Loadbearing Cold-Formed Steel for Mid-Rise Residential Buildings in Australia

Nicholas Paul Franklin
University of Wollongong

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Loadbearing Cold-Formed Steel for Mid-Rise Residential Buildings in Australia

Nicholas Paul Franklin

Supervisors:
Prof. Timothy McCarthy, Dr. Emma Heffernan and A/Prof. Lip Teh

This thesis is presented as part of the requirement for the conferral of the degree:
Doctor of Philosophy

The University of Wollongong
School of Civil, Mining and Environmental Engineering

August 2019
Abstract

Australia has a growing population, especially in dense city regions, which has led to an increase in the demand for mid-rise apartment buildings. At the time of writing, Australian mid-rise apartment buildings are typically constructed using reinforced concrete (RC) which often necessitates extensive time on-site. More efficient methods of construction, such as cold-formed steel (CFS), could improve productivity and enable the sector to meet the increased demand. CFS is used internationally as a loadbearing structure in mid-rise apartment buildings; however, its use for this purpose in Australia has been extremely limited.

The thesis approaches this issue through three studies. Firstly, a desktop study and a survey of international CFS industry experts are implemented to establish the state-of-the-art of mid-rise loadbearing CFS construction. Through triangulation of the various data collected, the advantages and disadvantages and recommendations for best practice are presented, providing a holistic view of loadbearing CFS construction. One of the key benefits identified was the lightweight of the loadbearing CFS system.

In order to quantify the implications of the lightweight system and investigate the structural feasibility of CFS in the Australian mid-rise residential market, a comparative case study was undertaken in two parts. A mid-rise residential archetype building was designed initially with a traditional RC structure to be typical of the sector, and subsequently with a loadbearing CFS structure. The CFS structure demonstrates a significant 60% reduction in weight as compared with the RC structure. This represents a reduction of up to 53% for the governing axial load combination and 58% for earthquake actions, based on designs for Sydney and Brisbane with two soil types.

In the second part of the comparison study, deep concrete bored pile and shallow pad footing foundations are designed for the RC and CFS structures using geotechnical data typical of the Sydney region. The resulting designs demonstrate a substantial saving in concrete and reinforcing steel volumes for the CFS structure in comparison to the RC structure. The volume of concrete and steel reinforcement for pile foundations is reduced by 42% and 45%, respectively, and the pad footings have a concrete and steel reinforcing reduction of 40% and 59%, respectively. The reduction in foundation material is significant as it can lead to savings in construction costs and a substantial reduction in earthworks such as rock drilling,
dewatering and waste remediation.

The third study addresses the need for more efficient modelling of CFS full-system behaviour. It presents an efficient modelling method that enables the practical and accurate 3D elastic analysis of a multi-storey CFS building structure to study its lateral behaviour within the serviceability limit state. Each shear or gravity wall is represented by an equivalent shear modulus four-node orthotropic shell element, determined from the experimental test of a representative wall panel, or the analysis of a finely detailed finite element model of the panel. The proposed modelling method is verified against full-scale shake table test results with respect to the natural period, the peak storey drift and the peak floor acceleration at two different construction phases. The method is applied to the archetype mid-rise apartment structure and demonstrates a significant reduction of storey drift under serviceability wind loading when compared to calculations using conventional methods.
Acknowledgments

Firstly, I would like to express my gratitude to my supervisors for their guidance and support throughout my PhD. Dr Emma Heffernan, you are an inspiration to me in many ways, thank you for always being available and giving me good advice. I admire your patience, though I’m sure you’ll be happy to hear the end of ‘Hey, Emma…’ over the desk divider, every couple of hours. Prof Tim McCarthy, thank you for your leadership and counsel. It has been all the more rewarding experience thanks to the opportunities you have provided me. Prof Lip Teh, I look up to your knowledge and dedication, you have certainly helped to improve my research skills. I’d also like to thank Dr Aziz Ahmed for the generous contribution to this project, thanks for being a great mentor.

I would like to thank my family, who have supported and believed in me throughout my many years at university. It wouldn’t have been possible without Dad’s distinguished thesis as a guiding light. I am grateful to my other family, the Backen’s, for their kindness and the enjoyable times we’ve had over the last few years. Special thanks to Irene, if it wasn’t for you, I would have surely starved by now. Chloe, thank you for the encouragement and understanding during my thesis writing. I look forward to what the future holds. Thanks to my friends who have been there along the way, especially to Remy for the trips to the crag and Tom for your words of wisdom and help with proof-reading.

Thanks to the Sustainable Buildings Research Centre for being a great environment and wonderful community of people to work with. Thanks to Dr MD Elliot, Steve and Refat for the friendship and good times (here and abroad). I’d also like to thank Robyn for keeping the SBRC in shipshape and always looking out for me.

I would like to thank the members of the Steel Research Hub, the University of Wollongong and other industry groups who assisted in my research. Thanks to Steve Avasalu for mentoring me and providing guidance for the industry-related aspects of my research. I would also like to acknowledge the financial support from the Australian Research Council Industrial Transformation Research Hubs Scheme (Project Number IH130100017) and BlueScope Ltd, without which this research would not have been possible.
Certification

I, Nicholas Paul Franklin, declare that this thesis submitted in fulfilment of the requirements for the conferral of the degree Doctor of Philosophy, from the University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. This document has not been submitted for qualifications at any other academic institution.

Nicholas Paul Franklin

9th August 2019
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<th>Definition</th>
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<tbody>
<tr>
<td>ABCB</td>
<td>Australian Building Codes Board</td>
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<tr>
<td>AISI</td>
<td>American Iron and Steel Institute</td>
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<td>AS</td>
<td>Australian Standards</td>
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<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
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<td>ASD</td>
<td>Allowable strength design</td>
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<td>ASI</td>
<td>Australian Steel Institute</td>
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<td>BCA</td>
<td>Building Code of Australia</td>
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<td>BCC</td>
<td>Building Codes Committee</td>
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<td>BIM</td>
<td>Building information modelling</td>
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<tr>
<td>CBD</td>
<td>Central business district</td>
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<tr>
<td>CFS</td>
<td>Cold-formed steel</td>
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<td>CFSEI</td>
<td>Cold-Formed Steel Engineers Institute</td>
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<td>CLT</td>
<td>Cross-laminated timber</td>
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<td>DSM</td>
<td>Direct strength method</td>
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<td>FE</td>
<td>Finite element</td>
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<td>FEA</td>
<td>Finite element analysis</td>
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<td>FEM</td>
<td>Finite element method</td>
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<td>FRL</td>
<td>Fire resistance level</td>
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<td>HD</td>
<td>Hold-down</td>
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<tr>
<td>HRS</td>
<td>Hot-rolled steel</td>
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<tr>
<td>LRFD</td>
<td>Load and resistance factor design</td>
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<td>MEP</td>
<td>Mechanical electrical plumbing</td>
</tr>
<tr>
<td>NASH</td>
<td>National Association of Steel-Framed Housing</td>
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<tr>
<td>NCC</td>
<td>National Construction Code</td>
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<tr>
<td>NEES</td>
<td>Network for Earthquake Engineering Simulation</td>
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<tr>
<td>OSB</td>
<td>Oriented strand board</td>
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<tr>
<td>OSM</td>
<td>Off-site manufacturing</td>
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<td>OT</td>
<td>Overturning</td>
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<tr>
<td>PEER</td>
<td>Pacific Earthquake Engineering Research Center</td>
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<td>PGA</td>
<td>Peak ground acceleration</td>
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<tr>
<td>PIS</td>
<td>Participant Information Sheet</td>
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<tr>
<td>PT</td>
<td>Post-tensioning</td>
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<tr>
<td>RC</td>
<td>Reinforced concrete</td>
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<tr>
<td>RL</td>
<td>Reduced level</td>
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<tr>
<td>SA</td>
<td>Standards Australia</td>
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<tr>
<td>SBRC</td>
<td>Sustainable Buildings Research Centre</td>
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<tr>
<td>SCI</td>
<td>Steel Construction Institute</td>
</tr>
<tr>
<td>SFIA</td>
<td>Steel Framing Industry Association</td>
</tr>
<tr>
<td>SHS</td>
<td>Square hollow section</td>
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<tr>
<td>SRH</td>
<td>Steel Research Hub</td>
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<tr>
<td>UOW</td>
<td>University of Wollongong</td>
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Chapter 1 - Introduction

1.1 Background

Mid-rise apartment buildings are becoming more common in the major Australian cities, with a higher growth rate than other types of dwelling (Martin et al. 2016). This can be attributed to several factors including an increasing population, decreasing land supply, better apartment affordability and a desire to live in established cities (Shoory 2016). Despite the rising market share, the productivity of this sector in Australia has remained mostly flat (Slater and Cater 2016). For this study, mid-rise is defined by a building height of between four and ten storeys. Mid-rise apartment structures are traditionally built with reinforced concrete (RC) in Australia through well-established supply chains. Generally, mid-rise apartment developments require a certain number of apartment pre-sales to secure financing, after which, construction can commence (Shoory 2016). This has resulted in a conservative industry, lacking in innovation. As innovation is a key driver of productivity, profitability and job creation (Senate Economics References Committee 2015), addressing this issue may lead to improvement of the productivity of the construction industry. This presents an opportunity for the introduction of innovative building structure solutions such as loadbearing cold-formed steel (CFS), which is the focus of the research herein.

CFS is used for many applications, including buildings, bridges, storage racks, and various types of equipment (Li 2016). In buildings, CFS is commonly used for non-loadbearing interior partition walls and exterior curtain walls (Peterman et al. 2016a). CFS members are manufactured by cutting a coil of galvanised steel into thin strips and passing these strips through a roll forming machine (Pringle 2014). The steel undergoes plastic deformation through a series of dies until the cross-section takes the general shape of a ‘C’ or ‘Z’ (Williams 2016). The CFS framing system consists of repetitive members such as studs and joists assembled into sub-systems such as walls and floors (Williams 2016).

There are a number of benefits associated with CFS that are favourable for construction, including its light weight, high strength-to-weight ratio, ductility which prevents brittle failure, consistent material quality, non-combustibility, recyclability and insusceptibility to termites and mould (Heywood et al. 2012; Williams 2016). The nature of CFS also gives it excellent properties for off-site manufacturing (OSM). OSM is the assembly of building elements such as panelised walls and floors which are manufactured and assembled at a different location to the building site. Through OSM, construction time on-site can be
significantly reduced, minimising construction costs in turn. The light weight of CFS also allows panelised components to be carried on-site by hand without the use of cranes. OSM is an innovative development in the construction industry with Australian reports highlighting the importance of its uptake for improving productivity in the sector (CRC Construction Innovation 2007).

In Australia, CFS has been used as a framing material for detached housing since the 1940s, with the first guidance on structural design issued in 1991 (NASH 2010). Australia has been a world leader in the use of high strength CFS sections over the last 30 years (Hancock and Pham 2016). High strength sections allow more efficient components and the potential for application under higher loads in mid-rise structures. Despite this, at the time of writing, the use of loadbearing CFS in Australia for mid-rise apartment construction is very limited with only one structure higher than three storeys (Dynamic Steel Frame 2015). In contrast, the use of loadbearing CFS in mid-rise residential construction has been increasing internationally, including in seismic regions in the USA (Schafer 2011). Robust structural and non-structural performance of CFS construction, as well as efficiency and economy, make it an excellent system for mid-rise construction (Torabian et al. 2016).

Due to the slender sections used in construction, the member behaviour is complex, with a number of buckling modes to be accounted for (Li 2016). Research into CFS has increased significantly in recent years, resulting in developments such as the direct strength method (DSM) for design, which has allowed for more efficient and higher capacity members (Camotim et al. 2016). The full-system behaviour of CFS structures is also complex and not well understood. Research is active in further developing the system-level understanding for mid-rise CFS framed construction, understanding roof and floor diaphragm behaviour and modelling of full building seismic response (Schafer et al. 2016). However, there is still a significant knowledge gap between system performance and current understanding, which is predominantly at the component level (Leng et al. 2017).

### 1.2 Research scope and significance

The literature review has identified the need for improved productivity and highlighted a lack of innovation in the current mid-rise apartment construction sector. The loadbearing CFS system has several associated advantages that could assist in productivity growth in the sector. Research is required to explore the potential of loadbearing CFS for the Australian mid-rise apartment market and is therefore the focus of this
thesis, from a structural perspective. Buyer perceptions are beyond the scope of the current thesis.

The literature review has also identified a gap in knowledge regarding the full-system performance of CFS structures, which acts as one barrier to the design. A better understanding of the full-system behaviour and simplification of the analysis and design methods could improve accessibility to practicing engineers and facilitate the adoptability of loadbearing CFS in mid-rise construction. In light of this, the development of an efficient modelling method to aid in CFS lateral serviceability designs is also addressed in this thesis.

The significance of this thesis is the use of a comparative study and its practical nature, which provides quantifiable evidence of the potential benefits of loadbearing CFS for mid-rise construction in an Australian context. The thesis seeks to have an impact on industry by developing the basis for a design tool that can be used by practicing engineers for CFS design.

This research is a part of the University of Wollongong-led Research Hub for Australian Steel Manufacturing, the centre point for collaborative steel research in Australia. The overarching Steel Research Hub Program B is based on market-focused steel product innovation, which is a driver for this study.

1.3 Aims and objectives

This research aims to explore the feasibility of mid-rise loadbearing CFS construction in the Australian market from a structural perspective, with a view to facilitating its further adoption within the Australian construction industry. The three focal aims are in bold with their objectives below:

Investigate the application of loadbearing CFS mid-rise construction in international markets.

- Explore the current state-of-the-art in mid-rise loadbearing CFS construction.
- Identify the advantages and disadvantages of loadbearing CFS.

Assess the structural feasibility of CFS in the Australian mid-rise residential market.

- Compare the structural systems of RC and CFS for a typical Australian mid-rise apartment.
- Determine the effect of a CFS structure on foundation design.

Facilitate the structural design process for mid-rise CFS.
• Develop an efficient method to simulate full-system serviceability behaviour of a CFS shear wall building.
• Demonstrate the application of the developed method to a mid-rise apartment structure.

1.4 Arrangement of thesis

The thesis consists of three core studies designed to meet the research aims. The initial study is broad and exploratory to obtain an overview of CFS construction. This is followed by a more detailed comparative study. Lastly, full-system CFS building modelling is addressed. The seven chapters are structured as follows:

Chapter 2 outlines the overarching methodology of the thesis, describing the research processes and linking them to the research aims for each study. It justifies the research paradigm and design and outlines the operationalisation of the thesis. Ethical considerations, research quality and dissemination are also presented.

Chapter 3 presents the exploratory study into loadbearing CFS. It provides a review of the literature regarding the use of loadbearing CFS. The desktop study and survey of international CFS construction professionals provide valuable practical insight into the use of loadbearing CFS in mid-rise construction. The results are triangulated to present the advantages, disadvantages and best practice for loadbearing CFS construction.

Chapter 4 comprises the first part of the comparative archetypal case study. An archetype structure is designed with a RC structure, typical of the Australian mid-rise apartment sector, and subsequently with a CFS structure for comparison. The design actions for both structures are determined and compared in order to assess the structural feasibility of mid-rise loadbearing CFS in Australia.

Chapter 5 presents the second part of the comparative archetypal case study, which extends on the findings from Chapter 4. In this chapter, shallow and deep foundations are designed for the archetype building with a RC and CFS structure. The volumes of concrete and reinforcing steel are compared to quantify the benefits of utilising a loadbearing CFS structure.

Chapter 6 presents finite element modelling of a CFS structure using equivalent shear modulus shell elements. The method aims to provide a lateral serviceability design check for practicing engineers. The
method is described and validated with two storey shake table tests, then demonstrated through application to a simplified mid-rise archetype building.

Chapter 7 provides collective conclusions for the preceding chapters. The research aims and objectives are discussed, and the main findings presented. Recommendations for future research are addressed, followed lastly by the contributions of the thesis and concluding remarks.

1.5 Publications and dissemination

Journal Articles

Peer-reviewed conference papers

Presentations
Cold-Formed Steel Case Study Review. Invited presentation by Nicholas Franklin to 2nd Annual Steel Research Hub Symposium. 16 November 2016, at the University of Wollongong, Australia.
Sub-Structure Structural Design. Invited presentation by Nicholas Franklin to 3rd Annual Steel Research Hub Symposium. 29 November 2017, at the University of Wollongong, Australia.
Equivalent Shear Modulus Shell Element Analysis of CFS buildings. Invited presentation by Nicholas Franklin to 4th Annual Steel Research Hub Symposium. 31 February 2019, at the University of Wollongong, Australia.
Efficient 3D Structural Modelling of CFS Building Frames for Serviceability Checks. Invited presentation by Nicholas Franklin to National Association of Steel-Framed Housing (NASH) 2019 National Engineering Workshop. 15 July 2019, at Western Sydney University, Australia.
Chapter 2 - Overarching methodology

2.1 Introduction

This chapter describes the overarching methodology for the thesis, providing a basis for the following chapters. The philosophical underpinnings are described, and the rationale and operationalisation of the research are justified. This thesis consists of three distinct studies designed to meet the overall aim. Table 2-1 details the relationship between the research aims, objectives, research strategy, methods and contribution for each study. A graphical overview is depicted in Figure 2-1 and detailed in Section 2.2.2.

The first study, presented in Chapter 3, was an exploratory investigation into loadbearing CFS construction in Australia and internationally, serving to form a knowledge base and guide the subsequent research. A desktop review of the grey literature relating to CFS case studies was followed by the development of multiple case studies on a smaller sample of buildings, with the intention to triangulate the findings. The second study, presented in Chapters 4 and 5, comprised a comparative case study with the aim to assess the structural feasibility of loadbearing CFS in the Australian mid-rise residential market. A hypothetical archetype mid-rise apartment building was designed firstly with a traditional RC structure and subsequently with a loadbearing CFS structure. The resulting design loads and foundation designs were compared. The final study, presented in Chapter 6, aimed to simplify the structural design process for loadbearing CFS structures in mid-rise buildings in a manner that is useful for practicing structural engineers. A novel method for lateral serviceability behaviour prediction was developed and verified against experimental results. Findings from all the studies are synthesised in the concluding chapter, Chapter 7.
Table 2-1: Research overview

<table>
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<th>Study Description</th>
<th>Aims</th>
<th>Objectives</th>
<th>Research Strategy</th>
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<td>CFS exploratory study</td>
<td>Investigate the application of loadbearing CFS mid-rise construction in international markets</td>
<td>Explore the current state-of-the-art in mid-rise loadbearing CFS construction; Identify the advantages and disadvantages of loadbearing CFS</td>
<td>Exploratory study</td>
<td>Grey literature review; Multiple case study</td>
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<td>Comparative archetypal case study</td>
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<td>Design action comparison; Archetype case study</td>
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<td>5</td>
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<td>Facilitate the structural design process for mid-rise CFS</td>
<td>Determine the effect of a CFS structure on foundation design; Develop an efficient method to simulate full-system serviceability behaviour of a CFS shear wall building</td>
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2.2 Rationale

This thesis uses a multi-strategy design with both natural and social science research methods, pure and applied in classification, as is common for the construction context (Fellows and Liu 2008). It is important to understand the rationale behind the methods used, initially with defining the paradigm through which the research was carried out.

2.2.1 Research paradigm

Fellows and Liu (2008) define a paradigm as a theoretical framework with a system by which people view events. Different research design and methods are used depending on which paradigm the researcher subscribes to. It is generally considered that two paradigms, positivism and interpretivism, lay on either side of a paradigm continuum (Holt and Goulding 2014). On the one end, the more traditional positivism assumes that a single reality exists (Tracy 2013) whereas interpretivism postulates that reality is constructed by the mind (Walliman 2014). Generally, physical and natural science research is undertaken with a positivistic approach. Within the field of construction management research, there has been a paradigm shift to a more qualitative, interpretivist approach (Fellows and Liu 2008). The research in this thesis is not management based, but it is within the field of construction. It uses both interpretivist and positivistic constructs, ranging from the heuristic knowledge of construction professionals to numerical simulation. Hence, the paradigm cannot be considered as wholly positivistic. A fitting paradigm for this research is one of post-positivism. Post-positivism is a branch of positivism that considers humans’ understanding of reality is inherently partial (Tracy, 2013: p. 39). Post-positivism developed as a philosophical view in the 19th century through challenging the positivist view of the absolute truth of knowledge (Creswell 2014). It is an appropriate paradigm for this thesis as it adapts a natural science approach to social science and has a reliance on quantitative methods (Robson 2011).

2.2.2 Research design

The research design refers to the style of research methods used, generally considered to be qualitative, quantitative and mixed methods (Bryman 2006). The design should maximise the chance of realising the research objectives; an overview of the design is depicted in Figure 2-1.
The research undertaken in this thesis is primarily quantitative in nature. However, construction is an applied field of research that can be viewed with interacting elements of both subjectivity and objectivity (Bourdieu 1990). People play key roles in most aspects of construction, meaning that social science research methods are applicable (Abowitz and Toole 2010). As this research is within the construction industry and built environment, a mixed-strategy design approach was deemed suitable (Robson 2011). In qualitative research, the object is to gain understanding and collect information and data such that theories will emerge. Thus, qualitative research is a precursor to quantitative research (Fellows and Liu, 2008: p. 8).

The research in this thesis was carried out sequentially. The initial CFS exploratory study has a flexible design, anticipating the research would develop over the data collection process (Robson 2011). The following comparative archetypal case study and efficient 3D lateral analysis study use fixed, theory-driven design methods. The rationale for implementing mixed methods in this thesis was for use in the triangulation of data and to ‘confirm and discover’ (Bryman 2006). Triangulation of the initial literature review with the findings of the desktop study and survey was used to meet the aim and objectives of the
first study by exploring the state-of-the-art and identifying the advantages and disadvantages of loadbearing CFS construction. ‘Confirm and discover’ refers to the use of qualitative data to generate hypotheses and quantitative research to test them (Bryman 2006). The findings from the exploratory study were used to generate a hypothesis that a mid-rise apartment building designed with a CFS structure would lead to a reduction in foundation materials as compared with traditional RC construction. Chapters 4 and 5 implement a quantitative comparative case study approach study to test this hypothesis.

2.3 Operationalisation

It is important that the methods selected to collect data to address the research aims are effective for their purpose. This section details the methods employed for each chapter and justifies their use.

2.3.1 Cold-formed steel exploratory study

The first study is considered to be exploratory, with the aim to investigate the application of loadbearing CFS mid-rise construction in international markets. This study served to guide the research direction of the thesis. It consists of two parts; a desktop study and a case study survey, the findings of which were triangulated and used for the following chapters.

2.3.1.1 Desktop Study

A literature review is essential in the first stages of a research project to gain an understanding of the current state of knowledge (Walliman 2014). A critique of the literature also leads to the articulation of research gaps. For this study, a number of case studies on buildings with loadbearing CFS structures were reviewed, providing an overview of the state-of-the-art in CFS construction. The focus was on mid-rise residential, student accommodation or hotel building typologies with a predominantly loadbearing CFS structure. Due to the lack of peer-reviewed research papers on CFS building case studies, it was appropriate to review grey literature. Grey literature can be defined in many ways; for the purposes of this research, it is considered to be publicly available unpublished research (Benzies et al. 2006). The advantage of using grey literature is that there are many more sources of data, relative to the limited volume in research-based literature. The reliability of web sources and risk of bias must be considered when using grey literature so, in order to minimise the effect of these issues, all information was sourced from reputable companies, magazines and steel institutes. Another challenge of using grey literature was that the inconsistency of information provided an incomplete set of data. Cases without enough meaningful information were discarded, and the case studies in the subsequent phase of the exploratory study were then developed to create a more complete
set of data on specific CFS buildings. As the market is relatively small, 23 case buildings were deemed to be a sufficient representation. Both qualitative and quantitative data were collected and used to design the questionnaire for the following case studies.

2.3.1.2 Case study survey
The second part of the exploratory study was the development of case studies on specific CFS mid-rise buildings. Case studies are a useful method to obtain more detailed information and a greater understanding of the drivers for loadbearing CFS use. They also have the benefit of providing a source of insight and ideas from construction industry professionals, making them not only exploratory but explanatory too (Yin 1994). A set of individual case studies of mid-rise apartment buildings with a loadbearing CFS structure were selected based on a set of selection criteria (detailed in Chapter 3). Due to the small size of the current CFS market, case studies were selected to be representative of the international CFS market, such that theoretical rather than statistical generalisations could be made (Fellows and Liu 2008). Case studies are focused on a phenomenon in context (Robson 2011); for this study, the opinions and views of construction professionals were taken in the context of the particular CFS building they were involved with.

Participants were asked to provide quantitative data on the building project, and qualitative opinions on the project, as well as project drawings and schedules. Collecting both quantitative and qualitative information provided a rich and holistic data set (Tracy 2013). The intent of the case studies was to be descriptive, which can be used to establish the state-of-the-art or norm in the area of research (Walliman 2014). When designing case studies, the availability of documentary information must be considered (Knight and Ruddock 2008). At the time of writing, due to the lack of mid-rise CFS buildings constructed in Australia, international cases were required. An internet questionnaire was the most appropriate method to gather data due to its simplicity of distribution and accessibility for international participants (Robson 2011). Surveys have commonly been employed in construction research in recent years (Fellows and Liu 2008). The survey in this study is titled: A survey of cold-formed steel (CFS) companies relating to specific mid-rise residential building projects. Interviews would often be used for this purpose in case study research (Knight and Ruddock 2008); however, this was considered to be unnecessary, because the majority of information is quantitative, and the burden of time on participants is lessened with an internet survey. Purposive sampling was used, meaning respondents were selected to ensure the target building type was explored. Data were collected using an online survey tool (Survey Monkey) for the period of September 2016 to December
2016; more details on the questionnaire are discussed in Chapter 3. The case studies were used to design the comparative archetypal case study in Chapter 4, and to corroborate the findings.

### 2.3.2 Comparative archetypal case study

The comparative archetypal case study presented in Chapters 4 and 5 aims to assess the structural feasibility of CFS in the Australian mid-rise residential market. Traditionally, mid-rise apartments have been built with a RC structure in Australia. It was therefore of interest to compare an RC structure type with a CFS structure. A comparative case study methodology was selected to compare the parallel situations (Walliman 2014). Case studies are a useful tool for research in the built environment (Groat and Wang 2013). A mid-rise apartment building, typical of the sector, was designed for the comparison by a professional architectural firm (Heffernan et al. 2017). A hypothetical building was appropriate as, at the time of writing, there are no CFS buildings over four storeys in Australia. This style of case study design has the benefit of variable control, much like an experimental study which attempts to isolate and control every relevant condition which determines the events investigated and then observes the effects when the conditions are manipulated (Walliman, 2014: p. 11). In this case, the independent variables are: i) the structure type for the building; and ii) the building location; and the dependent variables are: i) the resultant design actions (Chapter 4), and ii) the foundation design (Chapter 5). The archetype building was initially designed with a RC frame, fully compliant with regulatory requirements. Subsequently, the building was designed with a CFS frame, resulting in some changes in the floor plan layout to ensure compliance. The design and analysis of the archetype building is presented in detail in Chapter 4.

### 2.3.3 Efficient 3D lateral analysis study

The final part of the thesis was the development and validation of a model for CFS lateral serviceability behaviour. The aim was to develop a simplified method to accurately predict the lateral displacements of a loadbearing CFS structure under serviceability loads. This was operationalised with a computer simulation study, using numerical analysis; specifically the finite element method (FEM). This method is entirely quantitative, following the classic scientific method as depicted in Figure 2-2.
Simulation research design involves a simplified model representation of a system in which parameters can be modified to gauge effects (Wallimann 2014) and investigate the dynamics of a system (Hartmann 2005). In this case, a numerical model was used to represent a CFS building. The benefit of using a validated numerical simulation is that it allows the behaviour of a structure to be explored without the need to build and test the structure experimentally (Hartmann 2005). It is useful for complex systems such as buildings which are composed of many sub-systems such as walls and floors. The overarching purpose of this simulation was to represent the CFS system in a pragmatic and straightforward manner. CFS shear and gravity walls are represented by shell elements assigned an equivalent shear modulus derived from experimental results. This method is simple and can be easily understood for the purpose of being implemented in industry. ETABS 15.2.2 was used to build and analyse the model. The FEM is a general discretisation procedure of continuum problems posed by mathematically defined statements (Zienkiewicz et al. 2005). It allows complex structural systems to be discretised and analysed rapidly with the use of computers. For structural engineering analysis, it operates through solving equilibrium force-displacement equations for each node in the structure. The relationship between the force and displacement is related to the stiffness and a nodal force required to balance the loads acting on an element (Zienkiewicz et al. 2005).

