individual buildings

Tenants that demand low structural and non-structural damage, minimised disturbance of operations, or business continuity after an earthquake have a direct impact on the choice of structural systems for specific buildings, either by engaging in discussions for “tailor-made” buildings or by seeking building owners/developers who are willing to cater to their needs. Projects can also be developed on spec by developers guessing as to the expectations of this market segment. However, less sophisticated tenants have an equally strong impact, albeit indirectly, which is expressed through the lease-rates considered by developers when calculating return on investment for new buildings in the competitive market. In all cases, the speculative builder must assess the rates that the market can bear, have insights into tenants’ expectations for the targeted occupancy, and balance these demands with the risks implied with each investment. In most cases, even when considering resilient/low-damage construction, cost is important (cost was indicated to be the most important consideration for structural system selection by owners). This limits how much building designers can move towards improved building performance and towards the goal of fully operational structures immediately after a major earthquake, except for a few select buildings with less cost-sensitive owners.

Additional increase in seismic performance, if desired for all buildings, would need to come from government regulation

While the construction of individual buildings able to achieve high seismic performance can be driven directly or indirectly by tenants’ expectations, the seismic resilience of a community depends more on the common performance shared by most of the significant buildings in that community than on the stellar features of a few. Given that a region has a multiplicity of building owners with often diverging expectations and means, government regulations would be required to increase the resilience of a region (as well as that of the individual structures in that region), and decrease the likelihood of a few major structures designed to code minimums affecting access to many parts of an otherwise “low-damage” city. While insurance may be considered a means of providing regional resilience if the cover is sufficient and relocation of people and businesses is not considered to be problematic, this can only be relied on if it remains available, affordable, and adequate, and is purchased by the majority of stakeholders.

Context affects final decision outcome

The specific structural systems selected in the Christchurch rebuild could have been quite different from those that have been actually constructed for a wide number of reasons. For example, if there had been no significant damage or lessons learned from previous earthquakes on the seismic performance of some structural systems, construction may have continued in a similar manner to before the earthquakes. Also, if the relative costs of different materials at the time of the rebuild had significantly differed from that which existed, if some of the research had not progressed to a form easy to apply, or if marketing of specific seismic solutions had not occurred, then structural forms may have been different from what exists now.

The reconstruction experience has paralleled an increase in stakeholder knowledge

Stakeholders from all fields of the Christchurch construction industry have educated themselves to understand the key issues with the different systems. The industry is also sophisticated enough to be able to explain how it considers the large number of factors that influence the selection of a structural system. Considerations relating to not only the lateral-force-resisting system itself, but also the costs and benefits for the whole building, were clearly described by the range of people interviewed. This knowledge places the industry in a good situation to address future issues (such as revised seismic hazard maps, price fluctuations, new developments/technologies, and stakeholders' seismic performance expectations) in a clear and rational manner, as it balances performance, cost, and other issues in structural form selection decisions, in an environment sometimes requiring more than minimum governmental standards.

The Christchurch experience may be unique today, but it is likely to repeat itself in other similarly developed urban centres worldwide after future devastating earthquakes. As such, the Christchurch rebuilding experience is significant, providing a unique insight into some of the mechanisms that can dictate structural engineering decisions during the post-earthquake reconstruction of a modern city.

1 Introduction

1.1 Background

Christchurch, the largest city in the South Island of New Zealand, with a population of about 400,000, has steadily grown since 1856 (the year it was established as the country's first city, by a royal charter). Much of the flat land of Christchurch was reclaimed by draining swampland. Early buildings were constructed of timber and masonry, followed later by steel and concrete structures. At the time of the Canterbury earthquakes (2010–2011), central Christchurch had structures of various vintage, including unreinforced masonry structures, timber structures, RC structures (frames and walls), reinforced masonry structures, a few steel-framed structures, and others. Some had brick façades and brick infills. There were portal structures, tilt slab structures, and structures containing prestressed concrete. The majority of the modern multistorey structures were of RC or prestressed concrete, detailed for ductility. A few were structural steel buildings.

The strongest shaking in central Christchurch from the Canterbury earthquake sequence occurred on 22 February 2011, from an earthquake of magnitude 6.3 having its hypocentre at a depth of 5km and a horizontal distance of less than 10km from the city's Central Business District (CBD). Many unreinforced masonry buildings collapsed, as did some RC and timber buildings. Structural damage occurred to almost all multistorey buildings. The building damage, severe soil liquefaction across the city, and human casualties have been extensively documented (NZSEE, 2011). The few steel buildings also had some issues, but overall their behaviour exceeded that exhibited by many other structural systems and a larger proportion remained in use, albeit after some required significant repair (Bruneau et al., 2011).

After the earthquake events, the Christchurch CBD was "red zoned" with access severely restricted to different parts of the CBD for months or years as negotiations occurred with insurance companies about what to do with damaged buildings, while there were nearby building hazards, and as buildings were demolished. Over that period of time (and continuing at the time of writing), many of the buildings in the CBD were demolished.

The space created by the demolition has allowed the reconstruction to start. Although reconstruction is taking place throughout the broader Christchurch metropolitan area, much of the rebuilding of multistorey buildings is taking place at the heart of the city (Christchurch City Council, 2011).

At the time of writing, more than six years after the February 22, 2011 earthquake, one might have expected that the Christchurch reconstruction would be complete, but this is not the case. Nevertheless, significant reconstruction has occurred.

To the casual observer strolling the streets of the CBD, the new “heart” of Christchurch may seem quite different from the old one. Beyond the inescapable freshness in architectural expression that occurs when an entirely new city is reborn, one can also notice the fact that the city of Christchurch, which used to be “a concrete city” (i.e., where RC buildings dominated the urban landscape, with almost all multistorey buildings relying on RC frames or walls to resist earthquake shaking), is emerging as a city with a variety of structural forms and what appears, at first glance, to be a predominance of steel structures. Furthermore, the reconstruction appears to favour the use of a number of innovative and emerging structural systems introduced to make the new buildings of Christchurch more seismically resilient.

The Christchurch experience may be unique today, but it is likely to repeat itself in other similarly developed urban centres worldwide after future devastating earthquakes. As such, the Christchurch rebuilding experience is most significant, particularly because one of the declared goals of the city is to build “a city that will be stronger, smarter and more resilient to physical, social, and economic challenges” (Christchurch City Council, 2016a, 2016b). It may therefore provide lessons for other modern urban centres that find themselves in a similar post-earthquake situation in the future.

In light of what appeared to be a unique situation, the authors have attempted to quantify the shift in construction practice to date, and, more importantly, to document the decision-making process that has led to this shift. The present report is intended to be a “lessons learned” document, targeted primarily at structural engineers and stakeholders in the construction industry around the world, with the following aims:

- a. To describe the structural systems used in the Christchurch rebuild after the 2010–2011 earthquake sequence
- b. To quantify the extent to which each structural system has been used in new buildings as part of the Christchurch reconstruction effort
- c. To document the major factors that have been affecting decisions about the selection of these systems.

Although written primarily for structural engineers, construction contractors, and code-writing bodies in New Zealand and abroad, this report is also likely to be of interest to social scientists, policy makers, and other individuals interested in post-disaster urban reconstruction and disaster resilience.

1.2 Organisation of this report

This report has two distinct parts for which data was collected in different manners. The information presented in Sections 2 and 3 summarises knowledge and opinions available in the existing literature.

More specifically:

- Section 2 summarises some aspects of the insurance and legislative context that have had an effect on the reconstruction of Christchurch and that are helpful to recognise in the context of this study. It also provides a brief overview of past construction history, an outline of relevant seismic design requirements in New Zealand, and pertinent information on the fluctuation in the cost of steel on the international market in recent years.
- Section 3 illustrates a few selected examples of the main structural systems used in the reconstruction of Christchurch, as seen by the authors during “sidewalk surveys”, to highlight the broad diversity of structural systems used and some variations in the details expressed within each structural system.

Section 4 outlines the methodology followed by the authors to collect data on the Christchurch reconstruction, which is presented in Sections 5 and 6. Because some information shared is either confidential or proprietary, results and findings in some parts of the report are compiled and presented in a format that respects this confidentiality. More specifically:

- Section 4 provides details on the methodology followed to collect and summarise the data.
- Section 5 summarises the quantitative findings of this study, investigating variation in decisions and the resulting structural systems over time since the earthquake.
- Section 6 summarises additional information obtained during the interviews that provides context and explanations to the results presented in Section 5.

The findings from additional interviews conducted with engineers in Wellington and Auckland, to determine the impact of the Christchurch earthquakes there, are presented in Section 7. Perspectives on Christchurch’s reconstruction from an architect, a project manager, and a developer are explored in Section 8.

Conclusions are presented in Section 9. Appendix A lists the individuals interviewed as part of this study, who have agreed to be acknowledged for their valuable time and contributions to this research effort. A number of case studies, providing further insight into the decision process that led to the selection of specific structural systems, have been volunteered by a number of engineering firms and are provided in Appendix B. These case studies help to illustrate the diversity of professional opinions and approaches taken by the engineering community in answer to project-specific demands, as well as some of the recurring themes that have led to the final structural system selections.

It is acknowledged that the current report provides a “snapshot” of the state of reconstruction, approximately midway through the process. The findings presented here might benefit from being revisited in the future to determine whether the trends reported in this study will change in future years if: (i) memories from the earthquakes become less vivid, (ii) buildings serving different functions or of different heights and configurations are added to the inventory, and (iii) financial shocks drastically change the economics of building construction in New Zealand.

2 General Factors Affecting the Christchurch Reconstruction

In any post-disaster situation, decisions affecting rebuilding activities are made in the context of the local culture/environment and the perceptions and biases of the key actors and stakeholders. Typically, some of the factors influencing these decisions include:

- a. The political and legislative environment, with rules specifying how buildings are to be constructed. These explicit rules relate to both process and quality control. They have generally been formulated on the basis of local and international knowledge, information, and experiences;
- b. The local rules that define the operations and charters of various organisations and stakeholders, such as the city council, the banking industry, and the insurance sector;
- c. Cultural norms that influence how interactions occur between groups, such as between clients, engineers, councils, contractors, financial institutions, and others involved;
- d. The strength of linkages and relationships between many of those groups;
- e. Economic factors, such as the cost of particular construction systems;
- f. Perceptions about seismic performance that may be attractive to the stakeholders;
- g. "Momentum" in the construction field and the ability of the industry to change direction. "Momentum" is used here to describe the natural tendency of certain industries, contractors, experts, and/or people to do things as they were done before. This may be associated with the cost, uncertainty, and risk involved in changing practices and/or equipment, and doing things differently. Other, less obvious, reasons may relate to the complex relationships between the groups above and procurement systems, prior experiences with changes in momentum, and the naturally broad range of inherent receptiveness of different individuals to change;
- h. Novelty factors, which include the desire to innovate, for example, to construct the biggest or first structure of a certain type;
- i. The availability of resources to complete the work on time and on budget;
- j. Priorities, which may vary to some extent between the public and private sectors.

Some of the above factors may conflict, tending to drive decisions in opposite directions. These factors (and many more) have been studied and are further described in the existing literature (e.g., Loosemore, 2003; Earthquake Engineering Research Institute [EERI], 1998; Alesch and Petak, 1986). It is beyond the scope of the work presented here (and the qualifications of the authors) to investigate such matters, as they fall within the realm of social sciences.

However, two important matters that are most significant in the New Zealand context and that may have had an impact on the overall decision-making process in the years following the Christchurch earthquakes are described here, because

they have been critical in setting the stage for the extensive demolition and reconstruction effort in Christchurch. These relate to New Zealand's unique insurance situation and legislative culture, which are described in Sections 2.1 and 2.2, respectively.

In addition, for the benefit of international readers less familiar with seismic design codes, terminology, and practices in New Zealand, Section 2.3 summarises a select number of issues that are relevant in the context of this report, and Section 2.4 provides a brief overview of past construction history in New Zealand.

Finally, some background information on recent fluctuations in the cost of steel is provided in Section 2.5 as it has been mentioned by many engineers (as part of this project) that this has had an impact on New Zealand construction practice in the past decades.

2.1 Insurance situation

New Zealand has a wide insurance penetration/coverage for both commercial and residential structures. Approximately 80% of the losses in the Christchurch earthquake sequence were covered by insurance (Marquis et al., 2015). Some reasons for this include:

- The requirement of banks when lending funds that the buildings be protected by insurance.
- The New Zealand Earthquake Commission (a Crown entity) insurance programme, which automatically covers the first \$100,000 of any residential claim. This programme, funded from a proportion of mortgage payments, reduces the risk for insurers and reinsurers, and results in cheaper overall insurance costs to owners. While this does not apply directly to commercial structures, it has made a large number of insurers comfortable with the earthquake insurance market, resulting in a number of distribution channels allowing for relatively cheap insurance.

The terms of insurance have generally been for "reinstatement", meaning that if an insured structure is damaged, it is required to be returned to an effective "as new" condition, but there was generally an upper limit on the insured losses. This has been the source of much discussion following the 2011 earthquakes. For example, for structures that have sustained some cycles of deformations during an earthquake, and thus some low-cycle fatigue, requiring a structure to be "as new" generally means assessing whether the remaining low-cycle fatigue life (and/or the ability to resist earthquakes) is likely to be similar (or not) to that required or provided by a new structure. A difference in answers to such questions can translate into major differences in repair costs or lead to demolition and reconstruction. By contrast, damage consisting of hairline cracks in concrete has been less contentious and has been generally repaired as part of the settlements.

While insurance coverage was widespread, the insured value was seldom sufficient to fully repair the damaged structures. However, in some cases, due to the aftershocks, owners were able to obtain the full reconstruction costs from the insurance company. In many cases, cash payments were taken instead, providing owners with flexibility (Marquis et al., 2015), with some owners opting to sell “as is”, repair, or replace. Note that loss of rent and business interruption costs were add-ons that have been included in many insurance policies.

As a result of the widespread insurance coverage in New Zealand, reconstruction has been possible. It has not always been fast, as negotiations about the damage and payments by the insurance companies have taken significant time, especially considering the continued aftershocks. Negotiations to define the terms of insurance for the new facilities to be built have also been time-consuming.

Note that, according to the New Zealand Insurance Council (Lucas, 2016), the New Zealand insurance sector is unable to encourage the use of specific types of construction that would have seismic performance beyond code requirements, such as low-damage construction (described in a later section), or of better design/construction of non-structural elements. This is because the New Zealand market is only a very small fraction (< 0.1%) of the global reinsurance market, and other international drivers control the conditions in New Zealand. Lucas (2016) indicated that while insurance costs went up initially after the Canterbury events (as was also shown by Marquis et al., 2015), the lack of worldwide disasters over the past few years have led to a surplus of reinsurance funds, and this has resulted in an insurance cost decrease. Also, even five years after the earthquake, unreinforced masonry structures – possibly the most seismically vulnerable structural form – are still being insured in Wellington, which is the major city in New Zealand that is exposed to the highest seismic hazard and risk. However, Lucas (2016) also indicated that insurance is unlikely to cover all the losses expected from an event, and that designing low-damage structures was prudent.

Incidentally, some engineers stated that the cost of insurance, which spiked by 300% following the Christchurch earthquakes, has returned to the pre-earthquake levels.

2.2 Legislative context

The New Zealand Building Code (Ministry of Business, Innovation and Employment [MBIE], 2016), originally introduced in 1992, in Clause B1: “Structure”, describes some high-level functionality and performance requirements to safeguard people from injury and from loss of amenity or property due to structural behaviour/failure. These are written in simple, general terms.

The New Zealand Building Act (MBIE, 2004) provides the legislative framework to meet the New Zealand Building Code. It describes how the legislative system works, including penalties for non-compliance, as well as how Compliance

Documents, which comprise Acceptable Solutions and Verification Methods, can be established and used.

Acceptable Solutions are automatically deemed to satisfy the requirements of the New Zealand Building Code. Building Consent Authorities (BCAs) (which are often Territorial Authorities [TAs] such as local bodies/councils) are required to ensure such compliance. Generally, they do this by requiring that the design is completed by a chartered engineer and that “producer statement” forms be signed by responsible professionals (Institution of Professional Engineers New Zealand [IPENZ], 2013). The producer statement system is intended to provide BCAs with reasonable grounds for issuing a Building Consent or Code Compliance Certificate (“building consent” being equivalent to the building permitting process in North America), without having to duplicate design or construction checking undertaken by others.

A number of producer statements (PS) are available, including PS 1 – Design (signed by design engineer), PS 2 – Design review (signed by design reviewer), PS 3 – Construction (signed by contractor and often used by the installers of proprietary systems), and PS 4 – Construction review (signed by construction inspector) (MBIE, 2015b). These can be submitted to the council to support a building consent application, and they may be required by the council. Note that not all PS forms are requested for a particular project.

Compliance Documents for Acceptable Solutions, such as those by Standards New Zealand that have been adopted for use, provide guidance for the design of a number of common structural systems for life safety in the 500-year event for ordinary buildings. Verification Methods are also Compliance Documents but are dependent on the application of engineering procedures and some judgment.

Alternatively, compliance with the Building Code may be satisfied by providing an “Alternative Solution”. An Alternative Solution follows provisions that in whole or part are outside the scope of the Compliance Documents. In practice, an Alternative Solution may be deemed to comply with the Building Code, if the design is approved by a licensed building practitioner who is a chartered professional engineer and if the BCA is satisfied as to the procedures used.

As part of this, peer review by a single structural engineer will typically be required at the discretion of the BCA, and a signed PS 2 form is required. Since there are currently no Compliance Documents for many newer structural systems, such as those incorporating base isolation, buckling restrained braced frames, rocking walls, and other “low-damage” structural systems, they are considered as Alternative Solutions (see later section for definition of “low-damage” systems).

The Alternative Solutions approach used to satisfy the performance requirements of the New Zealand Building Code for low-damage construction is quite flexible. It allows new solutions to be implemented in actual structures without large disincentive. As a result, it is possible, for better or worse, for many new systems to be implemented in New Zealand structures. The onus is on the engineers and

peer reviewers to ensure that the system is satisfactory. There are cases where peer reviewers have rejected particular solutions for which they believe there is not sufficient evidence to indicate that they will perform well, and they have accepted others.

The New Zealand situation described above is different from that in other countries, such as Indonesia, Myanmar, Japan, and the USA, for special systems (e.g. tall, irregular, or very important buildings, and some buildings using new structural systems). In these countries, expert review panels rather than one individual reviewer are often used. In some countries, standing panels provide review consistency over a region. In NZ, review quality depends on the reviewer selected for a particular structure and his/her specialised knowledge and impartiality.

It is worth noting that after the 2010–2011 Canterbury earthquake sequence, standards did not generally change to encourage construction that is more resilient. Rather, the changes made included the following:

- a. The seismic zone factor increased from 0.22 to 0.30 for Christchurch and the surrounding area. This zone factor is somewhat equivalent to peak ground accelerations for a 500-year return period event, and ordinary structures are designed to resist spectral demands associated with these using a combination of strength and ductility.
- b. The serviceability level event shaking considered in design, which is resisted primarily by strength and near-elastic response, increased from 25% to 33% of the 500-year return period shaking for ordinary structures soon after the earthquakes, but in late 2016 was returned to 25% again.

As a result, for IL2 structures (defined below) constructed between 2012 and 2016, which includes the period of the rebuild, the design force considered for serviceability analyses increased by 80% (i.e., $0.30/0.22 \times 0.33/0.25 = 1.8$).

Note that the recommendations that resulted from the Royal Commission on the Canterbury earthquakes (Canterbury Earthquakes Royal Commission, 2012) were primarily intended to ensure that new structures be designed to meet the pre-earthquake performance objectives.

They concentrated on issues such as concrete structure detailing, floor diaphragm design and performance, appropriate consideration for the effects of building inelastic torsion, and appropriate consideration of buildings with different lateral force resistances in opposite horizontal directions due to structural or gravity-loading effects. Recommendations for some of these effects were developed soon after the earthquake, but they were optional (e.g., Structural Engineering Society New Zealand [SESOC], 2011). Other recommendations have been incorporated in the 2016 standards developed by Standards New Zealand. As such, these effects were not necessarily considered in the design of structures built before 2017. Further discussion about developments of the New Zealand standards and decisions made immediately after the earthquake are given in MacRae et al. (2011) and MacRae (2013a) respectively.

In light of the above, throughout this report, a building that meets 100% of the requirements of the Building Code is referred to as being at 100%NBS, where NBS indicates New Building Standard. As such, new buildings in New Zealand must be designed to have a capacity corresponding to at least 100%NBS.

2.3 New Zealand Seismic Design Requirements

The information presented in this section describes some of the design parameters used for seismic design in New Zealand. It is intended for engineers already familiar with the fundamental principles of seismic design, but who are not aware of New Zealand practice and terminology in this regard.

2.3.1 Building Importance Levels

The 1992 Building Regulations "Building Code, Schedule A3" in effect in New Zealand specifies that buildings be designed considering an Importance Level (IL) from 1 to 4. The minimum IL for a particular structure is determined according to NZS 1170 Structural Design Actions (Standards New Zealand, 2004a) considering risk to human life or the environment, economic cost, and other risk factors for a building in relation to its use as defined by the standard below. Clients may specify that their structures be designed to resist a stronger level of earthquake shaking (i.e., to have a higher IL) than the minimum required standard for the purposes of design – something that some have indeed requested.

The four different importance levels in NZS 1170 are:

Importance level 1 (IL1)

Buildings posing low risk to human life or the environment, or a low economic cost, should the building fail. These are typically small, non-habitable buildings, such as sheds, barns, and the like, that are not normally occupied, though they may have occupants from time to time.

Importance level 2 (IL2)

Buildings posing normal risk to human life or the environment, or a normal economic cost, should the building fail. These are typically residential, commercial, and industrial buildings.

Importance level 3 (IL3)

Buildings of a higher level of societal benefit or importance, or with higher levels of risk-significant factors to building occupants. These buildings have increased performance requirements because they generally house large numbers of people, or fulfil a role of increased importance to the local community or to society in general. These include most educational facilities with a capacity greater than 250 people, as well as buildings where more than 300 people congregate in one area.

Importance level 4 (IL4)

Buildings that are essential to post-disaster recovery or are associated with hazardous facilities, including most hospitals; fire, rescue, and police stations; communications centres; and air traffic control towers, to name a few.

2.3.2 Earthquake Shaking Levels Considered in Design

The document *NZS 1170.5:2004 Structural Design Actions - Part 5: Earthquake actions - New Zealand* (Standards New Zealand, 2004b) specifies the seismic design levels to be considered in building design in New Zealand. An Ultimate Limit State (ULS) and a Serviceability Limit State (SLS) level of shaking are both specified by NZS 1170.5. The ULS is intended to ensure that the probability of collapse (and therefore risk of loss of human life) does not exceed a certain level. The ULS consideration is primarily associated with consideration of large and relatively rare events. Structures are expected to resist this level of shaking by a combination of strength and ductility. The SLS is intended to ensure that the building system, including its non-structural elements, is designed to retain its structural and operational integrity, without requiring repair, after smaller, more frequent earthquakes.

The annual probability of exceedance and return period factor, R , by which the reference-specified design forces are scaled are given in Table 2.1. As seen from this table, for the 500-year earthquake (i.e., with an annual probability of exceedance of 1/500), the scale factor for the spectra design values, described as the return period factor, R , given in the NZS 1170.5, is 1.0. For lower annual probabilities of exceedance, the R factor is greater, increasing to 1.8 for a 1/2500 annual probability of exceedance).

Table 2-1: Design annual probability of exceedance and return period factors with building importance level (adapted from King et al., 2004)

Annual Probability of Exceedance	Return Period Factor, R	Building Importance Levels			
		IL1	IL2	IL3	IL4
		Low hazard structures	Normal buildings	Important buildings including schools	Critical post-disaster buildings
1/2500	1.8				ULS
1/1000	1.3			ULS	
1/500	1.0		ULS		SLS2
1/100	0.50	ULS			
1/25	0.25		SLS1	SLS1	SLS1

Table 2.1 shows the levels of shaking considered for design for ULS and SLS events for buildings of different importance levels, and the corresponding scale factor, R , considered. Note that there are two SLS levels. For normal IL2 buildings, SLS1 is used and a lower return period earthquake (1/25) is considered. In other words,

the SLS1 serviceability shaking considered for an IL2 structure is 0.25 of the ULS level of shaking. For IL4 buildings, the system is also designed to continue to perform its functions after a serviceability limit state 2 (SLS2) earthquake (1/500). For SLS, structures are expected to resist the design level earthquake effectively elastically, with almost no yielding.

The maximum NZS 1170.5 lateral-force-reduction factor, k_{μ} , is the minimum of:

- a. the value specified for a particular structural material and form according to the structural materials standards, and
- b. that from the ULS and SLS considerations above which give $1/0.25 = 4$ for an IL2 structure.

The SLS considerations often limit the maximum lateral-force-reduction factor for ductile systems.

From 2011 to 2016, during the initial rebuild period when the SLS1 return period factor, R_s , was increased from 0.25 to 0.33 for Christchurch, the maximum lateral-force-reduction factor, k_{μ} , for IL2 structures was decreased to $1/0.33 = 3$.

While the return periods listed above are those mentioned in the standards, most ductile structures are not designed explicitly for these levels. They are designed for a lower level of shaking, which may be as low as 70% of that given above. This is done using a structural performance factor, S_p , which may be as low as 0.70. Further discussion of this is given in a later section.

2.3.3 Maximum Considered Event (MCE):

A maximum considered event (MCE) is referred to in the commentary to the current standard (Section C3.1.4 in NZS 1170.5, Standards New Zealand, 2004b), but is not referred to in the standard itself. It is stated that a structure should have a small margin against collapse in the most severe earthquake shaking to which it is likely to be subjected. The maximum considered motions assumed to represent this level of shaking for normal-use (e.g., IL2) buildings have generally been taken as that with a 2% probability of exceedance in 50 years, or a return period of approximately 2500 years.

The return period of approximately 2500 years mentioned above is equivalent to an annual probability of exceedance of $1/2500$. This definition is consistent with the US definition. According to the table above, the R factor for this $1/2500$ probability is 1.8 times that for the $1/500$ probability shaking.

There is no specified requirement to explicitly consider MCE shaking, according to NZS 1170.5. However, higher levels of shaking than the ULS are sometimes used in the standard and other documents. For example:

- In NZS 1170.5, stair support lengths are designed for $2/S_p$ times the ULS displacements. For an IL2 structure, this is $2/S_p$ times the displacement

associated with S_p x the 1/500 probability shaking level. It is therefore 2 times the displacement associated with the 1/500 year shaking level.

- The concrete design standard, NZS 3101:2017, will use 1.5 times the ULS level for cases considering collapse (Standards NZ, 2017).

2.3.4 The 7500-year Return Period Shaking Event

In some instances, a 7500-year return period has been used for IL4 buildings (e.g. Oliver and Pettinga, 2015). According to this reference, this has been deemed to be associated with a Collapse Limit State (CLS), which would be equivalent to the MCE. The system ductility was limited to 2.0 for the example cited by Oliver and Pettinga, and the CLS was assumed to be 1.25 times the IL4 ULS magnitude. This is equivalent to 1.8×1.25 times the 1/500 shaking level = 2.25 times the 1/500 shaking level. This is also equal to 2.25 times the ULS shaking level considered for an IL2 building. In that example, a S_p factor of 1.0 was used so that the structure is explicitly designed for the level of shaking stated. It is not known if other designs at the CLS have followed the same approach.

2.3.5 Design Ductility

The New Zealand design actions standard, NZS 1170.5, reduces elastic-level forces both by an S_p factor (described in the next section) and by a factor to account for ductility. The design ductility is equal to the lateral-force-reduction factor for structures with a fundamental period greater than about 0.7s. This includes most multistorey structures.

For such structures, the design ductility, μ , may be considered equal to the lateral-force-reduction factor, k_μ (which is designated by R in some overseas standards).

2.3.6 Adjustment to Shaking Level – the Structural Performance (S_p) Factor

While Table 2.1 specifies the level of earthquake that a structure is designed for, this design is not explicit for ductile structures, where the shaking intensity (as mentioned above) is multiplied by a structural performance, S_p , factor, which is typically less than unity, and may be as low as 0.70. In particular, ductile IL2 structures are explicitly designed for 0.70 times the 1/500 annual probability of exceedance shaking event. This is approximately equivalent to a 1/150 event. There has been much contention about the need for and use of the S_p factor, which was introduced in 1992 at the time of the “internationalisation” of the New Zealand loadings standard, when the stated design level corresponding to a 1/150 shaking event was changed to 1/500 one, and an S_p modification was used to ensure that the member sizes did not change significantly. As such, the current specifications for New Zealand IL2 structures may be regarded as corresponding to a 1/150 shaking event, but there is a penalty factor (which relates to the higher S_p factor) for structures expected to behave in a less ductile manner.

The use of the S_p factor has been justified as being appropriate on the basis that some structures in past earthquakes have behaved better than expected. NZS 1170.5 states that the S_p factor accounts for structures expected to be subject to

fewer than the number of loading cycles considered at peak displacements in most test programmes, to soft soil effects, and to the presence of non-structural elements. However, these effects are not necessarily present in every structure (such as in car-parking structures on stiff soil sites), and it has been argued that it cannot be relied upon to reduce the structural response (e.g., MacRae et al., 2011). This can explain why many parking structures in Wellington suffered significant structural damage during the 2016 Kaikoura earthquakes. Note that if the S_p factor could be fully attributed to the factors described in NZS 1170.5, its value should not change with ductility, but it does in the current standard. It might be desirable to revisit this in future editions of the standard.

2.4 History of building construction in New Zealand

2.4.1 Construction Practice Prior to 2011

MacRae et al. (2016) provides an overview of some popular forms of construction that were used in New Zealand before the Christchurch earthquakes. For perspective, a few points are worthwhile to summarise here with respect to masonry, RC, and steel construction, as these have been the materials predominantly used in engineered buildings.

Unreinforced masonry buildings were popular until about 1940 for one- or two-storey buildings. They fell out of favour in light of their poor performance in earthquakes, particularly during the 1931 Napier earthquake that killed more than 256 people. Partially filled and lightly reinforced concrete blocks have been used since the 1950s, and structures with all blocks filled have been used since the 1980s. Heavy masonry or plaster cladding was used until about 1990. Russell and Ingham (2010) provide a historical overview of masonry construction practices in New Zealand.

RC construction, including that with concrete structural walls, has been used since the 1930s. Plenty of cement, water, and aggregate is readily available in New Zealand, allowing concrete structures to become popular. The aggregates are available from the river gravels around the country. Concrete tilt panel single-storey frames were used from about 1950 and precast concrete flooring systems have been used since about 1966. Concrete moment frames with masonry infill were common until about 1970. Non-ductile concrete moment frames were used until about 1980, and thereafter ductile concrete moment frames have been used. Multistorey tilt panel buildings were also used after 1980. RC construction dominated Christchurch before the earthquakes. The influences of the world-leading research of Park, Paulay, and Priestley at the nearby University of Canterbury helped cement concrete as the major construction material in Christchurch. At the same time, research in timber and steel structures did not have the same profile. Seismic isolation has been consistently used with concrete structures in New Zealand since the late 1970s.

Steel frames from riveted construction were popular from the 1910s until about 1960, but welded and bolted steel moment frames started taking over from the

late 1950s. These framing systems were used until the mid-1970s when significant constructional and industrial-relations issues stopped the use of steel in multistorey commercial and residential construction. The issues were related to an impasse between the union employees involved in welding steel structures and their employers. The construction of one landmark structure was stopped for more than five years, so the industry decided to avoid steel structures. Steel structures have gradually been regaining popularity since then, mainly with shop-welded/site-bolted construction. The most commonly used lateral-load resistance systems in multistorey steel frames were EBFs and moment frames. According to Steel Construction New Zealand, in 2009, structural steel was used for major framing elements in over 50% of the buildings being constructed nationwide, mostly for gravity frames and in the less severe seismic zones of the country. The last two buildings constructed in Christchurch before the earthquakes, the 23-storey Pacific Tower and the 11-storey Club Tower, were constructed using structural steel frames for both the gravity- and seismic-load-resisting systems.

2.4.2 Development of Low-Damage/Replaceable Technology Construction

The term "low-damage" design has been used extensively in New Zealand. As it will arise a number of times in this report, it deserves a brief introduction.

Over the past few years, there have been significant developments in methods to limit damage in structures subject to earthquakes so that they do not need to be replaced immediately after a major event. These have been termed "low-damage structures". In parallel, for structures that are damaged, methods have been developed to encourage such damage to occur in a few locations (often easily accessible) where replacement of damaged components may easily be conducted. This is termed "replaceable technology construction".

Descriptions of these methods and some of their implementation are given by MacRae and Clifton (2013) and MacRae et al. (2016) for steel structures. Further techniques are available for structures of other materials, such as concrete and timber (Buchanan et al., 2011). Even for these structural systems, the major energy-dissipating devices are often made of steel.

The term "low-damage" was introduced before the Christchurch earthquakes (MacRae and Clifton, 2010) and is used widely. However, while definitions for this have been proposed (MacRae and Clifton, 2013), there is currently no accepted definition of "low-damage". This means that many groups are able to claim that their structure is "low-damage", but there is no way for this to be verified. Recently, the Ministry of Business, Innovation and Employment (MBIE), in conjunction with the Structural Engineering Society New Zealand (SESOC) and the New Zealand Society for Earthquake Engineering (NZSEE), has put together a working group that will describe three levels of low-damage construction, in both general performance expectations (that a client could understand) and technical descriptions of the means to meet these expectations (for those involved in building planning and construction). The structural system, permanent non-structural elements, and fit-outs may be provided with different levels of

performance. The development of this document has been delayed by the Kaikoura earthquakes. It is likely to be completed by 2018.

Some engineers have argued that low-damage/replaceable technology structures include nominally elastic systems, conventionally designed systems properly designed and constructed, EBF systems with replaceable links, BRB systems, systems with axially yielding devices, systems with flexural yielding devices, systems using lead dissipators, systems using friction dissipators, viscously damped systems, base isolation (using sliding friction systems, lead-rubber dissipators, or both), and rocking systems (MacRae et al., 2016).

2.4.3 Development of Flooring Systems and Fire-Protection Considerations

For concrete buildings, precast flooring became popular in the 1980s. This generally consisted of precast beams with timber infills and a concrete topping, or hollowcore units with a concrete topping.

Composite steel decking has also been available since the 1990s; in that system, a cold-formed steel decking material was used as both formwork and tension reinforcement for a concrete topping/floor. This decking was originally designed to span 3.5m, but different decking profiles, with unpropped spans up to 5.5m and propped spans up to 8.5m, together with simple design tools, were developed in the late 1990s. These were implemented in several buildings around New Zealand before the Christchurch earthquake sequence. They are most suitable for steel buildings where through-deck welding is used to connect decking to the beams and place shear studs.

One significant cost in building construction, especially for steel structures, is fire protection. Methods were published in the UK that allowed secondary beams supporting floors to be left unprotected in certain situations (e.g., Huang et al., 2003). These methods use yield-line theory to assess the strength of a floor under fire and allow unprotected flooring systems to achieve specified fire ratings. These were further developed in New Zealand following observations of the Cardington tests (Building Research Establishment [BRE], 2004). This resulted in the slab-panel method, which reduces the amount of fire protection required for floor systems. New Zealand engineers are familiar with this approach and it is commonly used. Design guidance and software is available from HERA (2006) and Steel Construction New Zealand (2014).

2.5 The cost of steel

The relative economics of different structural systems is affected by the cost of raw materials. Figure 2-1 from Trading Economics (2017a) shows that in mid-2008, the price of steel peaked at a record high value of US \$1265 per metric ton and quickly decreased by mid-2009. Since then, it has fluctuated to a lesser degree, with prices always less than 50% of the peak 2008 value and the lowest point at about 10% of the peak price (record low of US \$90 per metric ton). The reduced price of steel naturally makes it more attractive as a building material.

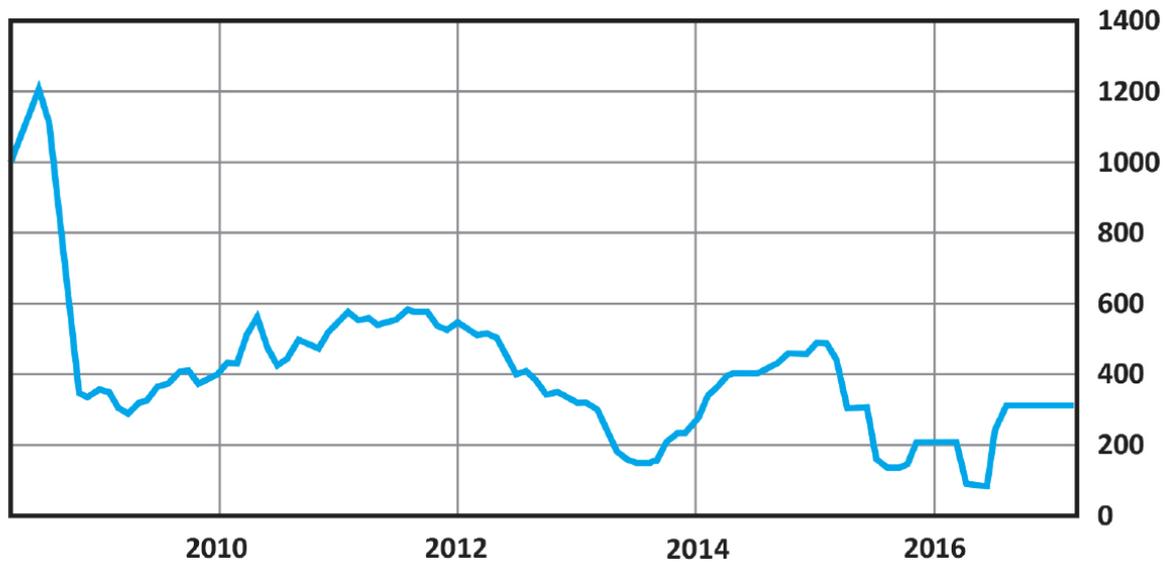


Figure 2-1: Steel price chart in US dollars per metric ton (Trading Economics, 2017a)

The price of steel plates, taken from a different source (SteelBenchmarker, 2017), follows similar trends, as shown in Figure 2-2, which also shows a significant difference in price between the US and Chinese steel producers. (Note: steel production in China increased 14-fold from 1990, to more than 77,000 metric tons in 2017, according to Trading Economics (2017b). This has resulted in accusations of China dumping steel (e.g., McBeth, 2016; Smith, 2016).