The modelling method was verified against published experimental data, and its application was demonstrated with the determination of storey drift for the archetype mid-rise building; further details are

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Figure 2-2: Modelling methodology process diagram (developed after Mihram (1972))
provided in Chapter 6.

2.4 Ethical considerations

The survey; *A survey of cold-formed steel (CFS) companies relating to specific mid-rise residential building projects*, presented in Chapter 3, involved participants from the construction industry. A number of ethical considerations are self-evident when carrying out research involving people (Robson 2011). Ethical approval for the questionnaire was obtained from the University of Wollongong (Human Research Ethics Committee, attached in Appendix C). Ethics most directly relates to the collection, storage, use and disposal of participant data (Fellows and Liu 2008). These issues were considered when designing the research methods so that they were addressed.

Participants were not subject to any deception or harm in the study and were ensured the right to withdraw. The participants were employees of a specific company, selected to meet criteria based on the research aims. Participants were not considered to be part of a vulnerable group. They were identified through Steel Research Hub (SRH) (Heffernan et al. 2017) contacts or publicly available information on the internet. Recruitment was via a professional email account, and participants were provided with a participant information sheet (PIS) (see Appendix A) before agreeing to take part in the study. Informed consent was obtained tacitly in the questionnaire; this approach was deemed suitable as the research is limited to non-sensitive information.

The participants were made aware that information provided would be shared with the SRH, but were given the option of anonymity in reporting of research data. Participants were advised and assured that they may choose to withdraw at any point before or during the completion of the survey. Or, within a month of completion, the participant was able to withdraw any data they had provided to that point, with no adverse effects.

Data collected in the questionnaire were securely stored on the SBRC computer system according to UOW archiving policies. After 5 years, the raw data is to be destroyed. The research was carried out under the UOW research ethics policy and the SRH guidelines.

2.5 Quality of research

The quality of the research can be judged by the concepts of reliability and both internal and external
validity. These concepts can differ between natural and social science research methods; for this thesis, they are explained as follows. Reliability is related to the consistency of measurements; a reliable test would produce the same results if carried out multiple times (Robson 2011). Validity can be judged by how well the research reflects the real world (Walliman 2014). Internal validity is how accurate or true the research is (Robson 2011), and external validity is a measure of how generalisable the results are to other cases (Walliman 2014). Table 2-2 provides an overview of how these issues of research quality were dealt with for each study in this thesis.

Table 2-2: Research quality considerations

<table>
<thead>
<tr>
<th>Study</th>
<th>Reliability</th>
<th>Internal Validity</th>
<th>External Validity</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFS exploratory study</td>
<td>Thoroughly documented methods (audit trail)</td>
<td>Pilot test questionnaire</td>
<td>Triangulation between case study and grey literature findings (theoretical generalisation)</td>
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<td></td>
<td>Professional participants</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Standardised questionnaire</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Comparative archetypal case study</td>
<td>Used well-established software</td>
<td>Designed to codes and regulations</td>
<td>Archetype building was designed to be typical of the sector by an experienced professional architect</td>
</tr>
<tr>
<td></td>
<td>Used methods from current practice</td>
<td>Design checked with professional engineers</td>
<td>Employ methods from current practice</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Analysis and design methods from reliable sources</td>
<td>Methods clearly defined</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Controlled and known conditions</td>
<td></td>
</tr>
<tr>
<td>Efficient 3D lateral analysis study</td>
<td>Used well-established software</td>
<td>Results compared with published models</td>
<td>Based on real experiments</td>
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<tr>
<td></td>
<td></td>
<td>Controlled conditions for variables</td>
<td>Used a common modelling tool</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Methods clearly defined</td>
</tr>
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</table>

For the initial exploratory study within social science, reliability was obtained through gathering data from reliable sources and selecting professional participants. The process was also well documented so that it is repeatable. The internal validity was obtained through a pilot of the questionnaire and discussion with colleagues to ensure the expected data were being collected. Due to the small sample size availability, statistical generalisation was not possible; however, external validity can be enhanced with methodology triangulation (Jick 1979). Triangulation was implemented for the grey literature case study review and developed survey case studies, allowing some level of theoretical generalisation, considering the small population of possible cases. As for the comparative archetypal case study, the research quality of both the
case study building design and the methods used in determining the design actions and foundation designs were considered. Reliability and internal validity can be achieved by using and designing to codes and regulations, documenting methods, reviewing designs by professionals and maintaining constant and known conditions between cases (Fellows and Liu 2008). This was ensured for the archetype case study building as the architectural designs were carried out by professional architects associated with the Steel Research Hub and the RC structural designs were advised through consultation with Australian structural engineers. The CFS structural design of the archetype building was undertaken by professional engineers with experience in CFS design in the USA (through association with the Steel Research Hub). The foundation design methodology was advised through consultation with Australian geotechnical and structural engineers and the researchers from the University of Wollongong.

The external validity can be considered as how typical the building design was for the Australian market and therefore, generalisable. This was ensured as the architect has experience and understanding of designing mid-rise residential buildings in Australia. For the final efficient 3D lateral analysis study, the experimental data used in the method were collected from published peer-reviewed journal articles, the computer program (ETABS 15.2.2, ETABS 18.0.1) and method are well established within the field, and results can be easily reproduced with the given data.

2.6 Dissemination

The data collected were disseminated through reports, presentations, academic conferences and journal articles as provided in Chapter 1. Published journal and conference papers are attached in Appendix A and Appendix B, respectively.

2.7 Summary

This chapter has provided the overarching methodology for the thesis. The research paradigm and research design were established, explaining how the three studies are linked. The research operationalisation was then outlined, justifying the selected methods for each chapter. Next, ethical considerations and research quality were addressed, followed lastly by dissemination. Chapters 3, 4, 5 and 6 describe each of the studies in this thesis in detail.
Chapter 3 - Cold-formed steel exploratory study

3.1 Introduction

This chapter details the first study of the thesis, an exploration into the use of loadbearing CFS for mid-rise residential buildings. A number of benefits to CFS construction have been identified in the literature such as high strength-to-weight ratio, light weight, portability, off-site manufacturability (OSM), recyclability, and termite and mould resistance (Heywood et al. 2012; Williams 2016). To advance the use of CFS and take advantage of these benefits in Australia, it is essential to learn from the construction industries in other countries implementing this type of construction method.

Initially, mid-rise construction in Australia is introduced, followed by an overview of CFS construction through a review of the literature. The methodology then outlines the desktop study review and case study survey research methods. The findings of the chapter are triangulated and categorised into themes. Finally, a summary of the advantages and disadvantages as well as industry best practice for mid-rise loadbearing CFS construction are presented, providing a holistic view of loadbearing CFS construction.

3.2 Mid-rise construction in Australia

Mid-rise residential construction is becoming increasingly prevalent in Australia, with the number of apartments constructed each year tripling since 2009 (Rosewall and Shoory 2017). This construction is beneficial as it contributes to both economic growth and employment; with the construction sector employing 9% of the Australian labour force, as of 2018 (Ai Group 2018). There are currently more detached dwellings in Australia than apartments, but the rate of growth for apartments of four stories or more is much higher than other dwelling types (Martin et al. 2016) with one-third of all building approvals in 2016 being apartments (Rosewall and Shoory 2017).

Detached housing is being replaced with multi-storey construction for a number of reasons including land supply, affordability and a desire to live in established cities (Shoory 2016). The reduced land requirement contributes to the more affordable price of apartments; 30% less than a house (Rosewall and Shoory 2017). The reduced travel time in major cities is another factor that contributes to the affordability of apartments. Much of the apartment development activity has been within the larger capital cities with over 75% of the apartment developments constructed since 2011 (Shoory 2016) and 90% of approvals in 2016 (Rosewall...
and Shoory 2017), located in Sydney, Melbourne and Brisbane.

Despite the increasing market share, the productivity of this sector in Australia has remained virtually flat (Slater and Cater 2016). Apartment building activity and new orders have also reportedly been steady or declining since 2017 (Ai Group 2018). It is therefore of interest to identify the sources of productivity loss for apartment construction. Böhme (2018) identified that construction of the substructure and superstructure are the two phases contributing most to the loss in productivity. Generally, the time between approval and completion of an apartment takes three times as long as a detached house (Rosewall and Shoory 2017). Slater and Cater (2016) also identified wet weather days to be a cause of productivity loss, and a longer time on-site results in more potential wet weather delays. To understand the factors affecting the construction time, the structure type is considered.

Building frames are commonly constructed with RC in Australia (Böhme et al. 2018; Mills 2010). Mid-rise residential buildings, in particular, are generally cast-in-situ and implement a flat plate system with a RC shear core (Menegon et al. 2017). As such, RC construction has well-established supply chains with materials and methods known by practising builders and tradespeople. Best practices for architectural and structural design are also established, leading to efficient design solutions. The structure can accommodate large spans so is flexible and adaptable to varying floor plans with non-structural partition walls easily moved around. Concrete is durable and dense, providing favourable acoustic and thermal properties. However, due to this density of structure, there is a large resultant weight on the foundations, as quantified in Chapter 5. This construction method often necessitates extensive time on-site for erecting formwork, reinforcement, and pouring and curing of concrete. Mills (2010) states that the greatest impacts on productivity for RC systems relate to formwork, reinforcement and labour management.

The increasing demand for mid-rise apartments draws the research attention to this building type. It is expected that the productivity of the sector could be increased by exploring other innovative construction methods. Innovation is a key driver of productivity (Senate Economics References Committee 2015), and loadbearing CFS construction is an alternative that can potentially provide many benefits over the traditional RC construction method. As the market for mid-rise residential buildings in Australia is strongly market-driven and highly conservative, the feasibility of CFS requires investigation.
3.3 CFS in construction

CFS is used as a loadbearing and non-loadbearing construction material in buildings extensively around the world, with the dominant growth being in the use of interior partition walls and exterior curtain walls (Peterman et al. 2016a). The use of CFS arose through the excess sheet steel from World War 2 and the realised efficiency it provided to construction (Schafer et al. 2016). In Australia, CFS has been used as a framing material for detached housing since the 1940s, with the first guidance on structural design issued in 1991 (NASH 2010). The CFS framing system consists of repetitive members such as studs and joists assembled into sub-systems such as walls and floors (Williams 2016). These sub-systems are fixed together to form the full building system, as depicted in Figure 3-1. Each of these levels of construction is described in more detail in the following three sections.

![Image: CFS system composition](image)

**Figure 3-1: CFS system composition**

3.3.1 Members

Members are the base elements of the CFS framing system, manufactured by slitting a coil of galvanised steel into thinner strips and passing these through a roll forming machine (Pringle 2014). The steel undergoes plastic deformation through a series of dies until the cross-section takes the general shape of a ‘C’ or ‘Z’ (Williams 2016). This process strengthens the steel and requires no heating to form its shape.
The steel is cut to the member lengths required for the construction purpose. Holes can be punched in the members to accommodate mechanical, electrical and plumbing (MEP) services. Member dimensions generally range from 100 – 200 mm in depth and between 40 – 50 mm in width. Thickness depends on the intended function of the member and other project logistics, ranging between 0.373 mm and 6.35 mm (Williams 2016). The tensile strength of CFS depends on the country in which it is produced; the British standards designate strengths of 280 MPa or 350 MPa (Grubb et al. 2001), whereas, in Australia, high-strength steel with a strength of 550 MPa is standard (Williams 2016).

Members are generally termed stud or joist, track, channel, furring and L-header, based on their application within the CFS sub-systems as depicted in Figure 3-2. A stud is a vertical wall member, and a joist is a horizontal floor or roof member. Studs generally have lipped flanges to increase the resistance to buckling.

![Figure 3-2: CFS stud, track and ledger](image)

The typical name designator system was developed in the USA for CFS member identification. This system is used internationally and defines the depth of the member, type, flange width and material thickness or gauge. For example, the member 600S162-54: ‘600’ is the web depth in 100th inches, ‘S’ refers to stud or
joist section, ‘162’ is the flange width expressed in 100th inches and ‘54’ is the minimum base metal thickness in 1000th inches. This naming convention is used throughout the thesis, with the exception of thickness, which is expressed in mm where specified.

3.3.2 Sub-systems
The CFS members are fixed together to create sub-systems such as walls and floors. A brief description of these components follows.

3.3.2.1 Wall
For mid-rise buildings, CFS is typically only used for non-loadbearing applications such as infill walls and interior partitions. However, the loadbearing capacity of CFS in mid-rise construction has been demonstrated internationally (e.g., (Westerman 2016)). Walls consist of a set of studs fixed to a top and bottom track; loadbearing walls also have noggins placed horizontally between studs to provide lateral restraint and prevent member buckling. The member dimensions and thickness can vary depending on the application; loadbearing studs require a greater thickness than non-loadbearing studs. In the UK, studs are placed at centres of 400 mm, 600 mm or 1,200 mm for compatibility with standard plasterboard widths (Grubb et al. 2001). In the USA, wall studs are generally spaced at 300 mm, 400 mm or 600 mm on-centre (Grupe 2015). Where required, insulation is placed between the studs, and plasterboard (or similar material such as oriented strand board (OSB)) is fixed on either side depending on the location of the wall. The number of layers of plasterboard is dependent on the fire and acoustic requirement of the wall; two layers are often used for mid-rise buildings. CFS walls can also be designed to resist lateral loads; walls designed for this purpose are referred to as shear walls and are generally strap-braced or sheathed with steel or OSB (Nakata et al. 2012). Figure 3-3 depicts a loadbearing gravity wall, strap-braced shear walls and a sheathed shear wall.
Figure 3-3: CFS wall types: (left to right) Gravity wall, strap-braced shear wall, sheathed shear wall

An alternative to stud wall systems is CFS wall cassettes which, instead of studs, use a series of structural linear trays connected together. Cassettes have fewer stability problems than slender studs, but this method is more applicable to low-rise buildings (Davies 2006).

3.3.2.2 Floor
CFS Floor systems usually consist of joists, ledgers, bridging, stiffeners and decking as depicted in Figure 3-4. Joists can be replaced with CFS trusses made from smaller sections.
The joists transfer the axial loads into the stud walls and can be C-section, Z-section or lattice joists. Bridging or blocking runs perpendicular to the joist direction, creating a load path which transfers lateral forces into the main lateral load resisting system (Elhajj 2011). Various flooring systems can be implemented on top of the joists such as OSB floorboards or metal deck. It is possible to prefabricate CFS flooring using a cassette design, but the size is constrained by transportation and manoeuvrability.

Other flooring systems that do not utilise CFS can be implemented in a CFS structure. Some of these solutions include composite dovetail, concrete hollow-core plank, precast concrete panel or RC (Grupe 2015). The concrete floors can be pre-cast or poured in-situ on a steel deck which can be beneficial to use as a working platform during construction. Composite slabs are much heavier than light steel floor joists but have better thermal and sound attenuation properties. The floors are supported either directly on the walls or attached to the side, depending on acoustics and risk of web crippling in joists. Structural steel beams can be used in large open areas where the spans are too long for the capacity CFS joists.

3.3.2.3 Roof
The various forms of steel roofing systems include C-section joisted flat roofs, purlins spanning between cross-walls, open roof trusses, lattice members and modular roofing systems (Lawson 2012). For low-rise and detached residential buildings, a pitched roof can be used, consisting of CFS trusses and rafters. For
mid-rise construction, a flat roof is more common, providing space for any plant such as air conditioners on the roof. A joisted flat roof may be constructed with C-section joists supporting a layer of vapour control material, waterproof membrane, insulation and concrete panels or similar. A slight pitch is required to avoid pooling of water.

3.3.3 Systems
The sub-system assemblies are fixed together to make a full CFS framed building system. The systems have been categorised into the gravity (axial) loadbearing system and lateral loadbearing system.

3.3.3.1 Gravity system
The loadbearing gravity system is designed to transfer the axial loads through the building into the foundations. Load from the floors and roof are distributed to the loadbearing walls. Studs may be doubled or tripled to increase the capacity in the lower levels of mid-rise construction where the gravity loads increase. The three main methods to assemble the walls and floors are ledger framing, balloon framing and platform framing, as illustrated in Figure 3-5.

![Figure 3-5: CFS gravity system configurations: (left to right) Ledger, platform, balloon](image)

The three framing systems can be implemented and combined to varying degrees in a single building. Ledger framing is considered as the most common in the USA (Nakata et al. 2012); it involves hanging the floor from the walls using a ledger or rim tracks. The joists frame into the ledger, which is then fixed directly to the wall studs. Storeys are stacked on top of each other with the studs in the upper storey bearing directly onto the studs in the lower storey, with the tracks between them creating a continuous load path between walls.

In platform framing, the floor bears on top of the wall below, meaning that the flooring is sandwiched
between the walls of the upper and lower storeys, making the walls discontinuous (Grubb et al. 2001). Axial load pathing distributes the load from the top track or studs to the floor joist and then to the studs in the level below. This method allows more construction tolerance than ledger framing as the storeys are assembled one at a time, but consideration must be made for the high stress imposed on the flooring as the weight of the storeys above is pathed through it.

Balloon framing is similar to ledger framing in that the flooring is attached to the flange of the studs with a ledger; however, the studs are continuous over multiple storeys. This method is useful for framing around floor voids and has continuous axial load pathing through walls (Grubb et al. 2001).

3.3.3.2 Lateral system

In CFS construction, the main lateral load resisting system is based around the transfer of lateral loads from the floor diaphragms to the CFS shear walls. AISI S400-15 (AISI 2015) details a number of CFS shear wall configurations including OSB, steel or gypsum-sheathed shear walls, strap-braced shear walls and CFS bolted moment frames. Integral or K-bracing is another bracing system, in which studs are placed diagonally to resist lateral loads; however, this method is more suitable for low-rise buildings (NASH 2010). Shear walls can be designed to be uncoupled (Type 1) or coupled (Type 2) with the other shear walls in the building. Chord studs are designed for the high compression and tension loads at either side of the shear walls. They are connected end-to-end throughout the height of the building. The load is transferred from the shear walls into the base of the structure through the use of hold-downs or tie-downs. A hold-down is an anchorage device used to resist overturning forces at the location of the chord stud (AISI 2015).

HRS can also be used in walls for lateral cross-bracing or in the place of shear wall chord studs, as end-posts when lateral loads are high. The use of reinforced masonry or RC shear walls at the lift core is also common in mid-rise CFS construction to resist lateral loads (Torabian et al. 2016).

Many components of the system have an impact on the lateral behaviour of the building and distribution of lateral forces, including the stiffness of the floors, walls and connections. However, design generally only considers the floor diaphragm stiffness and the shear wall stiffness (Madsen et al. 2011). Non-structural wall components such as plasterboard on gravity walls have been demonstrated to have an impact on building stiffness (Peterman 2014), which is presented in more detail in Chapter 6.
3.3.4 Construction methods

There are three primary methods to assemble a CFS framing system; stick, panelised and modular. These methods are described in the subsequent sections.

3.3.4.1 Stick

Stick building, also known as elemental, is when the framing is assembled on-site. Members are delivered in pieces for workers to fix together and frame in-place. Stick construction requires the least amount of technology, and no prefabrication factory is needed. Any changes in the construction details can be dealt with on-site; however, it requires skilled labour to carry out on-site jointing. CFS pieces can be packed together easily for transport and carried by a single person on-site for framing, negating the need for an on-site crane. Because of the amount of fixing, this method requires a large labour workforce. Stick construction is only used for smaller projects in Europe, as the labour is expensive (Baker 2006a). However, it is common in the USA because of the low labour rates, and buildings of up to 8 stories have been stick-built (Veljkovic and Johansson 2006). This method can be utilised for complicated designs that are impractical to be prefabricated.

3.3.4.2 Panelised

Sections of the building are assembled in a factory then delivered to site as a complete panel. Members are fixed together in ‘jigs’ to form wall panels or floor/roof cassettes and can be inbuilt with window and door openings. Panels are labelled before delivery to site, making it simple for workers to find the location for installation. Connections between panels are made on-site with screws or bolts, taking much less time than the assembly of the panel itself. It is typical that 75-80% of the work is complete when the panel assembly is ready to be delivered on-site (Williams 2016). The size of a panel can vary depending on the job and can either be erected by hand or by crane on-site accordingly. Wall panels up to ten metres in length can be lifted into place by a crane (Baker 2006a).

There are two different levels of pre-panelisation; open and closed. Open panel systems consist of the structural components of the system. Closed panel systems have the structural CFS parts as well as insulation and lining installed (Baker 2006a).

Panelisation allows for faster installation of CFS framing than stick-built construction; building schedules can be cut by approximately 75% with the use of prefabricated panels (Williams 2016). Labour costs are
also reduced as there does not need to be as many people on-site for erection. Pre-panelisation takes complexity from on-site to off-site, with the factory environment allowing for higher quality control and improved ergonomics and health and safety. The geometrical accuracy of the panel assembly is greater than stick build construction, but more elaborate detailing is required for approval (Allen 2014). Consideration of temporary bracing must be made for panelised construction, which may affect the construction cost.

3.3.4.3 Modular
Modular (volumetric) construction involves factory assembling three-dimensional structures with CFS. These modular units can be fitted-out to various amounts before being delivered to site and services can be pre-installed within the module. Residential building modules are usually 4 m wide and 10 m long in the UK (Lawson 2012). The units are delivered to site, lifted by a crane and stacked together. Many of the same benefits can be seen for modular as in panelisation, such as improved safety and less time on-site.

Different levels of modularity exist in the current international market, designed for different applications (Lawson 2012). Four-sided, or continuously supported, modules are fully enclosed, and lateral loads are distributed through the walls, meaning no structural steel is required. These consist of a floor cassette, four walls and a ceiling. Open-sided modules are designed so that the loads are transferred through corner posts. Structural steel sections are used for the corner posts, and parallel flange channel (PFC) beams are used in the floor cassette. These modules are usually used for low-rise buildings as a separate bracing system is required for stability (Lawson 2012). This design allows for bigger room and more variation in architectural design because it is not limited to an enclosed rectangular shape. Partially open-sided modules are a combination of continuously supported and open-sided modules and can incorporate intermediate square hollow section (SHS) posts for openings in the sides of the module. Bathroom pods, kitchens, and stair sections can also be modulated.

3.3.4.4 Connections
Connections in CFS structures can often fail before the members under lateral loads, so are an important part of the design. The connection elements consist of the members, connection components and connectors (SA 2018a). Connection components can be cleats, gusset plates, brackets and connecting plates; walls and floors are commonly connected using clip-angles (Zhang and Yu 2016). Connectors can be welds, bolts, screws, rivets, clinches, nails and adhesives (SA 2018a). The fixing method depends on the manufacturer and the tools available, with more advanced fixing methods able to be undertaken in factory conditions.
Riveting and self-drilling screwing are the most popular CFS fixing methods in the UK (Baker 2006a). Wall track is generally attached to the floor slab using power-driven pins, threaded anchor, epoxy or anchor bolts (Allen 2014).

3.3.5 Advantages and disadvantages

3.3.5.1 Advantages

CFS has a number of potential advantages over other construction materials. The advantages of its use can be associated with the steel’s material properties or its ability for prefabrication and OSM. Some material properties that are beneficial for construction include its light weight, high strength-to-weight ratio, ductility which prevents brittle failure, consistent material quality, non-combustibility, recyclability and insusceptibility to termites and mould (Heywood et al. 2012; Williams 2016). The light weight allows the CFS to be carried by hand on-site with stick or panelised construction and reduces the total permanent load on the building. A six-storey CFS residential structure is approximately half the weight of a conventionally framed structure and two-thirds the weight of a timber frame, as built in the UK (Baker 2006b). This can lead to savings in foundation construction as expanded on in Chapter 5. The ability of CFS to be used for both structural and non-structural purposes make it an excellent system for mid-rise construction (Torabian et al. 2016).

The speed of construction is largely due to the compatibility of CFS with OSM. OSM can be defined as the manufacture of components, elements or modules before installation into their final location. There are numerous benefits that OSM has over traditional methods; Gibb and Isack (2003) found that clients viewed the top three benefits of OSM to be time, quality and cost. The increase in construction speed can be up to 50% faster than traditional on-site methods in the UK (Lawson 2012). Reduction in time is due to less work required on-site as well as adverse weather having no effect on components assembled in a factory environment. This is especially relevant as weather delays are one of the causes of productivity loss in the Australian construction market (Slater and Cater 2016).

In 1998, the UK government reported that implementation of OSM is a solution to improving the quality and efficiency of the construction industry (Egan 1998). More recently, Mark Farmer (2016) reviewed the UK construction labour model and recommended investment in skills and technology to build new business models for pre-manufactured techniques. Reports have also been published that highlight the importance of OSM for progress in the Australian construction industry (CRC Construction Innovation 2007; Hampson
and Brandon 2004).

Quality is improved due to the factory setting, providing higher control and consistency. A report by CRC Construction Innovation (2007) states that OSM is perceived to have the ability to produce high volume, high-quality products due to the efficiencies of manufacturing principles. Another study by Tam et al. (2007) found through a survey of construction parties that better supervision on improving the quality of prefabricated products was the highest ranked advantage of prefabrication. Other benefits have been identified such as improved health and safety for workers, removing difficult operations off-site and fewer people required on-site, less noise and dust, fewer site vehicles, fewer commercial vehicle movements and reduced storage charges Heywood et al. (2012).

CFS construction also has benefits in terms of sustainability. Steel is a recyclable material with 82% of ferrous materials being recycled in Australia (Australian Bureau of Statistics 2016); the coil used for CFS in Australia typically has a recycled content of around 25% (BlueScope Ltd 2016). Due to the implementation of OSM, construction waste is significantly reduced with a CFS frame as compared with traditional construction; a study in the UK finding a waste volume reduction of 22% (Moulinier et al. 2008). Mao et al. (2013) also found that greenhouse gas emissions are reduced when using OSM.

### 3.3.5.2 Disadvantages

Whilst CFS has been reported to have many potential advantages as a construction material and system, some disadvantages to its use in mid-rise construction exist. Due to the high heat conductivity of steel, thermal bridging is an issue for CFS framing systems. Studs in contact with sheathing materials can create a thermal bridge allowing heat to flow through the walls, potentially resulting in higher heating costs or condensation problems. This can be alleviated through reducing the contact area of studs on the sheathing and using thermal breaks in areas where critical thermal shorts exist (Kosny and Childs 2002; Pringle 2014). Despite its high strength-to-weight ratio, as the building height increases, the loads in lower storeys can exceed the capacity of CFS studs. Studs can be doubled or tripled, but this leads to high fabrication costs and wall dimension changes. CFS is most effective when loadbearing walls are aligned through the height of the building. When architectural details require deviations from this layout, other materials such as HRS or RC may be more effectively used (Heywood et al. 2012). However, increasing the number of trades required for construction has the potential to reduce the efficiency of CFS construction (Torabian et al. 2016).
The cost can also be a factor for CFS use in areas where supply chains are in their infancy, such as Australia. To take advantage of the off-site manufacturability of CFS and pre-panelisation, substantial set-up productions costs are necessary. If the demand for construction in the area is not high enough, the payback period may not offset the capital start-up costs. Blismas and Wakefield (2009) found the four major constraints for the implementation of OSM to be process change, high capital expenditure, supply-chain restrictions and regulatory restrictions. Size regulations for transportation and craning of panels also must be considered (Baker 2006a). In the UK, to transport a module without a police escort requires a unit width of less than 4.1 m (Lawson et al. 2005).

### 3.3.6 Regulations

A number of regulations exist around the construction of apartments and the use of loadbearing CFS, including aspects such as architecture, structure, fire and acoustics among others. Regulations are a determining factor for the accessibility of loadbearing CFS design as demonstrated internationally where the use of loadbearing CFS in mid-rise buildings has progressed further than Australia. For example, the American Iron and Steel Institute (AISI) produce a number of design and construction standards for CFS in the United States, Canada and Mexico. The AISI standards are updated regularly; the most recent AISI S100 series was updated in 2016, as detailed by Schafer et al. (2015). One of the major initiatives for current developments in the standard is promoting CFS framing in mid-rise construction (Yu and Chen 2016). The link between academic research such as that of Schafer (2015, 2016), and the AISI standards is potentially one of the reasons for the growing market for mid-rise loadbearing CFS construction in North America.

Some of the Australian regulations are outlined below.

#### 3.3.6.1 The National Construction Code

The National Construction Code (NCC) (ABCB 2019) is the overarching document for construction in Australia. It is an initiative of the Council of Australian Governments to incorporate all on-site construction requirements into a single code. It is produced and maintained by the Australian Building Codes Board (ABCB) on behalf of the Australian government and each State and Territory government. The Building Codes Committee (BCC) is the peak technical advisory body to the ABCB with responsibility for technical matters. The Building Code of Australia (BCA) is Volume One (Class 2-9 buildings) and Two (Class 1 and 10a buildings) of the NCC. Mid-rise residential structures are typically classified as Class 2 and hence, covered by Volume One. The BCA is given legal effect by building regulatory legislation in each State and
Territory. The NCC overrules in any difference arising between it and any Standard, rule, specification or provision as listed in the NCC. The NCC is divided into Sections A to J covering the following topics:

- Section A - General Provisions
- Section B - Structure
- Section C - Fire Resistance
- Section D - Access and Egress
- Section E - Services and Equipment
- Section F - Health and Amenity – Sound Transmission and Insulation
- Section G - Ancillary Provisions
- Section H - Special use Buildings
- Section J - Energy Efficiency

Compliance with the NCC is achieved by satisfying the performance requirements. This can be done with a ‘performance solution’ or a ‘deemed-to-satisfy’ solution. A performance solution is assessed according to one or more of the assessment methods. A deemed-to-satisfy solution which complies with the deemed-to-satisfy provisions (which are given in each section of the code) is deemed-to-comply with the performance requirements. There are four assessment methods for a performance solution:

1. Evidence to support that the material or product meets a performance requirement of deemed-to-satisfy provision.
2. Verification methods in the NCC or other.

Recent updates to the NCC occurred in 2019 with some of the major changes affecting Class 2 structures. These include the requirement of sprinkler systems in Class 2 and 3 buildings between 4 storeys and 25 m in height, and a number of changes to energy efficiency requirements, among others. Structure, fire resistance and sound transmission and insulation are particularly relevant to CFS construction and hence, addressed in more detail below.
3.3.6.2 Structure

Structural design requirements are set out in Section B of the NCC. Multiple performance requirements must be satisfied, including adequate performance under permanent, imposed and construction activity design actions. The code also sets out verification methods for structural reliability and robustness. Deemed-to-satisfy provisions for resistance to actions references AS/NZS 1170.0 (SA 2002a). The actions also depend on the importance level of the building. The relevant standards as referred to by the NCC for determination of structural resistance of materials and forms of construction for steel are listed as:

- Steel structures: AS 4100 (SA 1998)
- Cold-formed steel structures: AS/NZS 4600 (SA 2018a)

AS/NZS 4600 (2018a) is the standard for the design of CFS structures, recently updated in 2018. Section 1 outlines the scope and general information. Section 2 of the code describes the methodology for determining properties of CFS sections by breaking them down into simpler elements and the effective width method. Section 3 covers member capacity under axial tension, bending, compression and combined loading. Structural assemblies are explained in Section 4; this includes built-up sections consisting of channels placed back to back, lateral restraints and wall stud assembly requirements. Section 5 shows calculations for the capacities of connections and Section 6 covers cyclic loading and fatigue life. The DSM is covered in Section 7. It is used for determination of nominal axial compression and bending capacities of CFS members. Tables are provided for limits of pre-qualified members. Finally, Section 8 details the requirements of testing for material properties.

A number of significant improvements have been made in the 2018 update, outlined by Hancock (2016). In summary, updates have been made to design rules for bolted connections based on Australian research and added to Section 8. A new Section 9 on fire and Appendix B detailing methods for advanced analysis have also been added. Appendix D now includes buckling moments and stresses for local, distortional and global buckling. Considerable revisions were also made to the DSM. The recent updates provide a sound basis for the development of mid-rise loadbearing CFS construction in Australia; as the member behaviour can be more accurately predicted, higher resistance factors can be used.
The NASH Standard for Residential and Low-Rise Steel contains information that is also relevant to mid-rise construction. It describes current construction practice with CFS as well as proprietary system solutions for low-rise structures. It outlines design recommendations for roof, wall and floor systems as well as topics such as bracing, connections, durability and fabrication.

### 3.3.6.3 Fire resistance
Fire resistance is covered in Section C of the NCC. Buildings are required to maintain structural stability during a fire and avoid the spread of fire, especially to sole-occupancy units and public corridors. The deemed-to-satisfy provisions consist of requirements for fire resistance and stability, compartmentalisation and separation, and protection of openings. Compartmentalisation and separation requirements prevent the spread of fire by segregating different parts of the building, such as lift shafts and different rooms in the apartment. These walls must have a particular fire resistance level (FRL), which refers to the grading periods in minutes for the following criteria:

- **Structural adequacy**: The ability to maintain stability and adequate loadbearing capacity as determined by AS 1530.4 (SA 2014).
- **Integrity**: The ability to resist the passage of flames and hot gases specified in AS 1530.4 (SA 2014).
- **Insulation**: The ability to maintain a temperature on the surface not exposed to the furnace below the limits specified in AS 1530.4 (SA 2014).

For example, the FRL requirements for a loadbearing internal wall are 90/90/90, meaning it must fulfil the FRL requirements for 90 minutes each. A Class 2 building with a rise in four or more storeys requires Type A construction. This is important for fire-resisting construction requirements as Type A is the most stringent. The specification for fire-resisting construction requires that loadbearing internal walls and loadbearing firewalls must be constructed from concrete, masonry or fire-protected timber. CFS construction is not listed as a deemed-to-comply type of construction in the specifications, meaning a performance solution must be undertaken, which is a more lengthy and costly process.

### 3.3.6.4 Sound transmission and insulation
Acoustics are covered within the NCC Section F. The two most important requirements are the sound insulation performance of a building element (airborne) and the noise impact performance of a floor
(impact). Sound insulation performance indicates the level of speech privacy between spaces and the noise impact performance is characterised by how much impact sound reaches a room from the space overhead. Building elements must meet different levels of these requirements, depending on where they are located. The standards specify how to test building elements for these values.

For CFS construction, it is important to note the discontinuity requirement; a wall must have an impact sound insulation rating (be discontinuous) if it separates a bathroom, sanitary compartment, laundry or kitchen in one sole-occupancy unit from a habitable room (other than a kitchen) in an adjoining unit; or a sole-occupancy unit from a plant room or lift shaft. Discontinuous construction refers to a wall having a 20 mm cavity between 2 separate leaves where there is no mechanical linkage between leaves except at the periphery (ABCB 2016). This reduces the impact sound travelling through the wall. The code also provides a table listing the acceptable forms of construction for walls and their sound insulation performances.

3.3.7 International CFS bodies

There are a number of international CFS bodies that provide information about CFS construction, making it more accessible. In the UK, the Steel Construction Institute (SCI) is an independent source of information in steel construction. SCI provides technical information and training on the use of steel in design, with consideration to codes and building regulations. They also provide many case studies for different CFS applications. Cold-Formed Steel Engineers Institute (CFSEI) is an institution in the USA that provides a number of resources for CFS construction including construction details, design guides, issue papers, research reports and technical notes. CFSEI aims to develop and improve industry standards and design methods. Steel Framing Industry Association (SFIA), another USA group, aims to expand the steel industry through promotion and providing technical guides. Their industry members include manufacturers, fabricators, suppliers, designers, constructors and steel producers. SFIA also certifies manufactured CFS products for code compliance.

In Australia, the Australian Steel Institute (ASI) and NASH are the central bodies involved with the CFS sector. NASH is engaged with regulatory processes affecting steel-framed housing, producing standards, guides, conference papers and technical notes for CFS construction.

3.4 Methodology

The research aim for this chapter is to investigate the application of loadbearing CFS mid-rise
construction in international markets, with the objectives to:

- Explore the current state-of-the-art in mid-rise loadbearing CFS construction.
- Identify the advantages and disadvantages of loadbearing CFS.

Alongside the above literature review of CFS in construction, both a desktop study and survey of CFS professionals were used to collect data on existing loadbearing CFS structures. Details of both are outlined below.

3.4.1 Desktop study

Initially, an exploratory desktop study was undertaken. The research includes 23 CFS buildings constructed in the USA, UK, Canada and Australia. This was considered to be a sufficient number of reviewed cases to be representative of the market. As there is a lack of academic literature in this area, grey literature was utilised, with information sourced from a number of publicly available domains, including steel institutes, associations, company websites and magazines. The cases were selected based on the availability of data; no additional data was found for the desktop studies other than what was available publicly. The cases were also selected to provide a range of results, with not all cases being specifically mid-rise apartment buildings. This method has been demonstrated to be useful in the timber industry by Smith et al. (2018), who used a study review to evaluate mass timber. The data collected through the desktop surveys were somewhat limited and positively biased in favour of CFS. Hence, the following case study survey was implemented to collect additional information.

3.4.2 Case study survey

A questionnaire was designed to generate case studies on mid-rise loadbearing CFS structures built internationally. Questionnaires are widely used as a data collection method in social science as they are straightforward, standardised and low cost (Robson 2011), making them an effective tool for the purposes of this study. The findings of the desktop study were used to develop the criteria for building and participant selection, and to design the questionnaire. Table 3-1 provides an overview of the criteria for building selection, participants and an outline of the questions.
Table 3-1: Survey selection criterion and question overview

<table>
<thead>
<tr>
<th>Building</th>
<th>Participants</th>
<th>Questions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential 4+ levels</td>
<td>CFS Manufacturers</td>
<td>Key data on the building including time, cost and size</td>
</tr>
<tr>
<td>CFS structural system</td>
<td>Developer</td>
<td>Type of construction for roof, walls, floor</td>
</tr>
<tr>
<td>Built within the last 10 years</td>
<td>Builder</td>
<td>Reasoning for design decisions, what did and did not go well throughout</td>
</tr>
<tr>
<td>Various different countries</td>
<td>Architect</td>
<td>the construction process</td>
</tr>
<tr>
<td>Mix of lateral stability types</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Construction companies and professionals with experience in CFS mid-rise construction were selected to participate in the online questionnaire. The rationale for using companies as a source of data is that they can draw on industry experience to provide a deeper understanding of CFS construction. Non-probabilistic, purposive sampling was used to select participants for the purpose of satisfying the research aims. Participants were provided with a Participant Information Sheet (PIS) (attached in Appendix A) and were asked to consider a CFS construction project they were involved with and answer questions regarding it. The questions prompted information such as key data on the building including time, cost and size, type of construction, the reasoning for the design and other questions regarding building information modelling (BIM), compliance, sustainability and on-site construction logistics. The building projects were selected through meeting criteria to suit research aims; these criteria included loadbearing CFS structural system, residential, a height of four or more storeys and completed within the last 10 years. However, due to the lack of available data, some participants selected a building that they thought best fit the criteria. Connections in the SRH (Heffernan et al. 2017) were used to minimise non-responses, which can be common for internet surveys (Robson 2011).

The construction of a questionnaire requires a number of considerations to ensure the reliability and validity of the collected data. There is the potential for bias and for positive conclusions to be focused on, as the participants were commenting on projects they were involved with. Questions were put as simply as possible, phrased in a way that the participant would understand, and leading questions were avoided (Knight and Ruddock 2008). Visual aids were provided for some questions; one of the advantages of the internet survey (Robson 2011). A pilot test study was carried out on colleagues and SRH members to ensure questions functioned as designed, and would be able to be completed within half an hour. Reliability was ensured by presenting respondents with the same set of questions, which were refined after the pilot test.
study (Robson 2011). Table 3-2 presents an overview of the questionnaire as provided to the participants.

Table 3-2: Questionnaire key data, resources and questions

<table>
<thead>
<tr>
<th>Key Data</th>
<th>Resources</th>
</tr>
</thead>
<tbody>
<tr>
<td>Year of completion</td>
<td>Technical drawings – Floor space layout</td>
</tr>
<tr>
<td>Build duration</td>
<td>Conceptual construction drawings</td>
</tr>
<tr>
<td>Cost</td>
<td>Product technical data</td>
</tr>
<tr>
<td>Total storeys (CFS Storeys)</td>
<td>Models</td>
</tr>
<tr>
<td>Construction type (modular, panelised, stick)</td>
<td></td>
</tr>
<tr>
<td>Wall system</td>
<td></td>
</tr>
<tr>
<td>Roof system</td>
<td></td>
</tr>
<tr>
<td>Floor system</td>
<td></td>
</tr>
<tr>
<td>Façade</td>
<td></td>
</tr>
</tbody>
</table>

**Questions**

1. Why was CFS chosen/how did it compare with other construction methods (concrete, timber – if company has experience with this)?
2. Was any structural steel used, if so, where?
3. Were any parts of the structure overdesigned?
4. How complicated was the design process? – specify
5. How much was BIM used throughout the construction process?
6. What modelling programs were used for architectural design and structural analysis?
7. How many different CFS sections were used?
8. How was fire and acoustic compliance achieved?
9. Were there any problems with storage/ insurance?
10. Was there any difference in floor plans for the different levels?
11. Was sustainability taken into consideration? – LEED rating
12. Were there any problems with on-site coordination (cranes, concrete pours, etc.)?
13. What safety measures were put in place on-site, and how did these affect the cost?
14. Overall, what went well with the construction?
15. Overall, what went poorly with construction?
16. How labour-intensive was the construction process?
17. Were there any problems with site coordination?
18. Were there any problems or constraints with transportation?

### 3.4.3 Data analysis

Two methods of analysis were implemented: a matrix, primarily for quantitative data, and thematic analysis, primarily for qualitative data. A matrix is an intersection of two lists displayed as rows and columns (Milles et al. 2013). It is a concise visual method of presenting simultaneous data that can be more effective than
extended text (Milles et al. 2013). A matrix of data is provided for the findings of both the desktop study and the survey, as a concise overview of the data. Some of the collected data was deemed to be more suitable for thematic analysis and are summarised into general themes. Thematic analysis is described by Braun and Clarke (2006) as a method for identifying, analysing and reporting patterns within the data. It is a simple method, useful for summarising key features and highlighting similarities and differences in a data set (Braun and Clarke 2006). Themes were identified based on the information provided by survey respondents and desktop studies. The thematic analysis allowed triangulation of the findings from both the desktop study and the survey. Based on the findings of the literature review, desktop studies and case study survey, a summary of the advantages and disadvantages of CFS construction is provided with an overview of the best practices.

3.5 Results

3.5.1 Desktop study

A range of projects from USA, UK and Canada (and one Australia) were identified, with the earliest dated project from 2004 and the most recent in 2017. The majority of the cases used CFS as the main loadbearing structure, with some incorporating structural steel. Some cases used CFS in a secondary manner but highlighted a generalisable benefit to CFS construction. Stick, panelised and modular construction types were implemented, and a variety of roof, floor and wall systems were used. Table 3-3 provides a matrix overview of the 23 projects, of which 16 are residential, six are student accommodation, two are hotels, and one is a hospital.
<table>
<thead>
<tr>
<th>Name</th>
<th>Country</th>
<th>Year</th>
<th>Type</th>
<th>Total Storeys</th>
<th>CFS Storeys</th>
<th>Primary CFS use</th>
<th>Wall details</th>
<th>Lateral load resisting system</th>
<th>Roof</th>
<th>Floor</th>
<th>Method</th>
<th>Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>AQ Rittenhouse, Philadelphia</td>
<td>USA</td>
<td>2014</td>
<td>Residential</td>
<td>12</td>
<td></td>
<td>Façade</td>
<td></td>
<td></td>
<td>Concrete</td>
<td></td>
<td></td>
<td>(CFSEI 2015a)</td>
</tr>
<tr>
<td>Elan Westside Apartments, Atlanta</td>
<td>USA</td>
<td>2014</td>
<td>Residential</td>
<td>8</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>CFS trusses</td>
<td>Composite dovetail</td>
<td>(CFSEI 2015b)</td>
<td></td>
</tr>
<tr>
<td>Plaza at Pearl City Assisted Living Facility, Pearl City</td>
<td>USA</td>
<td>2014</td>
<td>Residential</td>
<td>5</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td>Concrete</td>
<td></td>
<td></td>
<td>(CFSEI 2015c)</td>
</tr>
<tr>
<td>Streatham Hub</td>
<td>UK</td>
<td></td>
<td>Residential</td>
<td>6</td>
<td></td>
<td>Loadbearing</td>
<td>CFS studs: Thickness: 1.4 - 2 mm</td>
<td></td>
<td>CFS purlins</td>
<td>Composite dovetail</td>
<td>(Metek 2016a)</td>
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<tr>
<td>Harbour Court Student Residence</td>
<td>UK</td>
<td></td>
<td>Student residential</td>
<td>7</td>
<td>7</td>
<td>Loadbearing</td>
<td>CFS studs: Thickness: 1.6 mm</td>
<td></td>
<td>CFS strap-bracing</td>
<td>CFS cassette</td>
<td>(Metek 2016b)</td>
<td></td>
</tr>
<tr>
<td>Rodin Square Project</td>
<td>USA</td>
<td></td>
<td>Residential</td>
<td>9</td>
<td>9</td>
<td>Loadbearing</td>
<td>Heavy-duty CFS studs</td>
<td></td>
<td>CFS walls</td>
<td>Precast concrete</td>
<td>(Westerman 2016)</td>
<td></td>
</tr>
<tr>
<td>Colborne Street, Brantford, Ontario</td>
<td>CAN</td>
<td>2011</td>
<td>Residential</td>
<td>5</td>
<td>4</td>
<td>Loadbearing</td>
<td>CFS studs: 800S200-54 – 800S200-68 600S162-33 – 600S162-68</td>
<td></td>
<td>CFS trusses</td>
<td>Hollow-core concrete joists</td>
<td>(SFIA 2015a)</td>
<td></td>
</tr>
<tr>
<td>Sheridan College Student Residence, Trafalgar Campus</td>
<td>CAN</td>
<td>2013</td>
<td>Student residential</td>
<td>6</td>
<td>5</td>
<td>Loadbearing</td>
<td>CFS studs: 600S162-33 - 600S250-68 800S162-33 - 800S200-97</td>
<td></td>
<td>Hollow-core concrete joists</td>
<td>Hollow-core concrete joists</td>
<td>(SFIA 2015b)</td>
<td></td>
</tr>
<tr>
<td>Portland House, Exeter</td>
<td>UK</td>
<td></td>
<td>Student residential</td>
<td>7</td>
<td>3</td>
<td>Loadbearing</td>
<td>CFS studs</td>
<td></td>
<td></td>
<td>Stick</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Raines Court, North London</td>
<td>UK</td>
<td></td>
<td>Residential</td>
<td>6</td>
<td></td>
<td>Loadbearing</td>
<td>CFS studs</td>
<td></td>
<td></td>
<td>Modular</td>
<td>SCI 2003</td>
<td>(SCI 2003)</td>
</tr>
<tr>
<td>Fulham, London</td>
<td>UK</td>
<td></td>
<td>Residential</td>
<td>6</td>
<td></td>
<td>Loadbearing</td>
<td>CFS studs: Depth: 100 mm Thickness: 1.2 mm to 2.4 mm</td>
<td></td>
<td>CFS strap-bracing</td>
<td>CFS cassette Depth: 200 mm</td>
<td>SCI 2003</td>
<td></td>
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<tr>
<td>Century House, London</td>
<td>UK</td>
<td></td>
<td>Residential</td>
<td>20</td>
<td>2</td>
<td>Loadbearing</td>
<td>CFS studs</td>
<td></td>
<td></td>
<td></td>
<td>SCI 2003</td>
<td></td>
</tr>
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<td>Heathrow Hotel</td>
<td>UK</td>
<td></td>
<td>Hotel</td>
<td>4</td>
<td>4</td>
<td>Loadbearing</td>
<td>CFS studs: Depth: 100 mm</td>
<td></td>
<td>Composite</td>
<td></td>
<td>SCI 2003</td>
<td></td>
</tr>
<tr>
<td>Name</td>
<td>Country</td>
<td>Year</td>
<td>Type</td>
<td>Total Storeys</td>
<td>CFS Storeys</td>
<td>Primary CFS use</td>
<td>Wall details</td>
<td>Lateral load resisting system</td>
<td>Roof</td>
<td>Floor</td>
<td>Method</td>
<td>Ref</td>
</tr>
<tr>
<td>----------------------------------</td>
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</tr>
<tr>
<td>The 400, Washington State</td>
<td>USA</td>
<td></td>
<td>Residential</td>
<td>5</td>
<td>4</td>
<td>Loadbearing</td>
<td>CFS studs</td>
<td>Concrete shear cores</td>
<td>Composite</td>
<td></td>
<td>(Allen et al. 2007)</td>
<td></td>
</tr>
<tr>
<td>Harvard Yards</td>
<td>USA</td>
<td></td>
<td>Residential</td>
<td>4</td>
<td>3</td>
<td>Loadbearing</td>
<td>CFS studs</td>
<td></td>
<td>Braced frames</td>
<td></td>
<td>(Allen et al. 2007)</td>
<td></td>
</tr>
<tr>
<td>Victory Hall, Texas</td>
<td>USA</td>
<td>2004</td>
<td>Student residential</td>
<td>4</td>
<td>4</td>
<td>Loadbearing</td>
<td>CFS studs</td>
<td></td>
<td>CFS trusses</td>
<td>Composite</td>
<td>(SFIA 2014a)</td>
<td></td>
</tr>
<tr>
<td>City Green, Wisconsin</td>
<td>USA</td>
<td>2007</td>
<td>Residential</td>
<td>10</td>
<td>4</td>
<td>Loadbearing</td>
<td>CFS studs: 600S200-43 - 600S162-97 Spacing: 16” on-centre</td>
<td>CFS joists</td>
<td>CFS joists</td>
<td>(SFIA 2013a)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Holiday Inn Express Hearne, Texas</td>
<td>USA</td>
<td>2008</td>
<td>Hotel</td>
<td>3</td>
<td>3</td>
<td>Loadbearing</td>
<td>CFS studs</td>
<td>CFS joists</td>
<td>Panelised</td>
<td>(SFIA 2014b)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Saint Joseph Hospital, Colorado</td>
<td>USA</td>
<td>2014</td>
<td>Hospital</td>
<td>8</td>
<td>6</td>
<td>Façade</td>
<td>RC</td>
<td>Concrete</td>
<td>Panelised</td>
<td>(BuildSteel 2017)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Courthouse Condo Wheaton, Illinois</td>
<td>USA</td>
<td>2008</td>
<td>Residential</td>
<td>6</td>
<td>6</td>
<td></td>
<td>Structural cement</td>
<td>Stick</td>
<td></td>
<td>(SFIA 2014c)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>St Mary’s Village Studios</td>
<td>UK</td>
<td>2015</td>
<td>Student residential</td>
<td>4</td>
<td>4</td>
<td>Loadbearing</td>
<td>CFS studs: Depth: 100 mm Thickness: 1.6 mm</td>
<td>CFS joists: Depth: 200 mm</td>
<td>Composite: Depth: 170 mm</td>
<td>Panelised</td>
<td>(Metek 2016c)</td>
<td></td>
</tr>
<tr>
<td>Oval Quarter, Lambeth Regeneration, London</td>
<td>UK</td>
<td>2014</td>
<td>Residential</td>
<td>6</td>
<td>6</td>
<td>Loadbearing</td>
<td>CFS studs (double-leaf) Depth: 100 mm Thickness: 1.6 mm</td>
<td>CFS cassette</td>
<td>Panelised</td>
<td>(Metek 2016d)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>The iQ, Bristol</td>
<td>UK</td>
<td></td>
<td>Student residential</td>
<td>11</td>
<td>11</td>
<td>Loadbearing</td>
<td>CFS studs: Depth: 100 mm Thickness: 1.6 mm</td>
<td>CFS cassette</td>
<td>Panelised</td>
<td>(PAW Structures 2016)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Zinnia, Melbourne</td>
<td>AUS</td>
<td>2017</td>
<td>Residential</td>
<td>3</td>
<td>3</td>
<td>Loadbearing</td>
<td>CFS studs: Thickness: 0.75 - 1 mm</td>
<td></td>
<td></td>
<td>(BlueScope Ltd 2017)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.5.2 Case study surveys

Data were collected from seven survey respondents, with four of them providing more detailed information on the structural system of a specific mid-rise building as summarised in Table 3-4. Due to the low response rate for residential structures, data for hotel and student accommodation were also collected and analysed.

Table 3-4: Survey case study structural systems overview

<table>
<thead>
<tr>
<th>Case</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>USA</td>
<td>Canada</td>
<td>UK</td>
<td>UK</td>
</tr>
<tr>
<td>Type</td>
<td>Hotel</td>
<td>Apartments</td>
<td>Student</td>
<td>Student</td>
</tr>
<tr>
<td>CFS Storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth: 92 mm – 152 mm</td>
<td></td>
<td></td>
<td></td>
<td>Depth: 92 mm (Asymmetrical lipped C)</td>
</tr>
<tr>
<td>Loadbearing wall system</td>
<td>Depth: 162 mm - 203 mm</td>
<td>Depth: 92 mm</td>
<td></td>
<td>Depth: 92 mm</td>
</tr>
<tr>
<td>Gauge: Varied</td>
<td>Gauge: Varied</td>
<td>(Asymmetrical lipped C)</td>
<td>1 mm-1.5 mm</td>
<td>1.5mm, 1.2 mm and 1 mm</td>
</tr>
<tr>
<td>Roof system</td>
<td>Thermoplastic polyolefin rigid insulation over 12 mm metal deck &amp; CFS joist</td>
<td>Flat roof - CFS trusses - CFS joist</td>
<td>Flat roof - 254mm lipped C 1.5 mm S390 steel</td>
<td>Cassette – 254 mm x 1.5/2.5 mm joist, with 22 mm OSB decking</td>
</tr>
<tr>
<td>Floor system</td>
<td>40 mm gypsum concrete over 12 mm metal deck &amp; CFS joist</td>
<td>203mm hollow-core concrete</td>
<td>254mm lipped Cee Cassettes - 1.5 mm</td>
<td>Cassette – 254 mm x 1.5/2.5 mm joist, with 22 mm OSB decking</td>
</tr>
<tr>
<td>Bracing system</td>
<td>Sheet steel-sheathed CFS shear walls, structural steel</td>
<td>CFS strap-bracing and Masonry shaft</td>
<td>CFS strap-bracing - 80, 50 &amp; 40 mm wide flat strap</td>
<td>CFS strap-bracing</td>
</tr>
</tbody>
</table>

3.6 Discussion

As well as providing information on the systems used, survey respondents offered their views on the CFS construction process with some common themes arising. The three main advantages described by respondents were the speed of construction, cost-effectiveness and non-combustibility, which was in agreement with the literature (Williams 2016). The categories of structure, time and cost, BIM, height, fire and acoustics, and sustainability are discussed in the following sections.

3.6.1 Structure

The loadbearing wall systems reviewed consist of studs that vary in depth and gauge depending on the
purpose and manufacturing capabilities of available fabricators. From the survey responses, stud gauge was found to range from 1 - 2.4 mm and stud depth was found to vary from 92 - 203 mm. Respondents stated that within a single project, it was beneficial to keep the stud section profile constant while varying the gauge. Connections between members were generally made with self-tapping fasteners. A number of lateral load resisting systems had been implemented including CFS shear walls, structural steel bracing or a combination of masonry shaft and CFS strap-bracing as shown in Table 3-4. The Case A structure depicted in Figure 3-6 utilised CFS walls with a layer of thin sheet steel as well as some structural steel braced frames to resist shear.