The low prices of imported steel have been tempting to many contractors, but there have been significant issues with the quality of steel imported from certain parts of Asia, which has made headline news (Leaman, 2016). This is not just a New Zealand issue, as similar problems have been reported in a number of other countries, such as Australia and the UK (Cooper, 2015; Hannan, 2017; Tovey, 2015). As a result, some engineers and contractors will only allow the use of structural steel that is sourced locally or from pre-approved overseas mills. Others will rely on extensive testing to ensure quality when steel is shipped to New Zealand from countries with cheaper mills. Some engineers/contractors have relied on their own staff to provide quality-control inspections, both at the point of origin and in New Zealand. Large steel-importing companies have also been established to take care of quality-control issues from countries where mills have mixed reputations. In short, the New Zealand steel industry has been active in highlighting and addressing the issues (e.g., Steel Construction New Zealand [SCNZ], 2016). Note that the above relates to both raw and fabricated steel, as parts of steel structures are often fabricated in Asia before shipping to New Zealand for final site assembly.

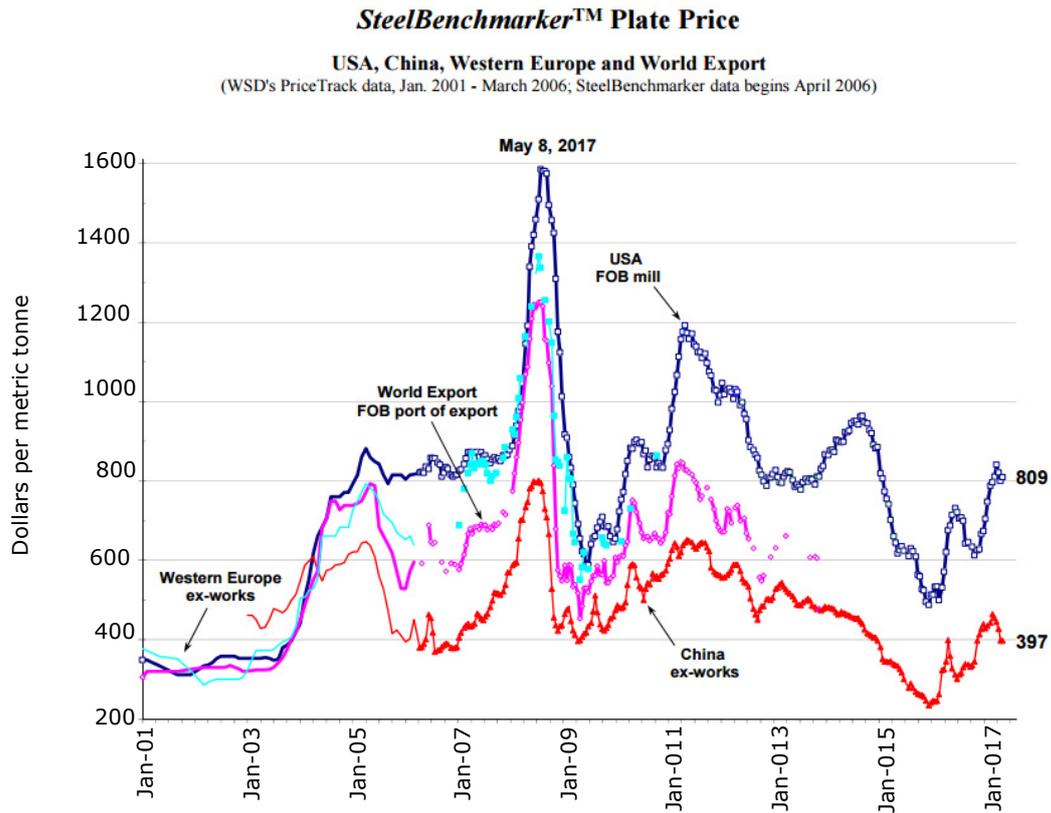


Figure 2-2: Steel price chart in US dollars per metric ton (SteelBenchmarker, 2017).

Note: The “ex-works” price of a consignment is the price at the plant/works gate with no transport included. The buyer pays all transportation costs. The free on board (FOB) cost includes manufacture, transportation to the port, and loading for shipment. All other costs are paid by the buyer (S&P Global, 2017).

3 Overview of Structural Systems Used in Christchurch

In 2016 and 2017, the Christchurch CBD could be described as a landscape of sprawling construction sites, with multiple new buildings being constructed, a few existing buildings being retrofitted, some buildings still in the process of being demolished, and a number of damaged buildings boarded up awaiting their fate. Anyone walking through the area could witness this activity, and structural engineers doing so could easily identify the various types of structural systems being used in the process.

The authors have selected some of the new structures being erected at that time in the CBD to illustrate the types of structural systems referred to throughout this report, while at the same time highlighting a sample of the range of structural details encountered across buildings sharing given types of structural system. Only information visually accessible to anyone strolling Christchurch (sometimes complemented by facts and figures from public websites) is presented here. Engineering comments in this section are solely those of the authors based on this informal survey and available public domain information, without access to structural drawings for the structures described, and should only be interpreted in this context. Correspondingly, commentary is kept to a minimum.

Note that the narrative assumes that the reader is familiar with the types of structural systems that have been used internationally in seismic design, and description of the structural systems is only provided here for those systems that are somewhat unique to New Zealand.

3.1 Base isolation

A number of new structures, either completed or in an advanced stage of construction, were observed to be steel frames supported on base-isolation bearings. Lead-rubber bearings as well as sliding-friction systems have been used. Some of those buildings are presented below.

3.1.1 151 Cambridge Terrace

The 151 Cambridge Terrace building (shown in Figure 3-1) was completed at the time of the survey. However, the friction pendulum base-isolation bearings were visible from the underground parking garage. The bearings were supported on concrete columns from the concrete foundation; such elevation accommodated the parking ramp detail. The supported structure was a steel frame throughout, except for the beams framing the parking ramp that provides access to the lower level.

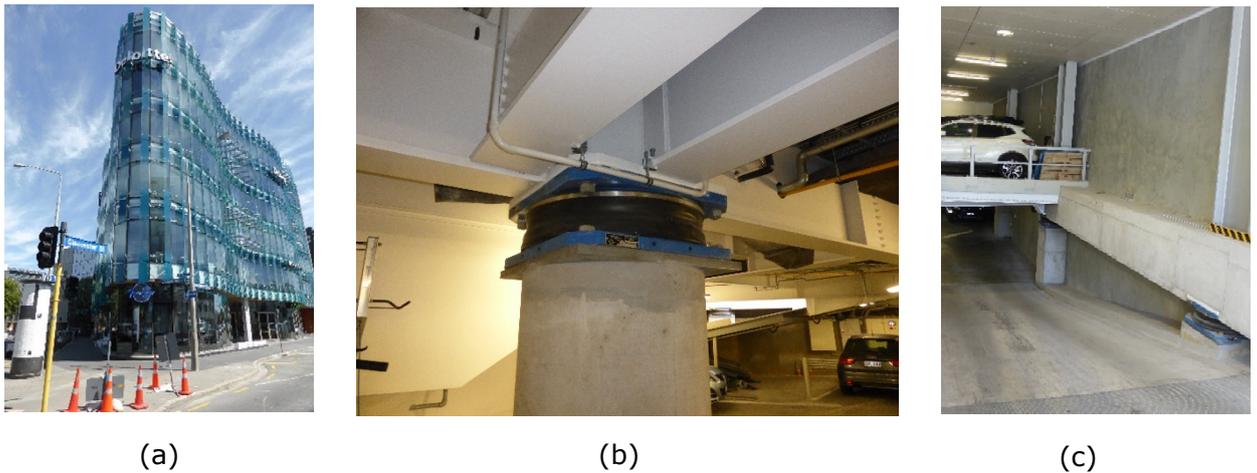


Figure 3-1: 151 Cambridge Terrace. (a) Global view, (b) Base isolator, (c) Details at parking ramp

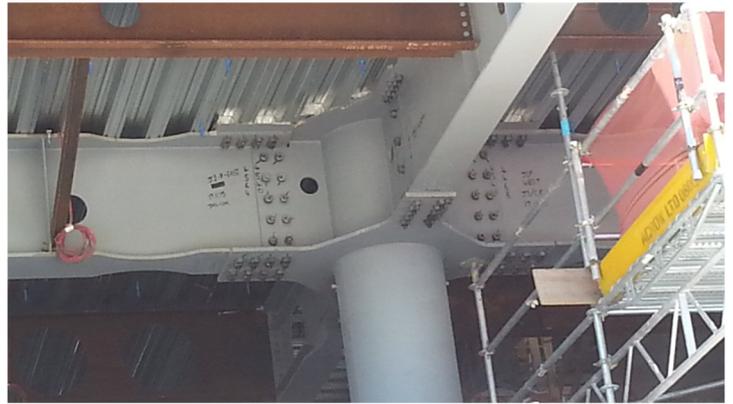
3.1.2 Justice Precinct, 121 Tuam Street

The Christchurch Justice and Emergency Services Precinct (a.k.a. the Justice Precinct) was the first of the high-profile “anchor project” constructions launched by the government as part of the Christchurch reconstruction plan. It is intended to house all justice and emergency services inside a single building, most notably the New Zealand Fire Service, the Ministry of Civil Defence and Emergency Management, the Canterbury Civil Defence Emergency Management Group, and Christchurch Civil Defence Emergency Management. Detailed information on the \$300m, 42,000m² project is provided on the following Ministry of Justice website: <https://www.justice.govt.nz/about/about-us/our-strategy/christchurch-justice-and-emergency-services-precinct/>

Base isolators are lead-rubber bearings, supporting a special ductile moment-resisting steel frame (Figure 3-2a), with circular concrete-filled steel-tube columns. The beams have reduced beam sections (RBS) as shown in Figure 3-2b, and are connected to beams with external diaphragm connections using bolted connections. A bearing above the column is shown in Figure 3-2c, and low-friction supports in Figure 3-2d.



(a)



(b)



(c)



(d)

Figure 3-2: Justice Precinct. (a) Global view of steel framing, (b) Special moment-resisting frame with reduced beam section connections, (c) Base-isolation bearing, (d) Slider detail on north-west elevation

3.1.3 Grand Central Building

The Grand Central Building is a base-isolated structure constructed on the site of the former Grand Chancellor Hotel. (The Grand Chancellor was demolished following the 2011 earthquake due to major damage and out-of-plane buckling of some of its RC shear walls, which left it notoriously leaning).

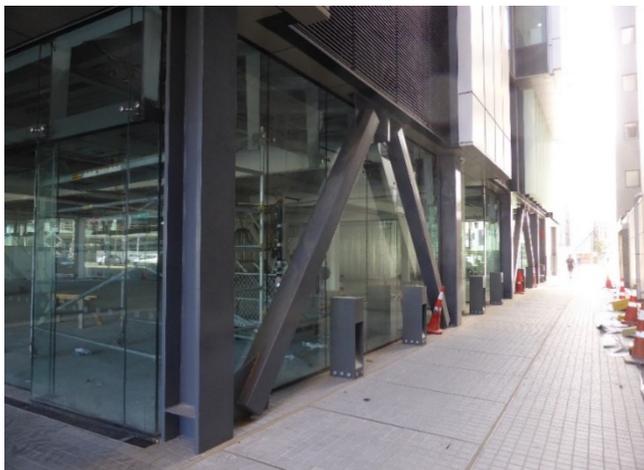
The Grand Central Building consists of CBFs and EBFs supported by base isolators (Figure 3.3). Information on the internet promoting the project and celebrating its completion indicates that the building cost somewhere between \$70 and \$85 million, providing office space to between 1,100 and 1,500 public servants from thirteen different government agencies (per McDonald (2017) and Fletcher Construction (2017)).



(a)



(b)



(c)



(d)

Figure 3-3: Base-isolated Grand Central building. (a) Global view during construction, (b) View of steel-braced frame and MRF inside first floor, (c) Centrally braced frame (CBF) architecturally exposed at ground level after building completion, (d) Brace connection detail

3.1.4 Acute Services Building

As indicated on the website of the New Zealand Ministry of Health, the Acute Services Building is the largest government project as part of the Canterbury rebuild, with 62,000m² of floor space. As shown in Figure 3-4, the structure consists of a ductile MRF relying on RBS for its moment connections, supported by base isolators and sliders located on top of short columns. Interestingly, the marking on the beams (Figure 3-4d) clearly delineates the zone of potential significant yielding (referred to as the “protected zone” in the US) over which trades cannot weld or drill/punch holes, as these could have detrimental effects on the development of cyclic plastic hinging during seismic response.



(a)



(b)



(c)



(d)

Figure 3-4: Acute Services Building – ductile MRF with RBS connections on top of base isolators and sliders. (a) Global view, (b) Base isolators, (c) Part of ductile MRF, (d) Protected zone at RBS connection

3.1.5 Christchurch Art Gallery

Although this report is about the Christchurch reconstruction, and thus focused on new construction, an exception is made here with a few words about the Christchurch Art Gallery, because this building served as the Emergency Response Centre during the 2011 earthquake. Interestingly, its glass façade, spanning the width and height of the building, was intact after the earthquake. In the years following the earthquake, extensive retrofitting work was performed, including geotechnical work to relevel the building, and structural work to repair damage to structural and non-structural components, and to base isolate it to better protect its art collections and provide reassurance for travelling collections (Strongman, 2015; Christchurch Art Gallery, 2017). The photos included in Figure 3-5 are provided (as a somewhat “off-topic diversion”) to illustrate how the base-isolator support columns were colourfully integrated in the underground parking garage of the Art Gallery.



(a)



(b)

Figure 3-5: Christchurch Art Gallery. (a) Global view, (b) Base isolators in underground parking garage

3.2 Viscous dampers

One building in Christchurch known to include US-produced viscous dampers was visited.

3.2.1 12c Moorhouse Avenue

The 12c Moorhouse Avenue building is an easily accessible example of this type of application because many of its viscous dampers at the first floor are exposed outside the building, and many others are visible through the glass façade (Figure 3-6).



(a)



(b)

Figure 3-6: Viscous damper implementation in the 12c Moorhouse Avenue building. (a) Global view during construction (from Google Street View, August 2015), (b) Close-up view of damper

3.3 Steel moment-resisting frames (MRF)

Beyond the steel MRFs that have been used in base-isolated structures (mentioned earlier), some were also used in regular structures.

3.3.1 41 Lichfield Street

In this building located on Lichfield Street (Figure 3-7), adjacent to the Justice Precinct, MRFs with bolted end-plate connections were used in one direction (N-S), while BRB frames were used in the other direction (E-W).



(a)



(b)



(c)



(d)

Figure 3-7: MRF and BRB frame, 41 Lichfield Street. (a) Global view of MRF, (b) Close-up of bolted end-plate moment-resisting connection, (c) Global view of BRB frame, (d) Close-up of braced frame connection

3.3.2 The Crossings, 71 Lichfield Street

The Crossings is a multi-building retail complex of new and retrofitted heritage buildings, filling the land bordered by Cashel, Colombo, Lichfield and High Streets. The specific building shown in Figure 3-8, with an irregular floor plan, provides a good example of ductile MRF having RBS and square concrete-filled columns.



Figure 3-8: MRF at the Crossings, 71 Lichfield Street. (a) Global view of space moment frame, (b) Close-up of RBS connection with bolted end-plate to moment-resisting connection to square steel section

3.4 Steel moment-resisting frames – with Friction Connections

The type of moment-resisting friction connections referenced here were developed in New Zealand and used initially in the Te Puni Apartment building at Victoria University of Wellington in 2007. The beam top flange in these moment connections are typically welded to the column, and detailed such as to allow rotation of the beam during sway motion of the frame. Beam moment capacity and energy dissipation is achieved when the bottom flange slides between clamping plates, themselves welded to the column (more information on this structural system is available in MacRae et al. (2010)).

3.4.1 The Terrace Project on Oxford Terrace

In the Terrace Project on Oxford Terrace (Figure 3-9), MRFs with sliding connections were used in two directions, using external diaphragm connections to the rectangular concrete-filled tube (RCFT) columns, as shown in Figures 3-9b to 3-9d. The base connection was a two-way connection with the column sitting on a central pin and oversized holes placed in the baseplate to allow flexural movement in both directions, as shown in Figure 3-9e.



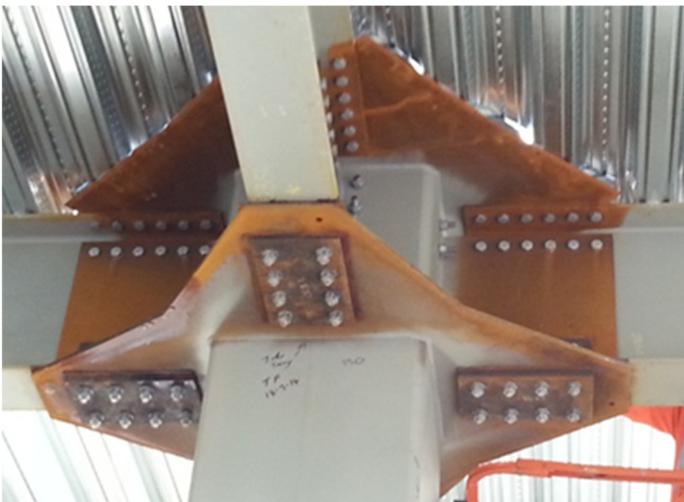
(a)



(b)



(c)



(d)



(e)

Figure 3-9: Details of the friction connections on the Terrace Project at Oxford Terrace. (a) Global view, (b) Bidirectional moment connection being constructed, (c) Unidirectional moment connection under construction, (d) Completed bidirectional moment connection, (e) Base connection detail

3.5 Steel concentrically braced frames (CBFs)

CBFs have also been used in some building instances.

3.5.1 222 High Street

The building at 222 High Street (Figure 3-10) uses CBFs in an X-configuration spanning two storeys. Braces are welded at their ends to gussets made integral with an extended segment of the beam. Braces are rectangular hollow structural shapes. The structure is base isolated.

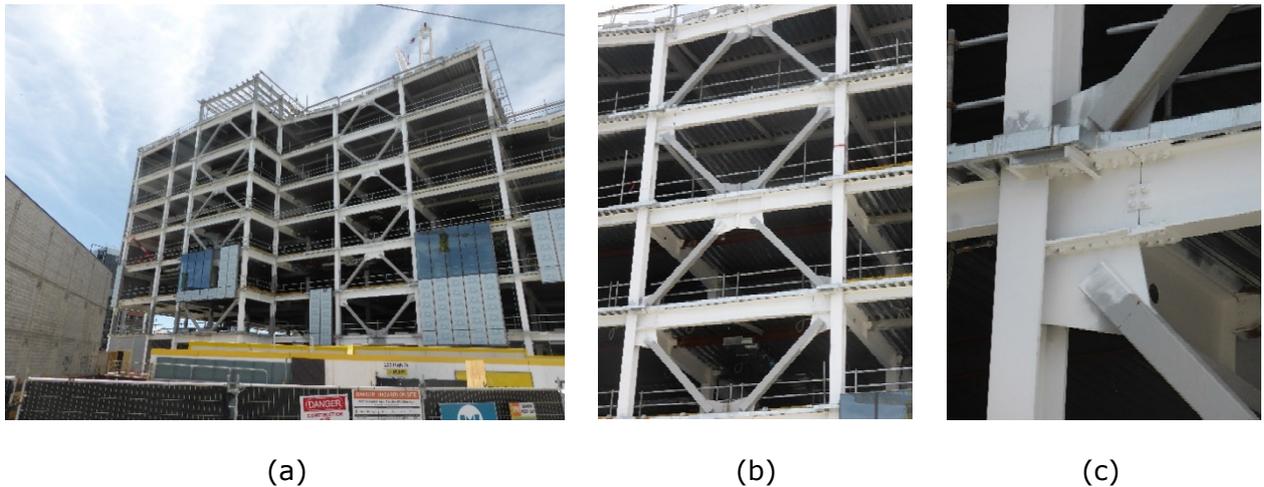


Figure 3-10: CBF, 222 High Street. (a) Global view, (b) Braced bay elevation, (c) Close-up view of gusset connection brace to beams and columns

3.5.2 124 Kilmore Street

CBFs in an inverted-V configuration have been used in the three-storey 124 Kilmore Street building (Figure 3-11). Braces are directly welded to the beams and columns.



Figure 3-11: Inverted-V CBF at 124 Kilmore Street. (a) Global view, (b) Close-up of brace-to-beam connection detail

3.5.3 60 Kilmore Street

CBFs have been used in the lower two storeys of the 60 Kilmore Street building (Figure 3-12).



Figure 3-12: 60 Kilmore Street. (a) During construction (Google Street View, August 2015), (b) Completed, as seen May 2016

3.5.4 Core Engineering Building, University of Canterbury

While outside the CBD, these interesting three-storey CBFs have been used for the University of Canterbury engineering hub building (Figure 3-13). It appears that the frames have been laid out such that braces in the first two levels are all oriented in the same direction.

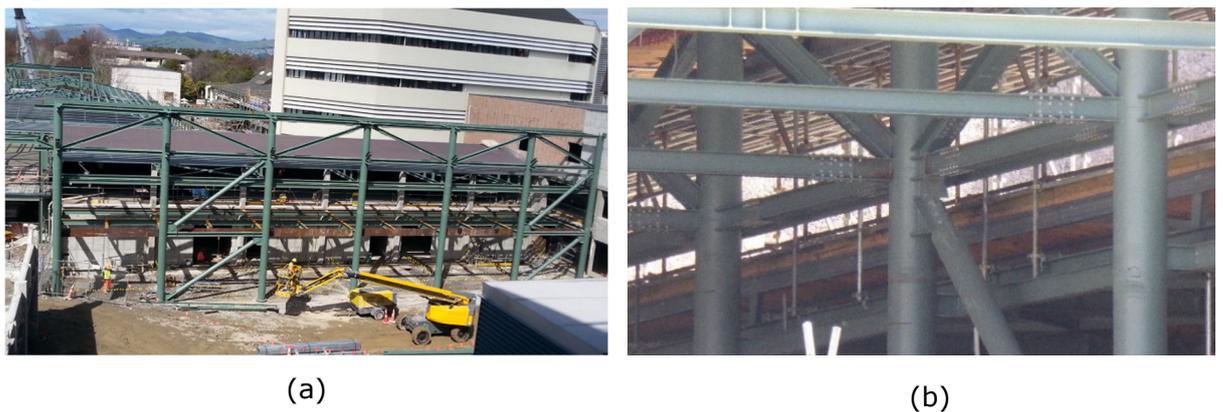


Figure 3-13: CBFs in the Core Engineering building, University of Canterbury. (a) Global view of braced frame, (b) Close-up of brace connection detail

3.6 Eccentrically braced frames (EBFs) with or without replaceable links

Many examples of EBFs could be seen in Christchurch, some with conventional detailing, others with replaceable bolted links. Examples of each type are provided below.

3.6.1 120 Hereford Street

EBFs were used in a number of bays of the 120 Hereford Street building (Figure 3-14). Both the inverted-V configuration and the D-configuration were used. In all cases, bolted replaceable links were used.

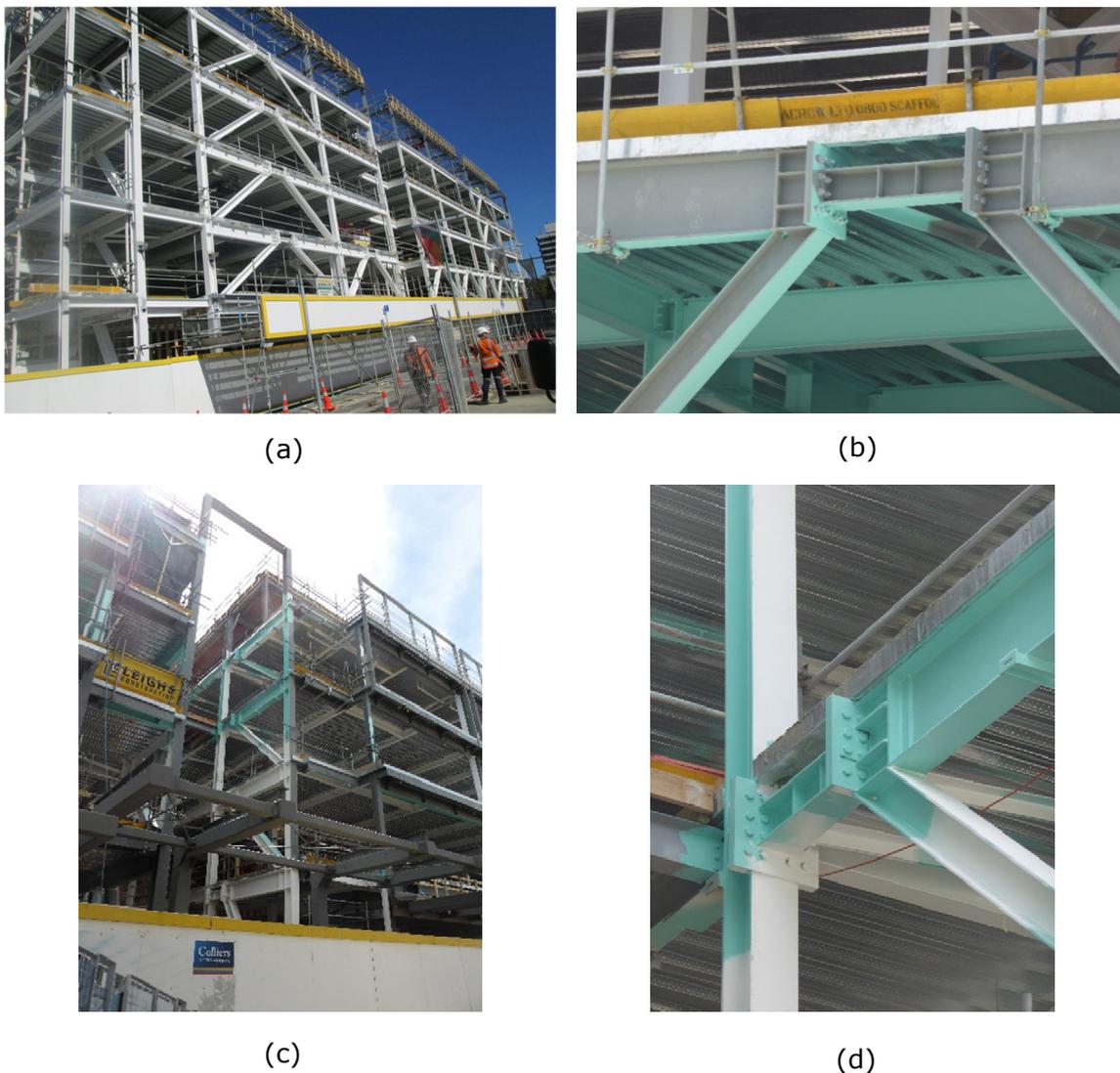


Figure 3-14: EBF, 120 Hereford Street. (a) Global view of inverted-V braced frame, (b) Close-up of link in inverted-V braced frame, (c) Global view of D-frame, (d) Close-up of link in D-frame

3.6.2 329 Durham Street

Inverted-V EBFs were used in the three-storey 329 Durham Street building (Figure 3-15). Here again, replaceable bolted links were used. The structural system was left exposed on the first floor as part of the architectural expression.



(a)

(b)

Figure 3-15: EBF, 329 Durham Street. (a) Global view of inverted-V braced frame, (b) Close-up of link in inverted-V braced frame

3.6.3 208 Barbadoes Street

Inverted-V EBFs with bolted replaceable links were also used in the two-storey 208 Barbadoes Street building (Figure 3-16). The structural system was left exposed over the height of the building façade, and distinctively painted orange as part of the architectural expression.



(a)

(b)

Figure 3-16: EBF, 208 Barbadoes Street. (a) Global view of inverted-V braced frame, (b) Close-up of link in inverted-V braced frame

3.6.4 Northwest Corner of Lichfield and Barbadoes Streets

Conventional inverted-V EBFs (meaning without replaceable links) were also used in the building on the northwest corner of Lichfield and Barbadoes Streets (Figure 3-17). In the first-storey façade, the braces and beams of the braced frame were left exposed and painted orange.



Figure 3-17: EBF, building on northwest corner of Lichfield and Barbadoes Streets. (a) Global view of inverted-V braced frame, (b) Close-up of link in inverted-V braced frame

3.7 Steel buckling restrained braced (BRB) frames

BRB frames are a relatively new structural system. They were developed in the mid-1980s in Japan and have been implemented in many buildings around the world since. A large number of buildings under construction at the time of the survey used BRBs as their structural systems. Some of those are presented below. This selection also illustrates the range of vertical distribution layouts (e.g., diagonal, super-X, or Chevron configurations) and the range of brace connection and gusset details used from building to building.

3.7.1 Science Annex, University of Canterbury

The new Science Annex building constructed at the University of Canterbury (not in the CBD) uses BRBs in multiple bays (Figure 3-18), and different types of braces and brace connections at different locations in the structure. Plates at the ends of the braces are stiffened and bolted to large gussets, having beam flange extenders to which the beams are connected. Columns are circular steel tube.



(a)



(b)

Figure 3-18: BRB frame, University of Canterbury Science Annex. (a) Global view, (b) Close-up view of gusset connection of BRB to beams and columns

3.7.2 14 Hazeldean Road, Addington

The five-storey building at 14 Hazeldean Road uses BRBs in a super-X configuration, spanning up to ten bays (Figure 3-19).



(a)



(b)

Figure 3-19: BRB frame, 14 Hazeldean Road. (a) Global view, (b) Close-up view of gusset connection of BRB to beams and columns

3.7.3 254 Montreal Street

The three-storey building at 254 Montreal Street uses BRBs in multiple bays of the structure, with braces pin-connected at their ends to gussets made integral with an extended segment of the beam (Figure 3-20). Some of the frames, both inside and outside, have been painted dark bronze and used for architectural purposes.



Figure 3-20: BRB frame, 254 Montreal Road. (a) Global view, (b) Close-up view of gusset connection of BRB to beams and columns

3.7.4 PwC Centre, Cambridge Terrace

The five-storey PwC Centre building on Cambridge Terrace has an irregular floor plan. It uses BRBs in multiple bays on the periphery of the structure, with braces pin-connected at their ends. Columns are circular steel tubes (Figure 3-21).

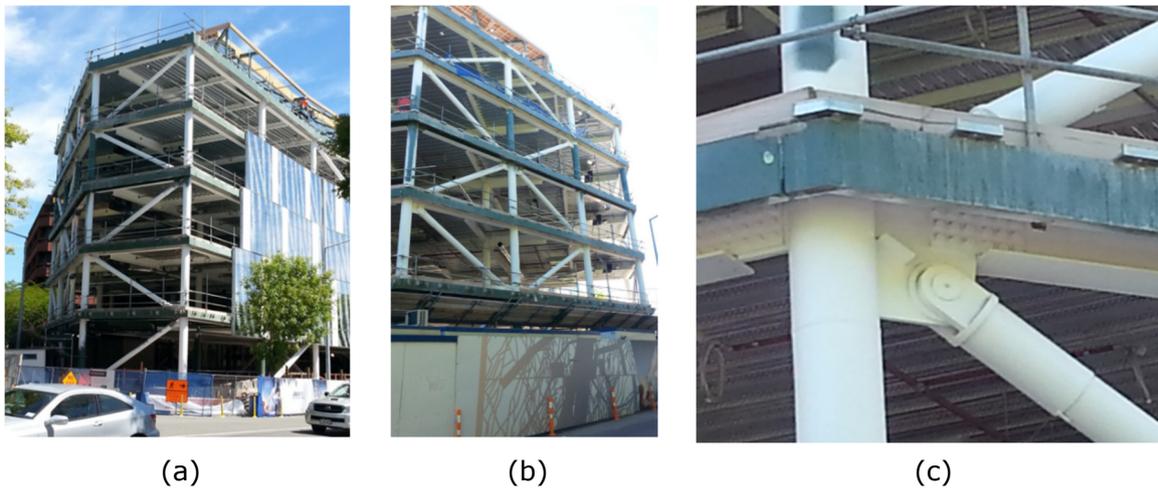


Figure 3-21: BRB frame, PwC Centre. (a) Global view, (b) Alternative brace configuration on other façade, (c) Close-up view of gusset connection of BRB to beams and columns

3.7.5 Building on Northeast Corner of Lichfield and Colombo Streets

The building constructed on the northeast corner of Lichfield and Colombo Streets used BRBs in a super-X configuration (Figure 3-22). Each super-X spans two-storeys/bays. Plates at the ends of the braces are stiffened and bolted to large gussets. The beams are spliced at the edge of the gussets.

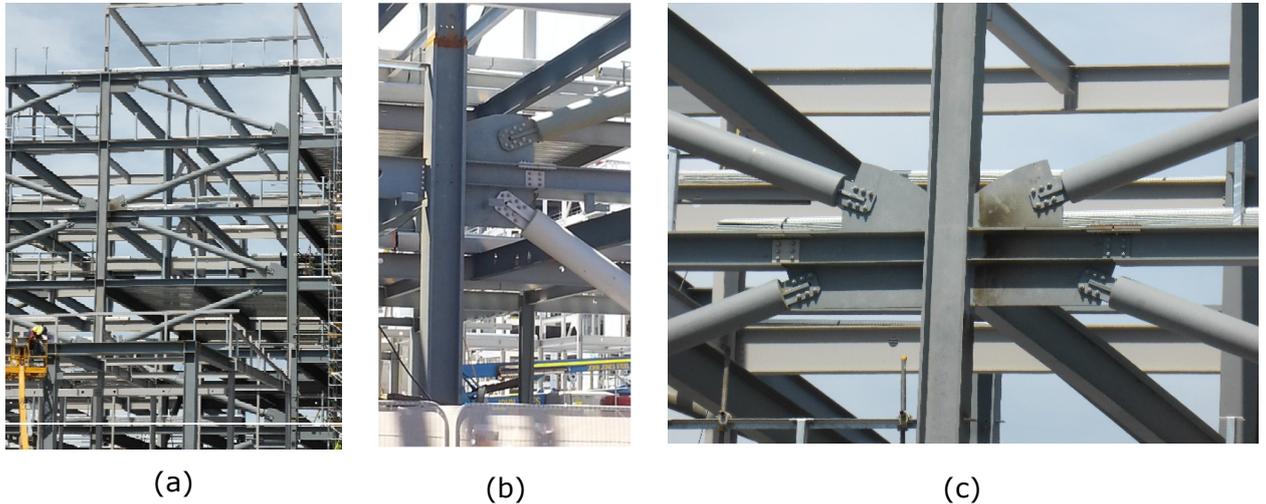


Figure 3-22: BRB frame, building on northeast corner of Lichfield and Colombo Streets. (a) Global view, (b) Connection to column at edge of braced frame, (c) Connection to column at mid-bay of braced frame

3.7.6 Lichfield Carpark (across from Justice Precinct)

The carpark being constructed on Lichfield Street, across the street from the Justice Precinct, was an example of BRB construction mixing two different types of BRBs and BRB connections in the same building. Figure 3-23 shows pin-ended BRB connections used for the BRBs at the ground floor, and bolted BRB connections on the floor above.



Figure 3-23: Lichfield carpark building, with BRB frame. (a) Global view, (b) Close-up view of BRB connections to column

3.8 Rocking frames/walls

Rocking steel CBFs with ring spring base holddown systems had been developed in New Zealand and implemented in a few buildings across the country prior to the earthquakes (e.g., Gledhill et al., 2008). Concrete walls detailed to rock about their base have also been implemented in New Zealand. This section describes some of those rocking frames/walls built as part of the Christchurch reconstruction activities.

3.8.1 Forté Health Building, 132 Peterborough Street

The first new rocking steel frame building constructed in Christchurch after the earthquakes was the Forté Health building, 132 Peterborough Street, which integrated a few different systems for seismic energy dissipation, including friction dissipators (Figure 3-24). Pairs of rocking walls were coupled in a manner such that seismic energy is dissipated during relative movement between the walls by lead extrusion dissipators. The rocking frames are also held down by tendons extending over their height. Vertical deformations of the rocking frame are also decoupled from the slab of the building while allowing horizontal force transfer.



(a)



(b)



(c)



(d)

Figure 3-24: Rocking frame system implemented in Forte Health building. (a) Global building view, (b) Guides at base of rocking frame, (c) Coupled rocking frames, (d) Energy-dissipating couplers between rocking frames

3.8.2 141 Cambridge Terrace

The rocking frames used at 141 Cambridge Terrace (Figure 3-25) are tied down at their base using large springs designed to limit the onset of rocking. Flexural yielding U-shape energy dissipators are placed between the columns of the rocking frame and the adjacent gravity columns. On the side of the façade, to ensure access to the building (i.e., without braces), rocking is designed to take place above a portal frame. The rocking frames have been left exposed after the completion of construction and are visible in the entrance hall of the building.



(a)



(b)



(c)



(d)



(e)



(f)

Figure 3-25: Rocking frame system implemented in 141 Cambridge Terrace building. (a) Global view, (b) Close-up view during construction, (c) Interior view after completion, (d) Detail at uplifting corner, (e) Portal on front side of building, (f) Sliding detail

3.8.3 Christchurch Central Library, Corner of Gloucester Street and Cathedral Square

Christchurch's new Central Library, being built on the corner of Gloucester Street and Cathedral Square, is heavily promoted on-site and online (<https://my.christchurchcitylibraries.com/central-library>). This information shows that its tall rocking precast concrete core walls will be surrounded by a steel frame (Figure 3-26).



(a)

(b)

Figure 3-26: Christchurch Central Library rocking walls during construction. (a) Global view, (b) Close-up view

3.9 Reinforced Concrete (RC) Structures

A number of buildings being constructed in the Christchurch CBD at the time of the survey were of RC construction. Examples of such construction are provided below.

3.9.1 Rakaia Apartments, 50 Kilmore Street

The five-storey apartment building being constructed at 50 Kilmore Street (Figure 3-27) consists of a steel floor frame with large precast concrete panels that serve as shear walls (supplemented by a few steel columns). A sign on the construction site advertised this building to be the first of a multi-building project; the architectural rendering on that sign depicted an adjacent similar-looking 11-storey building to be built later.



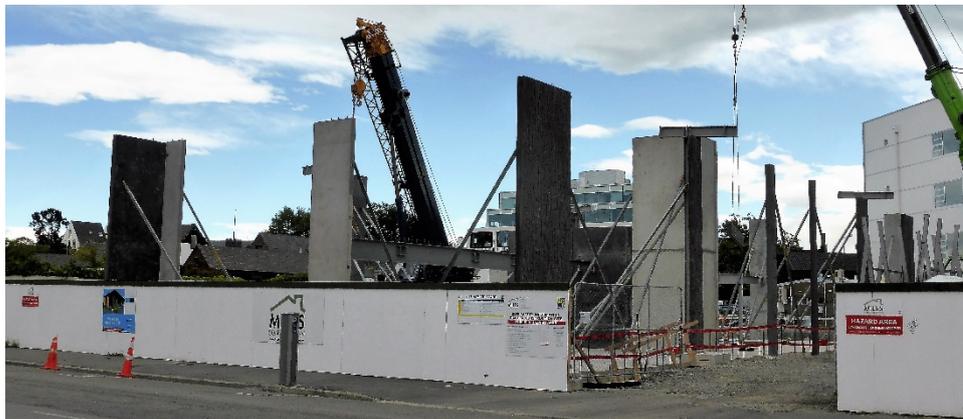
(a)

(b)

Figure 3-27: Precast concrete walls, 50 Kilmore Street. (a) Global view, (b) Close-up view of floor system

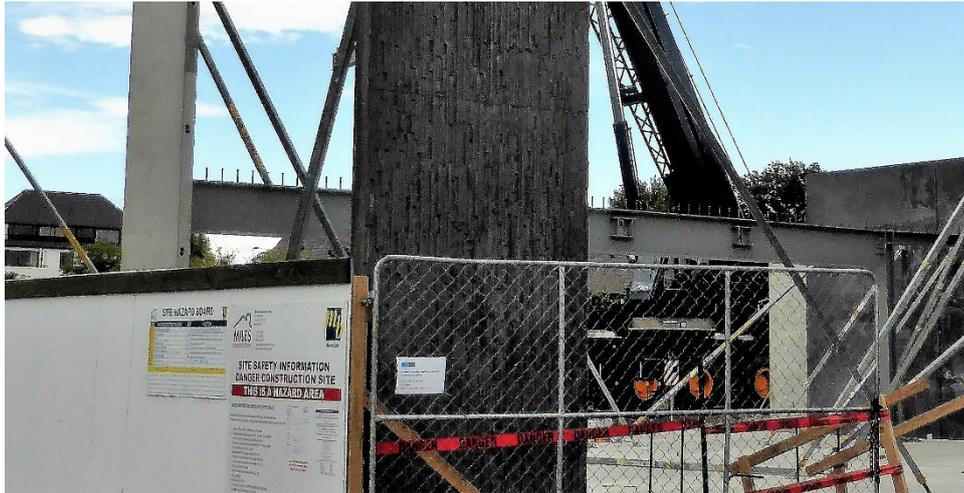
3.9.2 Canterbury Employers' Chamber of Commerce, 57 Kilmore Street

The two-storey office building constructed at 57 Kilmore Street for the Canterbury Employers' Chamber of Commerce (Figure 3-28) consists of a steel floor frame with large precast concrete panels that serve as shear walls.



(a)

Figure 3-28: Precast concrete walls and steel floor system, 57 Kilmore Street. (a) Global view, (b) Close-up of steel beam



(b)

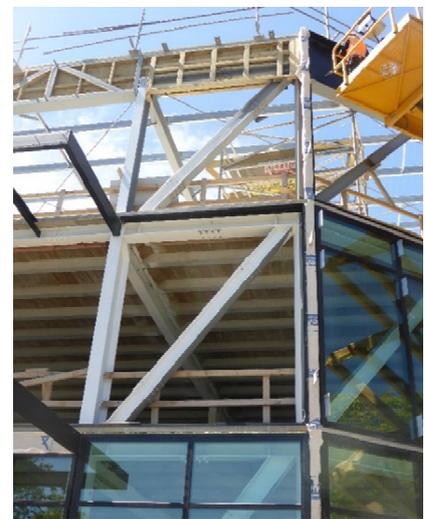
Figure 3-28 (continued): Precast concrete walls and steel floor system, 57 Kilmore Street. (a) Global view, (b) Close-up of steel beam

3.9.3 Corner of Colombo and Cambridge Streets

The three-storey building constructed on the corner of Colombo and Cambridge Streets shows precast panels along the alley-side of the building and CBFs along its main glass façades (Figure 3-29).



(a)



(b)

Figure 3-29: Combination of concrete walls and braced frames, building on the corner of Colombo and Cambridge Streets. (a) Global view, (b) Close-up view of CBFs

3.9.4 Forté Health Building, 132 Peterborough Streets

The three-storey building on the corner of Kilmore and Peterborough Streets (Figure 3-30) has precast walls in both principal directions to resist lateral loads, and a complete gravity steel framing system (beams, columns, and steel decking). Documentation on-site and online indicate that this building is intended to offer a mix of medical/dentistry and office space.

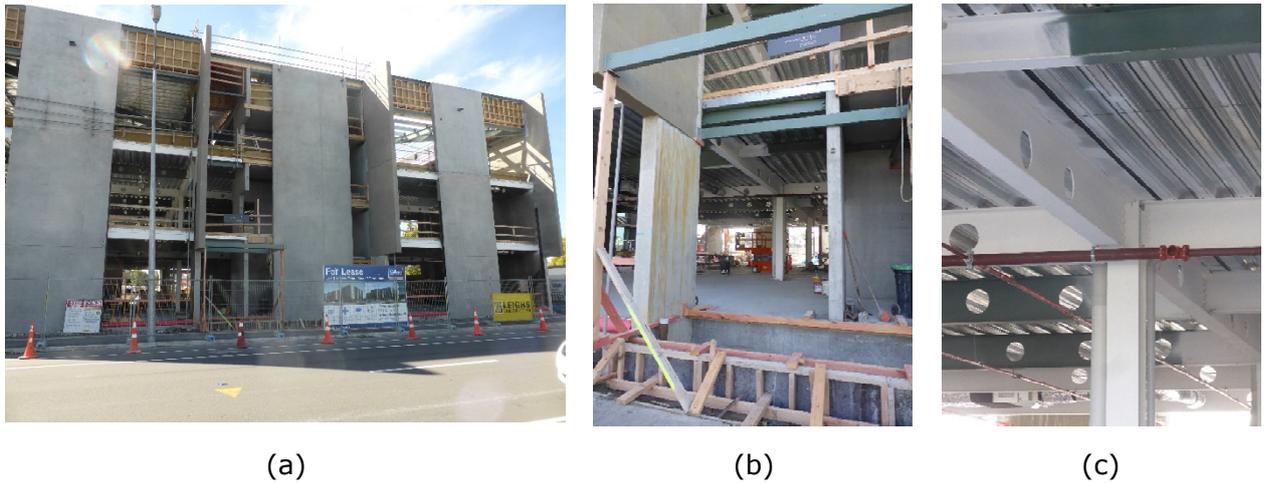


Figure 3-30: Precast concrete walls and gravity steel structure in the Forté Health building. (a) Global view, (b) Closer view of walls and gravity system, (c) Close-up of steel beams, columns, and decking

3.10 Timber construction

The authors did not notice any multistorey buildings having timber structural systems under construction in the Christchurch CBD during their visits. However, some completed projects have exposed timber structures, such as the Cathedral Grammar School buildings on Kilmore Street, shown in Figure 3-31.



Figure 3-31: Low-rise, open floor-plan timber structure used for the Cathedral Grammar School buildings. (a) Global view, (b) Close-up view of structural detail of timber beam and columns

3.11 Different structural systems in orthogonal directions

While many of the buildings observed during the “sidewalk surveys” had the same structural system in both directions, some had different structural systems in their orthogonal directions. The common example of that was when buildings shared divider walls along property lines. However, the use of different structural systems in orthogonal directions was also observed in buildings not sharing property lines with another building. An example follows.

3.11.1 160 Lichfield Street

BRBs were used in the north–south direction of the parking garage located on the southwest corner of Lichfield and Madras Streets (Figure 3-32). Core walls (around the staircases) as well as planar ones (along the parking ramps) are visible in the east–west framing direction.



Figure 3-32: Building at 160 Lichfield Street with BRBs in one direction and structural walls in the other. (a) Global view, (b) Wall view, (c) BRB connections

3.12 Hybrid structures

It is not uncommon, internationally, for buildings to have different structural systems in their orthogonal directions (as shown above). However, less frequently, buildings were encountered that had a combination of different structural systems in a given direction, or even different structural systems over their height. These are referred to here as hybrid structures.

3.12.1 Airport Hotel

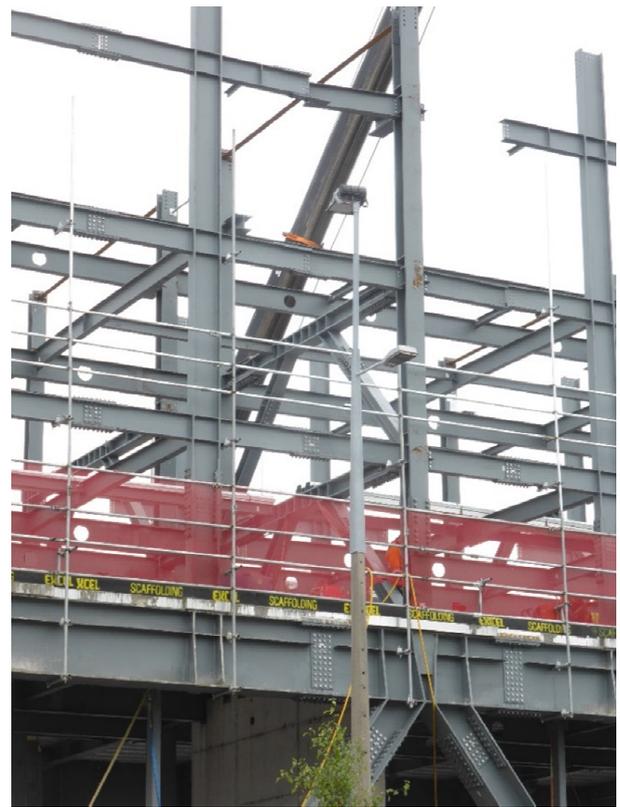
Not located in the CBD, but rather a stone’s throw from the Christchurch International Airport terminal, the hybrid structure shown in Figure 3-33 has concrete walls in the transverse direction and open braces in the longitudinal direction at ground level. At the other levels, EBFs are used in the transverse direction, and MRFs in the longitudinal direction.



(a)



(b)



(c)

Figure 3-33: Hybrid structure, Airport Hotel. (a) Global view, (b) View of concentric diagonals at ground level in longitudinal direction, (c) View of EBF in transverse direction and MRF in longitudinal direction about ground level

3.12.2 Awly Building, 287–293 Durham Street North

Interestingly, the base-isolated building located on the southwest corner of Durham Street North and Armagh Street (known as the Awly Building) could be considered a hybrid building, relying on MRFs at ground level and CBFs in the other floors above. Figure 3-34 shows the steel moment frames and braced frames. Figure 3-34b shows the same, somewhat hidden behind cladding, as well as the trench to accommodate base-isolation movements at the sidewalk level of the building.

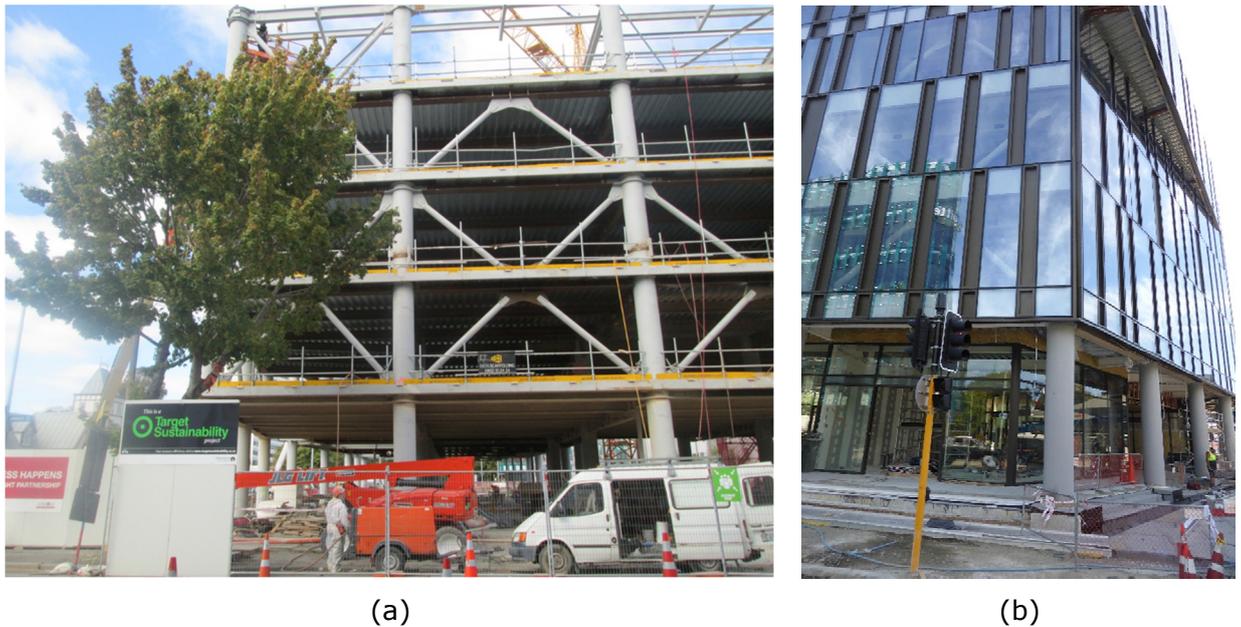


Figure 3-34: Hybrid structure example during construction, Awly Building. (a) In March 2016, (b) In March 2017

3.13 Architectural expression

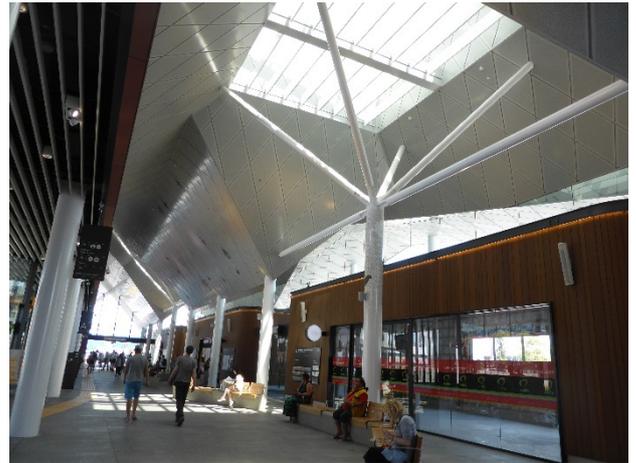
The structural systems of some buildings, while conventional in some ways, are best described separately as they are in-part defined by the architectural expressions of the building. An example is provided below.

3.13.1 New Bus Terminal

The new Christchurch bus terminal building consists of a freeform space-truss supported on either steel pipe columns or encased wide-flange columns. Select nodes of the space truss are rigidly welded to the top of the columns, as shown in Figure 3-35. Note that, as interesting as this structure might be, this is also a good example of a building that falls outside of the scope of this study, because single-storey buildings were not included in the data collected (as described in Section 4).



(a)



(b)



(c)



(d)

Figure 3-35: Christchurch new bus terminal. (a) Global exterior view, (b) Global interior view, (c) Encased steel column, (d) Connection detail at steel pipe column

4 Quantification Methodology

This section describes the methodology followed in conducting this study. This includes both the initially proposed methodology that was reviewed and approved by the UC Quake Centre, and small variations that occurred to the proposed methodology in light of new findings and understandings that emerged as the work progressed.

4.1 Project initiation

As may be gathered from the other sections of this report, this project was initiated because places in the developed world (i.e., modern cities in countries having well-developed seismic provisions) where there have been such significant post-earthquake rebuilding activities in such a short time are rare. More specifically, it was felt that the Christchurch experience could provide unique learning opportunities to earthquake engineers in other, similar countries because of the following several interesting factors about the rebuild:

- a. A significant amount of construction was done in a short period of time;
- b. The language of New Zealand is English, which facilitates the gathering of information while allowing other English-speaking people to easily understand the reasons for decisions made;
- c. New Zealand is a country with a “western” culture, and the seismic design approaches, standards, and forms have been developed in parallel and in a manner similar to that in some other western countries, such as Canada and the USA;
- d. New Zealand has been regarded as a leader in earthquake engineering in terms of both research and implementation of findings into New Zealand design codes and construction;
- e. Over the past 20 years, a large number of low-damage structural systems have been developed in New Zealand. Low-damage construction is applicable to buildings made with any type of structural material. Extensive research on such concrete, steel, and timber low-damage systems, most of which rely on steel dissipation devices, has been performed at the Universities of Canterbury and Auckland;
- f. The Christchurch earthquakes, together with the New Zealand insurance situation, provided the city with the land and finance to rebuild;
- g. Initial impressions were that the rebuild was dominated by steel construction. If that impression turned out to be true, this would indicate that the “new Christchurch” would be quite different from the concrete city that existed before the earthquakes;
- h. The open society of New Zealand, which has a relatively low level of liability compared to some countries in the world, makes it possible for engineers and other stakeholders to speak openly and candidly;
- i. The fact that Christchurch is home to the University of Canterbury, where almost all of Christchurch’s structural engineering practitioners have

graduated from, means that relationships and trust already exist between the University and practitioners;

- j. Various anecdotes were circulating worldwide, professing to explain the reasons for the apparent changes in structural material and structural forms used in Christchurch's reconstruction, but there was no study documenting:
 - i. What was actually being built in terms of structural material and form, and
 - ii. Why structures were being constructed the way they were.

In light of the above, the authors considered that there was a need to study this unique rebuild situation. This was to be done by quantitatively documenting what was built. Also, by collecting a sufficient number of individual stories from key stakeholders, the authors could develop a robust narrative indicating the range of factors that drove decisions related to the type of structural systems selected as part of the reconstruction activities. The support of the Quake Centre made the realisation of this project possible.

4.2 Methodology

The work conducted as part of this project was undertaken in four stages, namely:

- a. A street survey;
- b. A scoping exercise;
- c. Data collection; and
- d. Data analysis and writing of the report.

Some methodology issues related to each of these tasks are presented below. Recall that the scope of the study was limited to multistorey buildings in the Christchurch CBD and Addington areas. The Christchurch CBD is defined as the area bounded by the four avenues: Moorhouse, Bealey, Fitzgerald, and Deans.

4.2.1 Street Survey (2013–2016)

The street survey consisted of walking around Christchurch at various times, documenting and taking photos of the building construction. This was conducted by Bruneau when he visited New Zealand, as well as by MacRae on his frequent visits to the CBD over the time of major reconstruction. Photos were taken and a paper was published at the 2016 Australian Earthquake Engineering Society Conference (MacRae et al., 2016), describing some preliminary findings from this activity.

4.2.2 Scoping Exercise (2016)

The scoping interviews were held with ten different companies/individuals in their offices, as listed in the Appendix. Some of the companies approached for interviews had an existing relationship with the authors, and some were taken from a spreadsheet list of new buildings being constructed in Christchurch, provided by Steel Construction New Zealand (SCNZ). SCNZ developed this spreadsheet list under the leadership of Kevin Cowie. It was compiled by SCNZ engineer Zahid Hamid, who visited Christchurch every 3–6 months to drive around and identify new multistorey construction projects in the Christchurch CBD, as well as in Addington, where other significant multistorey construction was occurring. Based on the information collected during these reconnaissance visits, SCNZ requested specific building information for the addresses of the new buildings from the Christchurch City Council (CCC), to populate its spreadsheet list of on-going reconstruction projects. This spreadsheet list was kindly shared with the authors for this project. Information provided in the spreadsheet list included building name, site address, building function, structure type, and estimated floor area, as well as information on the engineer, architect, and builder, on the status of the project, and on the type of lateral-force-resisting systems and decking used. The spreadsheet list was expanded by SCNZ as construction continued. As mentioned above, the initial version of the spreadsheet list was used in part to identify some of the structural engineers to interview.

The goals of the interviews conducted as part of the scoping exercise were to:

- a. Work with the engineers to establish the best methodology to collect data towards the final project objectives;
- b. Obtain an initial indication of the likely drivers for the rebuild decisions;
- c. Determine how engineers could contribute to the work and make it relevant, other than providing information themselves;
- d. Consider methods to improve project quality and impact;
- e. Determine tasks to be conducted before the 2017 visit.

The interviews lasted an average of two hours each. The outcomes were as follows.

- a. Engineers considered the project to be worthwhile, and indicated that they were interested to hear how other stakeholders considered various issues. A number of options were discussed to collect the information and achieve the project objectives. Through the discussions, it emerged that the best way for engineers to provide quality information was not to meet at a seminar or conference and share information there (or other similar workshop-like approaches), but rather to be visited in their own offices to talk through their experiences with individual buildings. It was also suggested that focusing on the engineers with the greatest number of buildings in the spreadsheet list would best represent the industry.
- b. The authors became familiar with the language of the engineers and some of the factors influencing this rebuild. This helped them focus their questions in the 2017 interviews.

- c. Some engineers indicated that they liked the option to provide case studies of one or two particular structures that would help showcase specific reconstruction projects and that could be used to provide project-specific examples and decision drivers. These would be included with their company logo in the back of the report. This approach was also considered beneficial for the report, where individual building information would be anonymised, as it would explicitly feature real buildings with examples of different structural systems used as part of the Christchurch reconstruction.
- d. It was suggested that to get a wider view of the decisions affecting structural form, it was also necessary to speak to other key decision makers to obtain architectural, project management and client perspectives.

Summaries of all conversations were documented.

Discussions also took place with:

- a. The Quake Centre. The project was also advertised with featured interviews on the Quake Centre website (UC Quake Centre, 2016a; UC Quake Centre, 2016b);
- b. The SESOC executive manager;
- c. SCNZ, about the spreadsheet list;
- d. The CCC Chief Resilience officer.

4.2.3 Data Gathering

The data assembled by the authors on individual buildings and used for the quantitative part of this study was collected from a few different sources, including spreadsheet lists from the CCC and SCNZ, in-person interviews with structural engineers, field work, and web searches. No single source contained all the information needed for this project.

The CCC kindly provided information on building consents issued between 22 February 2011 and 8 March 2017 (equivalent to building permits in a North American context). As a subset extracted from the CCC's full database, focusing only on new buildings for which structural engineering design was needed and not including single-family houses and other constructions not of interest to this study, the information provided included the following: date building consent was issued, consent number, floor area, dwelling units, consent value, building type, intended use, site address, suburb, description of building work, and a "complexity" descriptor. The information was very helpful, but required sorting because it included information on all construction activities conducted over the entire area of jurisdiction of the CCC, discretised down to each individual permit, including small items such as temporary permits to put up marquees. The information received from the CCC was used to cross-reference and complement the information from the SCNZ spreadsheet list (itself, revised and updated by SCNZ with all information collected as of February 2017).

Hence, the final spreadsheet list used in this project was based on the SCNZ spreadsheet list, but included information from the CCC database, as well as some additional buildings identified during the interviews with structural engineers. As neither set of information (from SCNZ or the CCC) was complete, additional information was collected by (as necessary, on a case-by-case basis) conducting visual inspections of buildings; collecting information on specific buildings during interviews with engineers; and using Google Street View to verify the number of storeys of specific structures, validate street addresses, and/or observe stages of construction through time. Also, where information was unclear, this was highlighted in interviews/discussions with the companies at the data-gathering stage.

For the purpose of data analysis, the spreadsheet assembled by the authors contained, in addition to the information from the SCNZ and the CCC, details on the structural systems, obtained during interviews with the practicing engineers or subsequently confirmed via email. The columns of that spreadsheet provided information relating to:

- a. The type of project – indicating if it were public or private;
- b. The year of building consent;
- c. The building’s floor area in square metres;
- d. The number of storeys, obtained from either the CCC database, Google Street View (when it was up to date), the Christchurch Rebuild Facebook page, or site inspections;
- e. The type of structural system used, identifying the following categories:
 - *Base Isolation or Dampers*
 - *X-Direction Structural System Material*
Steel, Concrete, Timber, or Masonry
 - *X-Direction Structural System Type*
BRB, CBF, EBF, EBF with replaceable link, MRF, MRF with friction connections, MRF with RBS, RC Wall, RC Frame, Rocking Frame Steel, or Rocking Precast Walls; and whether or not the system was hybrid
 - *Y-Direction Material*
Steel, Concrete, Timber, or Masonry
 - *Y-Direction Structural System Type*
BRB, CBF, EBF, EBF with replaceable link, MRF, MRF with friction connections, MRF with RBS, RC Wall, RC Frame, Rocking Frame Steel, or Rocking Precast Walls; and whether or not the system was hybrid
 - *Gravity Structural System Material*
Steel, Concrete, Composite, or Masonry;

- f. The decision related to selection of structural system, indicating:
 - If the owner/client requested either: Base isolation, IL3 building, IL4 building, low-damage solution, no damage, fastest construction time, a showcase structure, or lowest cost construction,
 - If the engineer chose the structural system,
 - If the decision regarding type of structural system was influenced significantly by soil conditions, construction or fabrication cost, construction time, or building site layout;
- g. Useful websites providing information related to the specific buildings considered.

Interviews were conducted to obtain information on the decision-making process that led to the choice of structural systems for specific buildings and to validate the accuracy of the collected data. Time and resource constraints made it impossible to conduct interviews with all the structural engineering firms that have been involved in the reconstruction of Christchurch. Hence, to consider a large number of buildings while keeping the project manageable, the consultants to be interviewed were selected based on the number of buildings their firm had designed that were constructed or being constructed as part of the Christchurch recovery. (Note that, at the time of the study, many firms only had one or two such buildings.) The final ten consultants selected for interviews were responsible for over 65% of the multistorey buildings, and a slightly greater percentage of the total floor area, in the spreadsheet list assembled for the Christchurch CBD and Addington areas. (Note that the ten companies selected for the data-gathering activities were not the same as those interviewed during the 2016 scoping exercise, although there was significant overlap.)

Before each of the interviews, when applicable, a copy of the notes compiled from the previous 2016 meeting was sent to the consultant for review, to verify the accuracy of these prior conversations. An early draft of the observations from the "sidewalk survey" and a draft outline of this report was also provided to all consultants prior to the meetings.

Interviews with structural engineers were complemented by interviews with an architect, a project manager, and a developer/client, kindly arranged by John Hare of Holmes Consulting Group. These interviews addressed, from the respective perspectives of these stakeholders, general trends affecting the reconstruction activities in Christchurch, the way their industry works, interactions between different players, and specific buildings. Finally, interviews with engineers in Wellington and Auckland were kindly scheduled by Robert Finch of the Quake Centre.

Most interviews lasted approximately two hours (a few were longer), enabling up to four interviews per day, although in most cases, one or two interviews were conducted per day. It was emphasised during all interviews that information

related to specific structures would only be included in the report in an aggregated manner, and specific buildings would not be identified, except for those studied during the sidewalk survey presented in Section 3 or volunteered by the engineers as part of the case studies included in the Appendix.

Topics covered in these interviews included:

- General comments about the Christchurch rebuild and the drivers for decisions made as part of this process;
- Discussions about specific buildings, and determination of which factors (of the potential factors identified in the working version of the spreadsheet list) drove the selection of the structural system for these specific buildings;
- The opportunity for engineers to complete specific case studies for inclusion in the Appendix of the report. A template and example were provided for this purpose.

Anecdotally, an aftershock of the Kaikoura earthquakes occurred during one interview, leading the engineer to indicate that the sequence is still continuing, keeping earthquake issues in the minds of their clients.

During the interviews, engineers also brought to the authors' attention relevant buildings not included in the spreadsheet. This information was added to the working spreadsheet when the engineers provided the complete set of data for the missed buildings.

The "cooperation rate" for this study is defined as the number of firms/individuals who have agreed to meet with the authors and be interviewed for the purpose of this report, divided by the total number of firms contacted. The 2017 interviews were conducted with the ten engineering consultants in Christchurch for which quantitative data was collected, four firms in Wellington, two firms in Auckland, one architect, one developer/client, and one project manager. One Christchurch engineering consulting company who had done a small number of CBD buildings declined to meet. The cooperation rate for this study is therefore $(10+4+2+1+1+1) / (11+4+2+1+1+1) = 95\%$. Note that one engineering consulting company which had been responsible for many structures referred us to another company, which was formerly one of their offices. The office had decided to become independent of the former parent company and form a new company. The engineers in this new company held the experience and knowledge regarding the buildings of interest to this study, and agreed to interviews.

4.2.4 Data Analysis and Report Writing

Data from the spreadsheet list was reduced so that the number of specific systems built could be understood. The date of the constructed facilities was taken as the consent date. The number of systems, cumulative number as well as the percentage of different types of system, were extracted and plotted. In particular, for buildings with two orthogonal (or nearly orthogonal) frames in different directions, the system in one direction was attributed as half of a building, and that in the other was considered to be the other half of the building.

Specific spreadsheet columns were populated with the following information:

- a. The number of buildings constructed with a specific construction material against time;
- b. The floor area of buildings constructed with a specific construction material against time;
- c. The number of storeys of buildings constructed with a specific construction material against time;
- d. The number of buildings constructed with a specific lateral-force-resisting system against time;
- e. The floor area of buildings constructed with a specific lateral-force-resisting system against time;
- f. The number of buildings constructed with a specific lateral-force-resisting system against time, excluding the base-isolated structures;
- g. The floor area of buildings constructed with a specific lateral-force-resisting system against time, excluding the base-isolated structures;
- h. Owner and engineer decision drivers versus time.

To ensure that the information obtained from the interviews was complete and accurate, the portion of the spreadsheet list relevant to each company was sent to them to validate the accuracy of the reported data. (Even though the engineering firms were solicited to verify the collected data, accidental omissions are still possible, but hopefully minimised.) Also, summaries of discussions were written and discussed by Bruneau and MacRae to obtain the final document. Case studies were included in the document. The draft report, or relevant portions of it, was also sent to those interviewed for their review and possible comments, providing all those interviewed with a final opportunity to advise.

It was planned that excerpts and summaries from the report would be published and presented in refereed journals, conferences papers, and other avenues.

4.3 Project timeline

Some of the milestones in the project timeline are:

16 March, 2015	Authors approached Robert Finch, Director of the Quake Centre, regarding support for a project of this type
29 May, 2015	Formal proposal submitted to the Quake Centre
3 December, 2015	Proposal approved at Quake Centre board meeting
13–24 February, 2016	Bruneau visited Christchurch, to start project with MacRae; conducted trial interviews and developed methodology
10 February–27 March, 2017	Bruneau visited New Zealand to conduct work with MacRae on core of the project, including Stage 2 interviews in Christchurch (supplemented by interviews in Wellington and Auckland by Bruneau)
1 April–30 June, 2017	Authors finalised spreadsheets with feedback from engineers and information from CCC, and wrote draft project report
15–31 July, 2017	Draft report shared with those interviewed as part of this project, to ensure that anonymity of information has been respected
18 July, 2017	Final report completed and submitted to Quake Centre

4.4 Confidentiality

The list of individuals interviewed by the authors as part of this Quake Centre study is provided in Appendix A. Participation in the interviews was on a voluntary basis, and no compensation of any kind was provided for this purpose. All interviewees were given a copy of this report prior to publication and asked whether they agreed to be identified or preferred to remain anonymous. The list in Appendix A reflects their decisions. Finally, except for the publicly accessible information collected from the sidewalk survey presented in Section 3, and for information readily available on public websites, the findings in this report are purposely presented in a manner that does not explicitly identify the buildings for which specific decisions were made by those interviewed here or their clients. This was done to ensure candid discussions as part of the interviews.

5 Quantitative Findings

Data collected during the 2016 and 2017 interviews has been compiled into a spreadsheet, per the methodology described in the previous section, and used to generate the quantitative information presented below. As mentioned earlier, this study on Christchurch's reconstruction focuses on new engineered buildings of more than one storey located in the CBD and Addington areas, and the buildings considered (as indicated in the methodology) are those designed by the ten Christchurch engineering firms interviewed.

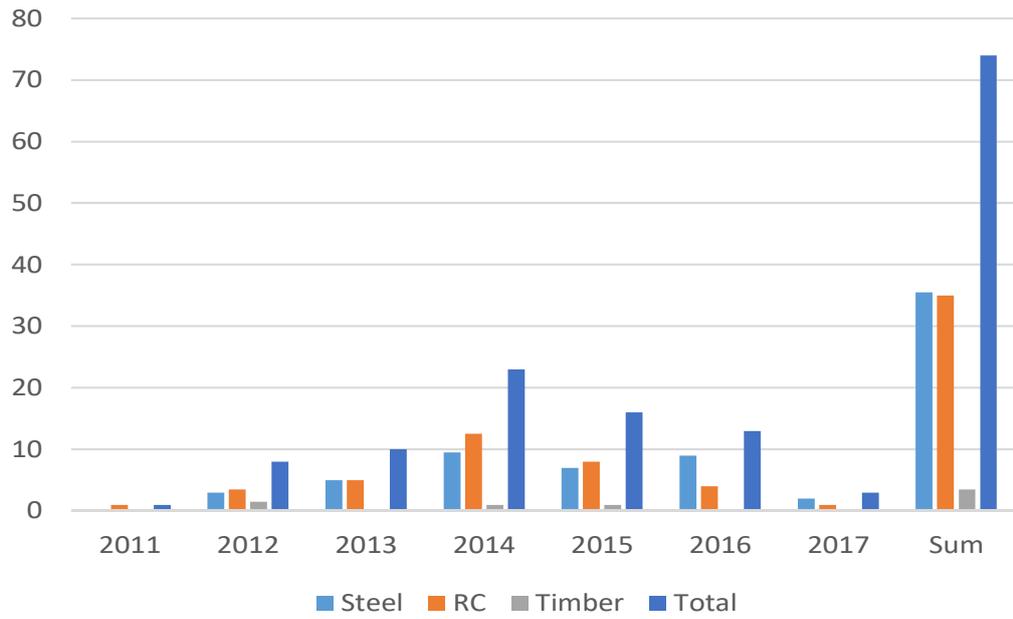
Note that for all figures below where information is presented as a function of year of consent, it must be recognised that results for 2017 are only for the first three months of the year (as data was collected, and last interviews were conducted, in March 2017).

Figure 5-1a presents the number of new buildings having steel, RC, or timber lateral-load-resisting structural systems. Data has been obtained for a total of 74 buildings, collectively covering a total of 482,317 square metres of floor space. For those buildings that had different types of structural systems in orthogonal directions, each direction was counted as 0.5 of a building when tallying the numbers. Also note that one building had masonry walls in one direction (counted as 0.5 building), but this small number was lumped together with the RC walls. Results in Figure 5-1a show that building consents were granted for almost an equal number of buildings having steel and RC lateral-load-resisting systems in each of the years following the earthquake, except for 2016 which saw 9 steel buildings versus 4 RC buildings. In total, over the 7-year period considered, for the 74 buildings considered, 35.5, 35, and 3.5 had steel, RC, and timber lateral-load-resisting systems, respectively.

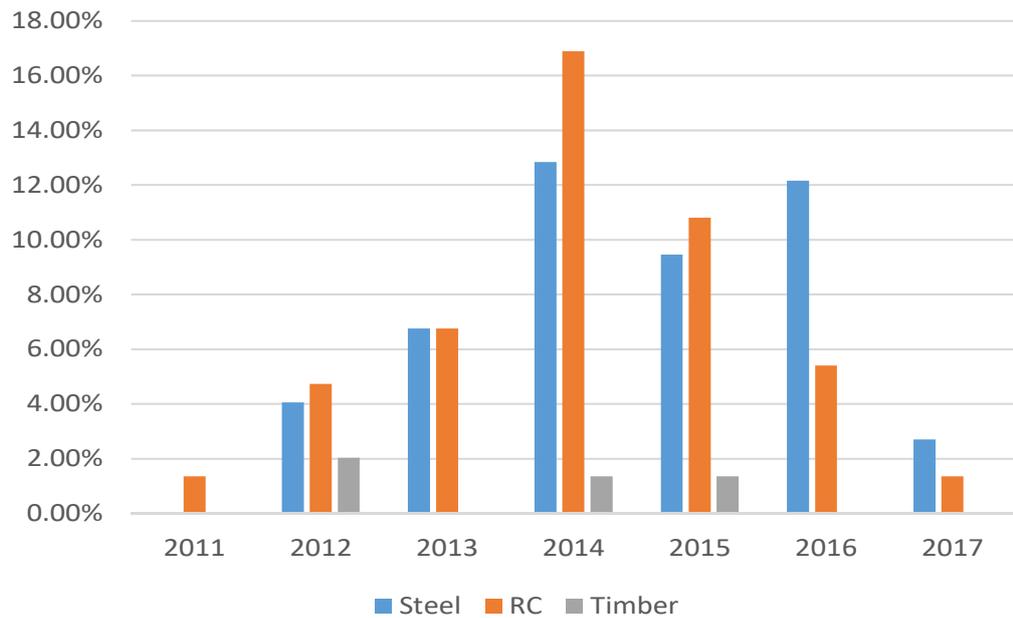
Figure 5-1b shows the same information in terms of percentages. Totals are not shown, but the 100% breaks down into a sum (for all the years considered) of 48%, 47%, and 5% for steel, RC, and timber, respectively.

Figure 5-1c shows the cumulative number of buildings having lateral-load-resisting systems of each material, progressively populating the parts of Christchurch considered in this study. Again, this shows steel and RC almost on par in terms of number of buildings.

The data gathered related primarily to the lateral-force-resisting systems. However, it was noted that gravity-resisting frame systems made of steel beams and columns were not only used in structures having steel lateral-load-resisting systems, but were also used in approximately three-quarters of the buildings having a lateral-force-resisting system that consisted of RC walls. For this reason, the number of buildings containing structural steel is significantly greater than that indicated by the quantitative results presented in this section.