![Figure 3-6: Rendering of the Case A building](image)

Figure 3-7 displays the third, and fourth-floor layout of Case A. The East-West direction has suites on the first floor, allowing the steel-sheathed shear walls to continue from the first floor to the roof. At the lower levels, the opposite side of the East-West corridor and the North-South wing are open areas, supported by structural steel. The upper storey shear walls on the North-South wing are supported by structural steel moment frames and braced frames. To simplify the foundations, the moment frames are aligned and spaced on every other party wall.
Figure 3-7: Case A shear wall plan with loadbearing (gravity) and shear walls

This structure was designed with the assumption of a flexible diaphragm for a metal deck and CFS joist flooring system. Another example of this style is the Pearl City Assisted Living Facility, which used sheet steel and gypsum composite shear panels for lateral resistance and a tie-down anchor system to prevent shear wall overturning. Harbour Court Student Residence used CFS walls that were strap-braced. On the lower ground, a moment-resisting frame with inclined struts was used for stability. Case B, C and D also use CFS strap-bracing. Harvard Yards used welded structural steel braced frames for lateral load resistance. Convent Hill made use of cast-in-place RC stairs, and elevator cores. The 400, Washington State building was provided with lateral resistance by RC shear walls designed for a high seismic zone. The loads were transferred through a rigid diaphragm concrete slab to the concrete shear cores.

Many of the cases noted the reason for selecting CFS was its light weight, which can be up to 50% lighter than that of a concrete alternative (BuildSteel 2017). The Zinnia apartments in Australia were reportedly between 33% and 50% of the weight of a comparable timber frame (BlueScope Ltd 2017). The light steel framing and composite floor slab system in the Streatham Hub project was reported to weigh only 60% of a concrete flat slab construction system. This can be vital when constructing on sub-optimal soils; for example, St Mary’s Village Studios was built on a steep slope with ground anchor stabilised soil, which
The importance of the loadbearing wall layout was highlighted by the Case B respondent, who noted that the architects were required to move walls to ensure alignment from floor-to-floor. Other cases had alignment issues which were solved through parametric design, architectural redesign and/or the introduction of HRS. Up to 100 tonnes of HRS was used in one case where there were long spans in the lobby area that CFS joists do not have the capacity to span. The respondent believed that coordination between CFS and HRS contractors is essential to reduce this type of overdesign. Alignment of the walls to ensure the load paths are continuous down the height of the building makes CFS design simpler and works to its strengths.

3.6.2 Time and cost

Time and cost are two of the critical factors determining the success of a construction project (Chan and Chan 2004). One survey respondent understood that the speed and cost benefits of CFS construction come, in large part, from the ability to prefabricate large amounts of the building: Panels were built one month in advance of need on-site (CFS manufacturer, Case B). Pre-panelised construction allows fast installation of the building envelope, which leads to earlier access for the following trades (Lawson 2012). Another respondent stated that their client finished the project two months ahead of schedule, which is significant in an industry where budgets and schedules are more often underestimated (Aibinu and Pasco 2008). Construction time can be further reduced because of the ability to concurrently phase CFS construction. Figure 3-8 depicts the Case B apartment structure, and Figure 3-6 depicts the construction schedule for Case B. The building’s structure took 90 days to erect, at a rate of roughly 10 days per floor.
It can be seen that, as the floor was still in construction, the loadbearing walls and HRS were erected. There is also a slight overlap between the walls for one level and the flooring above it. The Case B respondent noted that roof truss installation was 15 days, as compared with 5 days for hollow-core concrete flooring, demonstrating one of the benefits of hybrid construction (Heywood et al. 2012).

Another example of phased construction is Elan Westside Apartments, Atlanta. To improve the speed of
construction, each floor was divided into three phases to be completed simultaneously in different areas of the building footprint. Some cases demonstrated the benefits of CFS use under restrictive site constraints. Heathrow Hotel was built in a long narrow site with limited access. In this case, CFS wall panels were selected over precast concrete, due to the lightweight steel frames and the accuracy and speed of erection. The Sheridan College Student Residence in Canada opted to use CFS so that it was possible to complete the residence by the start of the university semester. RC construction requires lengthy times for concrete to cure in the colder months, so CFS was selected due to its significantly shorter construction time.

Reductions in cost as a result of less time in the construction phase are emphasised for Colborne Street, Brantford, Ontario. The developer of this four-storey residential CFS building believed construction time was reduced by 5-8 weeks. The time saving was due to the fast installation of CFS framing at a rate of one floor per week. It was claimed that an extra $100,000 (Canadian dollars) in rent was earned because of the early occupancy. CFS framing for Zinnia apartments in Australia was reportedly twice as fast to unload and assemble as compared with timber framing, with time-cost savings of $50,000 (Australian dollars). No concrete curing time or back-propping was also noted as a construction time benefit over RC construction; both factors are reported to reduce the productivity of RC construction (Mills 2010). Many cases noted that there were savings in insurance premiums for construction with CFS; due to the reduced risk of fire resulting from the non-combustibility of steel, as well as the short time spent on site by the workers, insurance premiums were lowered (BuildSteel 2018). Respondents stated that savings also came from lower construction financing charges, pared site supervision needs, and less labour.

3.6.3 Building information modelling

BIM has been demonstrated to improve the process flow of construction projects in the past (Azhar 2011). A McGraw Hill (2013) report stated that three-quarters of construction companies using BIM reported a positive return on investment from its implementation. One survey respondent stated that BIM was used in 80% of company projects. Preferred programs (software versions were unspecified) were Autodesk Revit and Autodesk AutoCAD for architectural design, Microsoft Excel, Trimble Tedds, IDEA StatiCa and Trimble SketchUp for structural analysis and in-house software for manufacturing. The developer of Elan Westside Apartments selected CFS because of the ability of the CFS engineer to create all of the structural framing drawings, layout drawings, shop drawings and material rolling information in a single model. For the Holiday Inn Express, it was noted that BIM and the use of CFS were required for everything to fit
together precisely. Plans showed the wall panel installation location and order. Holes for services were fitted into the framing during the roll-forming process and panels were assembled off-site accommodating the easy installation of services on-site. Another example of a project that used BIM for construction accuracy is the AQ Rittenhouse project. CFS was not used for any structural purposes; however, it was used for external framing over 12 storeys. The framing was installed in very tight quarters and within a high-traffic area. The panelised external walls came with stud, track, connection framing, exterior sheathing and finishes applied from the factory. Infill walling was not used to prevent the slab edge from being exposed, so HRS double bent angles were installed at each slab, with the panels welded to these. BIM was used for accuracy in placement of slots in bent angles for studs and jambs.

3.6.4 Height
Building height is a limitation for CFS construction compared with RC and structural steel construction. This is because the complexity of the construction system necessarily increases with building height, to ensure the capacity of the loadbearing system can satisfy design requirements. CFS studs often need to be doubled or tripled in the lower levels as the building height increases. As Australia currently has no deemed-to-satisfy provisions for apartments with three or more storeys using a fire-resisting CFS structure (ABCB 2016), a performance solution is required. The testing required to achieve this is costly, contributing to the lack of mid-rise CFS structures in Australia; at the time of writing, the highest loadbearing CFS building in Australia is four storeys (Dynamic Steel Frame 2015). However, international projects demonstrate that it is possible for much taller buildings to be constructed using loadbearing CFS. The iQ is an 11 storey student accommodation building in the UK; the loadbearing CFS structure was completed in less than 17 weeks. An example from the USA is the Rodin Square Project, a nine-storey residential building. The project uses heavy-duty structural studs with greater loadbearing capacity compared with standard stud shapes (Westerman 2016). Many of the ground floor studs carry over 215 kN, so there was no requirement for special structural steel elements. The walls also provide lateral bracing for the building, so no masonry core was required. These cases demonstrate that with efficient design practices, it is possible to build CFS structures with significantly more than three storeys.

3.6.5 Fire and acoustics
Fire and acoustic requirements for CFS construction are provided through prescriptive design, as there is a lack of understanding of the performance of CFS members, sub-systems, and systems (Batista-Abreu 2016).
Depending on the local standard used, there are different requirements for fire and acoustics. Projects in the USA, UK and Canada have all demonstrated solutions for their respective codes. The CFS manufacturer responding in Case C stated that two layers of 15 mm plasterboard were used for fire and acoustics, which was based on fire tests carried out in-house. It was noted that this method has widely accepted performance in the UK. Case B satisfied requirements using insulation, resilient channel and drywall. The Streatham Hub project in the UK used two layers of 15mm fire-resisting plasterboard on each face of the stud walls, attached via horizontal resilient bars to achieve a 60 dB sound reduction index. A composite slab and plasterboard ceiling system was sufficient for fire and acoustic requirements between storeys. In 2014, Oval Quarter was the largest CFS project in the UK at the time of construction, valued at over £100 million (Pound sterling). Sufficient acoustic insulation was achieved with a 21 mm deep screed board on top of CFS joists and two layers of 15 mm fire-resistant plasterboard with a service zone gap and a final layer of plasterboard ceiling below. The City Green project in the USA provided sound reduction through the combination of structural concrete floor sheathing panels topped with a poured-in-place underlayment. The underlayment had mats both above and below it, achieving sound attenuation scores of STC (Sound Transmission Class) 55.5 to 59.77 and IIC (Impact Insulation Class) 58.5 to 62.4. Many of the case studies noted that another reason for CFS selection over timber was its non-combustibility.

### 3.6.6 Sustainability

Some of the noted advantages from the desktop study and survey with regard to sustainability include the amount of recycled content in the steel, and reduction of construction waste. In terms of building energy sustainability, a survey respondent highlighted that CFS provided more space for insulation (as compared with solid timber framing) to reduce heating and cooling load on the building. One respondent stated that the CFS components achieve A+ and A ratings in the UK’s BRE Green Guide which ranks building materials and components from A+ to E, where A+ represents the best environmental performance (Building Research Establishment Ltd 2018). Other noted advantages were that CFS has a long lifespan as it does not have the shrinkage and moisture problems that exist with timber construction, and there are no requirements for termite proofing.

### 3.6.7 Summary of outcomes of exploratory study

Through triangulation of the various data collected in the studies reported in this chapter, the advantages and disadvantages of loadbearing CFS are presented in Table 3-5.
Table 3-5: Summary of advantages and disadvantages of loadbearing CFS mid-rise construction

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength-to-weight ratio – Lightweight construction allows wall panels to be handled by people on-site and reduces building weight on the foundations</td>
<td>Height limitations – As building height increases, higher loads are imposed on stud walls requiring doubling and tripling of studs, leading to higher costs and wall dimension changes or the introduction of HRS sections. However, loadbearing CFS is more than capable for use in mid-rise heights</td>
</tr>
<tr>
<td>Quality – Consistent material quality with no warping, as well as precision due to prefabrication allows tight construction tolerances</td>
<td>Thermal bridging – High heat conductivity requires considerations to prevent thermal bridging</td>
</tr>
<tr>
<td>Durability – Steel is non-combustible and insusceptible to termites and mould</td>
<td>Supply chain immaturity – CFS is only used in low-rise construction in Australia, so there is a lack of high production capacity. Significant set-up production costs would be required for mid-rise to be feasible</td>
</tr>
<tr>
<td>Prefabrication – Completely finished walls can be delivered to site, reducing on-site complexity. Can also be well-integrated with BIM through roll-forming fabrication</td>
<td>Code and permits – A performance solution is required for mid-rise fire-resisting construction with CFS in Australia. Further development of standards relating to CFS should provide deemed-to-satisfy fire and acoustics solutions, as proprietary systems incur testing costs</td>
</tr>
<tr>
<td>Speed – Prefabrication allows fast construction. Risk of inclement weather delays is reduced with factory conditions, and no curing time required. Construction can also be phased</td>
<td>Architectural flexibility – More stringent architectural requirements for loadbearing wall alignment and difficult to move walls around post-design</td>
</tr>
<tr>
<td>Cost-efficiency – An implication of the reduced time on-site means less on-site costs and earlier rent collection. The influence of unions in construction schedules is reduced. Insurance premiums are also less than timber due to non-combustibility</td>
<td>Disruptive to Australian industry – Requires a change in the Australian construction culture to introduce innovative solutions such as CFS. More logistics and planning required for new systems, inciting higher capital expenditure</td>
</tr>
<tr>
<td>Safety – Prefabrication takes a large amount of work off-site into safer factory conditions. On-site fall safety can be achieved using tie-off lines or scaffolding</td>
<td>Transport – Size regulations for transportation and craning of panels must also be considered when delivering panelised and modular systems</td>
</tr>
<tr>
<td>Job creation – Development of the use of loadbearing CFS in mid-rise would lead to job creation</td>
<td>Sustainability – Steel is a recyclable material, and off-site manufacturability reduces construction waste</td>
</tr>
</tbody>
</table>
A number of best practices were also identified based on the findings as listed below:

- CFS is most effective when loadbearing walls are in alignment through the height of the building. The continuous load paths simplify the design process and take advantage of the strengths of CFS. Repetitive floor plates are better suited to CFS for the wall alignment.
- Minimising the stud section profiles throughout the structure by maintaining a constant stud depth and varying the gauge can simplify the design and construction.
- Hybrid solutions can be integrated with loadbearing CFS such as the introduction of HRS moment frames in locations where CFS shear walls are not suitable. RC can also be introduced in flooring systems as precast concrete flooring, or for use in lateral force resisting systems as shear walls. The potential for integration allows the designer to work with the strengths of each material.
- Early collaboration with all players involved, including the CFS fabricator, can lead to more efficient construction. Coordination with HRS contractor is essential to reduce overdesign when implementing HRS.
- The use of BIM for design and manufacture can be useful to streamline delivery and ensure no clashes. Software interoperability is beneficial for the design and fabrication process.
- Avoid changes to wall locations on-site, however, penetrations for MEP purposes are easy to add on-site.
- Consideration must be made for transportation requirements when designing prefabricated walls and floor cassettes as trucks have limited width.
- Consideration of site constraints must be made for the delivery of panels; a large site allows storage of prefabricated components, however for a small site, on-time delivery and cost of cranes is critical.

3.7 Conclusions

This chapter has described CFS as a construction material and shown that loadbearing CFS construction is a viable method for mid-rise residential buildings due to a number of associated benefits. These findings demonstrate the state-of-the-art in mid-rise CFS construction through the presentation of a range of potential system types based around loadbearing CFS. The advantages found consistently in the case studies, such as speed and cost-effectiveness, are highly sought after in the construction industry and would
assist in addressing the lacking productivity growth in the Australian construction sector.

Other advantages include the ability to build over nine storeys high, reduce construction schedules as compared with traditional construction methods, satisfy fire and acoustic code provisions and achieve satisfactory sustainability targets. Some of the challenges with CFS construction such as fire and acoustic compliance in Australia were also addressed, but these can be seen as opportunities for the development of improved systems for Australia. Mid-rise CFS construction is still a relatively new method and can be improved through further investigation of current practice and development of new practices.

The following study in the thesis extends on the findings of this chapter relating to the lightweight nature of CFS, with the development of a case study archetype building. Where this chapter has explored a variety of mid-rise CFS structures and examined the use of CFS internationally, the purpose of Chapter 4 is to further evaluate the feasibility of loadbearing CFS in mid-rise structures in an Australian context.
Chapter 4 - Archetype design action comparison

4.1 Introduction

This chapter forms the first part of the comparative archetypal case study. The findings of Chapter 3 provide an insight into international loadbearing CFS construction, highlighting potential advantages over traditional construction methods, including speed of construction and cost-effectiveness. One of the key contributing factors to the advantages of CFS is the light weight of CFS, with cases reporting significant reductions in weight as compared with traditional methods (Baker 2006b; BlueScope Ltd 2017; Metek 2016a). This chapter aims to assess the structural feasibility of CFS mid-rise loadbearing structures in Australia, hence, an archetype building is designed with a RC structure to be typical of the current Australian market and subsequently designed with a CFS structure for comparison of design actions.

Firstly, a concise literature review on comparative studies is presented, followed by a methodology section outlining the eight design scenarios for the comparative study. The features of the archetype building are described, and an outline of the preliminary design of RC and CFS structural systems is presented. Permanent, imposed, wind and earthquake design actions are determined for all design scenarios. Lastly, the design actions are compared and discussed in relation to the findings from Chapter 3.

4.2 Literature review

The overarching purpose of this thesis centres on supporting the introduction of an innovative loadbearing CFS system into the conservative Australian construction industry. A similar endeavour has recently been undertaken by the timber industry in Australia, with the introduction of massive timber construction systems in the mid-rise apartment sector. Among other methods, a comparative case study approach was adopted (Timber Development Association NSW and Forsythe 2015). The case study is a useful method to compare cases within a context (Yin 1994); in this case, construction systems in a specific building sector. The comparative case study method is relatively uncommon in the construction industry as `like-for-like’ building projects are rare (Mills 2010). However; through the use of a model or hypothetical structure, research has been able to evaluate new construction systems in the industry.

WoodSolutions (Timber Development Association NSW and Forsythe 2015) produced a study that compared an eight-storey mid-rise cross-laminated timber (CLT) building with a traditional RC structure
in a theoretical location in suburban Sydney. The building used for comparison was designed to maximise the benefits of CLT construction. The study demonstrated cost savings as well as a reduction in weight of 50% for the timber structure; however, the effect on foundations was not quantified. A similar approach was used for a high-rise structure in the USA. Skidmore, Owings and Merrill, LLP (2013) undertook a comparative study of a 41 storey timber and steel hybrid structure with a benchmark concrete structure. Findings demonstrated a reduction in weight of 65% with an expected flow-on effect of decreased foundation costs, seismic loading and construction time. Another study in China compared an existing RC mid-rise residential structure to a CLT hypothetical design for the same structure for life cycle analysis (Liu et al. 2016).

Torabian, Nia and Schafer (2016) developed an archetype mid-rise building to assess the feasibility of a complete CFS structure at different storey heights, but no comparison was made with another structure type. The authors were unable to find comparative studies in the literature for CFS with alternative construction systems in the context of mid-rise apartment buildings, however; some of the findings from Chapter 3 reported savings in weight for CFS construction. Baker (2006b) states that a six-storey CFS residential structure is approximately half the weight of a conventionally framed structure and two-thirds the weight of a timber frame, as built in the UK. Streatham Hub, a mid-rise CFS project from the UK described in the desktop study review, was reportedly only 60% of the weight of that of a concrete flat slab structure (Metek 2016a). Some studies have compared low-rise CFS, Çelik and Kamali (2018) found that a CFS two-storey detached house structure was nine times lighter than an equivalent RC structure. The Zinnia apartments in Australia were reportedly between 33% and 50% of the weight of a comparable timber frame (BlueScope Ltd 2017).

Chapter 3 identified the benchmark building structure for mid-rise apartment structures in Australia to be RC. RC is used due to availability in the construction supply chain and current industry knowledge. However, the structure is heavy in comparison with the lightweight structure of CFS. The studies undertaken in the timber industry demonstrated that a lightweight structure provides structural benefits relating to both seismic loads and foundation design (Timber Development Association NSW and Forsythe 2015). Weight reduction can also be beneficial for difficult soil conditions such as collapsible soil that may have excessive settlement under additional load (Das 2007) meaning that a site that is unsuitable for a heavy RC building may be appropriate for a lighter CFS building.
Therefore, this study has a specific interest in quantifying the weight reduction and related benefits for a CFS structure compared with that for RC. The weight of a building is used in the calculation of design actions for permanent and earthquake load cases. Many authors have used a case study methodology to compare design actions. Paulson (2004) compared base shears due to wind and seismic actions on a prototype building in a number of locations in the USA, finding that generally, seismic base shear forces dominate over wind base shear forces for short and heavy structures. Panchal and Marathe (2011) compared design loads for a steel-concrete composite and RC high rise case study. Starossek (1998) compared concrete and lightweight structures for bridges in a weight-cost context. Comparison of wind loads for low, medium and high-rise buildings has also been carried out by Holmes, Tamura and Krishna (2009). However, no design action comparisons for CFS structures were found in the literature.

The timber industry has demonstrated the usefulness and potential impact of the comparative case study method as CLT construction systems have subsequently been implemented in Australia (Evison et al. 2019). The lack of literature highlights a gap for CFS mid-rise comparison studies. Hence, the present chapter implements a comparative case study method to compare the design actions for a CFS structure with a RC structure used as a benchmark building.

### 4.3 Methodology

The overarching aim of the comparative archetypal case study is to assess the structural feasibility of CFS in the Australian mid-rise residential market. The objective of this chapter is to:

- Compare the structural systems of RC and CFS for a typical Australian mid-rise apartment.

The design action comparison is the first part of the archetype comparative study depicted in Figure 4-1.
Due to the lack of loadbearing CFS structures in Australia, the case study was a hypothetical building. Figure 4-2 provides a general outline of the methodology.

The archetype building was architecturally designed to suit a RC structure and to be typical of the Australian market. A preliminary structural design was undertaken with both structural systems (CFS and RC) to ensure structural stability and relatively efficient design. Designs are limited to the superstructure in this chapter; the substructure is covered in Chapter 5. The permanent and imposed actions are independent of the building’s location, but the wind and earthquake actions depend on the geographic location and soil conditions. For this comparison, two locations and two sub-soil classes were selected as independent variables, resulting in four design scenarios for both structure types or eight scenarios overall, as depicted in Figure 4-3.
The locations of Sydney and Brisbane were selected as the majority of mid-rise development activity has been within the larger Australian capital cities (Shoory 2016), with over 75% of the apartment developments constructed since 2011 located in Sydney, Melbourne or Brisbane. The City of Brisbane is susceptible to higher wind actions than Sydney, whereas Sydney is in a higher seismic region. Sub-soil Class $C_e$ refers to shallow soil, and sub-soil Class $E_e$ refers to very soft soil (SA 2007). Permanent, imposed, wind and earthquake actions were calculated for all eight scenarios and compared with Australian limit state load combinations (SA 2002a).

### 4.4 Archetype case study building design

#### 4.4.1 Architectural design

The archetype building plan was designed with a RC structure for a notional site in Sydney, to be typical of the mid-rise residential sector in urban regions of Australia. The archetype was designed to comply with all necessary regulations at the time of design, including the NCC, Apartment Design Guide and BASIX (ABCB 2016; NSW Department of Planning and Environment 2015, 2017). A 3D architectural Revit model of the building, floor plan and architectural rendering are depicted in Figures 4-4, 4-5 and 4-6 respectively.
Figure 4-4: 3D Revit model of archetype building (Source: Steel Research Hub)

Figure 4-5: Archetype first-floor plan (Source: Steel Research Hub)

Figure 4-6: Artistic rendering of archetype building (Source: Steel Research Hub)
The building is seven storeys above ground level with a single underground basement car park level. There are 60 apartments ranging from 1-3 bedrooms and 22 underground car parking spaces. The ground floor was designed with a communal lobby and a higher ceiling than the storeys above. The top two storeys have a step-back as is common for a building of this size in Australia. In accordance with AS 1170.0 (SA 2002a), the building is considered to be of Importance Level 2 with an ordinary consequence of failure. The design working life was specified at 50 years, giving an annual probability of exceedance of 1/500, used for both wind and earthquake design. Further details are provided in Heffernan et al. (2017). Table 4-1 details the elevation and plan dimensions.

Table 4-1: Archetype building dimensions

<table>
<thead>
<tr>
<th>Floor</th>
<th>Floor-to-floor heights (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-G</td>
<td>2.7</td>
</tr>
<tr>
<td>G-1</td>
<td>4.0</td>
</tr>
<tr>
<td>1-2, 2-3, 3-4, 5-6</td>
<td>3.1</td>
</tr>
<tr>
<td>4-5</td>
<td>3.2</td>
</tr>
<tr>
<td>Total height</td>
<td>23.7</td>
</tr>
</tbody>
</table>

The architectural floor plan required slight changes for the adaptation for a loadbearing CFS structure. Adjustment of the wall thicknesses and their locations was required to accommodate the standard CFS stud walls and load path alignment. This impacted the compliance of the floor plan to the Apartment Design Guide (NSW Department of Planning and Environment 2015), hence minimal adjustments were made to the architectural plans for the CFS archetype building to ensure compliance.

4.4.2 Preliminary structural design

A preliminary structural design was carried out for both structures in the notional site; the resulting building designs were then used to determine the whole structure design actions for comparison. The two structures have significantly different load path mechanisms; for the RC structure, the loads are carried mainly through pinned column-slab connections with a laterally stiff core while the CFS structure predominantly uses loadbearing wall construction. RC members were designed using AS 3600 (SA 2018b). As part of the greater Steel Research Hub project, the CFS structural design was undertaken by professional engineers.
with experience in CFS design in the USA using the allowable strength design (ASD) approach from the North American AISI standards (AISI 2007, 2012). Table 4-2 provides an overview of the structural systems for the two structures, with further detail in the following subsections.

Table 4-2: System design for RC and CFS structures

<table>
<thead>
<tr>
<th>System</th>
<th>Reinforced Concrete</th>
<th>Cold-Formed Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>PT flat plate - 200 mm deep</td>
<td>CFS joists - 203 mm deep, metal deck and gypsum-based topping</td>
</tr>
<tr>
<td>Axial loadbearing</td>
<td>RC unbraced blade columns</td>
<td>CFS stud walls - 152 mm deep</td>
</tr>
<tr>
<td>Lateral load resisting</td>
<td>RC U-shaped shear core - 200 mm thick</td>
<td>CFS shear walls braced by steel sheathing.</td>
</tr>
<tr>
<td>Roof</td>
<td>PT flat plate - 200 mm deep</td>
<td>CFS joists - 203 mm deep</td>
</tr>
<tr>
<td>Façade</td>
<td>Lightweight cladding, brick and cement render</td>
<td>Lightweight cladding, brick and cement render</td>
</tr>
</tbody>
</table>

The CFS systems used are common for a USA context and comply with Australian requirements. It should be noted that some structural steel was required for the CFS building’s lower storeys to account for load transfer in the open area of the ground floor. An outline of the preliminary designs for each structural system is provided below.

4.4.2.1 Reinforced concrete structure

The RC structure is a flat plate construction, consisting of a concrete slab supported directly on columns as depicted in Figure 4-7. It uses a RC shear core for lateral load resistance, as is common practice for mid-rise apartments in Australia (Menegon et al. 2017). Flat plate systems allow for simple formwork and the accommodation of underfloor services within the ceiling as there are no beams spanning between columns.
The design was carried out using limit state design philosophy, with load and resistance factor design (LRFD). Predictability of applied loads is accounted for through load factors, and the variability of materials and construction is accounted for through resistance factors (Ellingwood et al. 1980). Load factors must be applied to the applied loads to express them in terms that are safely comparable to the ultimate strength levels. The magnitude of load factors applied to load combination equations in LRFD varies depending on load type and predictability (permanent, imposed, wind, etc.). The factored loads are kept below a computed member load capacity multiplied by a resistance factor.

The concrete had a characteristic compressive strength $f_c$ of 40 MPa and elastic modulus $E$ of 32.8 GPa for both columns and slabs (SA 2018b). Exposure classification for interior residential was A1, and exterior non-industrial temperate climate zone was A2, requiring a concrete cover of 20 mm to satisfy durability requirements. To satisfy minimum fire resistance of 90 minutes, a slab thickness of 200 mm was required (SA 2018b). Slabs were designed first, followed by columns and shear walls as detailed below.

**Slab**

Flat plate slabs are generally economical for spans up to 6 – 8 m (Cement Concrete & Aggregates Australia 2011). Post-tensioning (PT) is commonly used in flat plates when spans are similar in both directions and
is economical for spans of 7 – 9 m (Cross 2007). As the archetype building has a maximum span of 7.5 m, the slab was designed to be post-tensioned with bonded tendons, as is common in Australia (Cross 2007).

The initial depth of floor slabs in the architectural plan was 200 mm for fire requirements. For a maximum span of 7.5 m, this equates to a reasonable span to depth ratio of 37.5 (Cross 2007) and was therefore used for the preliminary design. Two-way slabs were designed in SAFE 2014 to satisfy the Australian requirements for flexural shear, bending, punching shear and short and long term deflection. A superimposed permanent action of 1 kPa was used for the preliminary design and pattern live loading was applied for the most adverse effect for both distributed and concentrated imposed loads. The amount and distribution of prestressing were determined using the load balancing method, with tendons designed to balance the entire permanent load. Under the balanced load, the slab remains plane, the remaining unbalanced load (imposed load) was used for service load behaviour calculations. Figure 4-8 depicts the layout of tendons (green) for a typical floor.

![Figure 4-8: PT layout for a typical floor of the archetype building](image)

The allowable mid-span deflection limit for serviceability was Span/300 for floors supporting plaster-lined walls (SA 2002b). Short and long term deflection was checked, using uncracked and cracked section analysis respectively, assuming 100% sustained permanent load and 25% sustained imposed load. The tendon layout and profile were reiterated until the transfer and long term stress limits were satisfied (SA 2018b).

Slabs were post-tensioned with standard 7-wire tendons in bonded flat ducts. Concrete strength at transfer
$f_{tp}$ was 22MPa (for 12.7 mm strand). For tendons less than 25 m long, jacking occurs at one live end with a jacking force of 560 kN or 616 kN shown in Figure 4-9, longer tendons were jacked from both ends.

![Figure 4-9: PT jacking values for a segment of the floor slab](image)

Tendon and reinforcement steel yield strength $f_y$ and ultimate strength $f_u$ are shown in Table 4-3. Reinforcement was designed to satisfy punching shear in regions around the columns.

**Table 4-3: Steel tendon and reinforcement properties**

<table>
<thead>
<tr>
<th>Application</th>
<th>Material</th>
<th>$E$ (GPa)</th>
<th>Unit weight (kN/m³)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebar</td>
<td>D500N</td>
<td>200</td>
<td>77</td>
<td>500</td>
<td>650</td>
</tr>
<tr>
<td>Tendon</td>
<td>7 wire – 12.7 mm</td>
<td>195</td>
<td>77</td>
<td>1,517</td>
<td>1,870</td>
</tr>
</tbody>
</table>

**Columns**

The structure was then modelled in ETABS, as shown in Figure 4-10. This was used to design the columns and shear walls of the structure.
Unbraced blade columns were designed to be continuous through the height of the building. Initial cross-section sizing was based on architectural requirements to fit within apartment walls. Slabs were modelled as rigid diaphragms and permanent, imposed, earthquake and wind actions were applied for analysis (SA 2002a). The columns were designed for the resulting axial forces and bending moments in accordance with AS 3600 (SA 2018b). Figure 4-11 depicts a 3D P-M (Force – Moment) diagram for an example column; all design load combinations were within the interaction surface.
Standard rectangular column dimensions were 1,200 mm by 200 mm and 1,200 mm by 250 mm, and circular columns had a diameter of 300 mm. Columns required a reinforcement percentage ranging from the minimum requirement of 1% in most cases to a maximum of 4% (SA 2018b); a typical column rebar cage is depicted in Figure 4-12.

Shear walls
The main lateral load resisting system for the RC structure was the RC shear lift core, as is the industry preference in Australia due to the fire rating (Concrete Institute of Australia 2014; Menegon et al. 2017).
pair of RC U-shaped limited ductile shear cores with a total of eight walls in the short direction, and two walls in the long direction were implemented for the archetype building. As with the majority of RC walls in Australia, they were designed and categorised as ‘limited ductile’ (Menegon et al. 2017). This category determines the seismic actions as explained in Section 4.5.4, and dictates that walls can be designed in accordance with the main body of AS 3600 (SA 2018b). Column-slab connections were modelled as pinned connections so the shear core was designed to take the entire lateral load. This also allows simpler detailing for column slab connections as no moment transfer is required. Reinforcement was designed based on the resultant reactions at the top and bottom of each wall segment. A wall thickness of 200 mm was sufficient with two layers of N16 reinforcement at 200 mm on-centre.

4.4.2.2 Cold-formed steel structure
The CFS structure used loadbearing CFS walls to carry axial and lateral loads to the foundations. A ledger framing system was implemented where the joists are fastened into the sides of the stud walls, and the walls are stacked vertically on top of each other, creating a continuous load path. A 3D Revit model of the CFS structure is depicted in Figure 4-13.

![Figure 4-13: CFS structural Revit model (Source: Steel Research Hub)](image)

CFS structural design was undertaken using the ASD approach. This approach compares the allowable strength of each structural component to the required strength, where the allowable strength is equal to the nominal strength divided by a safety factor (AISI, 2016). The structural steel had a minimum yield strength
of 450 MPa; the CFS grades are shown in Table 4-4.

Table 4-4: CFS yield strength

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>$f_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1</td>
<td>550</td>
</tr>
<tr>
<td>1 – 1.4</td>
<td>500</td>
</tr>
<tr>
<td>&gt; 1.5</td>
<td>450</td>
</tr>
</tbody>
</table>

For the CFS structure, firstly, it was necessary to identify a load path in order to determine which walls were required to be loadbearing. Next, the lateral load resisting system was designed, followed by the roof and floor and lastly walls and structural steel. The component design was carried out using both hand calculations and international software packages including ‘CFS’ (RSG Software Inc), CFS Designer (Simpson Strong-Tie), and RISA-3D (RISA Tech Inc). All CFS connections were assumed to be pinned unless they required specific detailing for a moment connection.

Lateral load resisting system

The lateral load resisting system is designed first as it has the greatest potential to cause design problems (J. Parr, personal communication, February 4, 2017). Lateral loads were resisted by Type 1 CFS shear walls braced with steel-sheathing. The lateral systems were designed to ensure a continuous path for shear walls to transfer the lateral loads from the diaphragms to the foundation. Loads were distributed to walls using the flexible diaphragm method, under the assumption that the metal decking alone makes up the diaphragm and the lightweight topping is non-structural (ASCE 2010). This means the lateral loads were distributed to walls based on the tributary areas of flooring connected to the shear walls. Shear walls were designed to resist the entirety of the lateral load with no account of the additional stiffness provided by non-structural wall panels or the rigidity of floor and wall connections. Deflection checks were satisfied for lateral earthquake and wind loads.

Shear walls steel sheathing had a thickness of 27 mil (0.686 mm) or 33 mil (0.838 mm) depending on load requirements. Some of the lower storey shear walls were designed as structural steel braced frames due to higher loading. A continuous post system was used to resist overturning on either end of the shear walls. The system uses a back-to-back post with one member being a single storey tall and the other member being
continuous over multiple levels. A continuous member would start at the foundation and stop in the third storey. Another continuous member would start in the third storey and stop in the sixth storey. Each continuous post would be screwed to the single-storey posts at each level to provide a load path for the tension. The shear wall posts were ranging from 600S162-1.5 mm single or back-to-back studs in the upper storeys to SHS for the lower floors when required. Hold-downs were designed for tensile loads at the base of CFS chord studs and embed plates for structural steel end posts.

**Floor and roof**

After resolving the lateral load resisting systems, the components of the gravity system were designed. Floors were one-way spanning and non-continuous over loadbearing walls. The roof and floor joists were designed for bending, shear, web crippling as well as total load and imposed load deflections. Deflections were limited to Span/240 for total load and Span/480 for imposed load. The roof was also designed to resist wind uplift. Shear was checked at joist ends and moment was checked in the centre for simply supported spans.

The flooring consisted of 203 mm deep CFS joists at 600 mm on-centre spacing. A 15 mm deep trapezoidal steel deck with a 25 – 40 mm gypsum-based floor topping was supported by the joists. Structural steel beams were required for joist to bear onto in areas with larger spans and where joist direction changed with no bearing wall below. CFS joists span between walls in the shortest direction as depicted by the typical floor plan in Figure 4-14.
The roof consisted of CFS joists at 900 mm on-centre, supporting a 40 mm top hat batten, glass fibre insulation and a metal roof deck with a total coated thickness of 0.47 mm. The roof panels were also framed into the side of the loadbearing walls. Clip angles were designed to connect joists to studs. For example, a 50 mm x 50 mm x 1.5 mm (450 MPa) clip angle x 150 mm long was used to attach a joist web and stud flange using 4 screws per leg.

**Walls**

CFS stud walls were designed to resist axial loads in the structure. The wall height and tributary width were used along with the wall weight to calculate the axial wall load. Pressure loads from the floor were converted into uniform line loads for wall design using the tributary area method with half the distance to the next loadbearing support, for each side of the wall panel. An excerpt of the engineering calculations for a corridor wall at Level 2 is provided in Figure 4-15. The loads are represented by variables; \( D \) for dead, \( L \) for live, \( L_r \) for roof live, \( W \) for wind and \( E \) for earthquake.
**D7: CORRIDOR BEARING WALL, LEVEL 2**

**3rd FLOOR DETAILS**
- LEFT SPAN LENGTH: 1.07 ft
- RIGHT SPAN: 9.08 ft
- LEFT LOADING TYPE: RESIDENTIAL
- RIGHT LOADING TYPE: CORRIDOR
- SPAN TYPE: SIMPLE SPAN
- SPAN TYPE: SIMPLE SPAN
- LINE LOAD MODIFIER: 1.00
- LINE LOAD MODIFIER: 1.00
- FLOOR SYSTEM DEPTH: 8 in

**LEVEL 2 WALL DETAILS**
- WALL TYPE: INTERIOR WALL (2+2)
- STORY HEIGHT: 10.17 ft
- WALL DL: 53.2 psf
- DEPTH: 6.000 ft
- STUD OC SPACING: 23.02 in
- DEFLECTION LIMIT: L/360

**USER SPECIFIED LOADS**

<table>
<thead>
<tr>
<th></th>
<th>D</th>
<th>L</th>
<th>Lr</th>
<th>S</th>
<th>W (psf)</th>
<th>E</th>
</tr>
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<tr>
<td></td>
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<td>pf</td>
<td>pf</td>
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</table>

**SERVICE AXIAL LOADS**

<table>
<thead>
<tr>
<th></th>
<th>D</th>
<th>L</th>
<th>Lr</th>
<th>S</th>
<th>W (psf)</th>
<th>E</th>
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<td>pf</td>
<td>pf</td>
<td>pf</td>
<td>pf</td>
<td>pf</td>
</tr>
<tr>
<td>LEFT SIDE OF WALL (psf)</td>
<td>34</td>
<td>39</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>CENTERLINE OF WALL (psf)</td>
<td>1187</td>
<td>1078</td>
<td>100</td>
<td>0</td>
<td>60</td>
<td>113</td>
</tr>
<tr>
<td>RIGHT SIDE OF WALL (psf)</td>
<td>120</td>
<td>320</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>TOTAL LINE LOAD (psf)</td>
<td>1331</td>
<td>1438</td>
<td>100</td>
<td>0</td>
<td>60</td>
<td>128</td>
</tr>
<tr>
<td>TOTAL AXIAL LOAD (lb)</td>
<td>2800</td>
<td>2830</td>
<td>106</td>
<td>0</td>
<td>0</td>
<td>253</td>
</tr>
</tbody>
</table>

*ROOF LIVE LOADS EXCEED SNOW LOADS AND WILL BE USED AS THE ROOF TRANSIENT*

**MEMBER DESIGN PARAMETERS**
- FLANGE BRACING: NONE
- REDUCTION FACTOR: ENTER VALUE
- ROTATIONAL STIFFNESS: ENTER VALUE

|   | 1 | 600 | 200 | 1.9 (450) | NOT IN LIBRARY |

**Axial**

<table>
<thead>
<tr>
<th>Axial lb</th>
<th>Ecc. Moment in-lb</th>
<th>ey in</th>
<th>Wind (psf)</th>
<th>Vymax lb</th>
<th>Mymax ft-lb</th>
<th>V / M Interaction</th>
<th>P / M Interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>D + L</td>
<td>5430</td>
<td>2102</td>
<td>0.396</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.073</td>
</tr>
<tr>
<td>D + Lr</td>
<td>2707</td>
<td>505</td>
<td>0.191</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.028</td>
</tr>
<tr>
<td>D + 0.75L</td>
<td>4970</td>
<td>1748</td>
<td>0.359</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.064</td>
</tr>
<tr>
<td>D + W</td>
<td>2099</td>
<td>505</td>
<td>0.187</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.038</td>
</tr>
<tr>
<td>D + 0.7E</td>
<td>2777</td>
<td>539</td>
<td>0.104</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.030</td>
</tr>
<tr>
<td>D + 0.75L + 0.75Lr</td>
<td>4943</td>
<td>1748</td>
<td>0.359</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.064</td>
</tr>
<tr>
<td>D + 0.75L + 0.505E + 0.75Lr</td>
<td>5092</td>
<td>1733</td>
<td>0.305</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.065</td>
</tr>
<tr>
<td>D + D + W</td>
<td>1392</td>
<td>303</td>
<td>0.222</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.024</td>
</tr>
<tr>
<td>D + D - 0.7E</td>
<td>1383</td>
<td>259</td>
<td>0.194</td>
<td>9.84</td>
<td>47</td>
<td>111</td>
<td>0.033</td>
</tr>
</tbody>
</table>

**SERVICEABILITY**

<table>
<thead>
<tr>
<th></th>
<th>Axial lb</th>
<th>Wind (psf)</th>
<th>Vymax lb</th>
<th>Mymax ft-lb</th>
<th>DEFLECTION in</th>
<th>Unity Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>ZONE 4 DEFLECTION</td>
<td>2000</td>
<td>9.94</td>
<td>45</td>
<td>119</td>
<td>0.015</td>
<td>L/7452</td>
</tr>
</tbody>
</table>

Figure 4-15: Level 2 corridor bearing wall design loads and combinations (Source: Steel Research Hub)

Axial load from above is assumed to occur concentrically, and loads from the adjacent floors occur half of
the stud depth from the stud centroid for a ledger framed system. The resultant moment from the right and left side is the eccentric moment, and the eccentricity distance is equal to the moment divided by the axial load. The wind load is used to check out-of-plane flexure of the studs with a limit of wall-height/360. Walls with similar loadings were designed for the worst-case to reduce complexity and save design time.

Axial loadbearing walls were framed with 152 mm deep CFS studs at 600 mm on-centre spacing. Flange depth was either 41 or 51 mm, and stud thickness was 1.5, 1.9 or 2.4 mm. Structural HRS members were required in regions of higher loading, illustrated in red in Figure 4-16. The stair and lift elevator shaft were also framed with loadbearing CFS walls.

![Figure 4-16: Structural steel and shear walls in archetype building (Source: Steel Research Hub)](image)

Lastly, the headers were designed for window and door openings.

## 4.5 Actions

Permanent, imposed, wind and earthquake actions were determined for both structures in each scenario. Figure 4-17 provides a reference for the coordinate system used to define the loading directions. Z is the vertical axis, X is in the direction of the long axis, and Y is in the direction of the short axis.
4.5.1 Permanent

Permanent loads act in the Z direction and consist of the self-weight of the loadbearing structural system and superimposed permanent load. The superimposed permanent load accounts for non-structural members such as floor cover, suspended ceiling and services. The self-weight of concrete is 24 kN/m$^3$, and this is increased to 25 kN/m$^3$ to account for the weight of the steel reinforcement and tendons in the concrete. Therefore the self-weight of the 200 mm slab is 5 kPa. Loadbearing CFS floor components are significantly lighter with a self-weight of 0.8 kPa. Superimposed on these loads for both structural systems is an allowance for partitions, floor cover, insulation, ceiling and MEP amounting to 0.88 kPa. An architectural drawing of the CFS floor system is depicted in Figure 4-18.

Table 4-5 shows a comparison of the permanent loads for the RC and CFS general residential floor systems, including the common components.
Table 4-5: RC and CFS floor permanent load components

<table>
<thead>
<tr>
<th>RC component</th>
<th>Load (kPa)</th>
<th>CFS component</th>
<th>Load (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200 mm PT slab</td>
<td>5</td>
<td>Gypsum cement</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Form deck</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CFS joists</td>
<td>0.12</td>
</tr>
<tr>
<td>Floor covering</td>
<td>0.10</td>
<td>Floor covering</td>
<td>0.10</td>
</tr>
<tr>
<td>Insulation</td>
<td>0.04</td>
<td>Insulation</td>
<td>0.04</td>
</tr>
<tr>
<td>Ceiling</td>
<td>0.12</td>
<td>Ceiling</td>
<td>0.12</td>
</tr>
<tr>
<td>MEP</td>
<td>0.24</td>
<td>MEP</td>
<td>0.24</td>
</tr>
<tr>
<td>Partition allowance</td>
<td>0.38</td>
<td>Partition allowance</td>
<td>0.38</td>
</tr>
<tr>
<td>Total</td>
<td>5.88</td>
<td>Total</td>
<td>1.68</td>
</tr>
</tbody>
</table>

The vertical structural components are also included in the permanent load. For the RC structure, these include RC columns and shear walls, and for the CFS structure, these are loadbearing CFS walls with a double layer of plasterboard on either side. External wall loads include the façade, which is the same for both structures. Figure 4-19 shows the plan layout of floors one to four based on the preliminary design. This was used to measure the lengths of the loadbearing CFS walls.

![Figure 4-19: Archetype floor plan with labelled walls (Source: Steel Research Hub)](image)

The floor composition varied depending on its purpose in the building. Table 4-6 shows the permanent loads for all floor and wall types in the CFS structure.
The floor and wall areas were used to calculate a total permanent load for both structures. The computed loads are compared in Table 4-7.

Table 4-7: Permanent loads comparison

<table>
<thead>
<tr>
<th>Type</th>
<th>RC  (kN)</th>
<th>CFS  (kN)</th>
<th>Reduction of CFS vs. RC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>40,134</td>
<td>11,537</td>
<td>71%</td>
</tr>
<tr>
<td>Wall/column</td>
<td>11,327</td>
<td>6,283</td>
<td>45%</td>
</tr>
<tr>
<td>Façade</td>
<td>4,288</td>
<td>4,288</td>
<td>0%</td>
</tr>
<tr>
<td>Total</td>
<td>55,749</td>
<td>22,108</td>
<td>60%</td>
</tr>
</tbody>
</table>

The reduction represents the difference between the CFS and RC loads as a percentage of the RC load. The most significant savings come from the floor at a 71% reduction. The wall/column comparison also shows a considerable reduction for the CFS structure as the loadbearing CFS walls are much lighter than the RC columns and shear walls. As the façade is the same for both buildings, there is no difference in load. CFS has a significant final permanent action reduction of 60%. The permanent weight of the CFS structure was in agreement with the data provided by the engineer of the CFS structure (Jay Parr, personal communication, 4/4/2018).

4.5.2 Imposed

The magnitude of the imposed load is dependent on the type of activity/occupancy and the location in the
building (SA 2002b). Table 4-8 depicts the surface loads applied in each location of the building; these are the same for both structural systems.

**Table 4-8: Imposed surface loads**

<table>
<thead>
<tr>
<th>Location</th>
<th>Load (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Residential</td>
<td>1.5</td>
</tr>
<tr>
<td>Corridor/stairs</td>
<td>4</td>
</tr>
<tr>
<td>Balconies</td>
<td>2.5</td>
</tr>
<tr>
<td>Roof</td>
<td>0.25</td>
</tr>
</tbody>
</table>

These uniformly distributed imposed actions were applied to the floor in SAFE 2014, as depicted in Figure 4-20. A live load reduction factor was applied in accordance with AS/NZS 1170.1 (SA 2002b) based on the area supported by an element (substructure). For the purposes of determining overall structural actions, point and line loads were not considered as distributed surface loads were governing.

![Figure 4-20: Imposed surface loads modelled in SAFE 2014 (units of kPa)](image)

Imposed loads were the same for both RC and CFS buildings as they have the same floor plan. The total reduced imposed load was calculated to be 6,331 kN.

### 4.5.3 Wind

Design wind actions were calculated in accordance with AS/NZS 1170.2 (SA 2011). For buildings less than 25 m in height, wind speed is assumed constant with height, and wind actions are approximated as an equivalent horizontal base shear and overturning moment. Base shear is the addition of positive external
windward force and negative external leeward force, the overturning moment is the moment about the base of the structure due to the wind forces, as shown in Figure 4-21.

Wind pressures are dependent on the design wind speed, aerodynamic shape factor and dynamic response factor of the building. The wind speed depends on peak gust wind data, Brisbane is in a different wind region than Sydney, giving a higher ultimate limit state design wind speed as depicted in Table 4-9.

Table 4-9: Wind location and hazard design

<table>
<thead>
<tr>
<th>Location</th>
<th>Brisbane</th>
<th>Sydney</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind region</td>
<td>B</td>
<td>A2</td>
</tr>
<tr>
<td>Average Recurrence Interval (years)</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>Terrain category</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Regional wind speed (m/s)</td>
<td>57</td>
<td>45</td>
</tr>
<tr>
<td>Design wind speed (m/s)</td>
<td>53</td>
<td>42</td>
</tr>
</tbody>
</table>

The natural frequency of the building can be conservatively estimated as 46 divided by the building height (SA 2002c). Using this estimate, the natural frequency is equal to 1.84 Hz, which is higher than 1 Hz.
Therefore, the design was based on static wind loads, and the dynamic response was not considered. Direction, topography and shielding multipliers were set as 1. The aerodynamic shape factor was the same for both structures as they have the same envelope. It is not necessary to calculate frictional drag for a building of these dimensions, and the calculation of torsional forces is only required for buildings over 70 m high. The calculated wind pressures are shown in Table 4-10.

Table 4-10: Wind pressures for Brisbane and Sydney

<table>
<thead>
<tr>
<th>Pressure (kPa)</th>
<th>Brisbane</th>
<th>Sydney</th>
</tr>
</thead>
<tbody>
<tr>
<td>External windward</td>
<td>1.04</td>
<td>0.65</td>
</tr>
<tr>
<td>External leeward</td>
<td>-0.74</td>
<td>-0.46</td>
</tr>
<tr>
<td>Internal positive</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Internal negative</td>
<td>-0.50</td>
<td>-0.31</td>
</tr>
</tbody>
</table>

The resultant forces were calculated by multiplying the external pressures by the tributary area they are acting upon. The surface of the balcony balustrades and external walls were considered as tributary area for wind pressure. The balconies take up roughly 25% of the building’s surface, so the building’s windward face surface area was multiplied by 1.25. Figure 4-22 depicts a segment of the third-floor exterior wall with areas considered as extra tributary area highlighted in red.

![Figure 4-22: Elevation and plan view of level 3 balcony balustrades](image)

Cladding forces were neglected as only the whole structure loads are of interest for comparison. The resulting actions are shown in Table 4-11 for wind loading in both the Y (acting on the large face) and X
(acting on the small face) directions.

Table 4-11: Wind action comparison

<table>
<thead>
<tr>
<th>Direction</th>
<th>Y</th>
<th>X</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Brisbane</td>
<td>Sydney</td>
</tr>
<tr>
<td>External windward force (kN)</td>
<td>1,719</td>
<td>1,071</td>
</tr>
<tr>
<td>External leeward force (kN)</td>
<td>-1,228</td>
<td>-765</td>
</tr>
<tr>
<td>Total external force (kN)</td>
<td>2,947</td>
<td>1,837</td>
</tr>
<tr>
<td>Overturning moment (kN\text{m})</td>
<td>34,921</td>
<td>21,765</td>
</tr>
</tbody>
</table>

The wind loads were the same for both the CFS and RC structure as the building dimensions are the same. Brisbane had a higher design wind speed than Sydney, causing a larger base shear and overturning moment.

4.5.4 Earthquake

The earthquake actions were computed using AS 1170.4 (SA 2007). Australia is a low-moderate seismicity region subject to intraplate earthquakes of low probability but high consequence (Wilson et al. 2008). There are three earthquake categories for design, depending on the parameters:

- Importance Level
- Probability Factor
- Building Height
- Hazard Factor
- Site sub-soil class

The archetype building has a height of 23.7 m and an Importance Level of 2, corresponding to a probability factor of 1/500 (SA 2007). The hazard factor $Z$ represents the peak ground acceleration for a return period of 500 years, ranging from 0.03 to 0.22 (Wilson et al. 2008). Hazard factors for Sydney and Brisbane are shown in Table 4-12.

Table 4-12: Earthquake hazard factors

<table>
<thead>
<tr>
<th>Location</th>
<th>Sydney</th>
<th>Brisbane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hazard Factor $Z$</td>
<td>0.08</td>
<td>0.05</td>
</tr>
</tbody>
</table>
There are five possible site sub-soil classes; hard rock, rock, shallow soil, deep soil and very soft soil. The soil type is used to determine the earthquake design category as well as the spectral shape factor; a softer ground condition leads to a higher design load (SA 2007). Two soil conditions were selected for each location; sub-soil Class C is shallow soil, and Class E is very soft soil.

The design actions are also factored by the structural ductility $\mu$ and structural performance factor $S_p$. These are based on the materials and type of lateral load resisting system. The structural ductility is an assessment of the structure’s ability to sustain cyclic displacements in the inelastic range, and the structural performance factor is an assessment of the building to survive earthquake motion (SA 2007). The seismic-force-resisting system for RC is specified as limited-ductile shear walls.

AS 1170.4 (SA 2007) does not have provisions for the CFS sheathed shear wall system used in this archetype. AS/NZS 4600 (SA 2018a) specifies a structural ductility factor for CFS seismic-resisting systems consisting of an assemblage of elements acting as a single unit or systems using semi-rigid connections of 1.25. This value is very conservative and likely for low rise buildings, built using low capacity and low ductility lateral load resisting systems. However, this may be increased to 4 if a special study is undertaken. The steel sheathed shear panels in the archetype structure designed by the structural engineer in the USA were based on tested values; thus, a ductility factor of 4 could be used. However, to err on the conservative side for the purposes of comparison, the ductility parameter for an undefined steel structure as given in AS 1170.4 (SA 2007), was selected for this archetype CFS building. This is essentially the same values as the RC structures, as shown in Table 4-13.

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Description</th>
<th>$\mu$</th>
<th>$S_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Structures</td>
<td>Other steel structures not defined above</td>
<td>2</td>
<td>0.77</td>
</tr>
<tr>
<td>Concrete Structures</td>
<td>Limited ductile shear walls</td>
<td>2</td>
<td>0.77</td>
</tr>
</tbody>
</table>

Based on these parameters, all design scenarios had an Earthquake Design Category of 2, meaning a static force-based analysis was sufficient for this comparison. Horizontal equivalent static forces were computed and applied to the building, as depicted in Figure 4-23.
The storey forces for Sydney Soil $C_e$ are shown in Table 4-14.

### Table 4-14: Earthquake storey forces for Sydney Soil $C_e$

<table>
<thead>
<tr>
<th>Storey</th>
<th>RC Force (kN)</th>
<th>CFS Force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>133</td>
<td>63</td>
</tr>
<tr>
<td>2</td>
<td>232</td>
<td>104</td>
</tr>
<tr>
<td>3</td>
<td>358</td>
<td>160</td>
</tr>
<tr>
<td>4</td>
<td>499</td>
<td>223</td>
</tr>
<tr>
<td>5</td>
<td>651</td>
<td>297</td>
</tr>
<tr>
<td>6</td>
<td>688</td>
<td>307</td>
</tr>
<tr>
<td>7</td>
<td>686</td>
<td>219</td>
</tr>
</tbody>
</table>

The resultant base shear and overturning moment calculated for each scenario are shown in Table 4-15, the actions are the same for both the X and Y directions.
The base shear and moments were much higher for the RC structure than CFS for all cases due to the higher seismic weight, which consists of the permanent action and factored imposed action for each floor. The earthquake actions in Sydney were higher than those in Brisbane and actions were greater in very soft soil than in shallow soil. It should also be noted that the resultant base shear and overturning moment were higher for earthquake loadings than the minimum resistance requirement for structural robustness (SA 2002a) for all cases.

4.6 Results

Actions were put into limit state load combinations for strength, stability or serviceability design from AS 1170.0 (SA 2002a). Load combinations factor the loads, accounting for uncertainty and the probability of each combination. The load combinations used for comparison are detailed in Table 4-16, where the long term factor \( \Psi \) is 0.4 for residential distributed and imposed actions.

<table>
<thead>
<tr>
<th>Location</th>
<th>Sydney</th>
<th>Brisbane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sub-soil Class</td>
<td>( C_e )</td>
<td>( E_e )</td>
</tr>
<tr>
<td>Structure</td>
<td>RC</td>
<td>CFS</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>3,247</td>
<td>1,372</td>
</tr>
<tr>
<td>Moment (kNm)</td>
<td>55,505</td>
<td>22,843</td>
</tr>
</tbody>
</table>

The axial loads, lateral base shear and overturning moments were compared separately, as they are in
different orientations. Chapter 5 details the resultant axial actions due to the overturning moment for foundation designs. The following sections compare the axial and lateral loads for each of the cases.