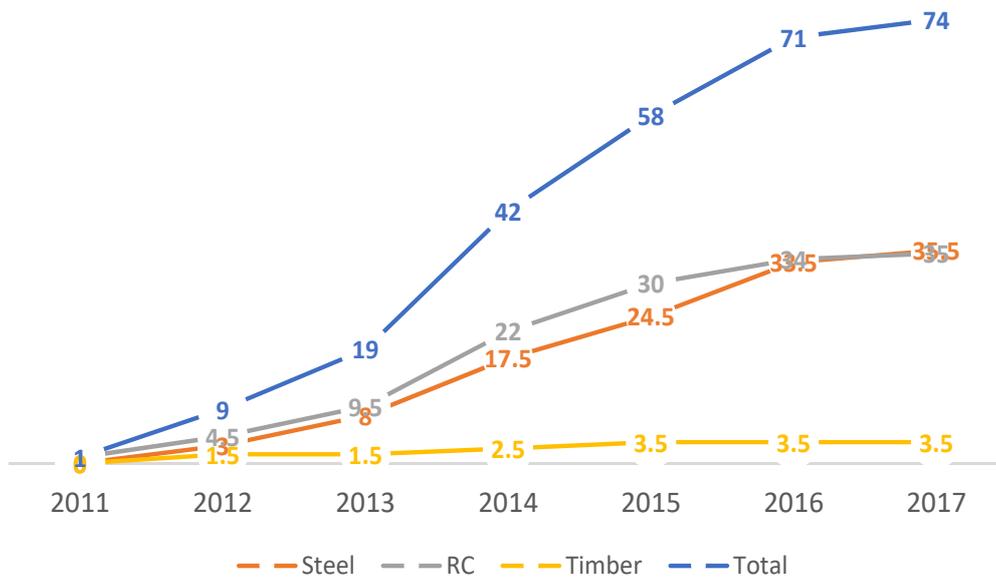


(a)



(b)

Figure 5-1: Number of new buildings having lateral-load-resisting systems of each material type. (a) Absolute numbers, (b) Percentages, (c) Cumulative numbers



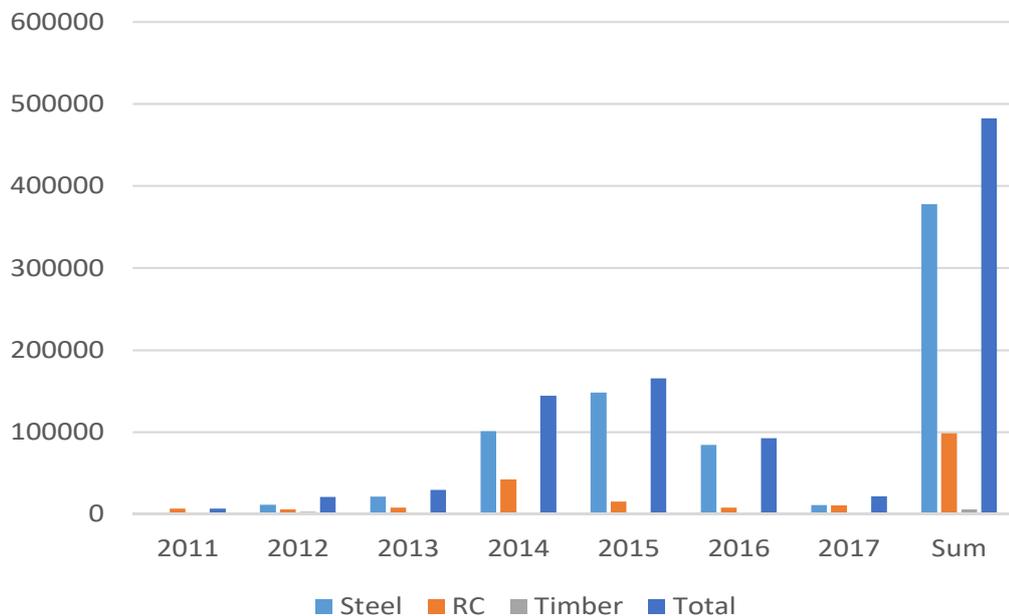
(c)

Figure 5-1 (continued): Number of new buildings having lateral-load-resisting systems of each material type. (a) Absolute numbers, (b) Percentages, (c) Cumulative numbers

Figure 5-2 presents the total floor area (in square metres) of new buildings having steel, RC, or timber lateral-load-resisting structural systems. In essence, for the same 74 buildings used in Figure 5-1, the results in Figure 5-2a show that steel, RC, and timber lateral-load-resisting systems have been used in buildings with floor space respectively totalling 377,929m², 98,572m², and 5,816m², for a total of 482,317m² of floor space. This corresponds to 78.4%, 20.4%, and 1.2% of the total floor area for the three materials respectively. If the gravity systems were to be included in the above numbers, the total floor area supported by structural steel would be further increased.

As shown in Figure 5-2b, in 2015 and 2016, building consents were issued for more floor area in buildings with steel lateral-force-resisting systems than buildings with RC lateral-force-resisting systems, at approximately 10 times more square metres of floor area. “Drilling” through the spreadsheet data indicates that most buildings relying on RC walls tend to be smaller buildings, and that the larger buildings have steel framing systems of one kind or another. This confirms the impression one gets when walking the streets of the CBD that Christchurch has had a “seismic shift to steel”.

Figure 5-2c shows that, in terms of floor area, the rate at which RC was built peaked in 2014 and decreased by more than half afterward (from floor areas of 42,500m² in 2014 to 15,500m² in 2015). For steel, rates peaked in 2015, with a decrease in the subsequent year (from floor areas of 148,000m² in 2015, to 84,500m² in 2016).



(a)

Figure 5-2: Floor area of new buildings having lateral-load-resisting systems of each material type. (a) Absolute numbers, (b) Percentages, (c) Cumulative numbers

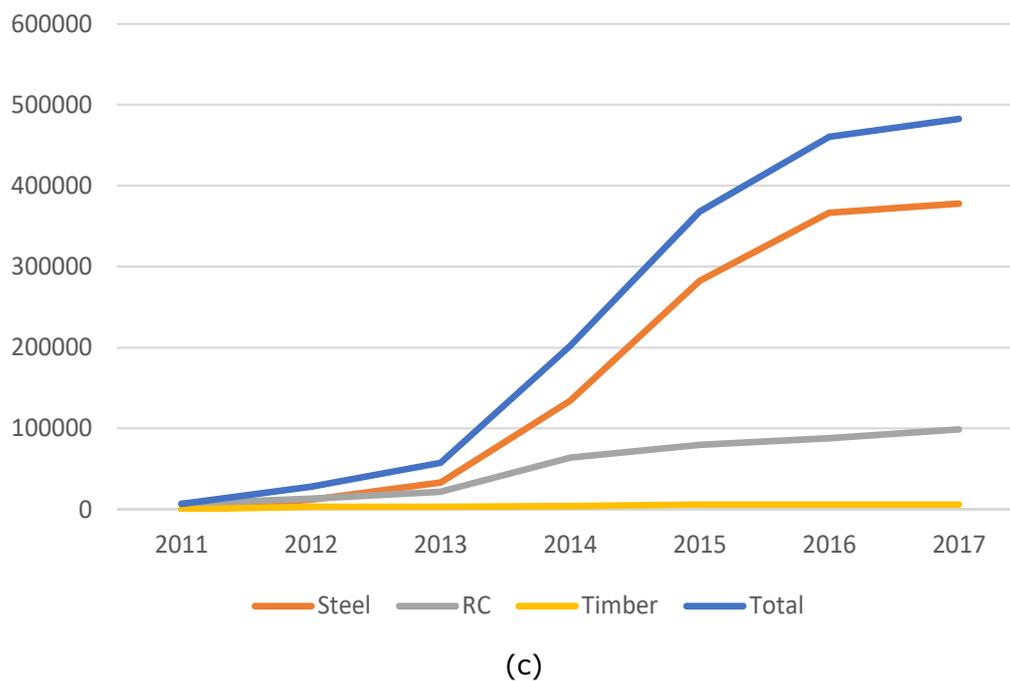
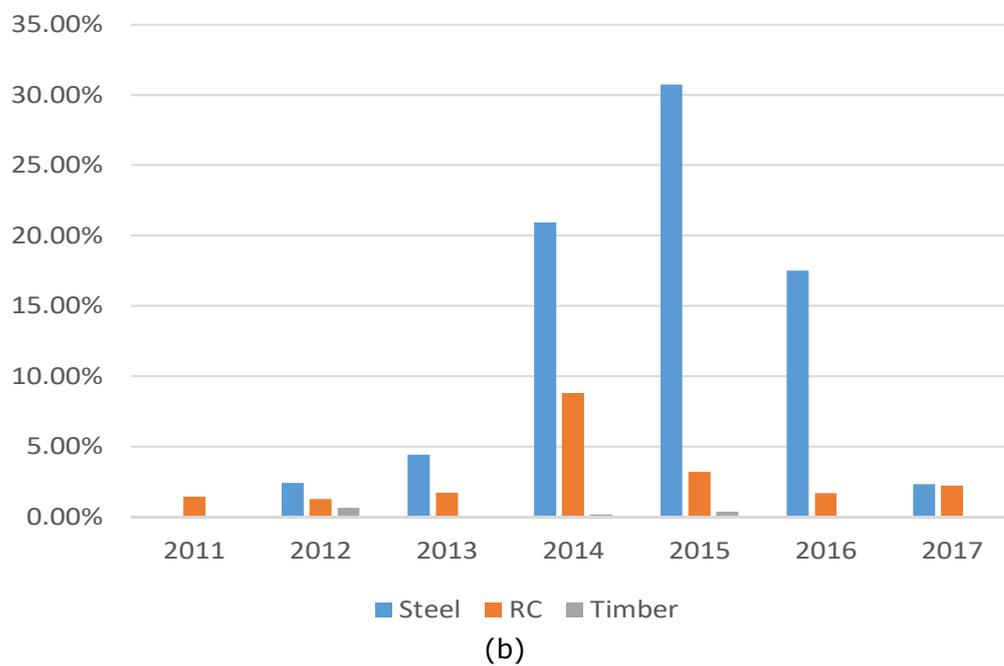


Figure 5-2 (continued): Floor area of new buildings having lateral-load-resisting systems of each material type. (a) Absolute numbers, (b) Percentages, (c) Cumulative numbers

Figure 5-3 presents the number of new buildings having different types of lateral-load-resisting structural systems. In this figure, structural systems have been broken down into the following categories:

- BRB = Buckling Restrained Braces (11 total)
- CBF = Concentrically Braced Frames (3 total)
- EBF = Eccentrically Braced Frames (2 total)
- EBR = Eccentrically Braced Frames with replaceable links (4 total)
- MRF = Steel Moment-Resisting Frames (9.5 total)
- MFF = Steel Moment-Resisting Frames with friction connections (1 total)
- MRF = Steel Moment-Resisting Frames with Reduced Beam Sections (4.5 total)
- RCW = Reinforced Concrete Walls (32.5 total)
- RCF = Reinforced Concrete Moment-Resisting Frames (0.5 total)
- RFS = Rocking Frame Steel (1.5 total)
- RFC = Rocking Frame Concrete Precast Walls (0.5 total)
- LVL = Laminated Veneer Lumber (2.5 total)
- B = Base Isolation (11 total)
- D = Dampers (2 total)
- H = Hybrid (7 total).

Data has been obtained on the same 74 buildings, but the building with masonry walls in one direction (0.5 masonry building) and a building with braced plywood walls (1.0 building) are not included in the figures, reducing the sum to 72.5 buildings. Also, for the results presented, a structural system on top of base isolators is counted twice (once as the structural system type, and once as a base-isolated structure). Likewise, buildings having steel moment-resisting frames and dampers are counted twice (once for frames, and once for dampers), as are hybrid buildings. This explains the higher total of 92.5 buildings obtained (=72.5 systems + 11 base-isolated buildings + 2 with dampers + 7 hybrids).

From the results, RC walls are shown as having the largest number of implementations, simply because this category (contrary to the others) has not been broken down into sub-categories. Data shows that there have been few MRF with friction connections, rocking systems, LVL systems, CBFs, and buildings with dampers. Incidentally, the 0.5 RCF is located on top of base isolators (for reasons described in Section 6).

For more clarity, cumulative results are shown for the most popular structural systems grouped together in Figure 5-3b, and without the RC results in Figure 5-3c, to better distinguish the other trends. In other words, in these figures, both types of EBF have been added together, and all three types of steel MRF have been also combined. On that basis, within the steel-frame systems, the most commonly used are BRBs (in 11 buildings), steel MRFs (in 15 buildings), and EBFs (in 6 buildings).

Results in Figure 5-3a show a rapid implementation of base isolation and rocking systems in the early years after the earthquakes, with fewer numbers in the past few years. Base-isolated buildings were first consented in 2012, with the number of building consents rapidly growing, then plateauing at a total of 11 by 2015 (i.e., 15% of the 74 buildings considered). BRBs were first consented in 2014, and the results show that they have since grown in number at a steady pace, reaching a total of 11 by 2017.

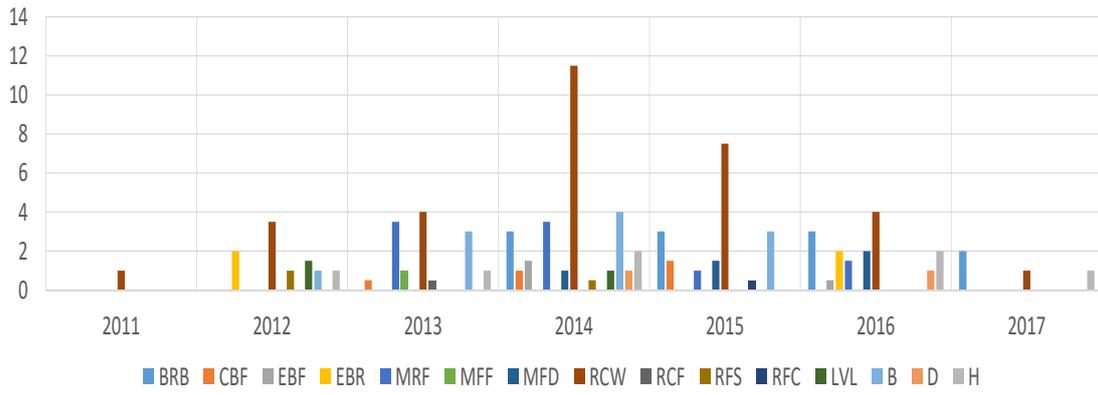
To better understand the significance of these results in terms of building size (by analogy to Figure 5-2), information on the same structural systems shown in Figure 5-3 is presented in Figure 5-4 in terms of floor areas (in square metres) per structural system.

Again, cumulative results are shown for the most popular structural systems grouped together in part b of the figure, and without the RC results in part c, to better distinguish the other trends.

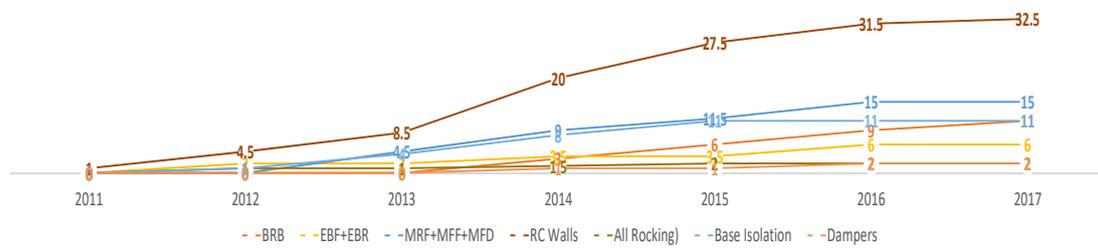
Results show that the following lateral-load-resistance systems have been used for buildings totalling the following floor areas:

- BRB: 111,000m² (23%)
- CBF: 38,500m² (8%)
- EBF+EBR: 27,500m² (6%)
- MRF+MFF+MFD: 202,000m² (43%)
- RCW: 80,400m² (17%)
- RFS+RFC: 15,000m² (3%)

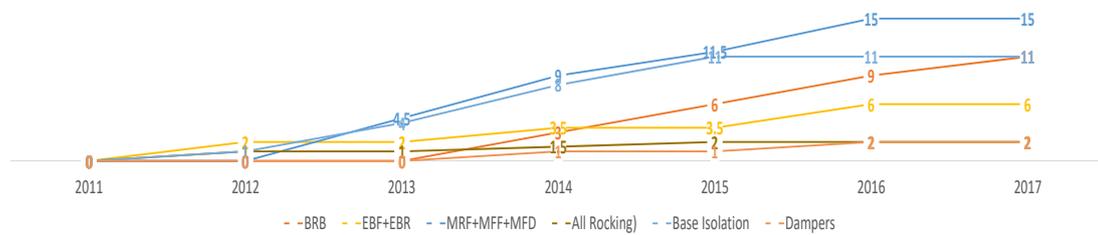
Interestingly, the 11 base-isolated buildings (15% of the total number of buildings) alone provide a total floor area of 190,000m², equivalent to 40% of the total floor area of the buildings considered in this study. This indicates that the base-isolated buildings are generally large buildings. Indeed, the two largest base-isolated buildings alone, built specifically for public-sector tenants, add up to more than 102,000m² (21% of the total floor area of the buildings considered here). Considering the three largest base-isolated buildings instead adds up to 129,000m² (and 27% of the total floor area). Also note the strong correlation between floor areas for base-isolated buildings and steel MRFs.



(a)



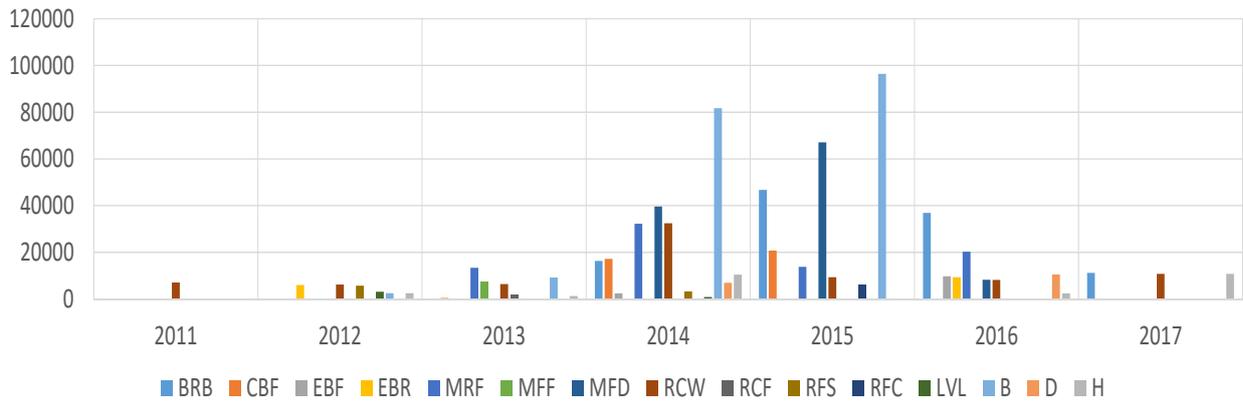
(b)



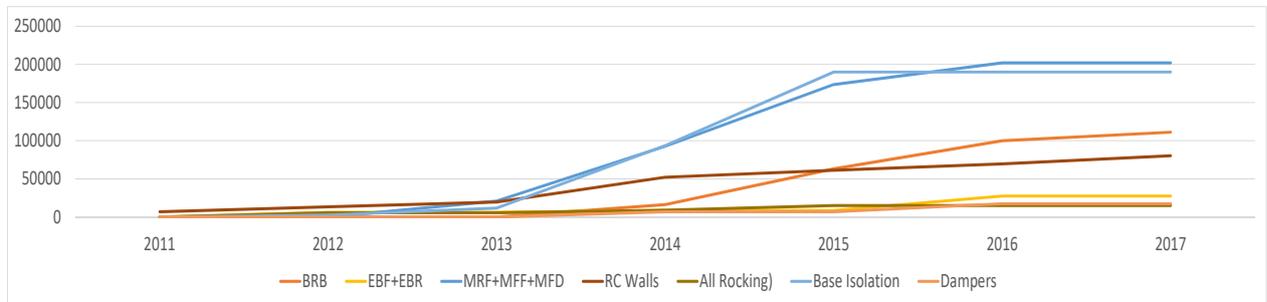
(c)

Figure 5-3: Number of new buildings with various types of lateral-load-resisting systems. (a) Absolute numbers, (b) Cumulative numbers for some structural systems (regrouped as shown), (c) Cumulative numbers for sub-set of structural systems

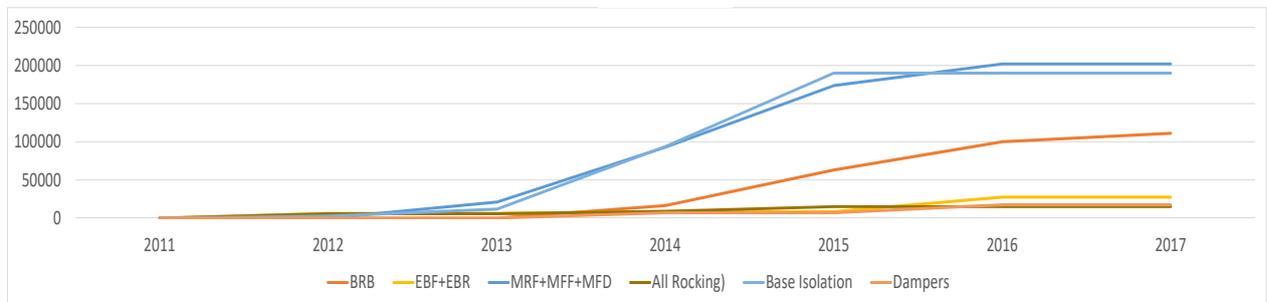
Quantitative Findings



(a)



(b)



(c)

Figure 5-4: Floor area of new buildings with various types of lateral-load-resisting systems. (a) Absolute numbers, (b) Cumulative numbers for some structural systems (regrouped as shown), (c) Cumulative numbers for sub-set of structural systems

To better understand trends in design, Figures 5-5 and 5-6 show the same results as in Figures 5-3 and 5-4, but for all structures that have not been base-isolated. This is worthwhile information for a number of reasons. First, given that findings from interviews (presented in Section 6) seem to indicate that base isolation has been more readily desired by owners in the years immediately following the earthquakes than in recent years, data on non-isolated buildings could be indicative of trends in a continued reconstruction scenario further away from the initial damaging event (assuming a reconstruction period free from other nearby damaging earthquakes “refreshing” the collective memory). Second, engineering firms that are comfortable designing types of structural systems that fall outside of the design standards (such as base isolation) might have been over-represented among the 10 engineering firms that have been interviewed. (These firms were selected because they were the most active in the Christchurch reconstruction, as described by the methodology presented earlier). Third, for its own sake, it is interesting to identify which structural systems have been more dominantly used when buildings have not been base isolated.

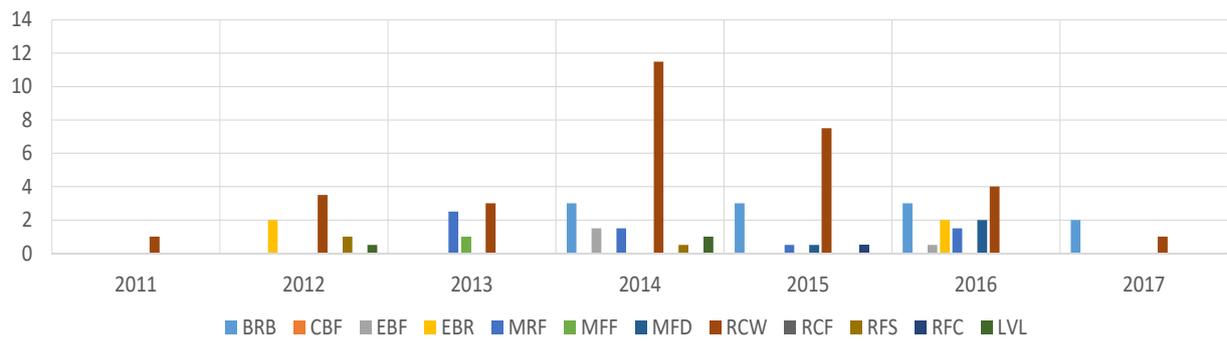
Figure 5-5 shows the results in terms of number of buildings, and Figure 5-6 provides the same break-down in terms of floor area.

Given that steel structures have been used in all base-isolated structures surveyed, except for one RC one (half walls and half moment frame), one timber one, and one with RC walls in one direction and CBF in the other direction, the number of buildings with RC walls is one less than before, but now constitutes a proportionally larger percentage of the total number of buildings ($31.5/63=50\%$). Figure 5-5 also shows that for all steel-frame systems, the most commonly used are BRBs (in 11 buildings), MRFs (in 9.5 buildings), and EBFs (in 6 buildings). This also shows that most base-isolated buildings had steel MRFs or CBFs.

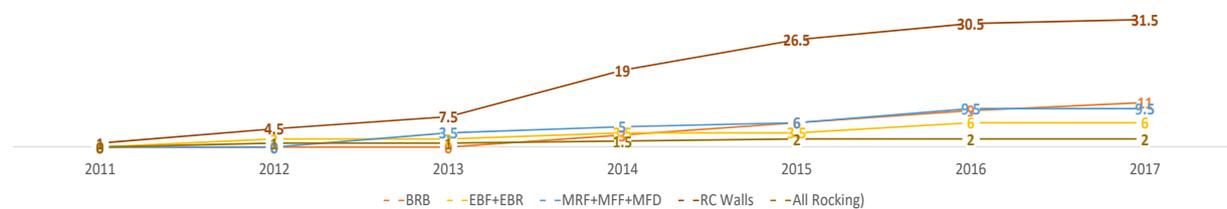
Similarly, Figure 5-6 shows that contribution of lateral-load-resistance systems to total non-base-isolated reconstruction floor area is:

- BRB: 111,000m² (38%)
- CBF: 0m² (0%)
- EBF+EBR: 27,500m² (9.5%)
- MRF+MFF+MFD: 57,000m² (20%)
- RCW: 78,000m² (27%)

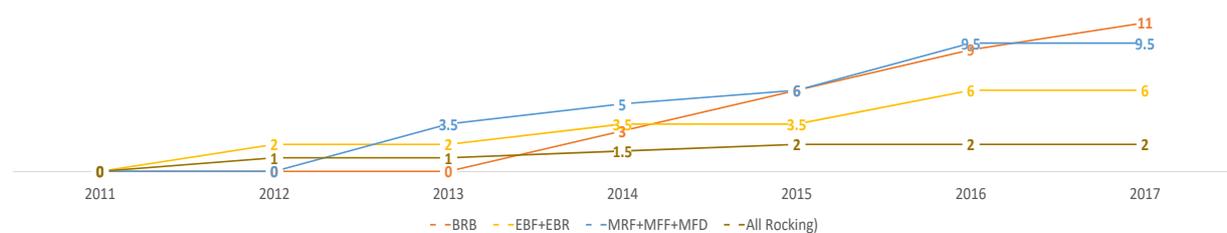
As such, with respect to new non-base-isolated buildings, RC lateral-load-resisting systems have been used for 27% of the floor area, and steel for 68% of the floor area.



(a)

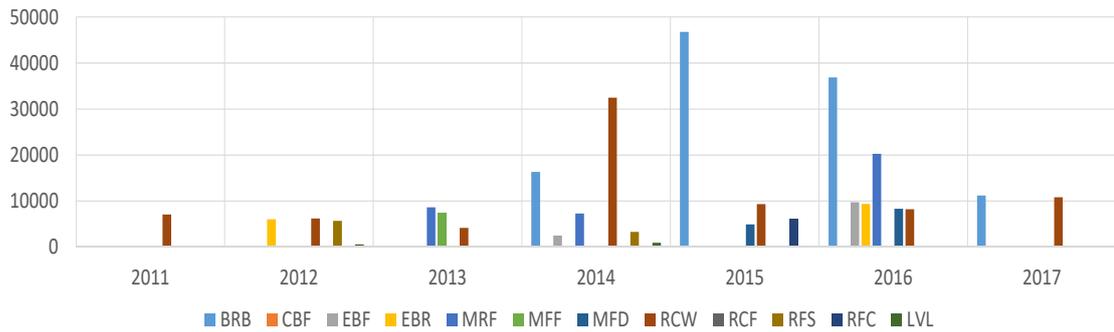


(b)

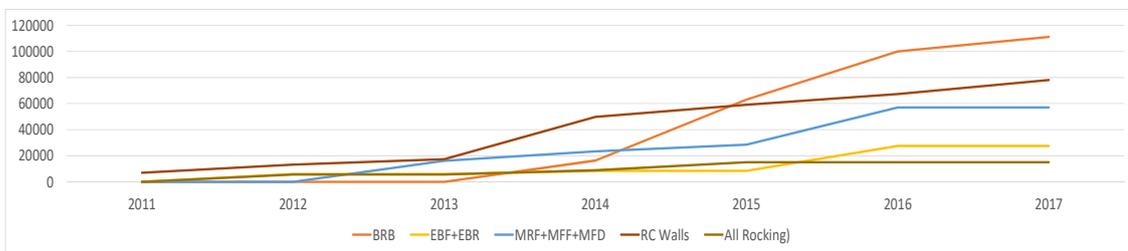


(c)

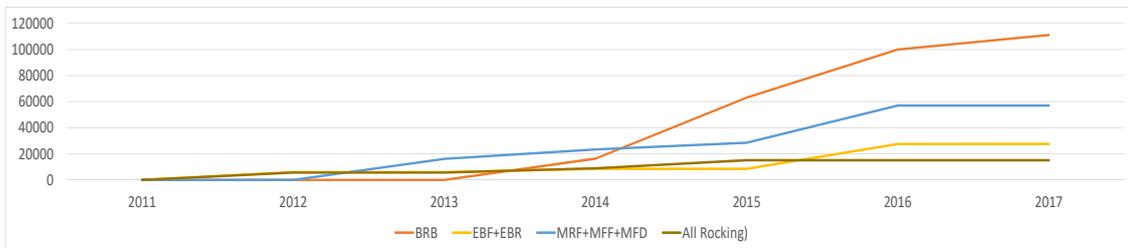
Figure 5-5: Number of new non-base-isolated buildings with various types of lateral-load-resisting systems. (a) Absolute numbers, (b) Cumulative numbers for some structural systems (regrouped as shown), (c) Cumulative numbers for subset of structural systems



(a)



(b)



(c)

Figure 5-6: Floor area of new non-base-isolated buildings with various types of lateral-load-resisting systems. (a) Absolute numbers, (b) Cumulative numbers for some structural systems (regrouped as shown), (c) Cumulative number for sub-set of structural systems

Figure 5-7 presents the factors that were identified as driving decisions regarding the choice of structural systems. The extensive interpretations presented in Section 6 provide an in-depth understanding of the data presented in Figure 5-7. However, in a preliminary way, results in Figure 5-7 show that:

- Owners specifically requesting base-isolated buildings, IL4 buildings, or “no damage” buildings is something that occurred more frequently in the early years following the earthquakes.
- There has been a “constant stream” of owners requesting low-damage solutions and/or IL3 buildings.
- In more recent years (i.e., more years away from the earthquakes), owners requesting fastest construction time and lowest cost have started to progressively dominate final decisions more.
- In most cases, decisions regarding choice of structural system were made by the structural engineer, and this trend has intensified in more recent years.

As indicated in Section 6, a number of factors not shown here as being dominant were however part of the “subtext” in discussions with engineers and were understood to be necessary conditions for the projects to proceed, so did not need to be explicitly stated. This explains the low numbers shown in Figure 5-7 for some categories.

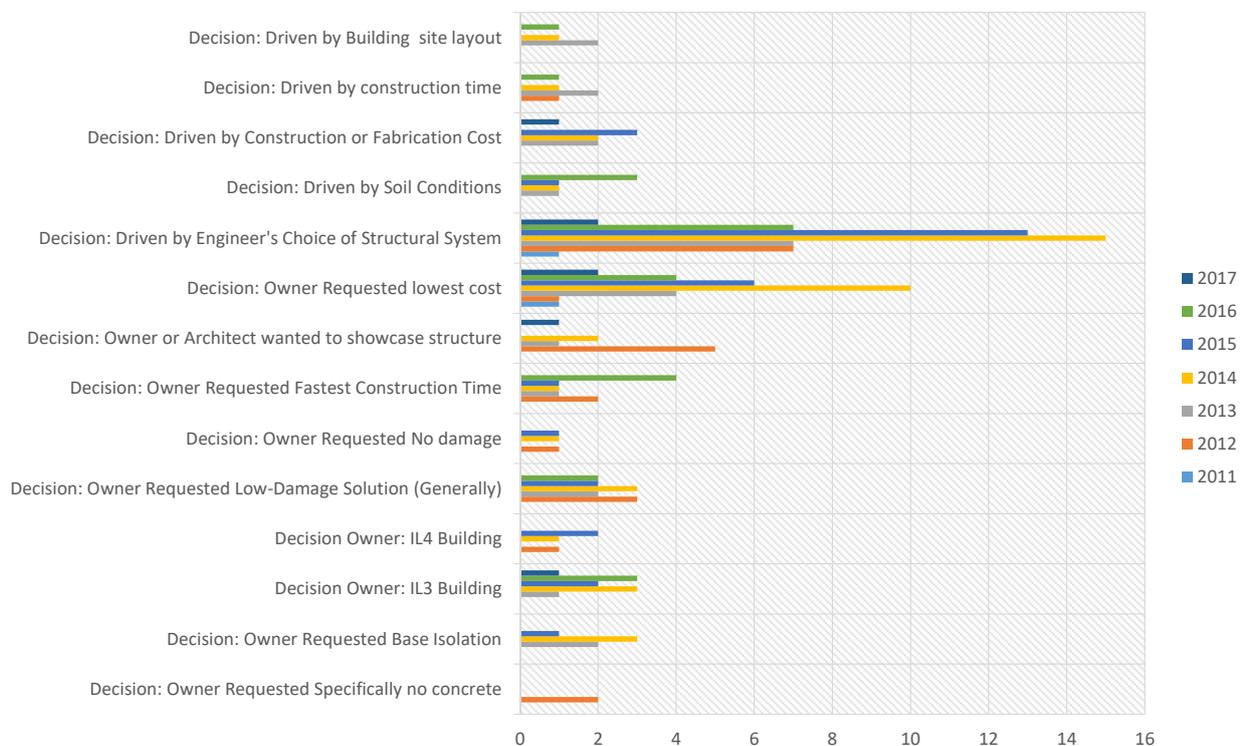
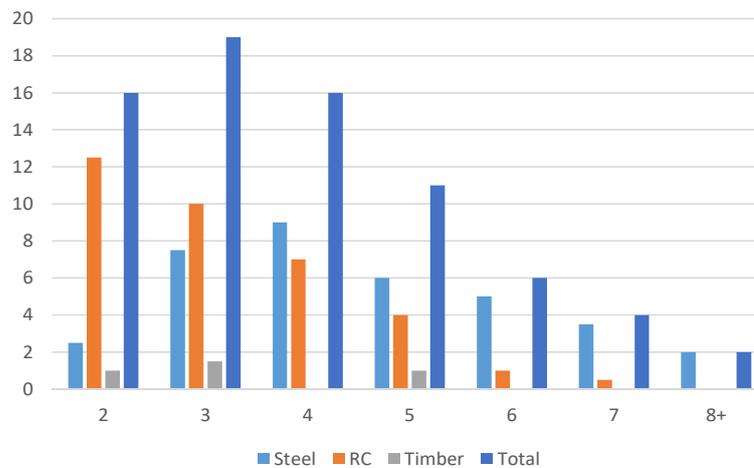
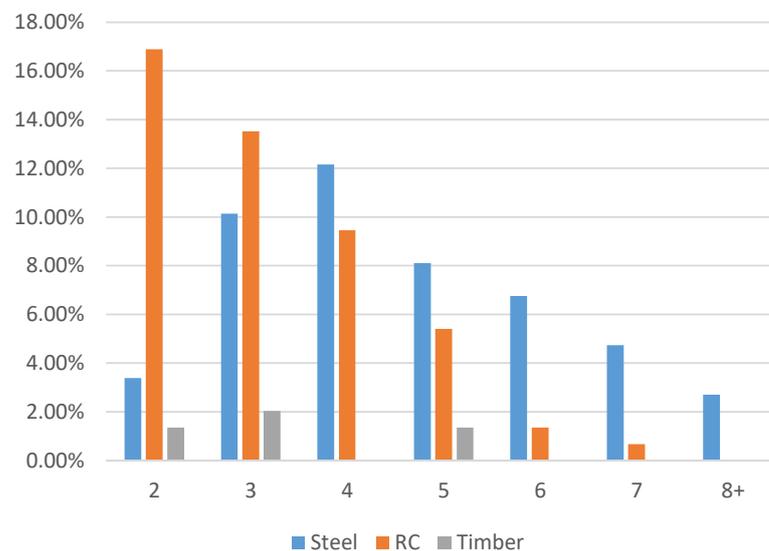


Figure 5-7: Factors driving choice of structural system

Figure 5-8 presents the number of new buildings having steel, concrete, or timber lateral-load-resisting structural systems as a function of building height expressed in number of storeys, both in terms of absolute numbers (in Figure 5-8a) and percentages (in Figure 5-8b) for the same 74 buildings considered in Figure 5-1. This complementary information shows that 80% (12.5 out of 16) of the two-storey buildings had RC lateral-load-resisting systems, and that the number of such RC buildings rapidly decreased with number of storeys. Results also show that steel lateral-load-resisting systems became dominant in buildings taller than three storeys. Results also show that 87% of the new multistorey buildings in Christchurch so far have ranged from two to five storeys (70% ranging from two to four storeys).



(a)



(b)

Figure 5-8: Number of new buildings with lateral-load-resisting systems of each material type as a function of building height, expressed in number of storeys. (a) Absolute numbers, (b) Percentages

6 Interpretation of Findings

The interviews with the Christchurch structural engineering firms provided detailed information on all the specific buildings considered in this study, from which the data above was tallied, as well as valuable overarching comments on the reconstruction process, going beyond the quantification process. Here, perspectives and interpretations are presented, based on these more in-depth conversations with the engineers. Factors identified as having a significant impact on the decision-making process of owners/tenants and structural engineers are described solely from the perspective of those interviewed. In other words, the authors have taken care to present only the opinions expressed by the engineers, and not the authors' own opinions (which may or may not coincide with any of the opinions presented here). Many of the words used are those of the engineers interviewed, and the authors have ordered the modified sentences to improve readability. Also, it may be seen that some of the opinions presented are contradictory to other opinions expressed, illustrating the diversity of opinions amongst those interviewed. The opinions listed were compiled in such a way that they could be traced back to individuals and groups, although these links are not shown in the document.