### 4.6.1 Axial actions

The axial actions due to permanent and imposed load combinations are depicted in Figure 4-24.

![Figure 4-24: Axial load combination comparison](image)

In all cases, the CFS load was significantly less than that for the RC structure, due to the reduced self-weight. The governing ultimate load combination for both CFS and RC is $1.2G + 1.5Q$ where the load was reduced by 46% for the CFS structure. The un-factored serviceability load combination was reduced by 44% and the long term imposed action serviceability limit state was reduced by 47%. This demonstrates that the lower the imposed load is in relation to the permanent load, the more significant the difference is between CFS and RC.

### 4.6.2 Lateral actions

Un-factored base shear and overturning moment due to wind and earthquake are compared in Figure 4-25. The wind actions are in the Y direction.
The base shear due to wind load was the same for both structures; the Brisbane load is much higher than the equivalent in Sydney due to the wind region. The earthquake loads on the CFS structure were reduced significantly by 58% as compared with the RC structure loads for all conditions. Depending on the location, wind or earthquake may govern. When comparing lateral loads for the X direction, earthquake governed all cases as the wind load was reduced for the smaller building surface area. Figure 4-26 compares the overturning moments about the X-axis for both structures.
The overturning moments were similar to the base shear; however, the moments due to earthquake actions were larger in relation to the moments due to wind actions. It is interesting to note that for Sydney sub-soil Class Ce, earthquake governed the overturning moment and wind governed the shear. This is due to the earthquake equivalent horizontal loads increasing in magnitude with height as compared with the uniform pressure distribution for the wind load cases. The governing load cases for each sub-soil class were determined and are provided in Tables 4-17 and 4-18. In both tables, the comparison shows the CFS loads as a percentage reduction relative to the RC loads.

Table 4-17: Governing lateral actions for sub-soil Class Ce

<table>
<thead>
<tr>
<th>Sub-soil Class Ce</th>
<th>Sydney</th>
<th>Compare</th>
<th>Brisbane</th>
<th>Compare</th>
</tr>
</thead>
<tbody>
<tr>
<td>Governing Shear</td>
<td>RC</td>
<td>CFS</td>
<td>Compare</td>
<td>RC</td>
</tr>
<tr>
<td>Earthquake</td>
<td></td>
<td></td>
<td>Wind</td>
<td>Reduction</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>3,247</td>
<td>1,837</td>
<td>43.4%</td>
<td>2,947</td>
</tr>
<tr>
<td>Governing Moment</td>
<td>Earthquake</td>
<td>Earthquake</td>
<td>Reduction</td>
<td>Wind</td>
</tr>
<tr>
<td>Overturning Moment (kNm)</td>
<td>55,505</td>
<td>22,843</td>
<td>58.8%</td>
<td>34,921</td>
</tr>
</tbody>
</table>
Table 4-18: Governing lateral actions for sub-soil Class Ee

<table>
<thead>
<tr>
<th>Sub-soil Class Ee</th>
<th>Sydney</th>
<th>Compare</th>
<th>Brisbane</th>
<th>Compare</th>
</tr>
</thead>
<tbody>
<tr>
<td>Governing Shear</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>Earthquake</td>
<td>CFS</td>
<td>Reduction</td>
<td>Earthquake</td>
</tr>
<tr>
<td></td>
<td>6,417</td>
<td>2,712</td>
<td>57.7%</td>
<td>4,011</td>
</tr>
<tr>
<td>Governing Moment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overturning Moment (kNm)</td>
<td>109,700</td>
<td>45,148</td>
<td>58.8%</td>
<td>68,563</td>
</tr>
</tbody>
</table>

For sub-soil Class C, earthquake was the governing action in Sydney except for CFS base shear, and wind was governing for all cases in Brisbane. For sub-soil Class E, earthquake governed all cases except for CFS in Brisbane. It can be seen that in most scenarios the same load is governing for both base shear and overturning moment; however for sub-soil Class C in Sydney, the base shear was governed by wind and overturning moment was governed by earthquake. As the wind loads are independent of the structure type, when wind was the governing load case, the lateral design loads were the same for both structures. When both structures were governed by earthquake, there was a significant load reduction of 58% for CFS. When the governing load cases were different for the two structures, the reduction was less significant. Figure 4-27 and 4-28 provide graphs depicting the governing base shear and moment in each scenario.

Figure 4-27: Governing base shear comparison
All cases show that the design loads for RC were higher than those for CFS; except in Brisbane with subsoil Class Ce where the design loads were the same and wind was governing.

4.7 Discussion

Through the use of multiple case studies, Chapter 3 provided an overview of the use of CFS in mid-rise construction and conveyed a number of benefits arising from its use. In order to assess the structural feasibility of loadbearing CFS in Australia, this chapter has compared the structural systems of loadbearing CFS and traditional RC for a typical mid-rise apartment structure. The comparison highlighted significant differences in the structural composition and design of the two systems. The CFS structure implemented a ledger framing system with single and double 152 mm deep loadbearing CFS stud walls as well as some HRS posts in the lobby and balcony areas. The flooring comprised of 203 mm deep CFS C-joists with a metal deck and gypsum topping. The lateral load resisting system was predominantly steel-sheathed CFS shear walls as well as some HRS braced frames in the lower storeys.

The mid-rise CFS construction system takes advantage of the high strength-to-weight ratio of CFS, but the system is more complicated than RC due to the number of components involved. Standardisation of components was found to be beneficial in streamlining the design process and minimising design work. As
practitioners recommended in Chapter 3, wall studs were kept at the same depth of 152 mm while the stud gauge was varied. Walls with similar loadings were designed for the worst-case to reduce the number of wall types for construction; however, this leads to a degree of overdesign.

The action comparison results found that the weight of the archetype building superstructure was significantly reduced by 60% with a CFS structure. This reduction is more significant than those presented in the literature, reporting a reduction in building weight of 50% (Baker 2006b) and 40% for Streatham Hub (Metek 2016a) as compared with traditional construction methods. The Streatham Hub project implemented a composite floor slab system, which would likely weigh more than the metal deck system used for the archetype comparison in this study. Both sources are from the UK and so may employ different traditional construction systems than Australia, and they do not specify if the lower transfer structure is included in the comparison.

The seismic weight was also significantly decreased with a CFS structure, as identified to be one of the benefits of CFS (Schafer et al. 2016). This weight reduction was shown to have an effect on the ultimate limit state design load combinations for the whole structure in all but one design scenario. For the actions due to the lateral loads, three conditions can arise:

- Wind governs both structures, and the resultant actions are the same
- Earthquake governs both structures, and the CFS actions are significantly less
- Earthquake governs RC and wind governs CFS, and the CFS actions are variably less

Based on the magnitude of actions, CFS is the most beneficial for conditions when earthquake is governing both structures, as it has the highest reduction in actions as compared with RC. Lower axial actions, base shear and overturning moment result in less demand on the lateral load resisting system and foundations. The lighter structure of CFS would be especially beneficial for soil garnering large settlements which might be unsuitable for the construction of a RC structure; the effect on foundations is explored in Chapter 5.

Consideration must be made for the design factors used in earthquake design, as the structural ductility and performance factor have a large impact on the equivalent horizontal actions. For example, the equivalent horizontal base shear as calculated with a structural performance factor of 1 and structural ductility factor of 4 is increased by more than three times when the structural ductility factor is changed to 1.25. This leads
to very conservative results when designing to AS/NZS 4600 (SA 2018a) without special testing. The USA standard for minimum design loads for buildings and other structures, ASCE 7-10 (ASCE 2010), provides Table 12.2.1, which details response modification coefficients and system overstrength factors, used to calculate seismic actions. Values are provided for both sheathed and strap-braced CFS systems. AISI S400 (AISI 2015), the North American standard for seismic design of CFS structural systems provides more detail on specific CFS systems, up-to-date with recent testing of the system behaviour (Schafer et al. 2015). The implementation of loadbearing CFS in mid-rise construction in Australia would benefit from the introduction of structural ductility and performance factors for mid-rise CFS lateral force resisting systems into the design standards. The values should be based on systems that can be used in Australia and would lead to more efficient designs and a simplified design process.

It should be noted that the graphs for lateral loads compare actions in the Y direction of the building. This direction has significantly higher wind loads as compared with the X direction due to the design of the building floor plan. It is common for mid-rise apartment structures to have an aspect ratio similar to the archetype due to architectural requirements. Earthquake actions govern all cases for load combinations in the X direction because of this. When designing the foundations, it must be checked in both directions; this is presented in Chapter 5.

The archetype building was designed to be a typical mid-rise apartment building in Australia. As these structures are traditionally built with a RC structure, the design is more efficient for a RC structure as opposed to a loadbearing CFS wall structure. The loadbearing CFS wall system benefits from having a repetitive floor plan with walls aligning down the length of the building. As the archetype structure was not designed with this consideration and only minor changes were made to the floor plan, the introduction of structural steel was required in a number of locations. The requirement for the addition of HRS adds design and construction complexity, as highlighted in Chapter 3. Therefore, the comparison presented could be considered less favourable for the CFS building. However, this would be a likely scenario for implementation in the industry at present, as most current architects in Australia design buildings for a RC structure. It is therefore necessary for a CFS structure to be able to adapt to these current mid-rise building designs. The findings of this comparison demonstrate that the adapted CFS structure still has a significant reduction in permanent loads and design earthquake loads. An optimised CFS structure would likely result in a more favourable comparison.
As the preliminary design for the CFS archetype building was carried out by a construction company in the USA, some of the solutions used in the archetype structure are common of the USA context, but not available in the Australian market, requiring the development of supply chains for the use in Australia.

### 4.8 Conclusions

This chapter first described the development of an archetype mid-rise apartment building used for the comparative study. The design of the preliminary structures for both CFS and RC was then detailed. Two locations and two soil conditions were selected to compare the effect of the structure on resultant whole structure design loads. Code-based analysis was carried out on the case study building to compare the permanent, imposed, wind and earthquake loads and resultant actions.

This study has demonstrated the beneficial effect on design actions due to the light weight of the CFS structure, which was shown to have a 60% lower permanent load than the RC structure. Earthquake loads were also significantly lower for the CFS structure. The imposed and wind actions, however, were the same for both structures. The governing axial ultimate limit load combination demonstrated a reduction of 53%. The governing lateral actions were also shown to be reduced for the CFS structure, with the greatest reduction of 58% base shear and overturning moment occurring in Sydney with sub-soil Class Ee. It was more advantageous to use CFS in a location with higher seismicity and poor soil conditions. The following chapter extends these findings with the use of the archetype building to understand the practical implications for the reduction in design loads through the quantification of the effect on foundations and earthworks for the two structure types.
5.1 Introduction

This chapter follows on from Chapter 4, forming the second part of the comparative case study. Chapter 4 demonstrated that the design actions are significantly reduced for the archetype building designed with a CFS structure than with a traditional RC structure. To understand the implications of these findings, in this chapter, the foundations are designed and then compared for the archetype building in two Australian locations.

Firstly, the literature review gives an overview of foundation construction and the associated costs, justifying the purpose of the comparison. The methodology section explains the design scenarios, and the subsequent sections present the archetype building design, locations for comparison and foundation analysis and design. Lastly, the resultant foundation designs are presented for both locations and discussed.

5.2 Literature review

Buildings consist of both a superstructure and substructure. The substructure is generally below ground level and has the purpose of transferring loads from the superstructure to the foundational soil or rock (Foster et al. 2010). The substructure is required to prevent excessive total and differential settlement of the structure whilst distributing the load over a large enough surface area in such a way that the founding material does not fail. Foundations are the component of the substructure that is in contact with the bearing material and can be categorised as shallow or deep (Foster et al. 2010). Shallow foundations are founded near the surface when there is sufficient bearing strength, deep foundations penetrate further below the ground surface and are required when the soil at the surface has poor loadbearing capacity (Das 2007).

The design of the substructure is imperative as there are significant risk and costs involved when excavating the ground (Chapman, 2005). Substructure costs can be up to 20% of the total structure costs (Chapman, 2005), representing a substantial capital expenditure for mid-rise building projects. There is also high risk associated with substructure works, with Chapman and Marcetteau (2004) reporting that ground-related problems account for between one-third and one-half of construction delays in the UK. Böhme et al. (2018) found, through a review of Australian mid-rise RC apartments, that the construction phase encompassing site works, substructure, basement and ground floor, took an average of 14 weeks to carry out, accounting
for 25% of the average total construction time of 55 weeks. Major time delays in the substructure
construction phase were commonly found to be caused by inclement weather during concrete pouring. The
impact of building substructure on construction cost and time highlights the importance of efficient
foundation design.

Chapter 4 demonstrated that the lightweight CFS structure led to a significant reduction in the design
permanent actions on the archetype mid-rise apartment building as compared with the RC structure. One
of the implications of this is the effect on the material quantities for the building substructure and
foundations, specifically reduced piling or smaller footing sizes (Timber Development Association NSW
and Forsythe 2015). However, no studies have been found that compare the foundation designs for the CFS
framed and RC structure. Hence, this chapter aims to fill the gap and quantify the effect of a lightweight
CFS structure on foundation designs.

5.3 Methodology

This chapter forms the second part of the comparative archetypal case study, as highlighted in Figure 5-1.

![Figure 5-1: Foundation comparison study within the comparative archetypal case study](image)

The overarching aim of the comparative archetypal case study is to **assess the structural feasibility of CFS in the Australian mid-rise residential market.** The specific objective of this chapter is to:

- Determine the effect of a CFS structure on foundation design.

For this purpose, a comparative case study approach was adopted. An archetype mid-rise apartment
building (outlined in Section 4.4) was designed for comparison of foundation designs in two Sydney
locations; Sydney Central Business District (CBD) and the suburb of Lindfield in North Sydney. These
locations were selected as they are likely locations for a mid-rise apartment structure, geotechnical reports with conditions suitable for deep and shallow foundations were also available. The archetype building was initially designed with a RC structure then with a CFS structure. Deep RC bored piles and shallow RC pad footings were designed in accordance with Australian standards for both structures as appropriate in each location, as depicted in Figure 5-2.

Figure 5-2: Foundation design scenarios; Sydney CBD and Lindfield

5.4 Archetype case study building design

The archetype case study building and preliminary designs of the superstructure for RC and CFS were detailed in Chapter 4 and depicted in Figure 5-3.

Figure 5-3: 3D models showing RC and CFS structures for the archetype building

This chapter is focused around the substructure design as outlined in the following sub-section.
5.4.1 Substructure design

The substructure consists of the transfer slab, basement columns, basement slab and foundations (piles/pad footings) as depicted in Figure 5-4.

![Building elevation highlighting substructure components](image)

Figure 5-4: Building elevation highlighting substructure components

The basement substructure design is the same for both structures with common components of a 400 mm deep RC PT transfer slab, RC columns, and 150 mm slab on ground. The retaining wall surrounding the basement level and basement slab-on-ground are not considered for design comparison as they are the same for both structures. Figure 5-5 depicts the architectural basement floor plan for both structures.

![Architectural basement floor plan](image)

Figure 5-5: Architectural basement floor plan (Source: Steel Research Hub)

To resist lateral loads, the RC structure has a RC shear core that is continuous from the basement to the roof. This was not necessary for the CFS structure as the lateral loads are resisted by CFS shear walls.
dispersed through the building. Hence, for the CFS structure, the basement shear walls were replaced with columns in each corner (four columns for each core) and non-structural walling in-between for fire-separation.

The transfer slab has the purpose of transferring the actions from the superstructure to the foundations. For the RC structure, the loadbearing columns and shear core were continuous above and below the transfer slab, meaning no eccentric loading. The loadbearing walls and shear walls in the CFS structure do not align with the basement columns in all cases, resulting in eccentric loading. Figure 5-6 depicts the worst-case permanent wall line loads from the preliminary design on the transfer slab; it can be seen that the columns below are not aligned with the loads.

Figure 5-6: Worst-case permanent wall line loads for CFS structure

Chapter 4 ascertained that the permanent loads for the CFS structure were significantly less than those for the RC structure, but due to the wall and column eccentricity, a relatively thick transfer slab was required for ultimate strength limit state design. A more efficient transfer slab would be possible for the CFS structure with rearrangement of the basement carpark plan and columns; however, the detailed design of the transfer slab was not within the scope of this thesis. For the purposes of comparison, an identical basement column layout was assumed for both structures, and a transfer slab of 400 mm was deemed appropriate for both buildings based on advice from consulting engineers, associated with the Steel Research Hub.
5.5 Design locations

Sydney CBD and the built-up suburb of Lindfield are urban regions, appropriate for multi-storey apartment structures. They are in the same wind region, so the wind pressures are equivalent. However, the subsoil classes differ, affecting the AS 1170.4 (SA 2007) earthquake design actions, as shown in Table 5-1.

Table 5-1: Foundation design location parameters

<table>
<thead>
<tr>
<th></th>
<th>Sydney CBD</th>
<th>Lindfield</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind region</td>
<td>A2</td>
<td>A2</td>
</tr>
<tr>
<td>Sub-soil Class</td>
<td>C&lt;sub&gt;e&lt;/sub&gt;</td>
<td>B&lt;sub&gt;e&lt;/sub&gt;</td>
</tr>
<tr>
<td>Foundation</td>
<td>Bored piles</td>
<td>Pad footing</td>
</tr>
</tbody>
</table>

Geotechnical reports were sourced from publicly available locations and simplified for foundation design (Table 5-2, Table 5-3). Sydney CBD has deep bedrock (Table 5-2), common in coastal Sydney regions (Douglas Partners 2015). RC bored piles are appropriate for this location, designed to transfer actions to the strong Class II shale rock layer. Cast-in-place construction avoids unwanted vibrations from pile driving operations, ideal for a site in close proximity to other buildings (Das 2007). Tables 5-2 and 5-3 display the soil profiles for Sydney CBD and Lindfield, the Reduced Level (RL) denotes the distance below the ground surface of the top of each stratum.

Table 5-2: Sydney CBD soil profile

<table>
<thead>
<tr>
<th>Strata</th>
<th>RL (m)</th>
<th>Unit</th>
<th>Layer thickness (m)</th>
<th>Ultimate Bearing Pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>Fill</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>Stiff to hard clayey silt</td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>Shale – Class II</td>
<td>-</td>
<td>10,500</td>
</tr>
</tbody>
</table>

Lindfield has a similar profile (Table 5-3); however, the founding rock layer is much shallower than Sydney CBD (Douglas Partners 2013). Shallow RC pad footings are typical for this type of location, bearing onto the low strength shale to prevent differential settlement.
Table 5-3: Lindfield soil profile

<table>
<thead>
<tr>
<th>Strata</th>
<th>RL (m)</th>
<th>Unit</th>
<th>Layer thickness (m)</th>
<th>Ultimate Bearing Pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>Fill</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Clay</td>
<td>1.3</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>2.3</td>
<td>Low strength shale</td>
<td>2.7</td>
<td>3,000</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>Strong rock</td>
<td>-</td>
<td>10,000</td>
</tr>
</tbody>
</table>

5.6 Foundation analysis and design

The design actions are transferred through suitable load paths in the superstructure and substructure into the founding rock, which in turn exerts an equal and opposite reaction back on the structure (Warner et al. 1998). Structural analysis was undertaken to determine the reactions used to design the foundations. LRFD was employed for foundation designs to satisfy strength, serviceability and durability requirements.

5.6.1 Design actions

The methods used to determine design actions for the superstructure were detailed in Section 4.5. The addition of the transfer slab self-weight for foundation design significantly increased the permanent actions of the structures. The weight of the transfer slab, basement columns and walls (non-structural for CFS) for both structures was 12,867 kN. The total resultant axial actions are provided in Table 5-4.

Table 5-4: Axial actions for RC and CFS structure

<table>
<thead>
<tr>
<th>Load</th>
<th>RC (kN)</th>
<th>CFS (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td>68,615</td>
<td>34,974</td>
</tr>
<tr>
<td>Imposed</td>
<td>13,046</td>
<td>13,046</td>
</tr>
</tbody>
</table>

Wind and earthquake equivalent base shear and overturning moment were calculated for both structures in each location. Lateral actions were applied to the positive and negative X and Y axes of the structure, as illustrated in Figure 5-7.
Table 5-5 presents the actions in the direction of the short (Y) axis. Wind actions were the same for all cases and earthquake actions varied by location and structure type.

**Table 5-5: Lateral actions for RC and CFS structure**

<table>
<thead>
<tr>
<th>Location</th>
<th>Sydney (Soil Cₜ)</th>
<th>Lindfield (Soil Bₑ)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Action</strong></td>
<td><strong>Type</strong></td>
<td><strong>RC</strong></td>
</tr>
<tr>
<td><strong>Earthquake</strong></td>
<td>Base shear (kN)</td>
<td>3,247</td>
</tr>
<tr>
<td></td>
<td>OT moment (kNm)</td>
<td>55,505</td>
</tr>
<tr>
<td><strong>Wind</strong></td>
<td>Base shear (kN)</td>
<td>1,837</td>
</tr>
<tr>
<td></td>
<td>OT moment (kNm)</td>
<td>21,765</td>
</tr>
</tbody>
</table>

**5.6.1.1 Design load combinations**

Foundation design actions were determined using load combinations for stability, strength or serviceability design as depicted in Table 5-6, where the long term factor \( \Psi \) is 0.4 for residential distributed and imposed actions, the combination factor \( \Psi_c \) is 0.4 for both distributed and concentrated actions, the earthquake combination factor \( \Psi_E \) is 0.3 for concentrated imposed actions and \( W_s \) is the serviceability wind action.
Table 5-6: Load combinations for foundation design (adapted from AS 1170.0 (SA 2002a))

<table>
<thead>
<tr>
<th>Loads</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stability</td>
<td></td>
</tr>
<tr>
<td>0.9G</td>
<td>Net stabilising permanent action</td>
</tr>
<tr>
<td>1.35G</td>
<td>Destabilising permanent action</td>
</tr>
<tr>
<td>1.2G + 1.5Q</td>
<td>Destabilising permanent and imposed action</td>
</tr>
<tr>
<td>1.2G, Wn, ΨeQ</td>
<td>Destabilising permanent, wind and imposed action</td>
</tr>
<tr>
<td>G, E∞ ΨeQ</td>
<td>Destabilising permanent, earthquake and imposed action</td>
</tr>
<tr>
<td>Strength</td>
<td></td>
</tr>
<tr>
<td>1.35G</td>
<td>Permanent action only</td>
</tr>
<tr>
<td>1.2G + 1.5Q</td>
<td>Permanent and imposed action</td>
</tr>
<tr>
<td>1.2G + 1.5ΨQ</td>
<td>Permanent and long term imposed action</td>
</tr>
<tr>
<td>1.2G, Wn, ΨeQ</td>
<td>Permanent, wind and imposed action</td>
</tr>
<tr>
<td>0.9G, Wn</td>
<td>Permanent and wind action reversal</td>
</tr>
<tr>
<td>G, E∞ ΨeQ</td>
<td>Permanent, earthquake and imposed action</td>
</tr>
<tr>
<td>Service</td>
<td></td>
</tr>
<tr>
<td>G + ΨQ</td>
<td>Serviceability permanent and long term imposed action</td>
</tr>
<tr>
<td>G, Ws</td>
<td>Serviceability permanent and wind action</td>
</tr>
</tbody>
</table>

An imposed load reduction of 50% was applied for all combinations as the basement columns have a large supporting area (SA 2002b). For earthquake load combinations, the actions in the X and Y directions determined separately were added by taking 100% of the horizontal earthquake forces for one direction and 30% in the perpendicular direction (SA 2018b).

Serviceability was checked with the axial long term imposed load and the un-factored permanent and wind action combination (SA 2002a). The annual probability of exceedance is 1/25 for the serviceability wind design case, resulting in a reduced design wind speed than that for strength (SA 2011). There is no earthquake serviceability load case, as structures designed to earthquake ultimate strength limit states are deemed-to-satisfy serviceability limit states (SA 2007). Accidental eccentricity was not considered in the archetype design, although it is required in the standards. The eccentric load would marginally increase the shear demands on the furthest walls from the shear centre.

5.6.2 Superstructure analysis

The load path used to distribute axial and lateral actions to the foundations differs between the two
structures. For the RC structure, the loads are carried mainly through pinned column-slab connections with a laterally stiff core while the CFS predominantly uses loadbearing wall construction. Linear elastic finite element analysis (FEA) was carried out in ETABS to determine the reactions for foundation design, as detailed in the following sections.

5.6.2.1 RC structure
The RC structure was modelled in ETABS as depicted by the 3D model in Figure 5-8. The blue columns and red shear walls are continuous from the base to the roof of the structure, with a minor increase in column dimensions at the base due to higher actions.

![Figure 5-8: RC structure in ETABS; 3D view (top), first storey plan view (bottom)](image)

Permanent and imposed actions were assigned as area surface loads on the floor slabs. The wind actions were resolved into a force at each diaphragm by multiplying the pressure by the tributary area of the storeys above and below. Lateral actions were applied as point loads at the centroid of each floor diaphragm as depicted in Figure 5-9 and scaled accordingly for each location.
As the relative stiffness of the core is much higher than the columns in a flat plate structure, it takes a majority of the base shear and overturning moment. The floors were modelled as rigid diaphragms to transfer the load to the shear core. The slab-column connections were modelled as pinned connections, so no moment and relatively small lateral reactions occurred at the base of the columns. Pin connections were also modelled at the ends of each shear wall. Figure 5-10 depicts a fourth storey plan view of the building under earthquake loading in the Y direction and X direction.
Figure 5-10: The deformed shape of the fourth storey floor due to earthquake loading in the Y direction (top) and X direction (bottom)

The torsional effects due to the eccentricity of the applied load to the shear centre result in large shear and overturning moment reactions at the base of the shear core.

5.6.2.2 CFS structure

The CFS structure directs superstructure actions through the CFS gravity loadbearing and shear walls into the RC transfer slab beneath. Due to the complex behaviour of the CFS structure, approximations were made for both the axial and lateral load distributions. The building was modelled using wall shell and floor slab elements as depicted in Figure 5-11.
5.6.2.3 Axial behaviour
Floors were modelled as shell elements and were assigned permanent and imposed actions as surface loads depending on floor type as detailed in Section 4.5.1. Loadbearing walls were also modelled as shell elements and assigned a self-weight in the same manner. Structural steel beams and columns were modelled in the lobby area of the first floor, as shown in Figure 5-12.
Figure 5-12: Structural steel in the first floor of the archetype structure

Loads were transferred through the loadbearing walls and structural steel columns into the transfer slab below. As there is no CFS wall option in ETABS, the wall shells were assigned an equivalent elastic modulus $E_{\text{shell}}$ to approximate a CFS wall in order to distribute the axial loads. The elastic modulus was calculated by equating the axial rigidity of the CFS wall stud and representative shell shown in Equation 5-1 where $A_{\text{stud}}$ is the cross-sectional area of the CFS stud, $A_{\text{shell}}$ is the cross-sectional area of the shell and $E_{\text{stud}}$ is the elastic modulus of the stud.

$$A_{\text{stud}}E_{\text{stud}} = A_{\text{shell}}E_{\text{shell}} \quad (5-1)$$

For CFS walls with 600S200-1.9 mm studs spaced at 600 mm, the representative shell was modelled with a thickness of 250 mm and elastic modulus of 657 MPa as depicted in Figure 5-13.
Shells representing the joist flooring were assigned an elastic modulus by equating the flexural rigidity of the floor joists to the floor shell as shown in Equation 5-2 in which $I_{\text{joist}}$ and $I_{\text{shell}}$ represent the second moment of area about the axis of bending of the joist and equivalent shell element, respectively. The elastic modulus of the joist $E_{\text{joist}}$ is the same as $E_{\text{stud}}$.

$$I_{\text{joist}}E_{\text{joist}} = I_{\text{shell}}E_{\text{shell}} \quad (5-2)$$

For a floor with 800S162-1.2 mm joists spaced at 600 mm on-centre, the equivalent elastic modulus of a 300 mm thick floor shell was calculated to be 286 MPa, shown in Table 5-7.

<table>
<thead>
<tr>
<th>Property</th>
<th>I (mm$^4$)</th>
<th>E (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joist</td>
<td>$1.93 \times 10^6$</td>
<td>200,000</td>
</tr>
<tr>
<td>Equivalent floor shell</td>
<td>$1.35 \times 10^9$</td>
<td>286</td>
</tr>
</tbody>
</table>

The joists are one-way spanning between walls, but all edges of the floor are connected to surrounding
walls. The walls parallel to the joist flooring take much less load, which was accounted for by reducing the bending stiffness of the floor shell in the direction perpendicular to the joist span as shown in Figure 5-14 where \( m \) is the bending stiffness. This allowed the floors to remain connected to all surrounding walls.

![Figure 5-14: Floor shell bending stiffness modifier](image)

Shells were meshed for analysis to form common nodes, giving an approximate load distribution from the floors to the walls. The RC transfer slab was modelled with a 400 mm thick slab element which distributed the loads to the basement columns.

As the load distribution is an approximation, the sensitivity of the assigned stiffness’s was checked. It was found that an increase or decrease of the floor and wall stiffness by a factor of 5 resulted in less than 10% variation in the reactions for each basement column. It is acknowledged that there are a number of other factors affecting the load distribution such as wall and floor finishes, structural steel elements and window and door openings, however, to model this would be cumbersome with likely limited refinement to the results. Hence, the assumptions made are considered appropriate for the purposes of this study.

### 5.6.2.4 Lateral behaviour

The lateral load resisting system consists of steel-sheathed CFS shear walls and CFS stud or structural steel chord posts. Walls are connected to the transfer slab at the base with embed plates and hold-downs to transfer tension forces, as shown in Figure 5-15.
The shear walls were designed as uncoupled, with hold-downs at the base of each shear wall chord stud. Figure 5-16 depicts half of the floor plan with shear wall locations highlighted in red.

The in-plane behaviour of the lightweight flooring system is neither fully rigid nor flexible, however; ASCE
7-10 states that for non-structural toppings with a depth of 40 mm or more, rigid diaphragm behaviour can be assumed (ASCE 2010). As the gypsum topping depth varied from 25 – 40 mm (detailed in Section 4.4.2), the flooring was assumed to act as a rigid diaphragm for this analysis. Under this assumption, the lateral forces are distributed according to the shear wall relative stiffness. Shear wall stiffness was assumed to increase linearly with length; hence, all the shear walls were assigned the same shear per metre.

The lateral load consists of both an overturning moment and a base shear which were applied separately to the model. The full building overturning moment $M_{OT}$ (earthquake and wind) was divided by the total shear wall length $L_{sw}$, resulting in a tension $T$ and compression moment couple at each shear wall, shown in Equation 5-3.

$$T = \frac{M_{OT}}{L_{sw}}$$

(5-3)

The tension and compression moment couple forces were applied directly to the transfer slab as depicted in Figure 5-17 with the green arrow showing the load direction, red lines showing the shear walls, and upwards and downwards blue arrows representing tension and compression, respectively.

Figure 5-17: Moment couple CFS shear wall actions on transfer slab
As the RC transfer slab has high in-plane stiffness, it was modelled as a rigid diaphragm, and the total base shear was applied at its centroid as illustrated in Figure 5-18 by the green and blue arrows for loading in the X and Y directions respectively.

![Figure 5-18: Base shear applied to transfer slab](image)

As there was no shear core in the CFS structure, the basement columns were fixed and designed for moment transfer into the pile caps or pad footings at the base. Figure 5-19 depicts the resultant moment and shear forces on the columns due to the ultimate load case for wind loading in the Y direction. This load combination governed moment for the circled column with a base moment of 188 kNm and corresponding shear force of 174 kN.

![Figure 5-19: Column reactions: moment (left), shear (right)](image)

The columns were dimensioned sufficiently to transfer the axial and lateral actions to the foundations with
minimum reinforcement of 1% (SA 2018b).

5.6.3 Foundations

The resultant base reactions from the superstructure analysis were used to design the foundations in accordance with Australian Standards (SA 2004, 2009, 2018b). Elastic analysis was deemed suitable for both piles (Pells 1999) and pads (Pack 2018) as they are bearing onto rock. Geotechnical assumptions included no; groundwater, hydrostatic water load on the basement, swelling, ground movements or soil contaminants. Concrete characteristic compressive strength $f_c$ was 32 MPa, and steel reinforcement yield strength $f_y$ was 500 MPa. The analysis and design methods for piles and pad footings are detailed in the following sections.

5.6.3.1 Sydney CBD - Piles

RC bored piles were designed for the Sydney CBD location in accordance with AS 2159 (SA 2009). Geotechnical, structural, serviceability and detailing requirements are presented in the following sections.

Geotechnical

Geotechnical provisions for ultimate bearing capacity were used to initially size the pile diameters. The geotechnical strength reduction factor $\phi_g$ was 0.65 in accordance with AS 2159 (SA 2009). Piles were socketed to a depth of 300 mm as per normal practice, with a clean socket and high-quality concrete at the base such that the full bearing capacity is engaged. Due to the short socket length, it is assumed that sidewall shear is not mobilised and therefore not included in strength and serviceability calculations. The design ultimate geotechnical strength of the pile group for compressive axial loading is equal to the sum of the individual pile capacities (SA 2009). Table 5-8 displays the capacity of each pile with diameters selected based on typical sizes in Australia (Pack 2018).
Table 5-8: Pile geotechnical bearing capacities

<table>
<thead>
<tr>
<th>Pile diameter (mm)</th>
<th>Base capacity (kN)</th>
<th>Factored capacity (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>450</td>
<td>1,670</td>
<td>1,085</td>
</tr>
<tr>
<td>600</td>
<td>2,969</td>
<td>1,930</td>
</tr>
<tr>
<td>750</td>
<td>4,639</td>
<td>3,015</td>
</tr>
<tr>
<td>900</td>
<td>6,680</td>
<td>4,342</td>
</tr>
<tr>
<td>1,200</td>
<td>11,875</td>
<td>7,719</td>
</tr>
</tbody>
</table>

Piles were also checked for uplift and detailed with steel reinforcement anchoring the base of the pile into the rock for cases with high overturning moments.

**Structural**

Piles were checked for structural adequacy, and reinforcement was detailed accordingly. AS 2159 (SA 2009) refers to AS 3600 (SA 2018b) column design for the ultimate strength of piles in bending and compression. A concrete placement factor $k$ of 1 was used, and the concrete cover was 45 mm for a non-aggressive exposure classification and 50-year design life.

To determine the lateral behaviour of the piles, they can be categorised as short or long. Short piles experience soil failure prior to structural failure, and for long piles, the reverse is true (Pack 2018). The 7.45 m embedment length of the piles was found to be longer than the critical length for short pile failure (Tomlinson 1994), hence all piles were assumed to behave as long piles, governed by structural failure. The piles transfer lateral loads through passive soil resistance against the pile shafts (Davisson 1970). An elastic soil model was used for the soil, which was represented by linear springs. Piles were modelled as multiple one meter long beam-column elements in ETABS to approximate the bending and shear forces. The spring stiffness was calculated by multiplying the soil subgrade modulus by the pile width. The subgrade modulus was assumed constant at 30 MN/m$^3$ for a stiff soil (Pack 2018). Table 5-9 displays the spring constants used for each pile diameter.
### Table 5-9: Spring stiffness for pile design

<table>
<thead>
<tr>
<th>Pile diameter (mm)</th>
<th>Linear spring constant (kN/m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>450</td>
<td>13,500</td>
</tr>
<tr>
<td>600</td>
<td>18,000</td>
</tr>
<tr>
<td>750</td>
<td>22,500</td>
</tr>
<tr>
<td>900</td>
<td>27,000</td>
</tr>
<tr>
<td>1,200</td>
<td>36,000</td>
</tr>
</tbody>
</table>

Isolated piles under columns were modelled with unrestrained free-head conditions, and piles under shear walls were modelled with rotation restrained fixed-head conditions under the assumption that the interconnecting shear wall ground beams provide restraint to the pile heads (Tomlinson 1994). Worst-case actions and design bending moment for out-of-position tolerance were applied at the top of each pile. Piles were then designed by treating them as a RC column (Pack 2018). Figure 5-20 depicts the resultant actions due to earthquake loading in the X direction and shear force and bending moment in the X plane for Pile 43 in the CFS structure (refer to Figure 5-26 for pile location).
The design was optimised to minimise the percentage of reinforcement and the number of bars to avoid congestion. Minimum reinforcement of 0.5% is required for fully embedded piles (SA 2009). For piles where more than 0.5% reinforcement was required, the pile diameter was increased up to a maximum of 1,200 mm. Table 5-10 shows the minimum reinforcement requirements for each pile diameter and corresponding helical confining rebar.
Table 5-10: Pile reinforcement data

<table>
<thead>
<tr>
<th>Pile diameter (mm)</th>
<th>Bar size</th>
<th>Area (mm²)</th>
<th>Bars required ( Nº )</th>
<th>Rebar (%)</th>
<th>Confining rebar diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>450</td>
<td>N16</td>
<td>201</td>
<td>4</td>
<td>0.51</td>
<td>5</td>
</tr>
<tr>
<td>600</td>
<td>N20</td>
<td>314</td>
<td>5</td>
<td>0.56</td>
<td>6</td>
</tr>
<tr>
<td>750</td>
<td>N24</td>
<td>452</td>
<td>5</td>
<td>0.51</td>
<td>10</td>
</tr>
<tr>
<td>900</td>
<td>N28</td>
<td>616</td>
<td>6</td>
<td>0.58</td>
<td>10</td>
</tr>
<tr>
<td>1,200</td>
<td>N36</td>
<td>1,020</td>
<td>6</td>
<td>0.54</td>
<td>10</td>
</tr>
</tbody>
</table>

Pile caps transfer the actions from the basement columns into the piles. As the foundation consists of isolated piles with one column and one pile per pile cap, the pile cap design was governed by reinforcement development lengths of pile and column starter bars. Pile caps were designed to extend at least 150 mm beyond the face of the pile (Cement Concrete & Aggregates Australia 2011). A minimum reinforcement was provided in both directions at the bottom face, and the vertical direction for the sides as no large flexure is present, depicted in Figure 5-21.

Ground beams underlie the basement shear walls for the RC building, whereas a tapered thickening of the basement slab to 300 mm under the lift walls was sufficient for the CFS building, as the lift walls are non-structural. For the RC structure, the ground beams were sized for development of shear wall starter bars. Reinforcement consisted of 4 N16 top and bottom longitudinal bars and N12 ties at a maximum spacing of 300 mm (SA 2018b).

**Serviceability**

Serviceability is related to the axial and lateral movement of the foundation. Load displacement behaviour of piles is usually linear for normal serviceability actions (Poulos 1987). Hence, the elastic settlement under vertical actions was calculated by the addition of the elastic deformation of the rock mass and the elastic shortening of the pile in compression (Das 2007). Soil parameters used in serviceability calculation were:

- \( E_{\text{pile}} = 30.1 \text{ GPa} \) according to AS 3600 (SA 2018b)
- \( E_{\text{rock}} = 1 \text{ GPa} \) - Class II shale (Pells et al. 1998)
- \( \nu_{\text{rock}} = 0.3 \) (Gercek 2007)

Where \( E \) is the elastic modulus and \( \nu \) is Poisson’s ratio. The settlement was calculated for single piles with...
the long term serviceability load case (Pells 1999) and compared with an allowable settlement of 5 mm provided in the geotechnical report. Lateral deflection of piles was checked using linear elastic analysis in ETABS with the equivalent line springs as described above. Serviceability wind load combination reactions were applied to the structure and piles checked for deflection under the resulting reactions.

**Detailing**
A standard detail for a column bearing onto a pile is shown in Figure 5-21 and a shear wall bearing onto a ground beam in Figure 5-21. Column and pile starter bars were detailed with sufficient development length for moment transfer in accordance with AS 3600 (SA 2018b). Table 5-11 depicts the straight development length, cogged development length and cog length. The required cog length was based on a pin of five times the bar diameter as is the most common in practice (Cement Concrete & Aggregates Australia 2011).

<table>
<thead>
<tr>
<th>Bar size</th>
<th>Straight development length (mm)</th>
<th>Cogged development length (mm)</th>
<th>Cog length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N12</td>
<td>450</td>
<td>225</td>
<td>180</td>
</tr>
<tr>
<td>N16</td>
<td>690</td>
<td>345</td>
<td>210</td>
</tr>
<tr>
<td>N20</td>
<td>950</td>
<td>475</td>
<td>260</td>
</tr>
<tr>
<td>N24</td>
<td>1,230</td>
<td>615</td>
<td>310</td>
</tr>
<tr>
<td>N28</td>
<td>1,530</td>
<td>765</td>
<td>360</td>
</tr>
<tr>
<td>N32</td>
<td>1,840</td>
<td>920</td>
<td>400</td>
</tr>
<tr>
<td>N36</td>
<td>2,150</td>
<td>1,075</td>
<td>450</td>
</tr>
</tbody>
</table>
Figure 5-21: Column bearing onto pile cap and pile detail

Figure 5-22: Shear wall and ground beam detail
5.6.3.2 Lindfield - Pad Footings
RC pad footings were designed to bear onto the shallow low strength shale for the Lindfield site. Geotechnical, structural, serviceability and detailing requirements for the pad footings are presented in the following sections.

Geotechnical
Pad footings were initially sized such that the factored ultimate load was less than the factored ultimate bearing capacity of the rock stratum. The ultimate capacity of the rock was factored by a geotechnical reduction factor $g$ of 0.65, according to AS 5100.3 (SA 2004). Linear elastic theory for combined bending and axial stress was used for analysis as the bearing material was non-yielding rock (Foster et al. 2010; Pack 2018). The maximum bearing pressure was calculated for axial loading combined with bi-axial moments. Figure 5-23 depicts the bearing pressure distribution due to the axial load $N$ and biaxial moments $M_Y$ and $M_X$. 
Starting with the minimum pad dimensions for required development length of column starter bars, the pad width was increased at 100 mm increments until an acceptable design was achieved, ensuring the maximum allowable bearing pressure was satisfied for all cases.

Checks were made in both horizontal directions for stability conditions including uplift, sliding and overturning (SA 2002a). Uplift only occurred at the shear core of the RC structure, for which reinforcement was designed to anchor into the rock. Sliding capacity was calculated based on the friction resistance of the pads against the bearing rock due to service loads as shown in Equation 5-4 (Pack 2018)

\[
F_{sub} = \phi_g \left[ \tan(0.667\phi)(N+W) \right] + A_{pad}c
\]
in which $F_{stab}$ is the stabilising force, the internal friction angle $\phi$ is 20° (Das 2007), $W$ is the weight of the pad, $A_{pad}$ is the cross-sectional plan area of the pad and the characteristic cohesion for shale $c$ is 3,000 kPa (Ghafoori 1995). Passive soil pressure was neglected for the calculation. The sliding capacity was factored by the geotechnical reduction factor and compared with worst-case lateral loads in both orthogonal directions. Lastly, overturning was checked to ensure the factored stabilising service load was greater than the worst-case applied moment on the pad.

**Structural**

Pad footings were considered to act as flexural members and were therefore designed for bending, flexural shear, punching shear and column bearing to AS 3600 (SA 2018b). A minimum cover of 40 mm was required for concrete poured against ground with a non-aggressive exposure classification in accordance with AS 2159 (SA 2009).

Bending moment capacity was calculated at a critical location 0.7 $a_{sup}$ away from the centroid of the column, as shown in Figure 5-24, where $a_{sup}$ is half the column width. A moment arm of 0.925 was multiplied by the pad depth to calculate moment capacity (Foster et al. 2010). The critical bending moment was calculated for the maximum bearing pressure reaction. Tensile reinforcement was designed to resist the critical bending moments at the base of the footing in each direction and satisfy minimum requirements for reinforced footings (SA 2018b).
Figure 5-24: Critical bending and shear check locations in pad footing

Punching shear was checked for pads where the critical perimeter was inside the perimeter of the pad, and flexural shear was calculated using the simplified method for beams in shear (SA 2018b) at the critical point, $d_o$ away from the column, where $d_o$ is the depth to the centre of the bottom reinforcement in the pad. Pad depth was increased to ensure that the concrete strength by itself was adequate, and no shear reinforcement was required (Foster et al. 2010).

**Serviceability**

Serviceability requirements of the pad footings were considered to be satisfied for pads that were sized based on the ultimate bearing pressure of the rock, settlement for pads proportioned in this way should not exceed 1% of the footing width (Douglas Partners 2013).

**Detailing**

A standard construction detail for the pad footings is provided in Figure 5-25. The reinforcement required for bending or crack control was continued on the side faces of the pad with U–shaped reinforcing bars.
5.7 Results

Concrete volume and steel reinforcement weight were calculated for both structures in each location. Figure 5-26 depicts the labelled foundation plan and axis with each number representing a foundation location. For the CFS structure, as there were no RC shear walls, foundations were not required at Locations 13, 14, 15, 16, 29, 30, 31 and 32.
The permanent and earthquake actions were significantly smaller for the CFS structure, due to the reduced self-weight. When comparing reactions of individual piles, for the RC structure, all lateral actions in both directions were governed by earthquake load combinations, while axial actions were governed by either earthquake overturning or axial load combinations. For the CFS structure, earthquake governed the X direction, whilst a mix of earthquake and wind governed the Y direction. This is because the earthquake load is the same in both directions, but the wind load on the short face of the structure is much less due to the smaller surface area.

Actions due to lateral cases were considerably higher at the base of the shear core for the RC structure, whereas the CFS structure distributed these actions throughout the basement columns and into the foundations with moment connections. The moments were generally small in comparison to the minimum imposed moment for construction tolerance. This is demonstrated with the example of the design actions for Column 23 in the Sydney CBD location as depicted in Table 5-12 for the RC structure and Table 5-13 for the CFS structure. The notation $F$ refers to force and $M$ refers to moment, followed by the axes, X, Y or Z in subscript.
Table 5-12: Column 23 load combinations for RC structure

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Direction</th>
<th>( F_X ) (kN)</th>
<th>( F_Y ) (kN)</th>
<th>( F_Z ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.35G</td>
<td>-Z</td>
<td>0</td>
<td>19</td>
<td>3,482</td>
</tr>
<tr>
<td>1.2G + 1.5Q</td>
<td>-Z</td>
<td>0</td>
<td>19</td>
<td>3,528</td>
</tr>
<tr>
<td>1.2G + ( W_u + \Psi_c )Q</td>
<td>-Z, -X</td>
<td>1</td>
<td>18</td>
<td>3,228</td>
</tr>
<tr>
<td></td>
<td>-Z, +X</td>
<td>-1</td>
<td>18</td>
<td>3,228</td>
</tr>
<tr>
<td></td>
<td>-Z, +Y</td>
<td>0</td>
<td>16</td>
<td>3,192</td>
</tr>
<tr>
<td></td>
<td>-Z, -Y</td>
<td>0</td>
<td>19</td>
<td>3,264</td>
</tr>
<tr>
<td>( G + E_u + \Psi_E )Q</td>
<td>-Z, -X</td>
<td>3</td>
<td>16</td>
<td>2,712</td>
</tr>
<tr>
<td></td>
<td>-Z, +X</td>
<td>-3</td>
<td>13</td>
<td>2,653</td>
</tr>
<tr>
<td></td>
<td>-Z, +Y</td>
<td>1</td>
<td>20</td>
<td>2,779</td>
</tr>
<tr>
<td></td>
<td>-Z, -Y</td>
<td>-1</td>
<td>10</td>
<td>2,586</td>
</tr>
</tbody>
</table>

The two strength combinations are in the negative Z (gravity) direction, the wind cases are applied in the positive and negative X and Y directions respectively, and the earthquake load cases have the primary load in the positive and negative X and Y direction with 30% of the secondary load in the perpendicular direction. The \( F_Z \) reaction is significantly higher for the column in the RC building, and the lateral reactions are very small as this column is not underlying a shear wall. The \( F_Y \) reactions are greater than the \( F_X \) reactions because the stiffer axis of the blade column is orientated in the Y direction. There are no moments |

Table 5-13: Column 23 load combinations for CFS structure

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Direction</th>
<th>( F_X ) (kN)</th>
<th>( F_Y ) (kN)</th>
<th>( F_Z ) (kN)</th>
<th>( M_X ) (kNm)</th>
<th>( M_Y ) (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.35G</td>
<td>-Z</td>
<td>2</td>
<td>75</td>
<td>1,573</td>
<td>-47</td>
<td>2</td>
</tr>
<tr>
<td>1.2G + 1.5Q</td>
<td>-Z</td>
<td>2</td>
<td>84</td>
<td>1,777</td>
<td>-52</td>
<td>2</td>
</tr>
<tr>
<td>1.2G + ( W_u + \Psi_c )Q</td>
<td>-Z, -X</td>
<td>20</td>
<td>72</td>
<td>1,500</td>
<td>-45</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>-Z, +X</td>
<td>-16</td>
<td>71</td>
<td>1,498</td>
<td>-45</td>
<td>-22</td>
</tr>
<tr>
<td></td>
<td>-Z, +Y</td>
<td>2</td>
<td>174</td>
<td>1,785</td>
<td>-188</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>-Z, -Y</td>
<td>2</td>
<td>-31</td>
<td>1,213</td>
<td>98</td>
<td>2</td>
</tr>
<tr>
<td>( G + E_u + \Psi_E )Q</td>
<td>-Z, -X</td>
<td>42</td>
<td>87</td>
<td>1,335</td>
<td>-72</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>-Z, +X</td>
<td>-39</td>
<td>32</td>
<td>1,147</td>
<td>-2</td>
<td>-52</td>
</tr>
<tr>
<td></td>
<td>-Z, +Y</td>
<td>14</td>
<td>149</td>
<td>1,542</td>
<td>-153</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>-Z, -Y</td>
<td>-11</td>
<td>-31</td>
<td>939</td>
<td>79</td>
<td>-14</td>
</tr>
</tbody>
</table>

The two strength combinations are in the negative Z (gravity) direction, the wind cases are applied in the positive and negative X and Y directions respectively, and the earthquake load cases have the primary load in the positive and negative X and Y direction with 30% of the secondary load in the perpendicular direction. The \( F_Z \) reaction is significantly higher for the column in the RC building, and the lateral reactions are very small as this column is not underlying a shear wall. The \( F_Y \) reactions are greater than the \( F_X \) reactions because the stiffer axis of the blade column is orientated in the Y direction. There are no moments.
in Table 5-12 as the columns in the RC structure are pinned. The column for the CFS structure has higher lateral reactions as the wind and earthquake loads are distributed to all the columns and the fixed base connection transfers a moment.

Design results are detailed below through comparison of the concrete and steel volumes required for each design. The concrete volume consists of the amount of concrete below the basement slab in cubic meters and the steel accounts for all steel below the basement slab excluding the column starter bars, in tonnes.

5.7.1 Sydney CBD pile results

Piles designed for the Sydney CBD demonstrated a significant reduction in material for the CFS structure as depicted in Table 5-14. The CFS structure required fewer piles with smaller diameters, resulting in a reduction of concrete volume of 42% and a reduction of steel reinforcement of 45%.

Table 5-14: Pile concrete volume and steel weight

<table>
<thead>
<tr>
<th></th>
<th>RC</th>
<th>CFS</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piles (№)</td>
<td>44</td>
<td>36</td>
<td>18%</td>
</tr>
<tr>
<td>Concrete (m³)</td>
<td>228.1</td>
<td>132.4</td>
<td>42%</td>
</tr>
<tr>
<td>Steel (tonne)</td>
<td>14.6</td>
<td>8.1</td>
<td>45%</td>
</tr>
</tbody>
</table>

Table 5-15 depicts the number of piles of each pile diameter for both RC and CFS, and Figure 5-27 provides a plan view of the pile diameters for the RC and CFS structures.

Table 5-15: Pile number comparison for RC and CFS

<table>
<thead>
<tr>
<th>Pile Diameter (mm)</th>
<th>Number of piles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RC</td>
</tr>
<tr>
<td>450</td>
<td>2</td>
</tr>
<tr>
<td>600</td>
<td>16</td>
</tr>
<tr>
<td>750</td>
<td>9</td>
</tr>
<tr>
<td>900</td>
<td>13</td>
</tr>
<tr>
<td>1,200</td>
<td>4</td>
</tr>
<tr>
<td>Total piles</td>
<td>44</td>
</tr>
</tbody>
</table>
Table 5-16 and Table 5-17 provide the pile diameters and magnitude of the worst-case actions for Piles 1 - 24 in the RC and CFS building respectively. As the building is symmetrical, the loads are the same for the other half of the piles and hence not shown.
### Table 5-16: RC piles with governing actions

<table>
<thead>
<tr>
<th>Pile</th>
<th>Pile diameter (mm)</th>
<th>$F_X$ (kN)</th>
<th>$F_Y$ (kN)</th>
<th>$F_Z$ (kN)</th>
<th>$M_X$ (kNm)</th>
<th>$M_Y$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>450</td>
<td>6</td>
<td>29</td>
<td>842</td>
<td>63</td>
<td>63</td>
</tr>
<tr>
<td>2</td>
<td>600</td>
<td>7</td>
<td>20</td>
<td>1,829</td>
<td>137</td>
<td>137</td>
</tr>
<tr>
<td>3</td>
<td>750</td>
<td>15</td>
<td>29</td>
<td>2,234</td>
<td>168</td>
<td>168</td>
</tr>
<tr>
<td>4</td>
<td>600</td>
<td>6</td>
<td>41</td>
<td>1,391</td>
<td>104</td>
<td>104</td>
</tr>
<tr>
<td>5</td>
<td>600</td>
<td>5</td>
<td>21</td>
<td>1,484</td>
<td>111</td>
<td>111</td>
</tr>
<tr>
<td>6</td>
<td>900</td>
<td>4</td>
<td>27</td>
<td>3,472</td>
<td>260</td>
<td>260</td>
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Table 5-17: CFS piles with governing actions

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<th>$F_Y$ (kN)</th>
<th>$F_Z$ (kN)</th>
<th>$M_X$ (kNm)</th>
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</table>

Each column is founded on a single isolated pile. For the RC building, the shear walls have a pile on either end, with large 1,200 mm diameter piles required at Locations 12, 20, 28 and 36 (refer to Figure 5.26 for these points) due to high torsional effects in earthquake load cases. These piles also required anchoring against uplift through the use of reinforcement embedded in the founding rock. For the CFS structure, four piles were sufficient for each lift core due to the lateral load distribution. The ultimate geotechnical bearing capacity governed pile diameter for most cases in the RC structure with only two piles requiring an increased diameter for structural requirements. This is due to the high lateral loads being localized at the shear core. The CFS structure required an increase in diameter for 13 of the piles due to the high distribution.
of lateral loads.

Minimum reinforcement of 0.5% was designed for all cases. All individual pile settlements were less than the allowable 5 mm as per the geotechnical report (Douglas Partners 2015), and maximum lateral displacement was less than 4 mm. Pile cap sizing was governed by detailing requirements including minimum development and cog length for both the column and pile starter bars. Pile caps were reinforced with a minimum of N16 bars at 200 mm on-centre for the base and sides.

5.7.2 Lindfield pad footing results

Isolated pad footing designs for the CFS structure resulted in a material reduction of 40% for concrete volume and 59% for steel reinforcement weight when compared with those for the RC structure as depicted in Table 5-18.