6.1 Factors driving decision by clients/tenants

6.1.1 Recent Earthquakes and Insurance

An overview of global influences already in place at the time of the Christchurch earthquake was presented in Section 2. As described in Section 2.1, the Christchurch earthquakes created an opportunity for redevelopment that would not have happened otherwise. The insurance coverage, with its wide penetration, provided funding to return many buildings to their "pre-earthquake" or "as new" condition, depending on the policy. The amounts of individual insurance settlements had an impact on owners' ultimate decisions during reconstruction, but that is not quantified here in financial terms. However, some indirect impact is reflected here through the interactions between the structural engineers and their clients, and expressed by final design choices.

Partly due to long delays in settling insurance claims, "downtime" has been a major issue in the Christchurch reconstruction. There has been much discussion about how to deal with damage in existing structures, particularly how to assess how much damage was suffered by various structural and non-structural elements and how to repair/replace this damage. This had a direct impact on the size of insurance settlements.

With respect to awareness of the need to prevent a re-occurrence of this damage in future earthquakes, as highlighted by one engineer, clients have had a natural tendency to forget the earthquake and return to the code minimum solution. However, continued aftershocks and other recent New Zealand earthquakes have

countered this natural tendency for now by sustaining awareness of the possible need for greater than minimum seismic performance.

6.1.2 Perception of Reinforced Concrete and Steel Buildings following the Christchurch Earthquake

Engineers indicated that many of their clients have the perception that RC buildings did not perform well during the earthquake. This reflects the general view of many people in Christchurch. A large part of the media news broadcast after the earthquakes focused on concrete buildings that had collapsed, that were severely damaged and leaning, or that had occupants unable to escape due to stair collapses. Beyond that, perceptions have been developed from pictures of damaged RC buildings showing buckled bars, plastic hinging, or spalling, and from reports that ductile RC buildings are hard to repair, even if they perform as intended for life safety. Many RC buildings that suffered relatively low damage overall developed beam plastic hinging and rebar elongation. They have been demolished after the earthquake, in part because beam elongation and rebar yielding is “irreparable”).

One engineer indicated that in both the Christchurch and Wellington earthquakes, the RC buildings had tenants flying across the room, presumably because of the pinched hysteresis effect of concrete structures (Lin, et al., 2012).

Engineers stated that most of the damaged RC buildings were 1980s vintage. They were often MRFs, as architects did not want any walls or braces. These were typically designed to a ductility of 5 or 6. This reduces weight and makes foundation design easier. Life safety was the only seismic performance objective. Buildings were therefore designed to sustain lots of damage, and this is what occurred. For the most part, they behaved as expected.

Based on observations, and the heavily promoted fact that the two tallest steel structures in Christchurch (the Club Tower building and the Pacific Tower building) exhibited satisfactory seismic performance and were reopened relatively fast, the perception of many tenants and owners following the Christchurch earthquake has been that steel structures are preferable. Coupled with the fact that the steel buildings in Christchurch have performed better and are still in service, this “lost confidence” or “fear” (as termed by different engineers) of tall RC structures, irrespective of the structural system used, has resulted in many clients “writing off” concrete as an option for structural systems. The safe solution for engineers was therefore to move to structural steel, which also has the advantage of being lighter (therefore also leading to lesser seismic forces and less weight on foundations).

This perception from the public (and thus tenants) that steel is a superior form of construction has been a major part of the Christchurch reconstruction, as the decisions made by developers, contractors, and owners towards steel largely reflect the “push” from tenants.

Concrete gravity systems are rarely seen now in Christchurch, although, as mentioned by some engineers, the shift to steel had started before the

earthquakes. The last two tall buildings constructed in Christchurch prior to the 2011 earthquake were steel construction. In addition to the people's perception that steel is better than RC, some engineers highlighted that while traditionally RC was perceived as being cheaper, the price of steel had dropped significantly in the years before the earthquake, making steel construction highly competitive. It was mentioned that projections show that the world steel price should start to increase in the global commodities market, leading to the question of "where will the tipping point be?" In other words, once the steel cost advantage reverses, the real test of the strength of public perceptions as a driver for post-Christchurch reconstruction will be determined by how much of a premium clients will be willing to pay for steel.

Countering that perception, some engineers have mentioned that older buildings with RC walls typically performed well during the Christchurch earthquakes. This is different from the behaviour of RC MRFs. They stated that mid-height RC buildings can still be built, although there is more work involved to raise seismic performance from the minimum requirements of the Building Code to the level of expectations of tenants.

While perceptions from clients/owners and tenants can be an important driver of decisions related to choice of structural systems, one engineer highlighted the fact that, after the earthquake but prior to 2016, there was a "race for gold tenants". These are government agencies, banks, lawyers, accountants, and large corporations, whose views and perceptions influence the decisions of developers and owners with respect to many things, including expected seismic performance in future earthquakes. Since 2016, the construction market has changed from commercial office projects to apartment buildings, healthcare and education facilities, and hotels. Some sectors of this upcoming market deal with smaller structures and they are more concerned with project cost than with tenant perceptions. These include foreign developers/owners who do not share the same perception/memory of the Christchurch earthquakes, those who want buildings for tenants seeking shorter-term leases, and those for whom insurance settlements are not a factor in development decisions.

6.1.3 Desire for Base-Isolated Building

New Zealand engineers have been pioneers in the development of base-isolation technologies and a number of buildings throughout the country have been base isolated. The New Zealand public is somewhat familiar with the concept because of news reports and a visitor gallery showcasing the isolators at Wellington's base-isolated Te Papa Tongarewa Museum of New Zealand. Whereas clients typically did not express any preference for specific structural systems when approaching engineers, base isolation is the exception to that rule. Some projects in the Christchurch reconstruction required base isolation at the initial request of the client. Some of these projects developed as intended, while others switched to a different structural system as part of the development and cost-assessment process.

Owners of buildings built (or retrofitted) with base isolation as part of the Christchurch reconstruction effort have often promoted the fact that their buildings are base isolated. For example, the Christchurch Justice and Emergency Services Precinct, which is a major public “anchor project” intended to revitalise the city, has both a dedicated webpage (Ministry of Justice, 2017a) and a Facebook page (Ministry of Justice, 2017b) to inform “people interested in the construction and development” of the project. On those pages, it is stated that “the precinct is the first major public building to be built in Christchurch by the government since the earthquakes of 2010 and 2011” and that “it is the largest multi-agency government co-location project in New Zealand’s history” with “offices for 2000 workers over 42,000 square metres”. It is also emphasised that it is “constructed using an advanced approach to seismic design, with base isolation and built to an Importance level 4 (IL4) standard”. One engineer described this project as expressing the architect’s and owner’s desire to create somewhat of a “showcase building” to highlight the government’s commitment to investing in Christchurch’s reconstruction.

Some engineers noted that some clients did not explicitly ask for base isolation, but rather for an IL4 building, which was then interpreted by the engineer as leading to a base-isolation solution.

Other motivations existed to prompt some owners to request base-isolated buildings (particularly in the early years following the earthquakes). In one case, a recognised businessman in Christchurch approached a reconstruction project from a philanthropic perspective and elected to build a prestigious building with the “best structural system”. That client wanted to be “a good corporate citizen”, and cost was not to be an obstacle. Another case involved the owner of a building that suffered significant cracking of its RC shear walls during the Christchurch earthquakes and ended up being demolished. The owner requested base isolation for his new building, constructed on the same site, because he explicitly wanted to avoid having his tenants displaced again by a future earthquake. In this case, the base-isolation option added only 5% to the cost, because the new building was built on top of the previous building’s foundation and therefore already had basement walls, on top of which the base isolators could directly rest.

One engineering firm underscored that, in some cases, owners insisted on having a base-isolated building even when the building was not well suited for base isolation or base isolation provided no benefits over other structural systems. For example, in the case of a taller building sited on softer soils, base isolation was provided to provide “peace of mind” to the tenants. However, to the contrary, in another case, the owner specifically did not want base isolation, because he had been advised by a professor that it was a bad idea to use base isolation in Christchurch.

In spite of the strong desire of some owners for base isolation, in a number of instances, owners objected to it on the basis that the resulting design reduced the amount of leasable floor space within a fixed lot boundary. Examples were also

provided of cases where the client initially wanted a base-isolated building, but eventually decided otherwise when project costs were considered.

When base isolation was used, owners typically wanted maximum flexibility of floor space for future contingencies. This was even true for buildings that were arguably never likely to change function. This often led to the use of MRFs for structures on top of the base isolators. Some of those were regular MRFs, while others had ductile RBS details in two-way frames. These were intended to provide ductile response during an extreme 7500-year return period event. In one case, the structure above the base isolation was an RC MRF in one direction and RC walls in the other. Concrete was used in that case because the extra weight of the superstructure (compared to a steel option) was needed to counter the buoyancy of the underground parking due to the height of the water table.

6.1.4 Desire for Low-Damage or Repairable Buildings

A number of clients, without asking for any specific type of structural system, either requested an IL3 or IL4 building design. Sometimes they specifically asked for a low-damage or repairable structural system. This was either requested as part of casual discussions, or as part of the project's specification brief.

The term "low damage" is understood by some clients to be equivalent to "extra protection" against future earthquakes. This typically prompts the engineer to either ask clients to better define their expectations when asking for low damage, and/or explain to the clients what this implies. Engineers describe the respective pros and cons of various types of structural systems. However, in its simplest form, many clients have interpreted "low damage" to correspond to requesting an IL3 building (recall from Section 2 that IL3 buildings are designed to lateral forces equal to 130% of the value required for normal buildings), or to requesting a SLS-2 design level (i.e., a serviceability design level for a 250-year event, instead of 175 years). Sometimes project specifications request damage prevention in more frequent earthquakes, as done, for example, for some critical government buildings. After the Canterbury earthquakes, the council has been requiring more rigorous peer review of PS2 forms.

In some instances, the motivation for developers to seek IL3 buildings has been to attract a tenant. This may involve the use of high-performing advanced structural systems (other than base isolation), such as rocking frames or other concepts. Examples of this marketing practice can be seen on some "For lease" signs when walking the streets of Christchurch, as shown in Figure 6-1.



Figure 6-1: “For Lease” sign visible from the street, emphasizing A-Grade Seismic Systems as one of the desirable features

In one specific example, the owner wanted a more resilient building that at the same time could provide the flexibility of open space (i.e., without braces), as it was perceived that this could provide a commercial advantage from the perspective of being both an asset to attract other tenants and in terms of future resale value. The owner was prepared to pay more for the building as this was also intended to serve as a showcase building for the company. This led to the use of a two-way MRF system with friction connections, used at all beam-to-column connections and base connections.

Some owners want protection against future code changes, especially if it is not too expensive. For example, one owner spent \$100k for extra reinforcing in walls to increase the resistance from 100% to 120% NBS. Another, with budget constraints, had to choose between obtaining a higher percentage of compliance with NBS or increasing the number of carparks, and chose the latter. In some cases, to prevent cost blowouts, an architect is given an incentive to produce a lower-cost system, and this can discourage the use of low-damage systems.

Owners typically asking for an IL3 structure generally plan to own their property for a long time, but do not necessarily wish for an IL4 building. Others are owners who originally considered base isolation but backed away from that decision when cost estimates exceeded their desired price point. This is particularly true for smaller buildings for which the cost-impact of base isolation makes the solution more uneconomical. Interestingly, in one example, the decision to use IL3 occurred exactly the opposite way: the client was a contractor who did not intend to use any innovative systems to keep costs down. A design with CBFs was therefore completed. However, when some tenants came with a 10-year lease prospect and asked for an IL3 building, the braces were substituted with BRBs. With a special layout of the BRBs across bays and storeys, the column sizes remained reasonable and the system ductility increased.

Either way, budget constraints have been an issue in driving the decision to request IL3 buildings; one engineer even added that while a number of owners initially requested IL3 buildings for a few years after the Christchurch earthquakes, this has been progressively removed from more recent design requirements as owners consider the increased costs. One engineer highlighted that tenants in buildings having different types of high-performance system might have unrealistic expectations of no damage following an earthquake, not realising that while the building itself may be fine, many other things will be “a mess” inside and outside of the building. While the engineer can discuss damage expectations with the client, it is not as easily done with the tenants as the engineer is often not involved in the project anymore when the tenants arrive.

One engineer stated that, for their practice, nothing major changed due to the earthquakes, except for the geotechnical requirements and the fact that clients are more open to low-damage systems. Another engineer mentioned that whether clients ask for low damage or not, some measure of low-damage concept is often brought into the structure by the consultants’ contemporary choices of structural system and design (as will be seen in Section 6.2).

6.1.5 Desire for Timber Construction

Even though New Zealand has a vibrant timber housing industry, because the scope of this study is limited to buildings of more than two storeys located in the CBD, only a few timber buildings were the subject of discussion with the engineering firms interviewed. In all of the cases where timber was specified, it was the client who insisted on timber construction. This decision was motivated by either an architectural aesthetic desire, a perception that using a timber structural system makes for a “Green Building”, or the fact (in one case) that the developer was also owner of a timber company with intellectual property in a new marketable product/concept. In that last instance, the owner wanted to see if using timber framing with that new product in a building with large open spans could be done as economically as for comparable RC or steel framing.

One high-tech company that had lost \$200k/day because of business interruptions due to the Christchurch earthquakes wanted the ability to be in immediate operation after a future major event. It chose a timber Presslam system that the engineer had used before. The system was reportedly fast to build, with frames in one direction and walls in the other direction. This building is not located in the CBD, so it was not included in the quantitative results presented in Chapter 5.

Experiences and views differ regarding timber structural systems. Some stated that while timber was specified for environmental reasons in a particular project, the timber for the building considered was shipped from Australia. One building was selected to be timber for fast construction, which was not realised. Others stated that the low stiffness is an issue and that timber can be expensive to fit out. Others strongly advocate timber, especially for structures of around three storeys. For example, one engineer likes it because of its light weight, and another believes that they can make timber building systems very economically.

6.1.6 Desire for Fast Construction Time

As succinctly summarised by one engineer, developers always seek the fastest possible construction, because time to completion has a direct impact on a project's financial return. However, it was mentioned that this has not been a governing driving factor for "quite some time" in Christchurch because the demands on the construction industry have exceeded some sectors of the industry's capacity to deliver. As a result, most buildings being built in Christchurch have not met (or will not meet) their target completion date. At the time of the interviews, fitting and glazing demands could not be met by the suppliers. Nonetheless, as mentioned by many engineers, even when not the primary driver on a project, construction time is always in engineers' and developers' minds. Also, it was stated that, irrespective of the structural system chosen, contractors generally find ways of optimising construction speed.

6.1.7 Design for Cost-Effectiveness

Much like for the previous point, many engineers mentioned that cost-effectiveness is an important driver in nearly all projects, as it is always in the client's mind, and the financial gains resulting from faster construction time are generally taken into account when comparing total construction costs for various alternatives. This is logical, as it is inherent to the construction business that costs can make or break a project. As such, cost-effectiveness is the client driver that is always there, implicitly or explicitly, even though not always the primary driver.

Demands from clients seemed to vary in this regard. Some engineers mentioned that they could not think of a client who said, "Give me the lowest cost!". They knew that their clients implicitly trusted them to come up with the most cost-efficient system. Other engineers mentioned that developers typically request the cheapest building possible, so the engineers need to find ways to "fine-tune" the superstructure. For example, this may mean developing a configuration that minimises the number of piles needed. Answering a client's demand for economy, some engineers strive to make it part of their market-differentiation strategy to be recognised for their ability to change the layout and optimise cost-effectiveness.

One engineering firm stated that, for a period of roughly two years following the Christchurch earthquakes, for projects within their portfolio, many decisions were made by engineers and funded by insurance companies without cost being an issue. However, in recent years, the process has returned to what it used to be, with cost driving the decision. This firm's clients nowadays have no preference for any structural system; focus is on the most cost-effective solution, which is not necessarily the lowest cost, as these clients will agree to a solution with better seismic performance if it is only slightly more expensive. In those projects (mostly office buildings, schools, and warehouses), the clients leave it to the firm's engineers to determine the best structural solution, and the firm honours that trust by ensuring reasonable trade-off and using well-proven structural systems.

Some notable comments indirectly related to client expectations with respect to costs in the Christchurch reconstruction context were also collected during the interviews. In particular, a few engineering firms highlighted the fact that the early reconstruction market consisted predominantly of office buildings for “premium” tenants, who typically sign long-term (10+ year) leases. These include government departments or large law firms that compete to attract the best employees. For these, cost was not a preoccupying factor (within reason). It was perceived that a large part of the building design activity was progressively shifting to the multistorey residential sector and to projects for developers for whom longevity of the building and attracting high-profile tenants are not priorities, with different implications on cost-effectiveness.

In a number of cases, while cost might not have been a factor initially, it became a driving factor as the project unfolded. For example, for a multistorey residential building, the structural engineer had proposed an EBF system, but the architects and client initially rejected it as they strongly favoured a design with precast concrete walls. However, when cost estimates for both systems were received, the client opted for the lower cost EBF option.

Specific to the Christchurch context, it was mentioned that some owners received limited insurance payments and must reconstruct within a tight budget. They therefore welcomed engineers proposing changes to the architectural configuration if they led to more efficient structural and non-structural systems and savings. For one engineer, this is done together with the client. By taking control of the whole project, the engineer believes that it decreases the risk. In other cases, owners have set quality targets to attract A-class tenants and welcome ways to achieve this objective within a set budget.

In some projects, costs of an RC wall option and a steel framing option are compared. In such cases, while the quantity surveyor estimates which system is least expensive at the point in time it is under consideration, the most cost-effective system cannot be predetermined as it depends on the market fluctuations of different materials. (Note: In New Zealand, the quantity surveyor is the person hired to estimate the cost of a construction project (New Zealand Institute of Quantity Surveyors [NZIQS], 2017)). In one example project cited by an engineer, “the contractor wanted to use concrete, but the winner was steel”, chosen based on cost alone. Other engineers also provided similar examples for which the steel option turned out to be cheaper. It was mentioned that concrete construction had already increased in cost before the Christchurch earthquakes, due to more stringent health and safety regulations and increased quality-assurance and quality-control requirements.

Finally, the need to determine cost using project-specific information was emphasised by one engineer, citing as an example the case of a project that had financial trouble because the quantity surveyor used the steel price from another project that (unknown to the quantity surveyor) was a loss-leader project using offshore fabrication. Another engineer stated that a lot of projects in Christchurch

have stalled, or not started, because developers can't make the expected return on investment work.

6.1.8 Awareness of Resilience and Business Continuity

"Resilience" has not been mentioned much as part of the interviews, and no reference was made to the resilience frameworks being considered internationally (e.g., Bruneau et al., 2003). Because of this, matters pertaining to resilience are not discussed in this report, other than to highlight the fact that the Christchurch resilience goals (Christchurch City Council, 2016a, 2016b) do not explicitly mention issues related to resilient buildings. Also, the New Zealand Building Code remains focused on life-safety goals. However, the concept of "business continuity" was frequently mentioned, reflecting an awareness of the importance of rapid return to operations for some tenants/owners.

Interestingly, buildings in the CBD that performed well during the Christchurch earthquakes were still unusable, since they were in the cordoned-off Red Zone after the earthquake. This kept owners away for months. As a result, one would expect the issue of business continuity to be front and centre. However, some engineering firms who underscored the importance of "business continuity" to owners/tenants, indicated that this was apparently not fully understood by the industry following the Christchurch earthquakes, but that this has been changing following the 2016 Kaikoura earthquakes (see Section 7).

Finally, in that context, while the Christchurch earthquake had an acute impact in raising the developers' awareness of the earthquake risk, some firms have indicated that this awareness is fading and that practice is returning to its pre-earthquake ways, with cost effectiveness being the major driver in most instances. The exception to this is critical infrastructure projects where the protection of both buildings and content is paramount.

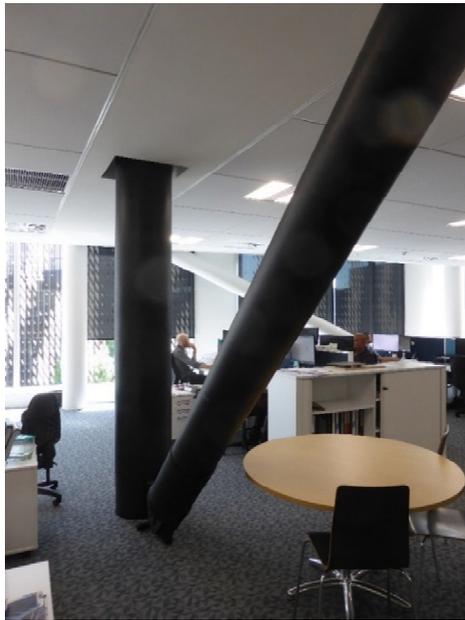
6.2 Factors driving decisions by structural engineers

6.2.1 Professional Culture and Client Relations

Seasoned structural engineers (similarly to other professionals) have typically developed a philosophy of practice from their years of experience. This philosophy of practice is influenced by the type of work conducted for clients, by experience and professional opinion on the respective benefits of various structural systems (which depends, to some degree, on opportunities provided by past projects), and by business relationships. The type of work includes issues related to project scale and building use, and whether or not the client is the owner. Some engineers satisfied with past experiences with a particular seismic system may also have a tendency to repeatedly use that system – and likewise work with the same people – rather than trying something else. The strength of relationships and trust between parties can also affect the costs. For example, when engineers are able to work with the contractor to determine the best details, this can benefit everyone.

Consciously or not, decisions are affected by the factors above, as well as multiple other factors that include formal professional-development activities, informal individual education by interpretation/synthesis of skills and information from various scientific and non-scientific fields, and professional ethical and moral obligations. The end result, not surprisingly, is that distinctive “cultures” exist from one engineering firm to the next – cultures that embody the engineering judgment, experience, and philosophy of their founders and/or subsequent leaders. Not surprisingly, these various professional cultures often drive the engineering process towards solutions that may differ from firm to firm.

The resulting breadth of valid engineering solutions can be regarded as the expression of differences in this culture. This, together with different professional opinions regarding: (i) the expected seismic performance of various structural systems, (ii) a hierarchy of priorities in rebuilding Christchurch, and (iii) how these various priorities can be best met for specific buildings, has affected structural engineering decisions. In other words, while some structural systems have been used more extensively than others during Christchurch’s reconstruction, there exists no “one-size-fits-all” solution in structural engineering. This is illustrated by the fact that structural engineering firms are also tenants in buildings. Of the ten firms interviewed, three companies either had their office in a base-isolated building or were about to move into one, two companies were in a building having BRBs (with the BRBs typically visible from inside their office, as shown in Figure 6-2), and one was in a building with viscous dampers. Nearly all of them designed the building in which their office was (or will be) located.



(a)



(b)



(c)



(d)

Figure 6-2: BRBs prominently featured in an engineering firm's office. (a) Inside working space (painted black), (b) Along the façade (painted white), (c) Close-up view of BRB end detail, (d) "Scratch" marker to indicate on BRB surface the maximum BRB elongation developed during the earthquake

Many engineers stated that the Christchurch earthquakes changed, in a major way, the relationship between all parties involved in their projects. Prior to the earthquakes, the architect would develop the conceptual plans and ask the engineer to fit a structural system to it. The selection was therefore driven by the architectural constraints and the geometry of the site. Now, contrary to practice prior to the Christchurch earthquakes, structural and geotechnical engineers are brought into the project at the same time as the architect, or sometimes sooner. For example, in one project, the architect was expecting a steel structure, but was not part of the selection of BRBs for the structural system. One engineer even mentioned that in some projects, geotechnical engineers were selected first, followed by structural engineers, and finally architects. This reversal of roles occurred because the Christchurch earthquakes have raised the awareness of clients to the fact that not all code-compliant structural systems will provide the same level of seismic performance.

Clients that do not have a specific initial request for any type of structural system are typically informed by the structural engineers of the pros and cons of various options. For example, one firm, as a general practice, does not advocate use of any specific structural system, but only describes to its clients the various possible structural systems (BRBs, EBFs, base isolation, shear walls, MRFs) and their relative merits in terms of resilience and seismic performance. Another firm also generally “walks” its clients through the list of possible structural systems, from MRFs to BRBs, but emphasises its preference for designs that eliminate eccentricities, for example by creating regularity in stiffness by correspondingly locating BRBs in bays and across the floor plan. A third engineering firm prefers rigid core structures and indicated that its clients (owners and architects) typically agree with the engineer’s recommendations. A fourth one has gone as far as developing special brochures to illustrate the list of options, with advantages/disadvantages, an overview of the corresponding cost premium and expected seismic performance for each structural system, and examples of recent implementation in Christchurch, to facilitate this discussion with clients. In some cases, this has facilitated the implementation of innovative systems. However, it has been often stated that more resilient structural systems (rocking frames, base isolation, etc.) generally entail a cost premium, and that many clients still prefer to rely on insurance to protect their assets rather than investing more into the structural system. Or, as one engineer said, some owners want low damage, but with established procedures – “leading edge; not bleeding edge”.

Note that in some cases, the client has practically been the tenant, as the owner is a contractor/developer tailoring the project to a specific high-profile tenant whose voice is important. Since the Christchurch earthquakes, from which many “campfire stories” resulted regarding the behaviour of structures, some of these tenants have gone as far as wanting to know who the engineer for the project is. Some have even been wanting to meet the engineer to have their specific questions answered directly. One engineer reported having made presentations to as many as five tenant groups in recent years.

However, a number of engineering firms have stated that client awareness is slowly fading as the years push the earthquakes further in the past, and have noted that some clients have reverted to the pre-earthquake practice of bringing the engineer into the project late – particularly for projects involving foreign developers – but that the 2013 Seddon and 2016 Kaikoura earthquakes (described in a later section) have slowed this creep to pre-2011 practice.

6.2.2 Soil Conditions and Foundation Design in Christchurch

Christchurch is, for the most part, located on deep alluvial soils. The geotechnical challenges arise not only because these soils are soft and contain liquefiable layers, but also because competent soil/rock is at a great depth (the Riccarton gravels are often about 20m down, with intermediate gravels at about 15m). As reported in the literature, a number of buildings suffered from differential foundation settlement or tilting of raft foundations.

Geotechnical design work is now taking substantially longer and designs are substantially more conservative because significant soil movement is less acceptable than in the past. Large estimated values of lateral spreading, due to liquefaction, increase the pile requirements. It was stated that some owners are willing to go beyond code requirements for geotechnical work nowadays in recognition that foundation problems after an earthquake are almost impossible to fix.

Changes in geotechnical design practice have also made foundation sizes more significant. For example, one engineer reported that an added compression load on piles due to down-drag must now be accounted for and that piles can now only rely on bearing and on friction-resistance below the lowest liquefiable level. In some conditions, resulting piles can be up to five times longer for the same soil than they were before the earthquakes, when this effect was not considered.

Given that geotechnical constraints and challenges affect most building designs in Christchurch as part of everyday practice, the engineers interviewed did not often identify foundation design to be a special driver as part of the reconstruction project, other than mentioning in passing a systematic preference for lighter structures. This lower mass reduces foundation costs and seismic (inertia) design forces. It was often mentioned that steel superstructures are lighter than RC ones.

A number of foundation design strategies were mentioned as part of the interviews. While it was not part of the study to document how frequently specific solutions were used, some anecdotal information was obtained. It appears that for deep foundations, Continuous Flight Auger (CFA) cast-in-place piles (a.k.a. drilled shafts) and screw piles (up to 25m long) have been broadly used. One engineering firm mentioned using 300mm diameter screw piles on 50% of its jobs; they are favoured for their cost-effectiveness and because driven piles are often rejected due to vibration concerns. (Screw piles can even be driven through asbestos-contaminated soils, and at an angle, as their lateral-load resistance is small when vertical). However, another engineer added that some geotechnical engineers do

not like screw piles because of the possibility of hitting large tree roots, and prefer ground-improvement approaches.

Other solutions mentioned by the structural engineers interviewed include gravel raft, raft with stone-columns (above liquefiable layers), driven precast piles used in the presence of 10m-deep gravel lens, timber piles to 5m used for short two-storey structures, and deep-soil mixing.

Some engineers considered raft foundations (popular in Christchurch prior to 2011) to still be a good solution, even though buildings on raft foundations may lean after an earthquake and would need to be straightened. One engineer indicated that on one project, the raft foundation was designed such that the weight of the building was equal to that of the soil removed, to minimise foundation problems on soft soils.

In any event, one structural engineer warned against over-conservatism in geotechnical design as “clients do not like pouring money into ground”.

6.2.3 Stiffness (Drift Control) and Design Ductility

While drift limits in the New Zealand Building Code have not changed following the Christchurch earthquakes, a number of engineers have indicated a preference for systems that can provide lower drifts under the design earthquake level, to limit non-structural damage. While this is not a general rule or standard practice, nearly all the firms interviewed have indicated a tendency to steer their new designs far from the 2.5% drift limit. Drift demands of 1% to 1.5% were often cited as a desirable outcome when possible. This drift demand depended on considerations relating to the specific details of each project. Not surprisingly, such low drift limits have been seen in the performance specification documents for some projects. It is more difficult to obtain these drifts with MRF, rather than braced or wall, structures.

One firm mentioned using in-house design guidance targets of 1% drift in ULS, which at the same time ensures no partition damage in SLS for serviceability factor R_s of 0.33 for an L/300 drift capacity. Likewise, another firm indicated having an in-house policy to limit drifts to 1.5% (less is preferred when possible), but is sometimes pushed to the code limit of 2.5% for some types of buildings. It was mentioned that the CCC’s 4.5m ground floor height requirement for the retail precinct can cause a problem for MRFs to avoid a soft-storey mechanism, which drives the engineers to use braced frame (unless strongly opposed by the architect).

One firm emphasised that limiting drift to 1.1% translated to a larger amount of leasable space in projects where the site boundary could not be crossed by the building at maximum drift, with the extra added benefit of limiting damage to façades and other non-structural elements.

Since the earthquakes, the New Zealand engineering community has become more appreciative of the need for redundancy, as this increases the likelihood of high

system ductility. Likewise, many firms expressed a preference for lower values of design ductility demand. Some firms indicated always using effective design ductilities of 3 or less as a consequence of SLS considerations governing the design of most buildings.

6.2.4 Base Isolation

This section focuses on base isolation when not originally requested by clients, but rather as one option evolving from discussions between engineers and their clients when the desire was for IL4 seismic performance. Interestingly, while some engineering firms were active promoters of base isolation, others had significant reservations. The impressions reported below illustrate, without identifying the specific projects, some of the arguments that drove decisions (or complicated matters) with respect to the possible use of base isolation.

The promise of superior seismic performance (and possible business continuity) typically drove the decision to use base isolation. If designed appropriately, it is regarded as a low-loss system with low total damage and low business interruption. It is different from replaceable technology systems for which damage (that may compromise the performance in future events) is acceptable, and replacement of elements may be required. In one example, an owner who had no preconceived notion about structural systems, when presented the pros and cons of various options by the structural engineer, retained the base-isolation option as a way to provide “a self-insurance policy” for the building and was prepared to pay a 5–7% premium on total construction cost to ensure satisfactory performance later. Some engineers stated that base isolation allows (above the isolators) stiff redundant structures with low drifts, which also provides benefits for façade detailing. While base isolation reduces the demands on the superstructure, and the type of demands required there, a number of engineers still detail it to provide some limited ductility.

A number of different arguments drove decisions against base isolation in some projects. While all understood the principles and potential benefits of base isolation, there was not unanimity among the engineering firms that base isolation was an appropriate structural system for application in Christchurch.

One firm mentioned that much confusion was initially created when GNS Science, Te Pū Ao (the New Zealand government agency responsible for seismological studies), issued a revised design response spectra with a “bump” at around 2 seconds, to account for soil effects as seen in the response spectra of many of the Christchurch earthquake records. However, on the basis of results presented in a paper by Whittaker and Jones (2013) showing that this modification to the spectra was not necessary for structures with significant damping (or ductility), an amendment to NZS 1170.5 2017 removed the “bump”.

In fact, one engineering firm who indicated being fully entrusted by their client to select the structural system (and thus drive decisions in this regard) expressed being not inclined to recommend base-isolated buildings because the one base-

isolated building in Christchurch at the time of the earthquake didn't behave much better than non-base-isolated buildings close to it. Another firm indicated that base isolation is fashionable in the post-Christchurch context, as client perception is often simplistic. That firm indicated that it lost jobs when advising against base isolation on some specific projects where the engineer felt it would not have been a good fit and/or was unnecessary. Another engineering firm similarly held that a base-isolated building on long period soils with a flexible MRF on top is a bad solution technically, and not an economical design. There were conflicting opinions on the use of base isolation with L-shaped buildings. One engineer indicated that it is effective and that seismic joints were not required, while another group considered that base isolation is not good for L-shaped buildings.

A number of engineering firms mentioned that designing base isolation can be "off-putting" because there is no design standard for that structural system in New Zealand. Therefore, base-isolated projects must be peer-reviewed, which can create challenges and delays, particularly when the respective advocates of lead-rubber bearing isolators and sliding-friction bearings are at odds in their recommendations. An engineer even mentioned that comments from peer reviewers with affiliations to competing bearing systems have bordered on unethical and have significantly delayed projects. This, from a distance, would seem detrimental to the base-isolation industry as a whole.

Beyond the above concerns, on a project-by-project basis, when the base-isolation option was declined by the client, it was either due to cost or due to a desire to maximise use of the site. Examples related to costs and cited by engineers include: (i) A case where base isolation was proposed, but the client was not interested in paying more to protect the building content (the base-isolation option was more expensive, in spite of the savings it would have provided to the cost of the MRF system that was specified for this project); (ii) A case where base isolation was discussed as a possibility for a private developer client that was aiming at government tenants, to be eventually rejected by the quantity surveyor; and (iii) A case where the client considered base isolation with a steel superstructure but decided against it when it was realised that additional mass needed to be added to the superstructure to get the isolation system to behave well.

One engineer underscored that while the cost premium for a base-isolated building could be 5–7% of the total cost for a large building like a hospital, it is actually a much higher percentage for an office building because the base cost of an office building is significantly lower than the base cost of a hospital. He also suggested that some private-sector clients are more cost conscious and less likely to favour a base-isolation solution.

A number of engineers noted that a large extra cost for base-isolated buildings (in addition to the cost of the isolators themselves) comes from the additional floor slab needed above the isolation level, which significantly adds to cost when there is no basement or underground parking. In some cases, the base-isolation system was located on top of columns to eliminate the need for this extra slab, but at the

cost of extra detailing for the building façade and suspended services (e.g., stairs, elevator cage, etc.).

Examples related to maximisation of land use include (i) A case where base isolation was not an option because the owner wanted to build up to the boundary line, and (ii) A case where an owner who had lost many buildings to the Christchurch earthquakes wanted to use base isolation to prevent damage in future earthquakes, but would have lost too much usable space on the small, irregularly shaped site, due to the set-back from the property line required to accommodate the base-isolation movements. The building ended up with massive cast-in-place RC walls and a steel gravity frame.

Engineering firms also expressed different preferences when it came to the type of base isolators used (i.e., lead rubber bearing versus friction isolators). Some firms emphasised that “not all bearings are similar”, as some will induce greater accelerations than others. Some firms expressed concerns about the reliability of performance of specific devices, the fact that some are tuned to a particular earthquake level, or the behaviour of the isolated structure under vertical ground accelerations. This related to the device’s initial strength, durability, and behaviour under cyclic loading. Sometimes these concerns came from the peer reviewer. However, in some projects, a specific type of bearing was used only because competitors arrived to the Christchurch market too late, or because some concerns remained unanswered with respect to the competing isolation systems at the time of implementation. For some critical structures, questions arose related to the performance of the system during a 7500-year return period earthquake, and RBS were introduced in the MRFs used above the base isolators as plastic hinging was expected at this extreme demand.

6.2.5 Dampers

At the time of writing, viscous dampers have been used in only one building to date in Christchurch. The client wanted a low-cost building, but something a bit above bare minimum, as the building location was upscale. The engineering firm, as the main tenant, was permitted by the client to design the structural system of their choice provided they met a specified budget comparable to that for a BRB system. Base isolation did not meet the budget, but an MRF with viscous dampers proved to do so, and using dampers allowed optimised foundations and reduced pile sizes. At the time of the interviews, it was found that viscous dampers were also slated to be used in MRFs for a new critical facility (to be located on an irregular site) that was designed to meet IL3 performance, but for which the base-isolation option was not retained.

Evidently, the number of buildings with viscous dampers in the inventory of the Christchurch reconstruction is limited. One engineer volunteered that, although he preferred viscous dampers to braces, adding a \$30k damper to a structure is a “hard-sell” when a comparable brace is \$4k, and that, while viscous dampers have been used in new buildings, the market for the devices in New Zealand might be more in the retrofit of older RC MRFs.

6.2.6 Steel Structures

Many commented to the effect that the Christchurch earthquakes gave the engineering community an opportunity to “brush up” and “get up-to-speed” on steel design, to the point that, while RC was the default option before the Christchurch earthquakes, there is nowadays “no objection” or “no resistance” to using steel. In other words, to paraphrase one engineer, while the local industry (engineers, contractors, quantity surveyors, etc.) was geared to do RC buildings out of habit and practice, the industry in Christchurch is now geared to do steel on a large scale. This is important because, as another engineer mentioned, for some developer-led projects, the use of steel or concrete is often decided by what the contractor is used to.

In describing specific projects, many engineers have commented that a specific building that was designed with either all steel or part steel “would have been an all-concrete” design prior to the earthquake. However, in one such project that would have previously been done in concrete, the developer blamed the fact that steel was used for the project going over budget, due to the expenses incurred at intertenancy walls to achieve fire rating and acoustic isolation. However, it was the opinion of the engineer that other factors at play (such as fit-out issues) were the real reasons for the budget issues.

One engineer stated that, before the earthquakes, the industry had already started to move away from RC frames because they were too slow to build, and that it was “the right decision” because steel has the advantage that damage is more visible when it happens. He emphasised that it is harder to determine damage in a RC building and it is not clear how to quantify strain damage.

Some engineers have commented that steel structures make on-site quality assurance easier, given that the entire structure is always on display, particularly with bolted connections, but many emphasised that a different type of quality control remains critically important, particularly for imported steels where large quality variations have been observed. One consultant stated that Chinese steel is often less than half the price of that obtained elsewhere, but, because of issues with quality control, they have their own quality control inspector in China. Some steel has been imported from China because the mills there produce sections, such as hollow sections, of sizes larger than can be easily accomplished in New Zealand.

Engineers highlighted a number of desirable features for their choice of steel construction, including lesser weight and lower foundation tie-downs (advantageous on the poor soil conditions frequently present in Christchurch), faster fabrication time (with at least two major local fabricators having fast automated systems with good welding procedures), and faster construction time (with up to seven storeys of frame being stood up from the ground at a time). They also stated that some clients perceive that low-damage solutions may be hard to implement in concrete and favour steel construction. However, it was also emphasised that there are a number of issues with steel that require special

attention, including fire protection (and the frequent use of intumescent paint), floor vibration control, and acoustic isolation.

Note that in the discussions with engineers who had designed steel structures, it was clear that it was not simply “steel” that they regarded as being good. It was well-designed steel structures. While many regarded the work of their colleagues highly, there was also discussion of some “cowboy” designs which may not be “up to scratch”.

6.2.7 Buckling Restrained Braced Frames

Some bespoke BRB designs have existed in New Zealand since 1991 when first applied in extensions to the University of Canterbury Geography building, but BRB became widely known to engineers only a few years after the Christchurch earthquakes. As indicated by the data presented earlier, a proportionally large number of buildings have been designed with BRBs as part of the Christchurch reconstruction activities. An engineer even ventured that most of the new office buildings still on the drawing board in Christchurch were being designed with BRBs (citing multistorey critical facilities, office buildings, and three-storey schools).

Most of the engineers interviewed have used BRBs in at least one project and were positive in their assessments. However, one engineer stated that he did not like BRBs for new designs (and accordingly had not specified them in reconstruction projects) on the basis that they lead to large concentrated storey displacements, but indicated that they would be “okay” for seismic retrofit.

Many reasons were provided to explain the emerging popularity of BRBs in Christchurch. First, there has been substantial promotion from BRB manufacturers following the Christchurch earthquakes. This was certainly an opportune enterprise at a time when engineers were looking for alternative “low-damage” designs (particularly by limiting drifts) and solutions that would allow rapid return to service (by being rapid to repair), while being lower cost than base isolation. Some firms expressed being more convinced of the ability to achieve those goals when using steel frames with BRB, rather than more conventional EBF, emphasising the fact that when replacing a BRB after an earthquake, “a lot of moment frame action and maybe more energy dissipation capacity still exists”, whereas moment frame action is completely lost when replacing the yielded link of an EBF. Some engineers also considered BRBs to be more easily replaceable after an earthquake than EBFs, as far as the replacement operation itself is concerned.

Second, engineers who favoured BRB frames indicated using them because they are stiffer than EBF, making it cost-effective to limit drifts to low values (targets of 1% to 1.5% drift were frequently mentioned). The greater stiffness provided by BRBs was underscored to be an advantage on the basis that many complaints after earthquakes are SLS-related issues and because tighter drift requirements have been specified on some projects. For example, one engineer cited a project that required prevention of damage to partitions, by adapting the $h/300$ limit in building codes for partitions. Low drift was also mentioned to be an enormous advantage

when restrictions required that the edge of the building at maximum drift does not cross the building site boundary.

Third, although engineers stated that BRBs are more expensive than a regular brace, they emphasised the benefit that their fabrication is quick and that they are considered to be a well-tested and robust system – although one engineer cautioned that “blindly accepting test results from California from years ago might not be the way to go”. Some BRBs in one Christchurch project were manufactured incorrectly and experienced problems when tested. (These problems have apparently been ironed out since). Some, but not all, engineers indicated that they are aware of current discussions/concerns in the profession about significant gaps and issues in design information for BRB systems, including those relating to both gusset plate design information and the lack of data on out-of-plane test performance of BRB systems.

A number of engineers have highlighted that, when they are performing as intended, BRB frames can be better “tuned” to demands such as eliminating unnecessary overstrength of the structure. This results in a stiff structure with limited load demands on the foundation, although one engineer expressed concerns that different testing configurations/regimes could give different brace overstrengths (up to 1.2 times higher was mentioned). Nonetheless, that same engineer expressed the opinion that the “most cost-efficient structural system in Christchurch would be BRB”, three to four storeys at most, with an accepting architect.

Fourth, many engineers stated that architects in Christchurch desire modern architecture and have showcased BRBs in many projects, considering the system to be suitable for the architectural requirements of modern office space, which calls for lots of glass façades. One engineer was of the opinion that architects prefer BRB frames to EBF ones when using bracing because they can be “snaked” along the building in different patterns, and because their connection gussets are nicer and can be shaped different ways, thus giving more freedom to architectural expression. In some projects, the architect selected BRBs over EBFs purely for aesthetic reasons, for buildings that “a year earlier” would have likely been EBFs. It was mentioned that while architects saw BRBs as a way to get away from EBFs, as too many EBF frames had been built before, some architects now want to get away from BRBs because many BRB buildings have been built in Christchurch by now. Another engineer also stated that while it was impossible to put braces in windows before, it is architecturally acceptable now. One engineer went as far as saying, “BRB is the vernacular of Christchurch”.

Among the projects having BRBs that were described during the interviews, beyond the large number of office buildings where BRBs were used, the following applications were noteworthy:

- Steel frames with BRBs were being considered for a new three-storey school (whereas two-storey schools are being designed with RC walls).

- When one owner who had lost two buildings during the Christchurch earthquakes asked for higher strength for his new commercial buildings, the engineer accomplished that by designing a BRB building.
- BRBs were used in a five-storey residential project where the underground carpark was of a layout that made using internal walls difficult. Thus, in spite of the strong preference for concrete walls in residential construction, the engineer suggested using BRBs at the periphery, the architect embraced the idea, and, in costing project alternatives, the BRB option was the most cost-effective solution. However, the engineer indicated that when using steel for residential construction, one must explicitly deal with fire issues (e.g., how to deal with exposed fire sprinklers), and with vibration and acoustics issues.

Some aspects of specific applications were also outlined:

- In one project, the client initially wanted IL2, which would have been accomplished with CBFs, but it was possible to change the braces to BRBs while keeping the same column sizes, so as to deliver IL3 performance for same cost.
- In some structures, BRB frames were used together with some concrete walls acting as continuous columns to better distribute yielding over the building height.

6.2.8 Eccentrically Braced Frames

EBFs have been used in a number of reconstruction projects. In some cases, “conventional EBFs” were used, detailed as done for decades prior. In other cases, EBFs with especially detailed replaceable links were used.

Some engineering firms stated that they considered EBFs to be the most cost-effective structural system to use. One engineer mentioned telling his clients that EBFs are what need to be done nowadays (i.e., post-Christchurch) when cost effectiveness is the driver. Another firm indicated that they used EBFs or CBFs on projects before they became familiar with BRBs.

Opinions were almost evenly split as to whether EBFs with replaceable links are better than conventional ones. Some firms expressed a strong preference for using bolted replaceable links and appreciated that doing so also facilitated the design of EBFs by decoupling the size of the link from that of the link-beam. Other firms held that replacing links in EBFs with replaceable links might not be performed as easily as theoretically foreseen. This is because of the presence of floor slab and residual drifts (i.e., the building may not be perfectly straight after an earthquake). Also, they were not convinced that the extra fabrication costs for EBFs with replaceable links could be justified. Many engineers held that replacing the link in a conventional EBF is not necessarily more difficult, and is relatively easy, given that yielding is localised and that even severely yielded steel links can be cut and replaced by new segments welded-in-place, as done in a number of post-

Christchurch repairs. They emphasised that buildings with traditional EBFs generally performed well during the Christchurch earthquakes (not even showing paint damage in the link in some less-reported cases). They also voiced concerns that some of the bolted replaceable links in some of the new buildings in Christchurch appear out of proportion with the rest of the frame and questioned if they might yield in smaller earthquakes. In that respect, some firms outright stated that they never designed a replaceable link EBF, as they did not believe in them, and that they designed conventional EBFs instead. In some projects with imported steel, the replaceable links were fabricated locally.

Anecdotally, in one project, EBFs with replaceable links were proposed to satisfy the developer's request for an "innovative" solution and strong preference for a reasonably fast to erect steel structure. Also, note that in some examples, EBFs were only added as secondary lateral-load-resisting elements to serve as drift-control frames, or to be used as an architectural showpiece and painted orange instead of being hidden by the building envelope.

6.2.9 Moment-Resisting Frames

Steel MRFs are more expensive (and not as light) as braced frames, but even in the post-Christchurch context, some clients insist on having open façades and more flexibility internally (i.e., without braces in one or more directions), even though they understand that using braced frames would be more economical. Other than when used as base-isolated frames, in three instances it was mentioned that steel MRFs were used because the building was triangular, either because (i) the building was not wide enough to accommodate braces (while still considered the best system to achieve maximum clear space to meet the architectural requirements), (ii) the architect did not want braces in this odd-shaped building that already had concrete walls on the boundary line (and the owner's brother-in-law was a steel fabricator, so using lots of steel was not a negative), and (iii) the project was driven by engineering constraints due to the site layout and the desire to get a robust building with lower damage using a reliable structural system (while size of the MRF members, detailed with RBS, was dictated by the 4.5m height requirement for the first floor, which incidentally brought the system close to drift limit).

At the other end of the spectrum, some engineers have indicated not liking steel MRFs, or having a preference to "shy away from" MRFs altogether following the Christchurch earthquakes. This is due to the miscellaneous damage issues created by large drifts. Another engineer indicated having used MRFs along a façade only to control torsional response when concrete walls were present on the other three sides of a building.

It was stated that engineers generally use universal column shapes in steel MRFs, but that tubes are used for columns if the architect asks for them. For fire rating, they are typically filled with concrete with reinforcing and/or are provided with intumescent paint. Such two-way steel MRFs with concrete-filled tube columns were selected in some instances to avoid braces and have flexibility of space.

Often, they had external diaphragm connections bolted to the beams (creating a net section area at the beam's first row of bolt holes). To ensure reliable energy dissipation, either friction connections or RBSs were provided. RBSs were also used with beam end plates held to the column with unbonded through-bolts in sleeves. These were tensioned after the concrete reached strength and calibration was conducted to ensure force was maintained considering creep.

Nobody considered designing an RC MRF building in the post-Christchurch earthquakes context (other than on top of base isolators, as mentioned previously).

6.2.10 Rocking Systems

A few buildings among the reconstruction inventory in wider Christchurch rely on lateral-load-resisting systems that consist of rocking steel frames, rocking RC walls, or rocking timber walls. The few engineering firms that have been involved in the design of such systems typically reported that owners in these projects were keen on the re-centring feature and saw that as a feature that could ensure post-earthquake serviceability.

For example, in one case, the client had had a prior conversation with an outside adviser who steered them away from base isolation due to uncertainty in design spectra and soft soils in Christchurch. Therefore, the owner wanted to rely on "a different type of damping" to account for uncertainty in spectral demands. Here, the engineer alluded to the "bump" in the spectrum at around 1.5 sec that had been seen in some Christchurch records. The owner specifically wanted something "tough" that could answer the tenant's need that all services be operational following an earthquake. This included the elevators, which had lost power in a different building after the earthquake. They did not want a repeat of that scenario because lifts were critical for the tenant's operations. Cost was not the dominant issue. The client trusted the engineer to select the system, which led to use of a steel-braced rocking frame. Interestingly, this design was not repeated by the engineer in other projects, as the rocking frames proved to be a big percentage of total project cost.

In another project (from a different engineering firm), the client liked the re-centring feature proposed by the engineer, and, once "fitted" with the architecture, it became a selling point to attract tenants. One engineer specified liking rocking systems that do not start rocking early, but carry significant force first.

During the interviews, it was discovered that some engineers have included in their buildings inconspicuous rocking parts, such as cladding panels that rotate on shear keys at their base to follow displacements, like a stiff continuous column.

6.2.11 Reinforced Concrete Walls

It was mentioned multiple times as part of the interviews that prior to the Christchurch earthquakes, many buildings had RC frame structural systems, both for gravity and for lateral-load resistance. None of the engineers indicated a desire

to design such systems in the future. An engineer cited the example of a building that had been designed as an RC MRF before the earthquakes, but had not started construction at the time of the earthquakes. It was redesigned as a steel MRF after the earthquake (although with precast walls on both side walls). Steel gravity frames also replaced the RC beams and columns in the non-seismic frames of the building.

However, contrasting with the above views on RC MRFs for the seismic system, many engineers commented that buildings with RC walls can provide excellent seismic performance in many instances. They are designing such buildings, often with steel gravity systems with rolled-shape columns and cold-formed steel composite floors. In particular, in many instances, large concrete walls are used to provide the required fire protection between buildings sharing a property boundary line (these concrete walls have a three-hour fire rating). In such instances, these walls are used for lateral-load resistance in that direction. Sometimes they are used with steel MRFs parallel to street. Given the typical length of the property lines and the relatively low building heights in Christchurch, these walls tended to be designed to remain elastic or for low ductility demands of 1.25. However, one engineering firm reported designing lateral-load-resisting walls over only part of the length of the boundary line, and detailing the walls over the remaining length of that boundary to not resist any lateral loads (i.e, making part of the length structural walls, and the rest of it like cladding).

Another engineering firm indicated a preference for core walls, designed to ductility of 2 or 3, as it was considered that, in its typical projects, using steel braces would have been more expensive by the time fire rating and acoustic isolation were taken into account (stating that “reinforced concrete is still cheap in Canterbury”). Further economy was stated to occur when making a regular standard grid layout in both directions, using the same member sizes, to make the building’s centre of mass and centre of rigidity coincide.

In another project for a design 30% “above code”, provided it could be achieved at a reasonable price, the same engineering firm recommended concrete core walls in both directions, with strong/stiff steel MRFs at the front to help non-structural elements by reducing drift demands. The owner instructed the architect to follow the engineer’s recommendations. Some other engineers also indicated a preference for buildings designed with stiff backs/spines. In some of these designs, the RC core was designed as “pinned” at the base and limited drift concentrations, while the lateral-load resistance was provided by BRBs along the building’s exterior.

With respect to type of implementation, a large number of engineering firms suggested that, unless dictated otherwise by special requirements, it is possible that future residential construction in Christchurch will be of concrete wall with steel gravity frames. During preliminary interviews conducted in February 2016, an engineer had suggested that the multi-unit, multistorey residential market might be reconstructed with all-steel structures, in contrast to the all-concrete structures used prior to the 2011 earthquake. This perception was driven by the fact that a

large residential project underway at the time used steel structural framing. However, the data collected in 2017 indicated that residential construction predominantly relied on concrete walls (either cast-in-place or precast) for lateral-load resistance. Depending on room sizes, gravity loads were resisted either by concrete walls alone or by a combination of concrete walls and steel columns. Many engineers commented that the main reason why the multistorey residential reconstruction in Christchurch uses RC walls is because, once the walls are up, one gets fireproofing and sound transmission class rating (i.e., acoustic isolation) automatically, and obviously, in some cases, intertenancy walls between apartments. By comparison, achieving an equivalent acoustics rating in intertenancy walls with cold-formed steel and drywall was said to require a double system with an airgap. However, as pointed out by one engineer, fireproofing has become more economical in recent years, so things could change. Another highlighted advantage of RC residential construction is that it allows lower floor heights, which brings economies in heating costs. As these buildings generally performed well during the earthquakes, because their RC walls were quite strong and behaved nearly elastically, many engineers foresee that a large percentage of residential buildings taller than two storeys will be built with RC walls.

One firm clarified that even though multistorey residential construction in Auckland has often been of structural steel, Christchurch has traditionally favoured RC or timber (although a large number of those were only two-storey buildings). The firm foresaw that this trend will likely continue. This is because Christchurch is a limited market for large apartment buildings with luxury units of the type commonly built in Auckland.

As mentioned earlier, the perception of some engineers is that the construction of office buildings in Christchurch is for the most part nearly completed for the near future. If this is true, then the market will next move to hotels and apartments, and this may result in an increase in the number of building projects considering RC seismic systems in Christchurch.

At the other end of the spectrum, one engineer highlighted that solid walls (irrespective of material) will not be an acceptable solution for many types of buildings; for example, it was mentioned that they are not desirable in carparks, for safety reasons, as they make it easier for attackers or homeless people to hide (highlighting how social factors can also drive choice of structural system).

As far as other wall construction types are concerned, a few engineering firms mentioned that reinforced masonry walls have been used in some instances as they can be cheaper than precast for buildings of three storeys or less. However, the price comparison depends on supply and demand at any given time, including how busy precasting yards and masons (of which there are a few) are in Christchurch, and on whether speed of construction is an issue (as precast elements dropped in place and bolted together is obviously much faster).

Finally, one engineering firm commented that for low-rise industrial buildings, tilt-up concrete walls remain an economical option (and that tilt-up walls performed well during the Christchurch earthquakes, even when they had brittle mesh), but that precast walls offer a better finish and lower needs for on-site management during construction. However, another firm expressed concerns that tilt-up construction has some quality-control issues. One engineer also mentioned that a new proprietary wall system, consisting of a cement fibreboard with rails in which concrete is poured, is also starting to be used in construction. The engineer considered it is as good as masonry but indicated that contractors do not like it.

6.2.12 Floors

The majority of engineering firms indicated a preference for composite floor systems with cold formed steel decking. It was stated that this steel floor system is fast to construct, light and cheap. It also allows pouring of the concrete topping over several levels at once. It was stated to work well with steel beams and to be often placed between secondary beams. In one case, it reduced construction time by 25%. In some cases, the concrete thickness has been increased to control floor vibrations. Fully unpropped composite floors are preferred.

One of the firms strongly advocated for concrete floors poured on timber infill prestressed rib floors, which they stated are light, cheap, easy to pre-camber, effective in providing some fire rating, and convenient for layout of services, although they can take longer to install in place than steel decking. Another company mentioned it as an option.

Hollowcore flooring was the most common flooring system before the earthquakes in concrete structures. It was also used in one building over steel gravity beams just before the earthquakes. However, questions about performance arose based on tests performed over the past few years. Similar issues have been raised about flange-hung double-T floors. Nevertheless, these have been incorporated in at least one building in Christchurch as a result of an architectural request. Many of these modern flange-hung double-Ts now contain a steel I-section to provide some toughness. In some timber structures, timber floors are used.

One of the costs for steel buildings is fire rating. The slab-panel method limits the amount of fire-proofing material needed for secondary beams and decking, thereby making buildings with composite cold form decks and secondary beams more economical.

On a related floor issue, note that some engineering firms do not explicitly design diaphragms for horizontal actions. Others have always considered them with strut-and-tie approaches.

6.2.13 Ceilings/Services

After the earthquakes, because there was significant damage to ceilings and services, the industry has focused more on these. Initially it was not clear who took responsibility for these elements, as they are not structural or architectural components, although they may be specified by either, or any other, group. Currently most of the responsibility falls back on contractor, who generally hires an engineer to sign a PS1 form. Generally, PS1, PS3 and PS4 are requested. The council checks PS4. In order to avoid ceiling issues, some engineers design buildings without suspended ceilings, where all services can be clearly seen.

6.2.14 Hybrid Designs

Hybrid buildings are defined as buildings where multiple structural systems are used together, even along a given elevation. This has been done either using combinations of conventional systems, such as RC walls and steel frames, or of innovative systems such as post-tensioned rocking frames, friction and lead dissipators, and yielding BRBs for the sake of showcasing technology (at the client's pleasure, and more expensively in the latter case).

In particular, one firm advocated the use of hybrid design meeting the above description in nearly all of its projects, including (in some projects) combinations of EBFs and RC walls, or precast panels hinged at their base supplemented by EBFs for drift control and with EBFs on the second floor resting on top of first-floor walls. The same firm mentioned using "clipped-on" RC cladding panels so as to not contribute to the lateral-load-resisting system, as a way to achieve symmetry in stiffness and strength of the structural systems. In another hybrid project for an IL3 building on bad soil conditions and an unusual site shape, four lateral-load-resisting rocking shear walls were used, together with a large number of "clipped-on" RC cladding panels, and supplemental steel MRFs to limit elastic drift.

It was commented that hybrid designs are primarily driven by cost, architectural requirements, and structural effectiveness.

7 Impact on Wellington and Auckland

7.1 General

Interviews were conducted with six structural engineering firms in Wellington and Auckland to determine if (and how) the Christchurch earthquakes of 2010–2011 had an impact on structural-engineering practice in building construction in those two cities. Auckland and Wellington are respectively the first and second largest cities in New Zealand, with populations of nearly 1.5 million and 400,000 people in their urban areas. (For comparison, Christchurch is the third largest with slightly fewer than 400,000 people, but is the largest in the South Island). Despite some similarities in the impact of the Christchurch earthquakes on these two cities, findings are presented below in separate sections because a number of important differences must be emphasised. While the number of engineering firms visited is relatively small compared to the study conducted in Christchurch, the interviews nonetheless provided valuable insights and perspectives from experienced engineers in respected engineering firms. The information presented here reflects the views expressed by the engineers met (and not those of the authors).

In the course of the conversations summarised in the subsequent sections, many engineers indicated that the recent Kaikoura earthquakes had an equal, if not more significant, impact on Wellington and Auckland to the Christchurch earthquakes. Given that these earthquakes have not received as much media attention as the Christchurch earthquakes (almost none internationally), possibly due to the small numbers of deaths and the media's primary focus on casualties and building collapses in such events, some of the characteristics of the magnitude 7.8 earthquake on 14 November 2016 are summarised here. Some information on the 2013 Seddon earthquakes is also provided, as nearly all of the engineers interviewed indicated that they also had a significant impact.

7.2 Impact of 2013 Seddon earthquakes and 2016 Kaikoura earthquakes

A shallow, magnitude 6.5 earthquake occurred on 21 July 2013, centred in New Zealand's Cook Strait, between the North and South Islands. It was followed a few weeks later, on 16 August, by a magnitude 6.6 earthquake on a nearby adjacent fault. Both earthquakes caused some moderate damage in Wellington, located at the southern tip of the North Island, 55 kilometres (34 miles) north of the epicentre, as well as in the town of Seddon and the wider Marlborough area of the South Island. After the Seddon earthquakes, police closed parts of Wellington's CBD, where the façades of a number of buildings were damaged and deemed potentially dangerous (Quilliam, 2013). One damaged building in Wellington was demolished.

The more recent 2016 earthquakes occurred in the northeastern part of the South Island of New Zealand, near the town of Kaikoura. The major event, on 14 November 2016, had a release of energy much closer to Wellington than suggested by the epicentre location (NZSEE, 2016). The magnitude was 7.8, and the

earthquake consisted of rupture on up to six faults. This was the second largest event in New Zealand since European settlement, and the largest since 1855. The shaking was felt around much of New Zealand. The rupture zone extended approximately 200km, north-northeast past Kaikoura. The shaking lasted approximately 90 seconds in some locations, which is significantly longer than the 20 seconds of the 22 February 2011 Christchurch earthquake. The land under the sea along the coast rose about 4m over a length of about 100km. The peak ground surface rupture, a right lateral slip of 10m, went through a house. Another two-storey house with unreinforced brick, Elms Estate, collapsed. Two people in Kaikoura were killed as a result of the earthquake.

Response spectra for ground excitations recorded near the fault indicate that the peak ground shaking was 1.27g in Ward (near the northern end of the ruptured fault), and the peak 5% spectral acceleration there reached 4g. While much of this region is sparsely populated, there was considerable damage to slopes and artificial structures.

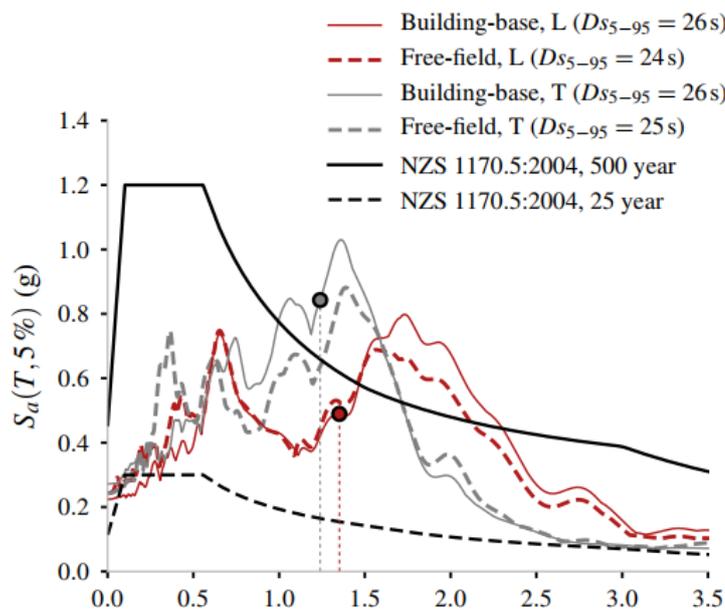


Figure 7-1: Spectral acceleration from 14 November 2016 earthquake, on a soft-soil site in Wellington (Courtesy of Dr. R. Chandramohan, University of Canterbury)

GeoNet (the official source of geological hazard information for New Zealand) estimated from reconnaissance flights that there may have been from 80,000 to 100,000 landslides. These blocked road access on the coastal route to Kaikoura. Also, damage along the inland route through Waiau cut off road and rail access to the town of Kaikoura. Over 600 tourists caught in the town had to be transported out by air and boat. The Clarence and Conway rivers were blocked by landslides, behind which lakes formed.

The shaking on a soft-soil site in Wellington is shown in Figure 7-1. Buildings with periods of about 0.8s–2.0s were most significantly affected, although the extent and significance of this damage, in many instances, was not publicly revealed. It may be seen that the shorter period structures, such as many of the older Wellington building stock, have shaking levels similar to the 0.25 ULS line (the line corresponding to the 25-year return period in the figure) and so did not experience extreme distress. Generally, for all structures with periods less than 0.8s or greater than 2.0s, the shaking was less than the ULS level (indicated by the line corresponding to the 500-year return period), and was therefore less significant than is expected in a design level “big one”.

Following the earthquake, many engineers were tasked to conduct building assessments to determine the significance and reparability of the damage, and whether this damage had reduced the resilience of the building significantly, irrespective of whether the building may have previously been categorised as “earthquake-prone” (i.e., vulnerable to earthquakes) (NZSEE, 2016). According to this document, a number of issues have been observed. These include:

- a. Damage to floor diaphragms in RC buildings with MRFs, particularly in buildings with precast floor systems, typically constructed since the late 1970s.
- b. Plastic hinge elongation effects that have cracked hollow-core units and reduced the seating of precast flooring. These raised concerns because seat lengths, particularly in pre-2000 buildings, may not have been large initially, which created a risk of floor collapse. At least one relatively new structure sustained such a collapse.
- c. Shear cracks in columns. One building found to have such significant shear cracks was consequently demolished.
- d. Single cracks forming at beam-hinge zones of frame structures possibly suggesting reinforcement yielding concentrated in one location.
- e. Compromised glazing systems.

While New Zealand engineers have experience in assessing building damage from previous events, because no national state of emergency was declared in Wellington following the earthquake, engineers or government representatives were not free to enter any building they wished. Entry was at the discretion of the owners. Given that a thorough inspection of structural elements, often hidden behind ceilings, cladding, or interior wall finishes, can cost several thousands of dollars in inspection costs alone, discovery and documentation of all damage was expected to be a lengthy process. By the end of 2016, the Wellington City Council required the owners of some 80 buildings in Wellington CBD to perform more in-depth inspections to determine the extent of damage (Devlin, 2016) and, presumably, if they are likely to be further damaged in future earthquakes. This included a number of buildings that had been closed after initial inspections. Buildings targeted by this ruling were typically those of 8–15 storeys (in the 1 to 2 seconds period range), on soft soils or ridgelines. They are typically located where development occurred in 1980s–1990s. Consequently, the inspection programme

targeted multistorey concrete buildings with precast floor systems. Note that many of these buildings were still operating while being evaluated. Also note that, generally, in both the 2013 and 2016 earthquakes, structures on stiff soils performed well (“Up the hill, people didn’t even get out of bed”, said one engineer).

On the day the Wellington interviews were conducted, the 80 buildings identified above were still the subject of more detailed structural engineering evaluations. Incidentally, on the day of the interviews, the newspaper headlined that four of those buildings located in the CentrePort district were to be demolished and three more were awaiting to be assessed (Rutherford, 2017). Findings from some of the structural investigations have been published since (Henry et al., 2017).

7.3 Impact on Wellington

7.3.1 Wellington Design Issues

Both seismic and wind design forces are larger in Wellington than in Christchurch. With respect to seismic forces, the value of the Z coefficient is 0.4 for Wellington, whereas it was 0.22 for Christchurch prior to the 2011 earthquake and has been raised to 0.3 since. The Z factor is used in New Zealand to obtain the elastic site design spectra. It approximately represents the peak ground acceleration as a percentage of gravity on stiff-soil sites. Wind forces can also drive lateral-load design, as many of the tall buildings in Wellington’s CBD are located on the waterfront. As anecdotally mentioned by one engineer, half a dozen buildings in Wellington are notorious for their noticeable movements during large windstorms. It was also mentioned by an engineer that the combination of winds and waterfront location made the use of exposed steel undesirable in Wellington, due to frequent sea spray that leads to corrosion.

It transpired from the interviews that, as a result of the higher seismicity, architects in Wellington have long been aware that seismic engineering considerations can drive decisions. As a standard practice in place even before the Christchurch earthquakes, they typically start discussions with the structural engineer from day 1 of the project (at least, for the type of large structures for which this matters). Likewise, many of the engineers mentioned having advocated and discussed with their clients, for years prior to the Christchurch earthquakes, the use of various low-damage designs. For example, there are notable examples in Wellington of base-isolated buildings and buildings having rocking frames with Ringfeder base connections built prior to 2011.

Although many buildings in the Wellington Harbour district and the CBD are on reclaimed land, competent rock is found at lesser depth than in Christchurch. This makes bored piles (a.k.a. drilled shafts) the preferred foundation system, as opposed to driven piles that create too much disturbance in an urban environment, or raft foundations, which are rarely used for new buildings in Wellington.

7.3.2 Construction Market in Wellington

As emphasised by all the engineers interviewed in Wellington, two important aspects of the construction market there must be taken into account before assessing the impact of the Christchurch earthquakes on structural design and construction practice there.

First, the economy of Wellington stagnated in the years following 2011. As a result, there was not much demand for new construction. To make matters worse, after the 2013 Seddon earthquakes, some private-sector companies decided to move their New Zealand headquarters from Wellington to Auckland (in part to lower their seismic risk). However, given the recovery of the commercial property sector in recent years (Harris, 2015; Stuff, 2016), and the damage in Wellington from the more recent earthquakes, it was stated that the 2016 Kaikoura event will have a more significant impact on Wellington than the 2011 Christchurch earthquake.

Second is the fact that the government is the largest tenant in Wellington. This is a tenant unlikely to move its operations to other cities (such as Auckland), which is an undeniable advantage to the commercial property sector, but which also has other ramifications. It was mentioned that in the 1980s, the Ministry of Works was the arm of the state that oversaw the construction of government buildings, and that these buildings were typically designed to be 20% above code requirements because the design concepts used for such buildings had to be approved by the government. (For example, the first base-isolated building in Wellington was built for the Ministry of Works itself). However, the Ministry of Works was eventually abolished, and the government started to lease from private owners, to the point where nowadays a dominant percentage of government agencies are tenants in privately owned buildings.

Many government agencies' leases will soon be coming to term and they will consider all the options available on the market. Government agencies are highly sensitive to business continuity and less intimidated by the cost needed to achieve this seismic performance. Therefore, in light of the recent Kaikoura earthquakes, many government agencies are likely to move away from older existing buildings, particularly buildings that are or will be assessed as providing only 67% or 80% of the seismic strength of new buildings. This has an impact on developers thinking about new construction. Big developers in Wellington are apparently increasingly considering base isolation for new building as an outcome of their discussions with heads of specific government departments/ministries. Other owners are more cautiously looking at high-performance buildings to provide a market advantage, but not necessarily using innovative technologies at any cost. They are seeking more "cost-neutral" solutions to be more cost competitive.

Incidentally, it was suggested by one engineering firm that an "exodus" of government agencies from older buildings, compounded by the fact that many buildings will be retired from the existing inventory when demolished as a result of the on-going post-Kaikoura engineering evaluations, could create a temporary shortage of available office space. This could lead to a construction boom, the

relocation of private-sector companies to other cities, or some of both. Given that many office buildings in Wellington were built in the 1960s–90s and are soon approaching or exceeding their 50-year life, and given that, in the post-Kaikoura context, many will consider it untenable to rent space in a building having a seismic strength assessed to be less than 70% of that provided by the latest edition of the loadings standard, owners of older buildings are contemplating their options to attract tenants. They will have to make decisions that are expected to affect the market.

7.3.3 Impact of Christchurch Earthquakes

All engineers met stated that, while the 2010–11 Christchurch earthquakes were a wake-up call to communities in the Wellington area, the 2013 Seddon earthquakes, and particularly the 2016 Kaikoura earthquakes, reinforced the message that extensive building damage will occur in earthquakes, and raised awareness locally that it might happen in Wellington sooner than later.

First, somewhat beyond engineering but affecting its practice, one aspect of the Christchurch earthquakes that had an impact on Wellington relates to the dynamic of earthquake response. One engineer mentioned that civil authorities in Wellington have been (and will continue to be) careful in their earthquake response activities, having witnessed the consequences of turning Christchurch’s CBD into a “restricted-access red-zone” that was effectively a Civil Defence area. Partly as a consequence of that approach, and partly because the damage in Wellington was nowhere near as severe as what was experienced in Christchurch, building damage was only found slowly in the days following the Kaikoura earthquakes. It was stated that the desire to prevent shutting down economically vital parts of the city (which would occur if many buildings there were “red-tagged” for future evaluation) might cause structural engineers to avoid being excessively conservative or hasty in declaring buildings hazardous during post-earthquake evaluations.

With respect to construction types, because the Wellington construction market was depressed in the years following the Christchurch earthquakes, it is difficult to ascertain how much any changes in construction have been influenced by the Christchurch earthquakes, and how much is due to the Kaikoura ones. The Christchurch earthquakes caused a large increase in seismic awareness, which translated into many owners requesting seismic assessments of their existing buildings, and in some cases led to seismic retrofit. After the Seddon earthquakes, there was still not a lot of new construction in Wellington, so impact on structural system decisions was limited.

However, in light of the aforementioned poor performance of simply supported concrete floor systems due to plastic hinge elongation in RC beams during the more recent Kaikoura earthquakes, all engineers commented that developers/owners now oppose the use of these floor systems. To structural engineers, this translates into an avoidance of RC frames – or, as one engineer put it, a “turn-off on RC frames” – due to beam elongation issues, and a conclusion

that using high-ductility systems (e.g., relying on a ductility of 6) to achieve a lower seismic strength is no longer a viable option. If ever required to design a RC frame again (“for whatever reason”), one engineer mentioned that he would likely use slotted beams, which consist of beams only connected to columns by a small region at their top bars and otherwise separated from the column by a “slot” (Muir et al., 2012). As a personal preference, it was mentioned that adding dampers aligned with the beam’s bottom bars would be the way to implement this solution. However, it was also commented that, because concrete walls are more likely to be used than RC frames in future RC construction, unless there were major architectural reasons making it necessary to use RC frames, this slotted beam system is unlikely to be implemented in Wellington in the near term.

One consequence of the Christchurch earthquakes, with respect to client perception, and observed by one engineer, is that more clients have started to ask for base isolation, or alternatively to request IL4 buildings. In the latter case, the engineer presents the pros and cons of a number of low-damage systems that could be considered, including base isolation as a possible solution. Interestingly, the engineer mentioned that, in one case, after the engineer described four or five different low-damage options that would have been appropriate for a specific tall building, and explained why base isolation wouldn’t be particularly effective if implemented in that case, the client stated: “You can say all the witchy nerdy stuff about how low-damage this is, but people go: ‘Ah, but that’s base isolation! Look, there’s the bearings’ and I can get \$50/m² extra for that”. This developer had an “anchor tenant” from the private sector who wanted a high-seismic-performance building.

As to whether base isolation, BRBs, rocking frames, or other systems are preferred, again, as in Christchurch, different “engineering cultures” exist. This is compounded by the fact that many engineering firms have offices in Christchurch as well as in Wellington. Some “cross-office pollination” occurs. Nonetheless, based on recent construction in Wellington, engineers have highlighted a few notable aspects of building design and structure types that are germane to Wellington and worth highlighting here.

- For non-base-isolated buildings, buildings in Wellington are much taller than in Christchurch, so overturning effects are large, leading to stiff core buildings.
- Design to the low drifts reported in recent Christchurch projects is less economical to accomplish due to the larger design seismic forces in Wellington. Seismic design drift demands on buildings are typically close to the code limits of 2.5%. However, some engineering firms are reconsidering the past practice of taking buildings to their drift limits in design.
- Large seismic demands in Wellington can translate into large base-isolation movements. It was mentioned that using a steel structure helps

reduce those displacement demands and that the use of bracing in tall buildings provides a good solution in that regard.

- In spite of the construction trend in Christchurch, architects in Wellington “still do not like braces”. In Wellington, braces have been used extensively in building retrofits (as shown for example in Figure 7-2), and architects there do not find braces particularly interesting or “fashionable” compared to architects in Christchurch. Yet, this does not mean that braces are not used. One of the largest construction projects on the waterfront at the time of the interviews (evidently designed prior to the Kaikoura earthquakes) consisted of a large base-isolated structure having a steel exoskeleton prominently displayed, as shown in Figure 7-3. (Also note the posted advertisements promoting the seismic features of the building).
- BRBs have been used in only a small number of new construction projects to date in Wellington, but engineers have indicated that they consider BRBs and EBFs to be well understood, low-damage, cost-effective solutions equivalent in seismic performance, and typically leave the choice to the architect. Similarly to Christchurch though, some engineers have stated the importance of careful gusset design, and the fact that BRBs may be more easily replaceable than EBF links (even for EBFs with replaceable links). Some mentioned not believing that replaceable links are a better solution.
- CBFs are typically only used on top of base isolation.
- One engineer expressed a concern over the use of sliding hinge joints, due to the possible change in friction coefficients over time, given the particular Wellington weather conditions.
- One engineer expressed a concern about the “over-reliance” on MCE being the upper limit for design for heavily strain-hardening systems (such as BRB), given that capacity design is done considering the level of strain hardening developed at the MCE, but that these demands would be larger for an earthquake exceeding the MCE.
- Older steel buildings with “less than desirable features” also deserve attention as part of seismic evaluations. One engineer reported that a steel-braced-frame building had also been damaged by the Kaikoura earthquakes and was subsequently demolished (details are confidential).
- Building codes are still intended to only ensure life safety. Although one engineer commented that “many engineers do not think beyond the code and how to improve performance”, some engineers are slowly turning to structural systems that can improve seismic performance to the extent possible without raising costs, recognising that most construction in Wellington is developer driven (“dollars drive compliance”, as stated by one of the engineers interviewed).



Figure 7-2: Example of Wellington building retrofitted with braced frames



Figure 7-3: Wellington base-isolated building with exoskeleton CBF. (a) Global view, (b) Promotional information, emphasising high seismic-design level, (c) Base isolators and gap at ground level, (d) Close-up view of brace connections and floor system

At the time of the interviews, it was obviously too early to tell with certainty what changes will emerge in building practice following the still-recent Kaikoura earthquakes, particularly considering that building assessments were still being confidentially performed. The following are some of the expectations that were expressed by different firms (not representing consensus opinions):

- Engineers will continue designing office buildings combining RC core walls and gravity steel frames, and residential buildings having RC walls.
- Rocking concrete walls are still considered to be good and desirable by a few engineering firms.
- For tall residential buildings, steel construction will likely continue to be used, due to the high seismic design coefficients in Wellington.
- Well-detailed steel structures will be the preferred choice, with well-controlled quality assurance of material and robust floor diaphragm.
- Steel construction in Wellington will be a mixed bag of different types of structural system, with no particular one foreseen to dominate all others.
- Interstorey drift will be a controlling design parameter.
- Timber construction, emphasised as using a renewable resource, will remain novel, as most developers are less confident about using timber in their projects.
- Viscous dampers will be included as an option worth considering for new buildings, even though implementation will remain a rare occurrence (it would not have even been considered an option a few years ago).
- “Not all clients will get base isolation (even though non-engineers can easily grasp the concept, which makes it popular), because it is not always practical.” It was stated that, for low-rise buildings, base isolation requires suspending the ground floor, which makes an enormous percentage difference to the budget and creates a loss of rental space on the outside, and for medium-rise buildings it is hard to make them stiff enough. Another firm mentioned the design challenge that, in small events, the base isolators remain elastic, with the result that the supported frames are subjected to forces corresponding to the lower period excitation, not to the high periods expected when the isolators will yield. The end result is that more non-structural damage can be experienced in a small earthquake than expected in the large design earthquake – something hard to explain to the owner after the small earthquake. It was mentioned that the market for base-isolated office buildings in Wellington is optimally for buildings taller than 10 storeys.

Finally, two noteworthy general issues were highlighted that will possibly have a bearing on future structural design practice in Wellington.

First, one engineer commented that one of the reasons that progressively more steel buildings are being built in Wellington is that the tradesmen highly skilled in placing concrete “are disappearing”, as “the good concrete builders from the 80s” are retiring. The new ones were deemed to be not as proficient. This has led, in some projects, to an increase in engineering fees to conduct more quality-assurance work than what was needed before. Apparently, the opposite is the case for steel fabricators, where workmanship quality is increasing due to computer-controlled fabrication. It was commented that this is somewhat of a role reversal of what occurred to the construction industry in New Zealand in the 1980s. Some local concrete precasters are also benefitting in the same way.

Second, one engineer expressed major concerns over a WorkSafe New Zealand policy (the Health and Safety at Work Act 2015 (MBIE, 2015a)), adopted as law in early 2016, which essentially requires “to eliminate risks to health and safety, so far as is reasonably practicable”, which the engineer interviewed paraphrased as “If you can eliminate a risk, you must”. Interpretation of the law when it comes to structural engineering and post-earthquake assessment is unresolved. This has significantly “clouded the field”. If a structural engineer, for example, finds a column that is damaged inside a building, hiring another engineer for a subsequent evaluation or a contractor to repair the damage could arguably expose the person entering the building to a risk (due to possible aftershocks) and therefore be construed as a violation of the law. If engineers or owners can’t send anyone inside a damaged building without risking a prison sentence for having violated the law, a lot of slightly damaged buildings that might otherwise have been salvageable may end up being demolished (from the outside). As a consequence, engineers must exercise extreme caution in not unduly calling something unsafe, as this would carry huge consequences, while at the same time upholding their duty to protect the public. The post-Kaikoura earthquakes challenges will likely determine if this law can/will be ignored in a post-earthquake perspective.

7.4 Impact on Auckland

A smaller number of firms were interviewed in Auckland as it was anticipated that the impact of the Christchurch earthquakes would have been far smaller there, given that buildings in the city are designed to much smaller seismic forces than most other large urban centres in New Zealand. The Z coefficient for Auckland is 0.13, which is three times lower than in Wellington, and more than two times lower than the new value used in Christchurch. Nonetheless, it was found as part of the discussions that “Christchurch was a watershed event”. This is because, before then, the engineers knew about the expected limit states of various structural systems during an earthquake, but the clients did not. After the Christchurch earthquakes, the clients understood that ductility equals damage and the challenges of how to repair damaged buildings (even when fully insured). As a result, a change in client perception occurred even in Auckland, particularly for

“owner-operators” who have a vested interest in the building over the long-term, as owners became more interested in buildings’ seismic performance. Whereas prior to the Christchurch earthquakes, discussions with clients on expected seismic damage rarely occurred (one engineering firm said “never occurred”), they are now always part of the conversation. Clients now have a readiness to listen to engineers, and, as a result, engineers are getting better at explaining things. Equally importantly, it was emphasised that further to the Kaikoura earthquakes, clients now understand that business continuity and disruption is not 100% affected by the structural system alone, but that it also depends on non-structural damage and on damage to adjacent buildings (affecting all surrounding neighbours). Life safety versus business continuity is now also part of engineers’ discussions with clients. Incidentally, it was mentioned that prior to the Christchurch earthquakes, when base isolation was considered as an option, it was often turned down on the basis that the extra cost could not be accommodated by the available budgets, even for critical buildings such as hospitals.

That being said, as one engineer said, it “never happened in my experience in Auckland that a client asked for a specific type of structural system”. Clients are typically more aware of the importance of proper seismic design/detailing of stairs (after having witnessed the collapsed staircases in the Forsyth Barr building in Christchurch). It remains the practice that consultants prepare structural options and present them to the client. Decisions at the concept level drive the material decisions and are made before the preliminary design. Cost is always a driver – but it comes after the concepts have been chosen. Furthermore, it was stated that clients in Auckland generally have no preference for structural materials. While the construction market was “flat” at the time of the Christchurch earthquakes, for totally unrelated reasons, it is now “overheated” and under pressure, so decisions with respect to construction material will often come down to availability issues (e.g., even supply of concrete floors can be a constraint). While it was mentioned that the market is split approximately 50/50 between RC and steel structures, and mostly driven by architectural issues and lowest bid, it was underscored that when schedules are tight and availability of material is considered, projects are a bit more likely to be framed in steel, and most buildings in Auckland have a large measure of steel. This is because concrete construction in Auckland still requires considerable on-site labour whereas more off-site work is possible with steel construction. In fact, significant steel fabrication in some projects is completed offshore. Many engineers indicated that procurement of fabricated overseas steel raises questions about the reliability of the steel material and fabrication procedures, and the importance of quality control.

An important driver in the above decisions is the fact that the design of taller buildings (>20 storeys) in Auckland is governed by wind, which leads to favouring the use of a concrete core wall system with jump forms, and steel gravity frames, even for apartment buildings. Lower rise apartment buildings tend more to rely on precast wall construction. Furthermore, because seismic forces are much lower, elastic seismic design is possible in Auckland. One engineer mentioned that

designing a RC frame as a nominally elastic structure (with a ductility of 1.25) can still be considered, and is “doable” in light of the lower seismic forces.

It was also indicated that the increase in the use of steel in Auckland can be seen as part of an evolution as technology changed over the past decades. After the boiler-maker’s dispute in the 1980s, which shifted practice to RC, the extensive marketing done by the steel industry over the next decade (emphasising the benefits of steel construction, developing different steel structural systems, providing seminar series, etc.) had a progressive impact on practice. For example, whereas a hospital designed in the 1980s would have had precast beams designed for high ductility, a hospital designed in 2000 in Auckland would likely have had a steel CBF. By the mid-2000s, more tall buildings having RC core walls with steel gravity frames were appearing. Hence, before the Christchurch earthquake, many steel buildings were being built in Auckland, particularly when there was a need for longer spans, shorter construction time, less mass, and benefits from off-site fabrication.

Given the above, a surprising number of buildings in Auckland have been built with BRBs in recent years (e.g, Auckland Structural Group, 2015). Even though it was mentioned that architects still hate braces (particularly at ground level where it impedes retail) and are still figuring out how to build around them, some engineers in Auckland have interacted with colleagues from Christchurch and have developed a taste for BRBs as a low-damage solution. It was stated that the ability to independently “tune” the strength and stiffness of BRBs is “handy” and that the ability to obtain the BRB overstrength from testing (and to use that advantageously in capacity design) provides a level of confidence. BRBs were stated by some Auckland engineers to have a more dependable ductility than links in EBFs, and are seen as an alternative to achieve high operability and damage limitation. This is beneficial when base isolation is difficult to implement due to site conditions (such as a stepped site or existing foundation issues), when a client has a budget to respect but is still wanting a limited/low-damage solution to limit non-structural damage and make the building serviceable after an earthquake, or for other reasons. BRB potential post-earthquake replaceability was also positively highlighted. In at least one instance, BRB frames have been used on the periphery of a large floor area core-wall building to limit torsional response.

It was considered that frames with BRBs may induce high accelerations, but the ability to provide lesser drift (thus less damage to façades) was perceived as an asset. In some projects, drifts of less than 1% were specified to ensure that façades will be serviceable post-earthquake, with all seals still workable. Note that, in Auckland, low deflection and stiff structures are preferred over flexible ones, even though the latter benefit from lower floor-accelerations. This is, first, because structures that drift beyond 1.5% are considered to require too much detailing to accommodate the drifts, and, second, because most of the damage to building contents in the recent earthquakes in New Zealand is perceived to have been drift-related.

However, it was stated that BRBs have not replaced EBFs in Auckland as a general trend, as EBFs are still dominantly used. For example, it was specifically mentioned by one engineer that EBFs provide a cheaper solution for carparks.

With respect to foundation design, it was stated that when buildings in Auckland are on reclaimed land, bored cast-in-situ piles are generally used, and there has been no change in practice in this regard due to the Christchurch earthquakes.

It should be noted that, as the comments expressed above are from two large Auckland engineering firms whose main clients are Listed Property companies interested in leasing buildings to large firms on 7- to 14-year leases, their opinions may be different from those of other firms with different clientele.

8 Perspectives from Architects, Developers, and Project Managers

To obtain wider perspectives about the factors that drove the selection of specific structural systems in various projects, interviews were conducted with an architect, a developer, and a project manager in their respective Christchurch offices in March 2017. Given that meeting a large number of individuals and firms from each of these fields of expertise was not feasible due to time constraints, the authors were referred to respected professionals deemed to have the experience and stature necessary to represent the broader views of their respective groups. These were:

Peter Marshall, Architect and Managing Director of the firm Warren and Mahoney Architects, which presents itself (per its website, <http://www.warrenandmahoney.com/en/>) as “an insight led multidisciplinary architectural practice with six locations functioning as a single studio”, with clients and projects throughout New Zealand and the Pacific Rim. The firm has “over 220 specialists working across the disciplines of architecture, master planning, urban design and sustainable design”.

Gordon Craig, Development Manager, and James Jackson, Project Manager, for the development company Ngāi Tahu Property Limited (NTP), which is described (per its website, <https://ngaitahuproperty.co.nz/>) as “a leading New Zealand property development and investment company with assets valued at approximately \$500 million”.

Matt Allen, Executive Director, RCP, which (according to the website <http://www.rcp.co.nz/>) “has been providing professional Project Management and Project Programming services to New Zealand’s property industry since 1996”. Also, according to Matt Allen, RCP is the largest project-management company in New Zealand, with 28 staff in Christchurch who have been involved with many of the major buildings constructed there since 2011.

The following subsections describe the role of each professional, in general terms and as part of the Christchurch reconstruction, and summarise how some of the decisions have been made by stakeholders through interactions with these different professionals. Given the challenge of describing the perceptions and expectations of entire professional communities and because different professional “cultures” presumably exist across firms within each profession (as was highlighted earlier for structural engineering firms), the findings presented below could conceivably have been slightly different if other equally respected professionals had been interviewed. However, the authors trust that the information gathered as part of these interviews remains helpful to further shed light on the motivations that drove the selection of structural systems as part of the Christchurch reconstruction.

The narrative below captures the views of the professionals interviewed, both by using their words and paraphrasing.

8.1 Architectural perspective

8.1.1 Background on Architectural Approaches

The role of the architect is to design spaces to accommodate the needs of users; in other words, they are primarily focused on people in the building and concerned with designing the space accordingly. They help the client develop the design brief for the project, and, in general, architectural design is developed from the “inside out”. That is, the space where the intended activity is to take place is important. For example, for an office building, making the workplace a pleasant environment requires the architect to consider the possible needs for an open plan, natural light, transparency (the ability of those outside the building to see activities inside), connectedness/togetherness, and overall visual outlook. There is also an urban design component, considering courtyards, pathways, and alleyways.

Architects are seeing an exciting convergence of the latest thinking regarding workplace design, engineering solutions, cost effectiveness, and speed of construction. There have also been many advances in glass-cladding technology, which have made glass façade construction possible. Furthermore, in many architectural companies, full 3-D renditions of all structures are developed. It was estimated that about 25% of buildings being constructed are fully developed on Building Information Modelling (BIM). This is a relatively recent development; before the earthquakes, most firms did not use BIM. It is foreseen as likely to eventually take over the industry. The above are seen to be major shifts in practice. For materials where design and fabrication can be automated, the above convergence is easier to achieve.

8.1.2 Overall Changes in Architectural Practice due to the Earthquakes

Before the 2010–2011 earthquakes, Christchurch had a “strength” in concrete construction. This was both an architectural and engineering strength. Consequently, a number of architectural companies had a tradition of featuring the concrete structure as part of the building’s architectural expression, be it beams, columns, walls, floors (e.g. flange-hung double tees), or other aspects. There was, and still is, among many Christchurch architects, a desire to show the structure for what it is.

During the earthquakes, there were problems with concrete connections, precast elements, and stairs. Many of these were related to reparability. As a consequence, after the earthquakes, clients generally wanted something cost-effective, fast to build, and repairable. To meet those combined objectives, engineers conducted studies and suggested braced steel structures. Architects embraced this, and the braced frames often replaced the concrete shear walls. This provided opportunities from an architectural perspective. Consistently with previous practice, the architects expressed the structure, but this time by integrating braces into the

architectural expression, also concentrating on expressing the steel frame connection details. The connections are important because, if they are visible, they are more readily inspected and repairable. Elegant connections are therefore sometimes designed, even if it entails a cost premium. The clients are aware and supportive of these changes.

Compared to concrete walls, architects find that braced structures allow lightness and flexibility, especially with open-plan work places. For these reasons, braced frame buildings are “spreading” across the country. While there are still many concrete walls built, the braced frames are here to stay and are not just a temporary reaction to the earthquakes. Nonetheless, as before the earthquake, architects still prefer the open space that moment frames can provide.

For residential structures, as opposed to commercial, issues of privacy, acoustics, and fire protection lend themselves to concrete walls. However, even for residential structures, there is a tendency for more glass and openness than before. A number of multistorey residential structures have been completed in steel, or a hybrid with steel, because of the advantages relating to their light weight, connectedness/transparency, and cost/time.

Incidentally, it was mentioned that the situation is different in Auckland, as there is still inertia there, with many construction companies preferring concrete structures. However, there is a worldwide tendency for a more highly glazed building, so it was suggested that the practice may possibly change there over time.

Structural forms have also changed in other ways as base isolation has been introduced in many buildings. There has been a learning curve related to the complexity of how to design the first level in base-isolated structures, as well as how to allow for movements. There has been a cost associated with this learning curve (as well as for some of the other new structural systems being used in Christchurch).

Finally, as a consequence of the earthquakes, architectural design has been less adventurous than before – for example, fewer cantilevers are being used.

8.1.3 The Decision Process

Since the earthquakes, client procurement of professional services has changed, in that architects and engineers now often come together a bit sooner in the design process. This was deemed to be due to time requirements to rebuild the city, especially due to time constraints on insurance policies. Discussions on the structural system are collaborative. Most clients do not require specific structural systems, but they now have a heightened awareness of damage levels (e.g., 33%NBS, 67%NBS, 100%NBS, and 120%NBS), and what they mean.

The extent of insurance coverage has had a significant effect on decisions, as much of the rebuild has been conducted using insurance funds. Owners in Christchurch at the time of the earthquakes had insurance policies that, depending on the policy, generally stated that their damaged buildings would be returned to a specific

condition, or replaced with something equivalent. There was quite a variance in policies and between insurance companies, and thus many negotiations. The CCC required all new buildings to be designed to at least 100%NBS. Now, therefore, to prevent overspending, the insurer often sits at the table during construction discussions.

Beyond the client, architect, engineer, and insurer, also sometimes present during the construction discussions are a quantity surveyor, to help understand the likely costs of newer systems and options, and other engineers, such as a fire engineer and a building services engineer, who is required to ensure proper seismic bracing of equipment/ceilings and some walls. While tying non-structural systems was already part of a building services engineer's mandate before the earthquakes, building services/ceiling issues are now taken more seriously after a number of failures, and clients are prepared to pay for this involvement.

Before the earthquakes, it was not always clear whether the structural engineer or the ceiling manufacturer was responsible for ceiling and equipment failures. After the earthquakes, ceiling manufacturers started to employ their own engineers.

Another major change following the Christchurch earthquakes is the cost of foundations. For example, foundations now can be deep rafts with lots of steel reinforcing. Sometimes the spacing between bars is small. For new construction, foundations may now be about 6% to 7% of the total cost, which is about twice what it was before the earthquakes. This means that, for a project on a fixed budget, there are less funds remaining for the rest of the project, such as the superstructure design and architectural expression. However, foundation costs vary significantly throughout the city, depending on location. In the end, greater costs would generally land on the tenants.

8.2 Developer perspective

8.2.1 Background on Developer Approaches

There are many different kinds of developers, focusing on different market segments and having dramatically different perspectives on how projects are approached. NTP's views, summarised below, represent those of a developer/investor with a long-term perspective. (Speculative developers possibly hold different views). At the time of the interviews, NTP was soon to have completed four significant new buildings as part of Christchurch's reconstruction.

A number of these buildings were developed together with the prospective tenants, as is often the case with developer/investors. Namely,

- In one building, the tenant was a structural engineering company that wanted a base-isolated structure to showcase its expertise and resilience.
- In another case, the government was the tenant, and they requested 100%NBS, but stated no preference for structural form. (The building ended-up using BRBs).

- In a third building, there was a large corporate tenant, who (as well as the developer) wanted no loss of income stream from future earthquake damage.

Note that some of the larger tenants (e.g. government and large organisations) “shop” for their developers. This is a time-consuming process that requires developers to respond to Requests for Proposals.

8.2.2 Overall Changes due to the Earthquakes

Developers/investors have observed damage to their properties due to the 2010–2011 earthquakes. They have seen concrete buildings with cracking, spalling, and, in some cases, collapse. They have also witnessed significant non-structural damage. As a result of this damage (even if only non-structural damage), the tenants left, which translated into a corresponding loss of rental income. Significant non-structural damage occurred to ceilings. Also, the gypsum board sometimes cracked, compromising the fire ratings. The possibility that tenants would vacate their premises due to non-structural damage was not considered much before the Christchurch earthquakes.

As a consequence of the above observations, after the earthquakes, corporate/government tenants on the market are “happier with steel”, and many buildings built as part of the reconstruction have had steel frames. As such, it can be said that the choice of structural system was guided by the tenants (in addition to the engineers).

As a developer, NTP indicated liking off-site fabrication of steel. NTP also expressed an interest in considering timber in the future, and is planning some apartment buildings which may be all concrete. NTP also does its own geotechnical assessments on projects, include site-specific spectra, as a way to make sure that design won’t be “over the top”.

Still, in the post-earthquakes context, there is a concern that some engineers are “gun-shy” and sometimes overly conservative. NTP believes there remains a balance needed between conservatism and economy, so they have always had their designs peer reviewed. This has sometimes resulted in significant savings.

After the earthquakes, the clients and CCC are both requesting a Producer Statement 1 (PS1) for design, and Producer Statement 4 (PS4) indicating compliant installation, for building services/non-structural elements. The legislation existed before, but now it is being enforced; ceiling and service duct restraints are now being undertaken in all NTP buildings. Sometimes, the ceiling installer wants to design the restraints, but developers often also require that they are checked by an engineer. Larger corporate tenants require this.

It was noted that obtaining insurance has become harder. The deductibles have become larger, and “loss of rent” insurance is harder to get and some companies limit it to 36 months. As a result, resilient buildings are desirable.

8.2.3 Factors Driving Decisions

As indicated above, tenants have had an important role in some decisions. In particular, some tenants are requesting to see the PS1 from the design consultants. As such, some tenants lead the decision-making process. For example, large legal/accounting firms look after their staff well and are prepared to pay more for some resilience. However, many smaller companies (<20 staff) simply require 100%NBS because they can easily move out after an earthquake and have employees work from home if need be.

In the decision-making process, the structural system performance experts are the structural engineers. They offer options to the developers. The selection of the structural form is a team decision, considering the requirement that the solution meets the brief (including cost, time, etc.). Lots of questions are asked at briefings. Developers/investors like to hear a “good story” as to why a particular system may be best, and are not just interested in the cheapest option. Since the earthquakes, there have been many more options for building systems and many more discussions. In the planning meetings, people around the table include the architect, the quantity surveyor, the structural and fire engineers, and sometimes a building services engineer.

From the perspective of a developer/investor, new monitoring and inspection technologies have become interesting as a way to rapidly provide tenants, owners, and engineers a level of assurance that the building is usable after a moderate, non-damaging event, and that the tenants do not need to move out. These technologies help to prevent loss of rent after an earthquake, and have already been installed in some buildings. Monitoring equipment signals obtained after small/moderate shaking are sent immediately to the engineer, who can immediately determine if the building has behaved in a certain manner, if a rigorous/invasive inspection and testing are not required, and if there is no need to evacuate. With respect to inspection, in one building, 300 inspection hatches were provided to facilitate post-earthquake assessment – one beside each beam-to-column joint. Not all buildings have received this level of technology implementation, but it is being potentially considered for all future tall buildings or for large corporate clients (e.g. government).

Note that when developers/investors conduct feasibility studies and assess market rates, they consider costs related to construction, tenancing, insurance, and maintenance (among many things), but they do not consider possible losses due to disaster (such as earthquakes, fires, or other risks). All risks are addressed/mitigated by insurance. Insurance costs increased significantly after the earthquakes, but they have moved down again to pre-quake levels. From this information, lease rates per square metre can be established, either using a basic rate plus operating expenses, or a gross lease-inclusive value (government tenants prefer the latter approach). In 2010, most rents were less than \$300/m²/year, with some up to \$400/m²/year. Now, rents are greater than \$400/m²/year as all the old

“C-grade” construction is gone, so it can be a challenge getting tenants into the new high-class buildings.

8.3 Project management perspective

8.3.1 Background on the role of Project Management Companies in New Zealand

Project managers are often hired by clients to represent them in dealing with the architect, engineer, quantity surveyor, contractors, and other professionals.

Project management developed as a discipline both to provide the client with an experienced outlook to assess the engineering and architectural advice, and, because the client often does not know exactly what is required for a particular building, to conduct a reverse briefing for the project. (Note: In a reverse briefing, the project manager proposes a fairly generic solution, based on prior knowledge or assumptions, and the briefing process works backwards from there to refine and finalise the brief and thus the design solution (http://www.workplacechange.org/resources/0000/1840/Guide_to_Strategic_Briefing_-_WCO.pdf). Hence, the key objective of this project-management approach is to “communicate reality” to the client, often doing so by helping the client ask the proper questions of the consultants, rather than by giving advice (which, incidentally, avoids liability on the part of the project manager). In essence, project managers rely on their prior experience and learning (and that of their staff) to duly advise their clients. For example, the RCP staff comes from diverse backgrounds, with about 1/3 from architecture, 1/3 from engineering, and 1/3 from either quantity surveying, construction, business administration, real estate, or the “school of hard knocks”. It was mentioned that, contrary to structural engineering, where there are minimum licensing standards, in project management there are no standards. This results in a huge variation in the quality of work and level of services provided, even though hourly rates do not differ significantly between project management companies. It was also mentioned that, until about 30 years ago, the “kingmaker” in terms of advising the client about new construction was the architect. Since then, project managers have begun to play a key role for large projects. (They are typically not involved in small projects). By 2017, it was estimated that project managers (employed within the company, or brought in as consultants) are relied upon to provide key advice in about 90% of large projects in New Zealand, and architects are used for the remaining 10%. However, in Christchurch immediately after the earthquakes, many clients had relationships with engineers as a result of the post-earthquake inspections, so the clients often went to the engineer as their first point of call for advice.

8.3.2 The Response of Landowners/Investors after the Earthquakes

Project managers are in a privileged position to witness the behaviour of investors over time. After the earthquakes, it was observed that some major landowners took their insurance pay-outs and left Christchurch, investing in other places instead, such as Auckland, Australia, or Canada. Other big institutional investors have not built in Christchurch. Some family trusts are waiting for the market to

return and the cost of land to stabilise before becoming involved in new Christchurch developments.

Of the local investors, it was stated that 80% have stayed and about 20% have left. Of those that have stayed, a number are in a “wait and see” position. They have the options of putting the funds received from insurance pay-outs in the bank and getting a guaranteed 2% return, or investing in a development project with the possibility of greater returns but the concurrent risk of losing it all. This is why many are waiting until all signs indicate a strong market demand before investing in building projects.

Some large overseas groups have been looking at the possibility of investing in Christchurch. They are typically interested in the possibility of \$500m development projects, but the lot sizes in Christchurch can typically support \$20m development projects, which is an insufficient number of tenants to keep these investors interested. One of the reasons for the low number of potential tenants is that not many companies have headquarters in Christchurch. As far as big developments are concerned, it was mentioned that the forthcoming convention centre may have a big influence on the Christchurch market, but that while it may not be the last of the big developments in the CBD, it may be the last for a while.

The major factor driving investment decisions is still financial return. Most investors/developers expect an initially calculated return of 20–30% to cover the risks in development and to ensure a 6–9% investor return in the worst-case scenario. In New Zealand, the owner assumes all the risk. Bankruptcy laws are different from those in other countries and this affects the decisions made. For a \$100m development, the owner may typically put up \$20m and borrow \$80m from the bank. There are obviously exceptions, as financial return is not the drive for all projects. For example, one investor in Christchurch was said to be making an “architectural monument” to his wealth.

When a developer plans a building project, they perform a financial feasibility study. The price depends on the location. For example, some premium office space may be leased at \$1000/m²/year, while another less prestigious part of a building (e.g., facing a carpark) may be \$400/m²/year (incidentally, it was mentioned that even though the current maximum height of new buildings of seven storeys seems to be more of a town-planning issue than an earthquake issue, the cheapest rents have been observed to be for space on the upper levels). As part of this feasibility study, the amount of space rented at different dates after construction is estimated. Anchor tenants are important, and government tenants are highly desirable as they seldom move. Anchor tenants can have significant influence on different aspects of the building that they are planning to move into. In the end, it is the tenants who dictate, directly or indirectly, the type of structures built.

8.3.3 Drivers for Structural Form

From a project manager's perspective, in general, a client wants optimised value. Cost is therefore important for all building components, not only for the structural system. There are different sorts of developments: some are speculative, others have a secure tenant prior to construction. They often have different drivers for value.

When it comes to structural systems, it was stated that if there were one that dominated with respect to optimised value, then everyone would use it. This is obviously not the case. The view from a project-management perspective is that concrete has been cheap in Christchurch – cheaper than anywhere else in New Zealand – and that the University of Canterbury supplied concrete structure design expertise. Therefore, before the earthquakes, concrete was a normal form of construction, typically relying on “big and strong” precast panels. Even more so, given that steel costs had been increasing and peaked in 2005–2006, but the market eventually corrected and steel became good value again, particularly considering that it is easier to specify and detail than RC, so faster for design and construction.

The choice of steel versus concrete does not seem to be a major issue for tenants/owners. However, sometimes the choice of structural system can affect the available leasable area, which can be a bigger factor. Different buildings not only have different costs, but also different structural forms as different tenants seek different types of space. It was mentioned that shear walls can reduce the rental rates and that moment frames increase the rental rates (but cost more to build). Buildings with braces have the lowest construction costs overall, however, as before the earthquakes, they reduce the “ability to lease”. The project manager helps clients quantify these trade-offs, while recognising that different clients want different things. For example, some clients will specifically request IL4 structures, or structures designed to 180%NBS, and hospitals want structural framing that will provide maximum flexibility (noting that they might be modified every 10 years and pulled down after 50 years).

It was mentioned that while the choice of walls or braces is sometimes cost driven, it is also sometimes driven by engineering issues (or other non-cost issues). The engineers have a large amount of influence/control over the system chosen. It was noted that when different structural forms are used, they depend significantly on what experience the engineer has developed. For example, there are engineering firms that specialise in base isolation using lead-rubber bearings, and others that prefer sliding-friction devices (e.g., the friction pendulum system) for base isolation; some firms like to “over-engineer” to limit risk (sometimes with simple things, like specifying a 10mm plate rather than the 8mm plate that may be required), while others like to put many “bells and whistles” into the structure. These things can add to cost.

It falls within the project manager's scope of work to consider the wider issues brought up by some of these solutions. The example provided was that some of the triple pendulum base-isolation units can require a lot of temporary bracing, which needs to be considered in the cost and construction timeline.

Along those lines, it was mentioned that low-damage construction is not always as attractive as it is sometimes purported to be. To support this statement, it was stated that:

- a. Some developers provided a high level of protection (e.g. base isolation) soon after the earthquake, guessing that clients would be prepared to pay more for higher levels of protection, but some of those developers have had trouble getting clients.
- b. There has been no evidence that insurers will give cheaper rates for base-isolated buildings. They consider macro-effects, and their different rates depend more on whether the building is designed to modern codes or not.
- c. When the relative risk of fire or earthquake are considered, the higher risk is probably fire. A significant number of buildings are constructed with alarms, to allow people to egress, but not with sprinklers (so, logically, people may be saved, but the building may be destroyed due to fire).
- d. There may still be business interruption for a number of reasons, as low-damage construction does not necessarily mean no damage, or that the building is immediately able to be occupied after an event.

9 Conclusions

This report describes and quantifies the types of structural systems that have been used as part of the Christchurch reconstruction after the 2010–2011 Canterbury earthquake sequence. It also describes the factors that have driven decisions affecting the choice of structural systems in multistorey buildings in Christchurch's CBD and Addington area. The methodology and findings are both provided.

This study was structured on the basis of the findings from an initial scoping exercise conducted with Christchurch professionals to determine how to best obtain information and compile it in a way that may be quantitative and useful to the profession. Interviews were conducted with individuals/groups in their workplaces to collect and verify data on individual buildings. Data was collected from the ten firms having designed the most new buildings as part of the Christchurch reconstruction, for a total of 74 buildings.

While the main phase of the work involved conversations/interviews with key structural engineering consultants working in Christchurch, information was also collected from the perspectives of architects, project managers, and client representatives. Discussions with professionals in Wellington and Auckland were also conducted. All these groups had an opportunity to review and comment on the relevant parts of the near-final draft of the report.

Major findings from this study are as follows:

- While before the earthquakes, almost all buildings in the Christchurch CBD and Addington areas had RC frames or walls as their structural systems, in the rebuilding of Christchurch that has taken place since 2011, the number of buildings with steel, RC, and timber lateral-force-resisting systems has been in the ratio of approximately 10:10:1. However, the floor-area ratios of the same buildings with steel, RC, and timber lateral force-resisting systems is about 79:20:1, because the steel systems tend to have been used in larger structures. Furthermore, for the above RC buildings, the internal gravity frames have been found to be of structural steel three-quarters of the time.
- Concrete structures in the rebuild were nearly all structural wall systems. Exceptions encountered were (i) a base-isolated building where RC moment frames were used in one of the building's orthogonal directions, and (ii) a building where rocking RC walls were used in one direction.
- Steel buildings have been constructed using a variety of lateral-load-resisting systems. The most frequently used systems, by decreasing numbers of buildings in which they have been implemented, are: BRB frames, traditional MRFs, MRFs with reduced beam sections, EBFs with replaceable links, CBFs, traditional EBFs, rocking steel frame systems, and MRFs with friction connections. Most new base-isolated buildings are

supporting either steel MRFs or CBFs. When considering only non-base-isolated buildings, BRBs frames have been used in buildings making up nearly 40% of the total new constructed floor area.

- The most common timber frames consisted of laminated veneer lumber, used in approximately 3% of the buildings.
- Of the 74 buildings considered, 9% of buildings used hybrid systems, 14% were base isolated, and 3% used viscous dampers.
- Beyond increased demands for both serviceability and design level earthquakes implemented in design standards following the earthquakes, there has been no mandatory requirement to use more resilient structural systems as part of the Christchurch rebuild. Nonetheless, many engineers and owners were aware of the benefits of more resilient systems and this has generally been part of the discussions in deciding the structural form. Many engineers stated that they also designed their buildings to have design level earthquake drifts much less than the maximums permitted in the standards. This was considered to limit structural and non-structural damage.
- The decision about which structural system to use for each specific building depends on many factors, including the person making the decision. From the survey conducted, it was found that the engineer chose the structural system in the majority of the cases. This was followed by the owner requesting lowest cost, the owner selecting a “low-damage” solution, and the owner requesting an IL3 building. While the structural engineer has a significant say, it became clear from the interviews that the decision about the system is made as part of a group that includes the client, the architect, and other parties (i.e., project manager, quantity surveyor, etc.), and that considerations of cost, construction speed, perceptions of structural performance and building post-event operation, tenants’ desires, engineering culture, time since the last nearby earthquake, cash flow of the client, and other factors are also significant. The decision varies by location throughout the country and is also affected by the local availability of construction skills, access to resources, and the strengths of relationships. As such, while some structural forms are more common than others, there is no single dominant form throughout the country. It is also worth noting that while Christchurch’s widespread insurance coverage has permitted the rebuild, with over NZ \$40 billion being reinvested in Canterbury, the insurance industry does not seem to be having a significant influence on the types of structural systems used in the rebuild.
- The shift towards steel structures was attributed to a combination of factors. These include the NZ legislative framework, which allows new systems; the perceptions of low damage and reparability of steel

structures after the Canterbury earthquake sequence; the low price of steel compared to several years before; the fast erection speed of structural steel; the availability of economical flooring systems that performed well and are compatible with steel buildings; the advent of economical methods to design for fire; the poor soil conditions in Christchurch, giving an economic advantage to light structures; the advent of “low-damage technologies”, which are easy to connect to steel structures; the availability of some systems with known strength (e.g. BRB systems) and therefore little section overstrength allowing economical design; and perceptions about performance and procurement issues with structural systems using some other materials.

Furthermore, on the basis of the above findings and discussions with those interviewed, the following key points can be drawn:

It is becoming a more widely held belief that preventing loss of life as a seismic performance objective is simply not sufficient for a good modern structure

While all structural systems designed according to modern New Zealand standards are believed by engineers, architects, clients, and other stakeholders in the construction industry to meet their design target of preventing loss of life during an earthquake, the industry (without governmental intervention) has generally moved away from code-compliant systems that will undergo high ductility demand, develop high displacement/drift (creating significant damage to the structure and non-structural elements), exhibit damage mechanisms not considered directly in most standard frame analyses (such as beam-elongation effects that produce floor damage), and are difficult to inspect/repair/reinstate after a major event. For that reason, RC moment frames as lateral-force-resisting systems, which were ubiquitous before the earthquakes, are practically non-existent in the CBD rebuild. Note that the only RC moment frame encountered in this study of the rebuild was supported on a base-isolated structure and was expected to sustain only low ductility demands. It will be interesting to track whether this practice will last as the effects of the earthquakes progressively become less vivid in the collective memory.

Structural engineers’ professional opinions impact the adoption of low-damage systems

In New Zealand, structural systems that are specifically designed to limit seismic damage in structures and that do not need to be fully replaced immediately after a major event have been termed “low-damage technologies/structures”. Not all low-damage systems are equal in terms of construction cost, expected performance (structural and non-structural), post-event inspection requirements/costs, or post-event reinstatement requirements/costs. These performance and cost issues relate to the whole

building (including structural and non-structural effects) for continued occupancy and use. There are differences in professional opinions regarding how some of these systems will perform under 3-D earthquake shaking or whether the total costs for these systems will be as low as stated by their promoters/advocates. For this reason, a number of engineers mentioned considering only “established” low-damage solutions.

While some of the systems require high technology, other ways mentioned to control building damage simply involved using some of the traditional systems while limiting drifts and ductilities.

Tenant expectations strongly impact choice of structural systems for individual buildings

Tenants that demand low structural and non-structural damage, minimised disturbance of operations, or business continuity after an earthquake have a direct impact on the choice of structural systems for specific buildings, either by engaging in discussions for “tailor-made” buildings or by seeking building owners/developers who are willing to cater to their needs. Projects can also be developed on spec by developers guessing as to the expectations of this market segment. However, less sophisticated tenants have an equally strong impact, albeit indirectly, which is expressed through the lease-rates considered by developers when calculating return on investment for new buildings in the competitive market. In all cases, the speculative builder must assess the rates that the market can bear, have insights into tenants’ expectations for the targeted occupancy, and balance these demands with the risks implied with each investment. In most cases, even when considering resilient/low-damage construction, cost is important (cost was indicated to be the most important consideration for structural system selection by owners). This limits how much building designers can move towards improved building performance and towards the goal of fully operational structures immediately after a major earthquake, except for a few select buildings with less cost-sensitive owners.

Additional increase in seismic performance, if desired for all buildings, would need to come from government regulation

While the construction of individual buildings able to achieve high seismic performance can be driven directly or indirectly by tenants’ expectations, the seismic resilience of a community depends more on the common performance shared by most of the significant buildings in that community than on the stellar features of a few. Given that a region has a multiplicity of building owners with often diverging expectations and means, government regulations would be required to increase the resilience of a region (as well as of the individual structures in that region), and decrease the likelihood of a few major structures designed to code minimums affecting access to many parts of an otherwise “low-damage” city. While insurance may be

considered a means of providing regional resilience if the cover is sufficient and relocation of people and businesses is not considered to be problematic, this can only be relied on if it remains available, affordable, and adequate, and is purchased by the majority of stakeholders.

Context affects final decision outcome

The specific structural systems selected in the Christchurch rebuild could have been quite different from those that have been actually constructed for a wide number of reasons. For example, if there had been no significant damage or lessons learned from previous earthquakes on the seismic performance of some structural systems, construction may have continued in a similar manner to before the earthquakes. Also, if the relative costs of different materials at the time of the rebuild had significantly differed from that which existed, if some of the research had not progressed to a form easy to apply, or if marketing of specific seismic solutions had not occurred, then structural forms may have been different from what exists now.

The reconstruction experience has paralleled an increase in stakeholder knowledge

Stakeholders from all fields of the Christchurch construction industry have educated themselves to understand the key issues with the different systems. The industry is also sophisticated enough to be able to explain how it considers the large number of factors that influence the selection of a structural system. Considerations relating to not only the lateral-force-resisting system itself, but also the costs and benefits for the whole building, were clearly described by the range of people interviewed. This knowledge places the industry in a good situation to address future issues (such as revised seismic hazard maps, price fluctuations, new developments/technologies, and stakeholders' seismic performance expectations) in a clear and rational manner, as it balances performance, cost, and other issues in structural form selection decisions, in an environment sometimes requiring more than minimum governmental standards.

Finally, although nobody wishes another series of powerful earthquakes to strike Christchurch in the future, it will happen. (New Zealand, after all, is a landmass that has been created by the forces of nature). If this happens within the life-cycle of the buildings currently being constructed, it will provide a unique opportunity to compare, side-by-side, the seismic performance of a large number of different structural systems and design strategies for buildings of the same vintage, that have been designed to the same design codes and standards, using 21st-century structural engineering technology. Given the high concentration of new buildings within the few blocks of Christchurch's CBD that will have been built within the span of a decade or so, a number of them instrumented with strong motion accelerometers, the results from a life-size experiment created by such an earthquake are bound to forever

change the practice of earthquake engineering, probably in unexpected ways, even more so than the 2011 events. At the time of writing, with Christchurch only in the “set-up” stage for this experiment, while hoping this future damaging earthquake never happens, the authors can attest that all the professionals interviewed as part of this project have strived to achieve the most positive outcome when it does happen.

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Appendix A – Sources of Data

List of individuals interviewed in 2016 and 2017

Findings in this report represent the opinions expressed by a number of individuals on the factors that drove decisions in the selection of structural systems for new buildings as part of the Christchurch reconstruction. Those who have generously shared their time and professional opinions to make this project possible are listed below (and thanked again for their valuable and candid insights).

Date	Company/Organisation	Individuals
2016	Christchurch	
Feb. 16	Steel Construction New Zealand (SCNZ)	Alistair Fussell, SCNZ Manager Kevin Cowie, Senior Structural Engineer
Feb. 16	Opus International Consultants	Jan Stanway, Principal Structural Engineer Will Parker, Technical Principal – Earthquake Engineering and Building Structures
Feb. 17	Aurecon	Stephen Hogg, Technical Director, Buildings
Feb. 17	Beca	Samir Govind, Technical Director, Structural Engineering
Feb. 18	Engenium Consulting Engineers	Alan Reay, Consultant Doug Latham, Senior Structural Engineer
Feb. 18	Holmes Consulting Group	John Hare, Director Bruce Galloway, Technical Director Stu Oliver, Technical Director
Feb. 19	Calibre Consulting	Sean Gardiner, Business Unit Leader, Structures
Feb. 19	Lewis Bradford Consulting Engineers	Craig B. Lewis, Managing Director Tim Shannon, Technical Director
Feb. 22	Structural Engineering Society New Zealand (SESOC)	John Snook, Executive Officer/Treasurer
Feb 23.	Christchurch City Council	Mike Gillooly, Chief Resilience Officer
2017	Christchurch	
March 1	Lewis Bradford Consulting Engineers	Craig B. Lewis, Managing Director Tim Shannon, Technical Director
March 1	Ruamoko Solutions Consulting Structural Engineers	Julian Ramsey, Director
March 1	Engco Consulting Engineers	Mike Cuisel, Structural Engineer
March 2	Aurecon	Stephen Hogg, Technical Director, Buildings

Date	Company/Organisation	Individuals
March 3	Engenium Consulting Engineers	Alan Reay, Consultant Grant B. Coombes, Director Jeremy Mitchell, Senior Structural Engineer Chris Urmson, Senior Structural Engineer
March 3	Quoin Structural Consultants (formerly Structex)	Brett Gilmore, Director Gary Haverland, Director
March 6	Opus International Consultants	Will Parker, Technical Principal – Earthquake Engineering and Building Structures
March 6	Beca	Samir Govind, Technical Director, Structural Engineering
March 8	Holmes Consulting Group	John Hare, Director
March 10	Kirk Roberts Consulting Engineers	Jade Kirk, Managing Director Nick Calvert, Structural Manager/Senior Structural Engineer,
March 24	Warren and Mahoney Architects	Peter Marshall, Managing Director
March 24	RCP (Project Management and Project Programming services)	Matt Allen, Executive Director
March 24	Ngai Tāhu Properties (property development and investment)	Gordon Craig, Development Manager James Jackson, Project Manager
2017	Wellington	
March 15	Aurecon	John Finnegan, Technical Director, Buildings
March 15	Beca	Rob Jury, Senior Technical Director, Structural Engineering
March 15	Dunning Thornton Consultants	Alistair Cattanach, Director
March 15	Spencer Holmes	Peter Smith, Managing Director
2017	Auckland	
March 21	Holmes Consulting	Jeremy Austin, Technical Director
March 21	Beca	Richard Built, Senior Technical Director, Commercial Structures

None of the individuals interviewed have requested to remain anonymous.

Appendix B – Case Studies by Engineering Firms

As explained in Section 4 (Methodology), during initial interviews with practicing engineers to determine the best approach for this study, many engineers indicated a willingness to volunteer short case studies to illustrate the factors that drove decisions on some specific Christchurch reconstruction projects. These case studies are collected in this Appendix.

The decision to provide (or not provide) case studies, the selection of buildings to showcase as part of these examples, and the specific reasons for choosing these specific buildings, were entirely left at the discretion of the engineering firms interviewed. The authors only requested that each case study be limited to one page, follow a specified layout, and address factors that drove the client and the structural engineer to select the structural system specific to each building.

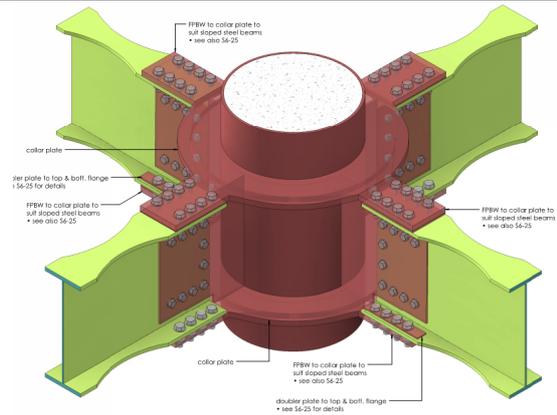
The case studies are presented here in the order they were received from the engineering firms. They provide a significant and representative portion of the structural systems that have been used as part of the extensive on-going Christchurch reconstruction. These case studies are valuable in illustrating the diversity of professional opinions and approaches taken by the engineering community in answer to project-specific demands, as well as some of the recurring themes that have led to the final structural system selections.

Case Study Example – Ballantynes Redevelopment

Address/Buildings Name: 43-47 Lichfield Street, Christchurch

Client: J Ballantyne & Co. Ltd

Year of Consent (Construction Permit): 2017



Description of Structural System:

Lateral load resisting system consists of two way steel moment resisting frames with boundary concrete shear walls in one direction. The frames comprise of concrete filled steel CHS columns, and welded beams with reduced flange widths at hinge zones. The gravity load resisting structure consists of comflor steel deck flooring, steel secondary beams, a suspended concrete flat slab ground floor structure with RC beams and RC basement structure on a shallow raft above intermediate piles.

Owner/Developer/Tenant's Requests or Specifications:

The owner did not specifically request a 'low damage' structure or dictate the structural form. Their building brief did drive the structural form however, with many constraints; adjoining existing floors of Ballantynes Building, car park access from adjacent separate building, basement with column setout to suit loading dock, retail floors to remain open and flexible with no diagonal braces or walls, and upper levels to suit car parking. As the building replaces a building that was demolished following the Christchurch earthquakes, resiliency was required, the tight programme requires fast construction time, and cost-effectiveness was also strongly required.

What Drove Engineering's Decisions:

The reasons described above, particularly the constraints to develop a structure that suits basement, retail and car park layouts drive the bay spacings. The lack of any desired braces or walls then meant a moment resisting frame was required, with structural steel the only option considered. Base isolation, given the basement was not considered an option given the size and space required for boundary offsets to adjoining buildings and to the internal seismic joint between the new structure (building straddles a boundary, and was therefore split into two seismically separate structures). Steel deck flooring was chosen for its relative lightness and speed of construction. Resiliency is incorporated though relatively low ductility demands, and design to a higher importance level.

Other Comment:

The boundary conditions and other constraints have largely governed the structural design of the building. Major efficiencies could have been achieved without the internal seismic joint, and by incorporating BRB or EBF bracing.



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Case Study Example – Hereford Street Car Park

Address/Buildings Name: 152-158 Hereford Street, Christchurch

Client: Calder Stewart Construction

Year of Consent (Construction Permit): 2015/16



Description of Structural System:

The lateral load resisting system for the eight-storey structure consists of Buckling Restrained Braces (BRB's), located with two opposing braces on each side of building on each level (64 braces in total). The gravity load resisting structure consists of unpropped steel deck flooring, unpropped secondary steel beams and propped primary beams. The foundations are shallow RC foundations above a hardfill raft above piles, with the raft and piles designed to suit multiple existing conditions on site (remnant piles and basement from previous demolished buildings).

Owner/Developer/Tenant's Requests or Specifications:

The client's brief was largely focused on cost and programme efficiency. No 'low damage' solution was requested, and no special requests were made for resiliency. Design brief required as open and column-free ground floor retail space as possible whilst also achieving a user-friendly car park layout above.

What Drove Engineering's Decisions:

We were engaged by the contractor as part of a design-build contract with the owner, and many decisions were contractor led. The foundations were designed to maximize the existing piles and basement structures on site, removing the need to expensive removal, and a detailed study was undertaken to achieve uniform performance across multiple different conditions. For the superstructure, 10 floor options were developed ranging, with the final solution a balance between efficient car park layouts, minimal columns and internal braces at ground floor, cost and constructability. Long span heavy concrete floors were ruled out, and multiple steel options considered. The cheapest option was not selected based on the implications on retail and car park layout. Steelwork construction was the obvious solution, with the bay geometry driving BRB braces rather than EBF's. Precast cladding panels were detailed to ensure they provide no stiffness.

Other Comment:

Other than complex foundations, the building is relatively straightforward construction with steel beams and columns, and a regular layout of braces around the building perimeter.



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Case Study Example – Three35 Lincoln Road

Address/Buildings Name: 335 Lincoln Road, Christchurch

Client: Cadaques Investments Ltd

Year of Consent (Construction Permit): 2012



Description of Structural System:

Lateral load resisting system consists of steel Eccentric Braced Frames (EBF's) utilizing bolted shear links. Gravity load resisting system consists of steel beams supporting steel purlins at the roof, steel composite beams supporting suspended rib and infill flooring at suspended floors, and a suspended concrete ground floor slab supported on concrete pilecaps, supported by steel screw-piles. First known use of replaceable-link EBF's in NZ.

Owner/Developer/Tenant's Requests or Specifications:

Owner specifically requested an innovative structural solution in light of recent earthquakes. They wanted a steel building that would be perceived by tenants as an appealing system in light of the post-earthquake environment at the time. The client specifically requested:

- A low-damage structural system, easy to repair after an earthquake, and provide minimal or no post-earthquake disruption of business operations
- A cost-effective structural system
- A building that would have a rapid construction time to cater for high demand for office space at that time

What Drove Engineering's Decisions:

A steel framed building with replaceable-link EBF's supported on steel screw-piles was chosen due to the following:

- Overall perception that steel was a more "resilient" option in the post-quake environment
- Speed of construction of steel at that time faster than concrete
- Proven track record of EBFs in recent seismic events
- Insurance and bank approval of easy to repair structural seismic system, and reduced downtime following earthquake and better assurance of repairs
- Ease of erectibility
- Lower mass than concrete system
- Deep screw-piles to match premise of easy to repair structural seismic system

Other Comment:

The use of this replaceable link EBF system was award winning and a first in NZ. This system has gone on to become commonplace for commercial buildings in post-earthquake Christchurch, and industry guidelines have been published based partially on the experiences with this project.



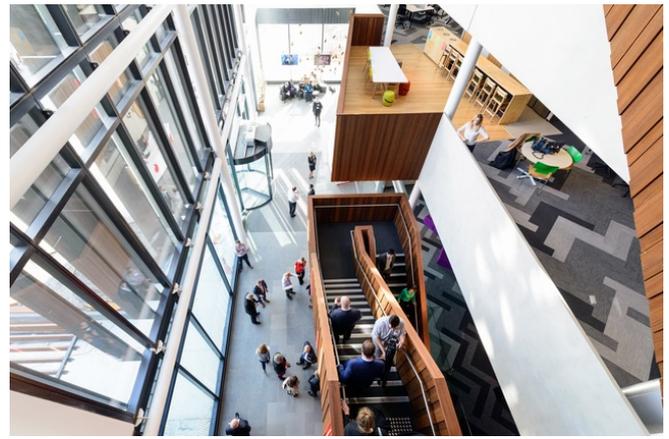
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Vodafone Head Office

Address/Buildings Name: 213-221, Tuam Street, Christchurch

Client: Calder Stewart Construction Limited

Year of Consent (Construction Permit): 2015



Description of Structural System:

The gravity system is comprised of steel purlins on portal frames at roof level, precast concrete panels, metal deck tray flooring system (rib and infill at level 1), steel supporting beams and concrete filled CHS columns. The gravity loads of the floor structure will be supported by a steel-concrete composite superstructure. The vertical CHS columns are filled with concrete to minimise the application of intumescent paint for fire requirements. The lateral system of the building is made up of concrete shear walls in the core and around the perimeter of the building. Due to the open atrium a seismic gap has been installed in the level 1 floor which required an internal EBF frame from ground to first floor.

Owner/Developer/Tenant's Requests or Specifications:

The client and the tenant had the following requirements:

- Cost-effective building –the client had a budget requirement to work to
- Open plan spaces
- Open plan atrium
- Cantilevered feature stair

What Drove Engineering's Decisions:

The building was designed as a design build project with early collaboration between tenant, developer, contractor, engineer and architect forming the principal consulting team. The main innovation came by overcoming the geotechnical issues by applying ground improvements solutions to the foundations for the building, essentially removing liquefaction risk (potential) and strengthening the geology of the site at the same time.

We designed the building layout, by positioning the core to ensure the centre of mass of the structure and centre of rigidity (the point at which the building rotates around) to align with each other, to allow us to create a perfectly symmetric building in regards to seismic loads. We then applied shear wall panels to the external façade to ensure in major events capacity of the structure will always be maintained.

Other Comment:

Kirk Roberts business philosophy revolves around designing and delivering the most cost-effective solutions to our clients through completing high levels of engineering analysis. This higher order of analysis has seen us engaged on a number of recent projects which had stalled due to budget concerns – our involvement to re-engineer and in some cases re-plan the developments have allowed the projects to move ahead and allowed the projects to be delivered on or under budget. This approach is our standard and we pride ourselves on delivering the best value for project to ensure that developments that we are involved with “stack up”.

This involvement in projects where innovative engineering design has been the make or break on a project means that our clients gain the benefit and cost savings of our past projects for their projects.

Having multiple engineering disciplines in-house means that our delivery is integrated allowing us to deliver the most cost-effective solutions available through past experience and a high quality and capability of our staff.



Young Hunter House

Address/Buildings Name: 36 Victoria Street, Christchurch

Client: Tony and Vanessa Merritt

Year of Consent (Construction Permit): 2012



Description of Structural System:

The gravity system consists of timber HyJOIST purlins to the roof spanning onto infill timber walls (at roof level) resting on the LVL post-tensioned moment frames. Precast concrete walls provide the gravity system in the orthogonal direction to the LVL portal frames. The gravity system to all floors comprises of a TCC (timber composite concrete) flooring system with LVL joists, plywood, and concrete slab spanning onto the LVL frames at each level.

Owner/Developer/Tenant's Requests or Specifications:

Following the earthquakes in Canterbury the client had a key requirement for a safe building and a building that would sustain low levels of damage in future earthquakes.

What Drove Engineering's Decisions:

Seismic post-tensioned and dissipated LVL frames resist lateral loads in the north-south direction. They are based on PRESS technology and are designed as limited ductile with the design governed by deflection. At a design level earthquake, drifts of 1.1% are expected, but due to the post-tensioned system, the building can sustain drifts greater than 1.8% (or 1.64 times greater than those expected from a code level event).

The lateral stability and capacity of the frames at the joints is provided through two mechanisms: 1) post-tensioned strands and 2) ductile steel dissipaters. The post-tensioned strands provide re-centring ability while the ductile steel dissipates the earthquake's energy. The post-tensioning strand has been stressed to less than 60% of capacity, ensuring the strand will not yield even for earthquakes well above code level events. As drift levels increase with larger seismic forces, the axial forces in the strands increase allowing the building to re-centre. The lower initial stressing force also mitigates losses due to creep. The column is isolated by steel plates reducing crushing on the column joint due to axial loading from the strand.

The dissipaters have been externally mounted to the beams, allowing them to be easily accessed, inspected, and replaced after an earthquake. The link has been designed to be easily and quickly replaced if yielding occurs.

Other Comment:

Kirk Roberts is leading the industry in the development, research and implementation of new seismic technologies such as PRESS technology, friction dampers, isolated structures, displacement based design and the like. This has allowed us to design and create low damage, performance based structures able to withstand seismic events well in excess of code. The success of these technologies is proven because we can achieve low damage, performance based buildings which are cost competitive with conventional designs.

Another recent technology that Kirk Roberts can deliver is in cross laminated timber (CLT) construction. While CLT is an emerging material we have completed design competitions which have shown that CLT is competitive and in some cases more cost-effective than other products. One advantage of CLT is the ability to fabricate off site to allow construction on site to proceed very quickly.

The benefit of the KR philosophy of providing innovation is to bring projects on or below budget, or alternatively increasing the amount of money able to be spent on the building fit-out where the budget will be most beneficial for meeting the purpose of the spaces



Case Study Example – 141 Cambridge Terrace

Address/Buildings Name: 141 Cambridge Terrace

Client: Private Owner

Year of Consent (Construction Permit): 2014



Concentric rocking braced frame



Ringfedder hold down springs combined with UFP plates

Description of Structural System:

Exterior precast concrete load bearing walls located at the site boundaries provided elastic bracing structure in the long direction. There are two rows of structural steel beams and columns supporting interior spans with 450mm deep double tee precast flooring.

The special features of the structural design consist of full height concentric bracing frames in the across direction that are designed to rock under seismic overload conditions. The frames are held down with high compression ringfedder springs combined with yielding flexural plates attached to a non-rocking gravity only column. The system re-centers the building after a large earthquake returning the building back to vertical.

Owner/Developer/Tenant's Requests or Specifications:

The building located at 141 Cambridge Terrace is a 6-storey office building designed in accordance with the New Zealand Building Code AS/NZS1170, the building is designed for 100% NBS Seismic Design Actions for an importance level 2 structure. The primary lateral load resisting system incorporates 'resilient design principles', The purpose of this type of resilient design is to prevent seismic damage to the primary structure thus allowing the building to remain operational after a major seismic event and avoid the need to demolish. It will also will facilitate rapid damage assessment of the primary structure and post disaster recovery back towards re-occupancy, reducing the operational downtime for tenants.

What Drove Engineering's Decisions:

A long narrow site and a desire to maximize site coverage whilst including a resilient structure design drove the need for incorporating the full height boundary wall and elastic bracing elements in the long direction. The architecture required full height atriums either side of the building for internal light which limited the ability to place bracing structure. 5m long concentric braced frames were located each end of the atrium and at several other internal grid locations. The resilience was provided by allowing the frame to rock under severe seismic loads and to separate the rocking structure from the gravity load bearing structure to minimize secondary damage to floor diaphragms. The building also needed to incorporate car stacker lift pits. The foundation system utilised the benefit of a naturally occurring gravel subgrade layer which was found 3.0m below ground level and corresponded with the base of the car stacker pits. The site was excavated to accommodate the car stackers and the entire shallow foundation beam system was incorporated with the re-compacted ground around the car stackers. The building was able to be designed for shallow foundations and avoid the requirement for rocking frame hold down ground anchors due to the inclusion of the car stacker pits.

Other Comment:

Challenges included keeping the integrity of the resilient design detailing consistent and true to the resilient building principals. These included the design of seismic load paths and separation of gravity structure to avoid damage to floors when rocking occurs and transfer of storey shears to elastic shear walls.

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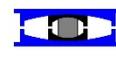
Level 2 Iwikau Building
93 Cambridge Terrace
Christchurch 8013

Case Study Example – Grand Central New Office Building

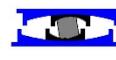
Address/Buildings Name: Grand Central, 161 Cashel Street, Christchurch

Client: Private Owner

Year of Consent (Construction Permit): 2014



Triple Pendulum Bearing
Three Seismic Isolators



First Pendulum
Optimized for
Content Protection



Second Pendulum
Optimized for Non-Structural
Component Protection



Third Pendulum
Optimized for
Structure Protection

Description of Structural System:

The lateral resisting system consists of moment resisting frames on all grids, running in the east-west direction. Concentrically braced frames provide lateral resistance in the north-south direction and are located at the perimeter, core and atrium. At ground floor level the steel structure is supported by sixty-one isolation bearings which in turn are supported by short reinforced concrete plinths located in the sub-floor space. The ground floor steel beam grillage resists the column moments and the p-delta actions

Owner/Developer/Tenant's Requests or Specifications:

The Client has advised a number of key drivers should be considered for the structural design of this project:

- ❖ Speed, build-ability and repeatability of the structural solution
- ❖ Consideration of a seismically resilient or low damage solution (base isolation).
- ❖ Consideration of rapid construction focussing on offsite prefabrication and efficient erection
- ❖ A functional, cost effective and comfortable facility
- ❖ Smart selection and use of suitable structural systems for future flexibility

The **Resilience target** for the Grand Central Building was aspirational, to design for 90% continued operation functionality while targeting less than 10% total building loss after an event.

What Drove Engineering's Decisions:

Structural steel was selected to reduce weight and increase the speed of erection. The good performance of structural steel buildings in the Canterbury earthquakes was also considered. The decision to base isolate the building was driven by the Client in order to improve occupant safety and to help future proof. The benefits of base isolation are:

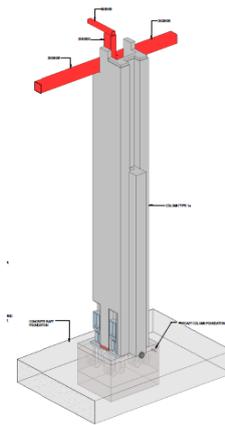
- ❖ Significantly reduced forces acting on the structure
- ❖ Significantly reduced inter-storey deflections
- ❖ Minimal structural damage expected
- ❖ Minor damage to non-structural elements expected
- ❖ Significantly reduced contents damage expected
- ❖ Business continuance as long as services to the building are still operating and access to the building is possible
- ❖ Straight forward to re-level the building in the event of any foundation settlement
- ❖ Enhanced occupant safety as a result of the above

Case Study Example – Knox Church Reconstruction

Address/Buildings Name: Knox Church Reconstruction

Client: Private Owner

Year of Consent (Construction Permit): 2014



Description of Structural System:

The rebuild of Knox Church used post tensioned rocking buttresses using plug and play replaceable energy dissipaters at the base of the column piers. The existing roof had a new ply diaphragm overlay and a structural steel ring frame was installed at the eave height to distribute seismic load to all buttresses.

Owner/Developer/Tenant's Requests or Specifications:

The Knox Church wanted to rebuild the damaged building but transform it into a state of the art resilient building that would ensure its seismic performance would preserve the building for future generations.

What Drove Engineering's Decisions:

The original building was supported by unreinforced masonry buttresses and any reconstruction work should not interfere with the original historic fabric of the building. The solution became obvious that a modern form of cantilever buttresses anchored into a new concrete raft foundation would provide the resilient building structure required.

The heritage timber roof was to remain in place on site and the structure was required to be constructed under it. The cantilever buttresses and anchor blocks were precast, assembled and prestressed of site then cast into the new raft foundation. The roof weight was then transferred to the buttresses.

Other Comment:

The heritage drivers to retain the original roof heavily influenced the rebuild solution. The solution aligned with the desire for the Knox Church to have a resilient building to preserve it future for future generations.

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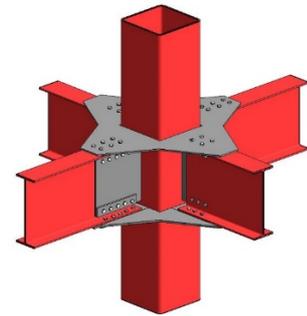
Level 2 Iwikau Building
93 Cambridge Terrace
Christchurch 8013

Case Study Example – The Terrace Hospitality Precinct

Address/Buildings Name: The Terrace Buildings

Client: Private Owner

Year of Consent (Construction Permit): 2014



Description of Structural System:

Two way moment frame using sliding hinge joints in each direction. Columns are 600mm square full height with plate thickness varying between 20mm, 16mm and 12mm. Secondary beams span to the moment frame with comflor 60 tray flooring over. All buildings are founded on reinforced concrete raft foundations.

Owner/Developer/Tenant's Requests or Specifications:

The expectations of the owner on this project was to have a 'low-damage' structural system that would protect the primary structure from irreparable damage. The building was not allowed to have diagonal braces at ground floor level.

What Drove Engineering's Decisions:

The buildings are accessible from all sides and the ground level is all hospitality and servicing use. The use of diagonal braces of any form was undesirable due to the building use and flexibility required to suit tenant requirements in this A Grade quality development. The buildings are also generally irregular vertically and in plan driving the seismic bracing solution to a two way moment frame. There was a strong desire to implement a low damage system to provided tenant confidence in the new building development which started design soon after the 2011 Canterbury earthquakes.

Other Comment:

The use of top and bottom flange plates in two way moment joints introduces significant complexity and steel weight into the connections by comparison with other types of concentric or eccentric bracing systems. The cost of plate weight was partially offset by the use of SHS steel tubes with different wall thickness manufactured and imported as a project specific order.

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Level 2 Iwikau Building
93 Cambridge Terrace
Christchurch 8013

Case Study Example (“Call Me Snake” Sculpture)

Address/Buildings Name: “Call Me Snake” is a sculpture by New Zealand artist Judy Millar.

Client: SCAPE Public Art

Year of Consent (Construction Permit): 2015



Description of Structural System:

The sculpture comprises 5 large frames that have been slotted and welded together. The steel frames are made of PFC perimeter elements and SHS main elements. The frames have internal timber stud framing and are clad in treated plywood. The artist then applied her large artwork to the plywood as stickers in a wallpaper fashion. The foundations consist of reinforced concrete ground beams.

Artist/Owner’s Requests or Specifications:

The sculpture needed to be robust enough to withstand wind loading and people climbing on it. It was to be on the site temporarily and hence needed to be able to be shifted by a crane in one piece.

The installation had a tight time frame so accurate electronic drawing files were shared with the structural steel worker for the production of the shop drawings and then in turn with the surveyor setting out the hold down bolts. The sculpture was installed in one morning much to the delight of the contractor.

What Drove Engineering’s Decisions:

A precast concrete option was considered but discounted due to the desire for the sculpture to be as light weight as possible and for it to have as small a foundation system as possible.

The precision available with structural steel construction was key to the success of the project.

Other Comment:



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Queenstown:
Third Floor, 17 Church Street

Case Study Example (King Edward Barracks Building 2)

Address/Buildings Name: King Edward Barracks Building 2

Client: Ngai Tahu

Year of Consent (Construction Permit): 2015



Description of Structural System:

Five storey structural steel frame building, with suspended precast concrete floors.

The superstructure gravity load system consists of prestressed rib and infill concrete floors supported on 310UC primary floor beams that are embedded within the depth of the precast floor system (overall floor depth of 310mm). The primary steel beams span a regular column grid of 7.8m in each direction between 310UC steel columns. The superstructure is supported on a shallow bearing foundation grillage system, overtop an existing basement structure (the imploded Central Police Station basement) that was backfilled with compacted hardfill to form a 3m engineered gravel raft over an intermediate gravel layer.

The primary lateral load resisting system consists of concentric Buckling Restrained Braces (BRB's). There are three braced bays in each direction, with each bay geometrically arranged into a diagonally opposed double bay. The brace nodes have sliding hinge joints at the leading edge of each gusset to form a pin in the collector beam at the gusset face.

Owner/Developer/Tenant's Requests or Specifications:

A building with a performance threshold and robustness ideally higher than a standard minimum code structural solution. Not necessarily a 'low-damage' structural system, but a pragmatic solution that would offer good performance (i.e. higher than 100%NBS), be repairable in the event of a large earthquake, and be cost effective to build within a set budget. Speed of construction was a significant motivation as a pre-determined tenant handover date was contractually in place at the time of design that allowed relatively modest timeframes for design and construction.

What Drove Engineering's Decisions:

The primary gravity structure (rib and infill embedded within the steel 310UC support beams) was elected to provide a clear services plenum for maximum flexibility of services and to minimise the floor to floor heights of the building to save façade, services and general vertical building cost for each plan square metre of tenancy space. Due to the minimal co-ordination required with services with this slim-floor system the structural design could proceed ahead of the services and architectural design for a staged building consent process necessary to start construction as soon as possible.

Visible steel bracing around the perimeter of the building was considered the most efficient solution for this building at an early stage due to large perforations in the stair and lift cores, and the inefficient plan locations (structurally speaking) of the lift and stair cores. BRB's and Eccentrically Braced Frames (EBF's) were considered for this purpose, and each system provided comparable performance at similar cost. The BRB system was chosen primarily for aesthetic appeal but also because it provided greater flexibility in geometric configuration, and has the added benefit of enhanced reparability after a large earthquake, albeit it at a modest cost premium to the EBF option.

Other Comment:

Full scale tests of the proprietary Buckling Restrained Braces were undertaken for this building prior to installation, and test samples were also randomly selected from the completed structure as part of the structural specification. The tests included the gussets and pins as designed and constructed. We experienced no issues with the pins and gussets, and getting braces in and out of the building. However generally we found overstrengths of the BRB's were more variable and in some cases higher than anticipated based on historic test data.



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Queenstown:
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OPUS HOUSE AMHERST PROPERTIES 2015

THE STRUCTURAL SYSTEM

Opus House is located approximately 2km south west of the centre of Christchurch at 12 Moorhouse Avenue and occupies a gross area of approximately 7,000 square meters. The new IL2 building is a five storey viscous-damped, steel moment resisting frame structure on a network of reinforced concrete ground beams on piles. The building form gives an open-plan office space, with fluid filled viscous dampers (FVDs) forming diagonal braces on the perimeter in both principal directions of the frame. Unlike conventional ductile steel design, the use of FVDs can permit significant damping of seismic loads but with little or no damage to the structure.

The gravity system comprises a steel roof structure supporting a lightweight roof on cold-formed purlins. The steel moment frames support the composite Comflor floor system 160mm thick via secondary beams on a 7.5m x 9.0m grid. Foundations governed by gravity loads comprise bored piles 24m down to the Riccarton Gravels, due to poor ground conditions. The ground floor spans onto a grid of ground beams which transfer loads to the piles.

The lateral system design has been based on performance based design principles and utilises a low-damage design philosophy throughout to achieve the stipulated seismic performance criteria. To achieve these criteria, fluid viscous dampers were used in combination with a steel moment frame in both directions to reduce the peak response of the frame and the storey shears forces that would have been required for a conventional moment frame or braced frame design. This reduction in response and significant damping of base shear demand enabled substantial cost savings in the foundation and superstructure to be realised. As a result, the foundations were governed by gravity loads.

The use of fluid viscous dampers in combination with steel moment frames provide substantial benefits in terms of building performance. Whilst this technology has seen little application in New Zealand it has wide acceptance as part of a seismic system in the US, Asia and parts of Europe. With a move to greater focus on building performance and low damage design in New Zealand, the use of FVD's offers a solution that is cost effective and merits greater consideration by designers, both in new builds and seismic retrofit projects.



REQUESTS OR SPECIFICATIONS

The tenant was Opus, and therefore the brief was to incorporate a resilient structural system in the building, while keeping to budget constraints for the building owner.

WHAT DROVE THE ENGINEERS DECISIONS?

The response to the brief considered the best structural system that could be achieved while meeting the client and tenant requirements, architectural layout and budget.

AWARDS

The building has recently won an ACENZ 2017 Award of Merit.



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TRIMBLE NAVIGATION BUILDING BIRMINGHAM DRIVE PROPERTIES 2012

THE STRUCTURAL SYSTEM

Located in Christchurch, this innovative building, with over 6,000m² of office space over two levels, utilises Laminated Veneer Lumber (LVL) Pres-Lam frames in one direction and Pres-Lam walls in the other to resist seismic loads. It also utilises timber-concrete composite floors on a 6.4m x 8.6m grid. This provides a lightweight, resilient structure which is very quick to erect, and is very cost effective. The building is split into two structures with a seismic gap in-between to achieve the desired layout.

The gravity system consists of timber trusses supporting the HyJoist purlins and lightweight roof. The trusses are supported on two storey Pres-Lam timber frames with supplementary yielding dissipators at the column bases and beam-column joints. The suspended floor is a timber-concrete composite floor using LVL spanning 6.4m. Foundations are shallow with a groundbearing slab tied into a grid of groundbeams on the frame lines.

The lateral system is Pres-Lam utilising the post tensioned frames in one direction and including yielding mild steel dissipators mounted externally. In the other direction, the steel roof bracing, and concrete diaphragm transfer loads to the nine pre-fabricated, coupled Pres-Lam shear walls via steel collector beams and a pin connection, which transfers load through a steel plate screwed to the wall. Post tensioning is provided by three 50mm diameter Macalloy bars, which are connected to CFA piles for transferring high tension loads under seismic actions. U-shaped flexural plates provide the energy dissipation for the walls.

Using a displacement based design approach, and utilising emerging technology based on research at the Universities of Canterbury and Auckland, this damage limiting Pres-Lam design minimises damage in seismic events through the dissipation of energy and controlled rocking of the structure. Effective damping in excess of 10% was achieved with these externally mounted dissipators, which can yield reliably in multiple events, but can also be easily replaced to mitigate any repair costs after a significant earthquake. The timber is high grade laminated veneer lumber (LVL) fabricated in New Zealand from sustainable, locally grown timber. In the shear walls, this LVL is cross-banded to provide additional capacity.



REQUESTS OR SPECIFICATIONS

The design of the building set out to deliver on the owner's (Birmingham Drive Properties) and the tenant's (Trimble Navigation) key drivers of sustainability, resilient seismic performance, innovation, flexibility and economy. The final form realises these lofty objectives, delivering a structural system that sets the benchmark for rebuilding Christchurch using sustainable timber products to achieve a high performing, but affordable, earthquake-resilient building stock.

WHAT DROVE THE ENGINEERS DECISIONS?

The choice of the system was agreed early in the design as the best system to meet the specific brief however, some design decisions were influenced by the ground conditions, ease of construction, and the need to optimise fabrication costs and keep within programme. Collaboration with UoC and the suppliers was a significant part of this design implementation.

AWARDS

NZ WOOD Awards 2012: Highly Commended

NZ WOOD Awards 2015: Engineering Excellence, Commercial Winner & Runner Up to the Supreme Winner

IStructE 2015: Commercial and Retail Structures, Commendation

ACENZ 2015: Award of Merit



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Christchurch Hospital—Acute Services Building

Address/Buildings Name: Christchurch Hospital—Acute Services Building (ASB), 2 Riccarton Ave, Christchurch Central, Christchurch 8011

Client: Ministry of Health

Year of Consent (Construction Permit): 2015/2016



Description of Structural System: The Acute Services Building's structure comprises a 10 storey capacity designed steel moment frame—base isolated over a thick reinforced concrete raft foundation. Gravity frames use welded columns and simple un-propped composite construction on a relatively wide grid. Cellular secondary beams are used, and slabs are composite steel decked. The isolation system uses a mix of lead rubber bearings and flat sliders. Isolators are located directly below the first suspended level, supported on circular concrete columns. The 1.4m thick structural raft sits on a natural gravel raft 5-10m thick, overlying sands and silts, at a depth of 20m.

Owner/Developer/Tenant's Requests or Specifications: At 60,000m², the Acute Services Building is one of the significant government anchor project in the Canterbury rebuild and is an Importance Level 4 facility containing essential post-disaster functions. Clinical and acute services (including an emergency department, operating theatres, intensive care and high dependency units) are located in a three level podium structure. Above this there are 6 levels of ward accommodation split across two linked towers with a rooftop helipad. The client aimed to create a functional, flexible healthcare environment for staff, patients, and visitors, in a building that would be resilient and enduring—this was a governing principle in the design process:

- a 'low-damage' structural system; a resilient building that would provide minimal disruption to service delivery following a significant disaster;
- a system focused on high-performance, efficiency and resilience in servicing and long term healthcare delivery - delivered efficiently; rather than at lowest first cost;
- a design supporting the planning requirements of modern healthcare delivery, now and in the future.

What Drove Engineering's Decisions:

- The natural gravel raft reduces the risk of liquefaction related ground damage, and made the site well suited to a shallow thick raft foundation.
- Base isolation is a fundamental part of the strategy to deliver the performance required of an Importance Level 4 facility (and the SLS2 requirements for continued functionality post-disaster)
- The building is relatively tall for an isolated structure, so it suited distributed lateral resistance in frames rather than localised bracing elements, to spread the overturning.
- A lightweight steel superstructure was favoured to limit the overturning actions and to make a low fixed base period easier to achieve for more effective isolation.
- Gravity framing has been made quite deep (rather than being optimised for minimum depth), in order to minimise floor liveliness and to ensure that large cells are provided for reticulation of services.
- A regular and relatively wide un-interrupted grid was both efficient for isolation system optimization, and suited a long-term loose fit philosophy in terms of providing least possible constraints on future re-use and redevelopment.

Other Comment:

Holmes

Holmes Consulting, 254 Montreal Street, Christchurch, NZ

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