Table 5-18: Pad footing concrete volume and steel weight

<table>
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<tr>
<th>Structure</th>
<th>RC</th>
<th>CFS</th>
<th>Reduction</th>
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<tbody>
<tr>
<td>Pads (#)</td>
<td>44</td>
<td>36</td>
<td>18%</td>
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<tr>
<td>Concrete volume (m³)</td>
<td>62.5</td>
<td>37.7</td>
<td>40%</td>
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<tr>
<td>Steel weight (tonne)</td>
<td>5.2</td>
<td>2.1</td>
<td>59%</td>
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Figure 5-28 depicts the pad footing design for RC and CFS. Plan dimensions of the blue pads were governed by detailing requirements, whereas red pads required increased dimensions to satisfy geotechnical bearing requirements.
Table 5-19 and Table 5-20 provide the pad plan dimensions (x in the X direction, y in the Y direction), depth, maximum axial load and critical moments in the X ($M_x^*$) and Y ($M_y^*$) direction for Pads 1 - 24 in the RC and CFS building respectively.
Table 5-19: RC pad dimensions governing axial load and critical moments

<table>
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<tr>
<th>Pad</th>
<th>x (m)</th>
<th>y (m)</th>
<th>Depth (m)</th>
<th>Fz (kN)</th>
<th>Mx* (kN/m/m)</th>
<th>My* (kN/m/m)</th>
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</table>

Pad footings were designed under each of the columns and wall-ends with ground beams connecting pads under the wall spans. Plan dimensions of the pads were typically governed by detailing requirements in the CFS structure with only eight pads requiring increased area to satisfy bearing capacity. Thirty of the pads required increased dimensions for bearing capacity in the RC building as depicted in Figure 5-28.
Pad depth was governed by column starter bar reinforcement requirements in most cases; however, 12 pads in the RC structure were thickened to satisfy the flexural shear requirements without the use of extra shear reinforcement. Punching shear and column bearing was not critical in any cases. Stability limit state checks did not govern the size of any footings, though extra anchorage reinforcement embedded in rock was required in the RC structure for pads at Locations 12, 20, 28 and 36.

Bottom reinforcement was provided in both directions as the pads are subject to two-way bending. In most cases, the area of steel was governed by minimum requirements for the CFS structure with only five pads

### Table 5-20: CFS pad dimensions governing axial load and critical moments

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<th>Pad</th>
<th>x (m)</th>
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<th>Fz (kN)</th>
<th>Mx* (kN/m/m)</th>
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<td>1.76</td>
<td>0.56</td>
<td>1,785</td>
<td>81</td>
<td>146</td>
</tr>
<tr>
<td>24</td>
<td>0.76</td>
<td>1.76</td>
<td>0.56</td>
<td>1,581</td>
<td>71</td>
<td>118</td>
</tr>
</tbody>
</table>
requiring more. Twenty-eight of the pads in the RC structure required more reinforcement to increase the moment capacity.

5.8 Discussion

This chapter has extended on the findings from Chapter 4 through a comparative study of the foundation design for a RC and a CFS structure. The value in this comparison is the basis in an Australian context and the quantification of the benefits of a lightweight structure. The locations selected are representative of the larger Australian capital cities, where the majority of mid-rise development activity takes place (Shoory 2016). Both the Sydney CBD and Lindfield scenarios demonstrated material savings for foundations with a CFS structure. This is in agreement with literature suggesting foundation savings for lightweight structures sizes (Timber Development Association NSW and Forsythe 2015).

The results demonstrated that the volume of materials required for pad footings (Table 5-18) was much less than that for the pile design (Table 5-14), however, the percentage reductions between the two structures were similar. The savings for CFS over RC in concrete volume and steel reinforcing for the piles were similar at 42% and 45%, respectively, as the piles all had the same reinforcement rate of 0.5%. The pad footings had a more significant difference, with a reduction of 40% and 59%, for concrete volume and steel reinforcement, respectively, for the CFS foundation design. The percentage of reinforcing steel in the pads for the RC structure was significantly higher because 28 of the pads required more than minimum reinforcement. The depth of the pads could have been increased to improve their moment capacity; however, this would require digging deeper into rock, which could be expensive in comparison to the cost of additional flexural reinforcement.

The reduction in concrete volume was slightly less significant for the shallow foundations at the Lindfield site as compared with the piles at Sydney CBD. Figure 5-29 shows the percentage of pad footings that were governed by detailing requirements for column starter bars, which was the same for both structures. The two factors governed a different dimension of the pad, as shown in brackets. A higher percentage means more pads were governed by detailing, indicating a less efficient design.
Figure 5.29: Percentage of pads governed by bearing and shear

Geotechnical bearing determined the plan area of the pad; 80% of pads in the CFS structure were governed by minimum size for column reinforcement requirements, whereas only 30% were for RC. The depth of the pad was determined by shear requirements, which also had a higher percentage for the CFS structure. This demonstrates that the CFS structure has a larger margin to increase design efficiency than the RC structure and that the material saving differential is greater when the ultimate limit state bearing capacity is governing rather than minimum detailing requirements. This could be achieved by minimising the size and number of columns in the basement through a more optimised design.

The importance of substructure design can be highlighted by the impact it has on excavation, trucking and landfill costs. Excavation in the Sydney region would generally require an excavator for removal of overburden soils and weathered rock, a large dozer with a ripper for excavation into stronger rock and boring equipment for piles. Excavated material would likely be removed to landfill incurring high trucking and landfill costs. Excavated soil is classified depending on the level of contamination, with soils with higher contamination levels requiring disposal at a licensed waste facility or remediation on-site. Soil at the highest levels of contamination, classified as hazardous waste, can cost around $1,000 per tonne to dispose of (Geo-Logix 2016). In cases where dewatering of the soil is required, wastewater must be treated before being discharged, incurring further costs and written approval (Sydney Water 2019). For the present
The lateral behaviour of lightweight flooring systems is not well understood, with diaphragms behaving somewhere in between rigid and flexible depending on the stiffness of building components. This has a significant effect on the distribution of loads to shear walls. Designing for both a rigid and flexible diaphragm envelope would be conservative, so a rigid diaphragm assumption was considered suitable for the purposes of this comparison. Research into the behaviour of the CFS structures with lightweight flooring is explored further in Chapter 6. The ability for CFS structures to transfer the loads through the RC transfer slab also requires further analysis. For a building designed using purely CFS sections for lateral resistance, chord studs are required to be large built-up sections, as large as 800S250-97 for a four-storey building (Torabian et al. 2016). Hold-down resistance is also important to consider as the tension forces in chord studs are directed through them into the transfer slab.

As explained in Chapter 4, the archetype building design was developed to be typical of Australian mid-rise residential buildings; it is more suitable for a RC frame than loadbearing CFS construction. As the concrete shear core in the basement of the CFS structure was redundant, it was replaced with four corner columns. Further optimisation would be possible for the CFS substructure to take advantage of the CFS strengths, which would reduce the permanent actions imposed on the structure. As the CFS shear and loadbearing walls were not designed to align with the basement carpark layout below, a deep podium was necessary to transfer the loads. The podium and carpark constitute a significant 37% of the permanent actions imposed on the foundation of the CFS structure. An optimised design of the transfer structure could lead to a reduction in slab depth, column dimension and number of required columns, further minimising pile and pad footing volumes for the CFS building. However, the current design represents a realistic
situation in the current Australian market, as mid-rise residential buildings in Australia are generally designed for a RC frame. Therefore, the loadbearing CFS system is required to be adaptable to accommodate designs that are not optimised for CFS.

5.9 Conclusions

This chapter compared deep and shallow rock bearing foundation designs for an archetype mid-rise apartment building with a RC and a CFS structure in two locations. Due to the smaller permanent load of the CFS structure, the permanent and earthquake design actions are significantly less than those for the RC structure, leading to less demand on the building foundations. The material savings were quantified with foundation designs for the CFS structure demonstrating a reduction of concrete volume and steel reinforcement of up to 42% and 59%, respectively. This can lead to an increase in construction productivity as less time is required on-site for substructure works.

This study establishes that interchanging the structure of a building from RC to CFS can lead to considerable material savings for mid-rise apartment building projects. For a building designed to exemplify a CFS structure, this effect could potentially be amplified to achieve greater savings. This research could be extended through comparison of an exemplary mid-rise CFS structure with a RC structure and other structures such as CLT with more holistic comparison criteria.

Overall, the archetype study of Chapters 4 and 5 has established that in terms of design loads and foundation designs in an Australian context, the CFS structure compares favourably with the RC structure. The following chapter of this thesis details a study aiming to better understand the serviceability behaviour of a loadbearing CFS structure and develop a method to simplify the design process of these structures to encourage mid-rise loadbearing CFS use in Australia.
Chapter 6 - Efficient 3D lateral analysis study

6.1 Introduction

This chapter presents the final study of the thesis which aims to facilitate the structural design process for loadbearing CFS. As expressed in the previous chapters, the use of CFS members for loadbearing applications in mid-rise buildings has been limited in Australia. Improvements to the analysis and design procedures, in terms of efficiency and accuracy of CFS buildings could lead to greater uptake in the Australian construction industry. Non-structural components contribute significantly to the lateral stiffness of a CFS building structure but are cumbersome to model explicitly in the structural analysis. They are therefore commonly ignored in a 3D structural analysis, and their benefits to the design are lost. Hence, this chapter proposes an efficient modelling method that incorporates the full-system behaviour and can be used in practice for serviceability limit state checks.

A literature review reveals the previous experimental and modelling work undertaken in the field, from component level to full-system behaviour. The methodology section describes the development of the proposed equivalent shear modulus method. Verification against seismic tests of a two-storey CFS structure in the USA is then detailed. Lastly, the storey drift under wind loading is checked for a modified version of the archetype building to demonstrate the application of this method.

6.2 Literature review

Historically, CFS lateral load resisting design codes were based on component-level design (Peterman et al. 2016b), which does not take into account the stiffening effect of non-structural wall sheathing and other components which are essential for fire and acoustic compliance. The AISI standards in the USA were updated in 2015 to include system-level design; however, they do not encompass the full-system behaviour (Schafer et al. 2015). In Australia, plasterboard can be used for lateral bracing in CFS structures; however, this is only for low-rise and detached residential buildings (NASH 2010). Research is active in improving the understanding of roof and floor diaphragm behaviour and modelling of the full building seismic response (Schafer et al. 2016). Experimental testing and characterisation of CFS framed building components such as sheathed and strap-braced walls have progressed significantly, as detailed in Schafer et al. (2016) and Sharafi et al. (2018).
Within the timber industry in the USA, full-scale building testing has been carried out (van de Lindt et al. 2011), resulting in improvements to design standards and modelling methods. Since then, full-scale tests of CFS building structures were undertaken in China (Li et al. 2014), Italy (Fiorino et al. 2017) and at the University at Buffalo, USA, as a part of the National Science Foundation-funded Network for Earthquake Engineering Simulation (NEES) project: CFS-NEES (Schafer et al. 2016). The NEES project involved full-scale two-storey shake table tests of a CFS building at its different stages of construction. The fully completed building’s response was up to 16 times stiffer than that of the earlier construction, which was without non-structural components (Leng et al. 2017). The significant knowledge gap between system performance and current understanding, which is predominantly at the component level, was noted (Leng et al. 2017). This result demonstrates the considerable and beneficial effect of non-structural components and highlights the importance of modelling non-structural components in the structural analysis.

To capture the global behaviour of the structure, FEA is required (Martínez-Martínez and Xu 2011). This is associated with a high computational cost, so a number of research efforts have been undertaken to better understand the behaviour and simplify the modelling methods. Shear walls in CFS building structures are commonly sheathed with OSB, which is rigid in-plane while the wall panel deforms through the flexibility of the sheathing connections. This behaviour has been replicated by Buonopane et al. (2014) with the shear walls being modelled using a rigid diaphragm attached to nonlinear spring elements simulating the fasteners. Bian et al. (2015) extended this research, using the model to analyse shear and gravity wall systems. A number of other studies have used a similar method involving the modelling of shear wall connections (Bian et al. 2015; Buonopane et al. 2014; Derveni et al. 2019; Fiorino et al. 2018; Niari et al. 2015; Telue and Mahendran 2004; Zhou et al. 2010). This fastener-based method can accurately simulate the behaviour of a shear wall but involves a large number of degrees of freedom which may be impractical for full-scale multi-storey building simulations, and may not be applicable to strap-braced shear walls.

To simplify the modelling of shear wall behaviour, shear wall panels have been modelled using linear frame elements and nonlinear diagonal braces as an equivalent truss (Fiorino et al. 2018; Fülöp and Dubina 2004; Kechidi and Bouraha 2016; Leng 2015; Shamim and Rogers 2013). Fülöp and Dubina (2004) developed a trilinear hysteretic model using frame elements to simulate single shear walls. Leng et al. (2017) produced a number of models of varying fidelity using OpenSees for the CFS-NEES building with the characterisation of nonlinear behaviour for shear walls, gravity walls, interior walls, and semi-rigid floor
and roof diaphragms. Wall components were modelled, taking into account the pinching effect in the hysteretic response on the basis of design equations and sub-system level tests. Leng et al. (2017) extended the use of equivalent beam-column elements by modelling sheathing subpanels. Li et al. (2014) created a shear wall model based on the restoring force of wall fasteners, represented as an equivalent bracing. This method has reduced degrees of freedom compared to the fastener-based method; however, many elements are still required for each wall in the building.

Further simplification of the shear wall representation has been developed by Martínez-Martínez and Xu (2011) with the use of a 16-node equivalent modulus shell element model without hold-downs. The equivalent modulus is a representation of the sheathing and stud properties. A three-storey building was modelled using this method, and the results for lateral displacements compared favourably against a conventional FEA model. However, no comparisons were made against laboratory test results. A more in-depth review of proposed numerical models can be found in a recent paper by Usefi et al. (2019).

This study proposes the use of an equivalent shear modulus four-node orthotropic shell element to model each wall for the purpose of simulating the elastic sway behaviour of a CFS multi-storey building, incorporating the lateral stiffness contributions of shear walls as well as non-structural components. The FE modelling herein will focus on accurately simulating the elastic stiffness of the system with a view to facilitating the serviceability limit state design of CFS mid-rise buildings under lateral loading such as wind.

The advantages of the proposed modelling method are twofold. Firstly, the proposed method can be used whether the sheathing is OSB, gypsum or steel sheeting, and can also be used for strap-braced shear walls. Secondly, the proposed method is significantly more efficient than existing modelling methods (Bian et al. 2015; Buonopane et al. 2014; Fiorino et al. 2018; Fülöp and Dubina 2004; Kechidi and Bourahla 2016; Leng et al. 2017; Li et al. 2014; Niari et al. 2015; Shamim and Rogers 2013; Telue and Mahendran 2004; Zhou et al. 2010).

6.3 Methodology

The efficient 3D lateral analysis study is the final part of the thesis, with the aim to facilitate the structural design process for mid-rise CFS and objectives to:

- Develop an efficient method to simulate full-system serviceability behaviour of a CFS shear wall
Demonstrate the application of the method to a mid-rise apartment structure.

It is believed that a contribution can be made through simplification and improvement of the structural design of CFS structures. Hence, the research presented in this chapter aims to support the improvement of the structural design process of CFS structures with the development of an efficient modelling method for serviceability design checks.

Each shear or gravity wall is represented by an equivalent shear modulus four-node orthotropic shell element, which incorporates the lateral stiffness (or flexibility) contributions of all components including sheathing, braces and fasteners as present in the wall. The equivalent shear modulus can be determined from the experimental test of a representative wall panel, or from the analysis of a finely detailed finite element model of the panel.

Due to the absence of quasi-static tests, the resulting building model was verified against the results of shake table tests undertaken at low-level seismic excitations (Schafer et al. 2016), with respect to the natural period, peak storey drift and peak floor acceleration. These measures are used for verification as they relate to the stiffness of the structure, which is used for shear wall design and serviceability drift calculations.

Analyses were conducted at two different construction phases in order to demonstrate the lateral stiffness contributions of gravity walls and their elements. The model is subsequently demonstrated through the application for the archetype building. The model and its components are detailed in the following subsections.

6.3.1 Shear and gravity walls

An equivalent shear modulus four-node orthotropic shell element is used to model each of the CFS shear and gravity walls. Figure 6-1 depicts a four-node shell element of height $h$ and width $b$, modelling a wall of the same dimension. The thickness $t$ of the shell element is the same as that of the wall. The deflection of the shell element (i.e. wall) under a horizontal load $F$ is also depicted in the figure, where $\delta_s$ is the deflection due to shear deformation of the shell element and $\delta_{HD}$ is the upward deflection of the hold-down.
The total deflection $\delta_T$ of the shell element is given in Equation 6-1 where $k_T$ is the total lateral stiffness

$$\delta_T = \delta_S + \frac{h}{b} \delta_{HD} = \frac{F}{k_T}$$  \hfill (6-1)

The hold-down deflection $\delta_{HD}$ is equal to

$$\delta_{HD} = \frac{h}{b} \frac{F}{k_{HD}}$$ \hfill (6-2)

in which the $k_{HD}$ is the stiffness of the hold-down. The deflection $\delta_S$ due to shear deformation is given by Equation 6-3 where $G_{xz}$ is the equivalent (in-plane) shear modulus of the shell element

$$\delta_S = \frac{F h}{G_{xz} tb}$$ \hfill (6-3)

Equations 6-2 and 6-3 are substituted into Equation 6-1 to give the equivalent shear modulus

$$G_{xz} = \frac{h}{tb \left( \frac{th^2}{k_T} + \frac{bk_{HD}}{b} \right)}$$ \hfill (6-4)
The total stiffness $k_T$ can be obtained from a laboratory test using Equation 6-1, as illustrated in Section 6.4.1. Alternatively, it can also be obtained from the analysis of a finely detailed finite element model of the wall panel.

### 6.3.2 Hold-downs

Hold-downs were modelled using spring and gap-link elements at the two base nodes of the shell element. Hold-downs have very high stiffness in compression, which is simulated by the gap element. The gap element is massless and is assigned a high stiffness of $10^7$ kN/m in the Z direction. In tension, the spring acts with a constant stiffness of 9.93 kN/m for the shear wall analysed in the present work (Leng 2015). The hold-downs for gravity walls were modelled with one-tenth of the stiffness of the shear wall hold-downs (Bian et al. 2015). Shear anchors were not modelled as the hold-downs are restrained in the horizontal directions.

### 6.4 Verification

To verify the proposed methodology, a two-storey building (Schafer et al. 2016) was modelled in ETABS at two construction phases, and the analysis results were compared with the experimental data and the beam-column analysis results of Leng (2015) for the natural period, peak storey drift and peak floor acceleration (Phase 1: A1-3D-SDa, Phase 2b: A2b-3D-SDa). Shake table test results were used for verification due to the lack of quasi-static tests of full-scale CFS buildings.

### 6.4.1 Tested wall stiffness values

The wall stiffness values were derived from the shear and unsheathed gravity wall tests undertaken as part of the CFS-NEES project (Liu et al. 2012, 2014). The tested components had construction details consistent with those used in the full-scale shake table test (Schafer et al. 2016). The experimental shear wall specimen with OSB sheathing is shown in Figure 6-2.
A number of CFS wall types were subjected to monotonic and cyclic loading under displacement control (Liu et al. 2014). The specimens were 2.74 m high and 1.22 m or 2.44 m long, with either OSB sheathing, gypsum sheathing or no sheathing. The walls were framed with 600S162-54 studs and S/HDU6 hold-downs with a specified stiffness of 9.93 kN/mm. In the current study, the stiffness was estimated at a deflection equal to the highest expected deflection of the full-scale building when tested under low-level seismic excitation (Peterman 2014). The total stiffnesses $k_T$ of the 1.22 m and 2.44 m long OSB sheathed shear walls with ledger were found to be 0.785 kN/mm and 2.47 kN/mm, respectively (Liu et al. 2012).

**6.4.2 CFS-NEES shake table test**

Shake table testing was undertaken for a full-scale two-storey CFS framed building as part of the CFS-NEES project (Schafer et al. 2016). The building was designed using the allowable strength method for a location in Orange County, California. The specimen was 7 m by 15.2 m in plan and 5.8 m in height. There were several CFS shear walls and gravity walls with and without window openings. OSB sheathed CFS joist floor and roof panels were used in a ledger framing system. The shear walls utilised back-to-back chord studs and were designed as laterally decoupled, requiring a hold-down at both ends of each wall segment. Simpson Strong-Tie S/HDU6 hold-downs (Simpson Strong-Tie Company Inc 2019) were used at the base of the shear walls on the ground floor. Steel straps were used to transfer horizontal forces from the upper to the lower chords.

Testing was undertaken at a number of different construction phases (1, 2a, 2b, 2c, 2d, 2e) aiming to
quantify the effects of non-structural components on the building’s structural performance. Two of the phases were analysed for verification, denoted as Phase 1 and Phase 2b.

![Figure 6-3: Phase 1 isometric drawing (Source: (Leng et al. 2017))](image)

Phase 1 consisted mainly of structural elements only: single-sided OSB sheathed shear walls, roof, floor and unsheathed gravity walls, as illustrated in Figure 6-3. Phase 2b had the addition of exterior OSB sheathing on the gravity walls (i.e. sheathed on one side only). However, the two phases were designed to have similar total weights on the shake table through the use of supplemental concrete blocks and steel plates. The only (primary) variable affecting the frame’s responses under shaking was, therefore, the system stiffness. Table 6-1 details the construction differences between the two phases. Field stud refers to any stud that is not a chord stud.
Two ground motion records of the 1994 Northridge earthquake were used in the tests (Schafer et al. 2016) at a number of scaling levels, being from the Canoga Park and the Rinaldi receiving stations as provided by the Pacific Earthquake Engineering Research Center (PEER). The unscaled ground motions are shown in Figure 6-4, while the actual peak ground accelerations (PGA) analysed in the present work are given in Table 6-2. As indicated in the table, different ground motions were applied in the short and long directions.

### Table 6-1: Construction details for Phase 1 and Phase 2b buildings

<table>
<thead>
<tr>
<th></th>
<th>Phase 1</th>
<th>Phase 2b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing</td>
<td>OSB</td>
<td>OSB</td>
</tr>
<tr>
<td><strong>Floor Diaphragm</strong></td>
<td>Joist: 1200S200-97</td>
<td>●</td>
</tr>
<tr>
<td><strong>Roof Diaphragm</strong></td>
<td>Joist: 1200S200-54</td>
<td>●</td>
</tr>
<tr>
<td><strong>Shear wall 1</strong></td>
<td>Chord: 2 x 600S162-54</td>
<td>●</td>
</tr>
<tr>
<td></td>
<td>Field: 600S162-54</td>
<td>●</td>
</tr>
<tr>
<td><strong>Shear wall 2</strong></td>
<td>Chord: 2 x 600S162-54</td>
<td>●</td>
</tr>
<tr>
<td></td>
<td>Field: 600S162-33</td>
<td>●</td>
</tr>
<tr>
<td><strong>Gravity wall 1</strong></td>
<td>600S162-54</td>
<td>○</td>
</tr>
<tr>
<td><strong>Gravity wall 2</strong></td>
<td>600S162-33</td>
<td>○</td>
</tr>
<tr>
<td><strong>Interior wall</strong></td>
<td>362S162-54</td>
<td>○</td>
</tr>
</tbody>
</table>

● = OSB present; ○ = OSB absent

### Table 6-2: Seismic record data

<table>
<thead>
<tr>
<th>PEER Record</th>
<th>CNP106</th>
<th>CNP196</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Direction</strong></td>
<td>Y</td>
<td>X</td>
</tr>
<tr>
<td><strong>Axis</strong></td>
<td>Short</td>
<td>Long</td>
</tr>
<tr>
<td><strong>Actual Phase 1 Excitation PGA (g)</strong></td>
<td>0.061</td>
<td>0.083</td>
</tr>
<tr>
<td><strong>Actual Phase 2b Excitation PGA (g)</strong></td>
<td>0.061</td>
<td>0.085</td>
</tr>
</tbody>
</table>
In the present work, the natural period was determined using free vibration analysis in ETABS. Nonlinear time history analysis was used to determine the peak storey drift and the peak floor acceleration. Nonlinear analysis was required for the hold-down elements to act in both tension and compression. The time step used for time-history analysis was 0.001 seconds, one-tenth of the input acceleration time history data and a damping ratio of 5% was used for both phases. Analyses were undertaken using factored seismic records to match the actual Phase 1 and Phase 2b PGA from the shake table tests (Peterman 2014).

### 6.4.2.1 Modelling walls, floor and roof

Figure 6-5 shows the present building model in ETABS, with the blue colour denoting shear walls and the grey colour, gravity walls. In the tests, chord studs were connected between storeys with a steel strap, this was modelled with a shared node between the upper and the lower shell elements.
The shear moduli for various lengths of shear walls in the building were determined through interpolation between the 1.22 m and 2.44 m wide shear walls obtained from the laboratory tests (Liu et al. 2012), shown in Figure 6-6.
For the sheathed gravity walls, due to the absence of laboratory tests, the elastic stiffness was assumed to be the same as the OSB sheathed shear walls. Walls shorter than 1.2 m were assumed to have the same stiffness per meter value as the 1.2m wall.

Sheathed gravity walls with openings naturally have a reduced stiffness. As there is no test data available for such walls, the shear modulus of a wall with an opening was factored by the square root of the proportion of the wall area that is sheathed. For example, a wall with a 55% sheathed area has a shear modulus equal to 74% of that of a fully sheathed wall that was tested (Liu et al. 2012), as illustrated in Figure 6-7. It will not be appropriate to linearly factor the shear modulus as an opening is typically located centrally within the wall, without reducing the wall width.
The vertical elastic modulus of the shell element $E_z$ was initially assumed to be 5 GPa as it was around the typical value for OSB panels (Cai and Ross 2010). However, the value can also be justified through the concept of effective shell areas in compression and tension due to overturning moment. An ‘effective area’ is the area on either the windward or leeward side of the shell element that has an equivalent axial resistance to the chord stud. For a 164 mm thick shear wall with back-to-back 600S162-54 chord studs, the effective width is equal to 157 mm, which the author considers reasonable. Figure 6-8 depicts the equivalent shell and effective lengths in tension and compression for this example.

For the gravity wall with a single 600162-54 stud as an end post, used in the shake table test (Liu et al. 2012), the vertical elastic modulus was equal to 2.7 GPa for the same effective width. For both shear and
gravity walls, the horizontal elastic modulus was assumed to be 500 MPa, or one-tenth of the vertical elastic modulus of a shear wall as the wall’s horizontal stiffness does not benefit from the presence of studs. Tables 6-3 and 6-4 list the properties of the wall components in Phases 1 and 2b, respectively.

Table 6-3: Phase 1 wall components

<table>
<thead>
<tr>
<th>Component</th>
<th>t (mm)</th>
<th>Gxz (MPa)</th>
<th>Ez (GPa)</th>
<th>Mass (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Wall 1</td>
<td>164</td>
<td>17.9 – 24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Shear Wall 2</td>
<td>164</td>
<td>17.9 – 24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Gravity Wall 1</td>
<td>152</td>
<td>0.3</td>
<td>2.7</td>
<td>77</td>
</tr>
<tr>
<td>Gravity Wall 2</td>
<td>152</td>
<td>0.3</td>
<td>2.7</td>
<td>34</td>
</tr>
<tr>
<td>Interior Wall</td>
<td>92</td>
<td>0.5</td>
<td>2.7</td>
<td>65</td>
</tr>
</tbody>
</table>

Table 6-4: Phase 2b wall components

<table>
<thead>
<tr>
<th>Component</th>
<th>t (mm)</th>
<th>Gxz (MPa)</th>
<th>Ez (GPa)</th>
<th>Mass (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Wall 1</td>
<td>164</td>
<td>17.9 – 24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Shear Wall 2</td>
<td>164</td>
<td>17.9 – 24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Gravity Wall 1</td>
<td>152</td>
<td>19.3 – 28.1</td>
<td>2.7</td>
<td>103</td>
</tr>
<tr>
<td>Gravity Wall 1 - opening</td>
<td>152</td>
<td>14.3 – 20.8</td>
<td>2.7</td>
<td>103</td>
</tr>
<tr>
<td>Gravity Wall 2</td>
<td>152</td>
<td>14.4 – 21.1</td>
<td>2.7</td>
<td>56</td>
</tr>
<tr>
<td>Gravity Wall 2 - opening</td>
<td>152</td>
<td>10.7 – 15.6</td>
<td>2.7</td>
<td>56</td>
</tr>
<tr>
<td>Interior Wall</td>
<td>92</td>
<td>0.5</td>
<td>2.7</td>
<td>65</td>
</tr>
</tbody>
</table>

For simplicity, parapets were not explicitly modelled, but their mass was accounted for as described in the next paragraph. The floor and roof were modelled as rigid diaphragms using four-node shell elements that correspond to the wall panel edges. Due to the ledger framing system, the joints between the floor and the walls were assumed to be pinned.

The masses of the walls, floor and roof were modelled in ETABS through the use of elemental mass and additional mass. Elemental mass is the self-weight of the components. The elemental masses of the shell elements are determined by calculating the mass per volume based on the information provided by Peterman (2014). In calculating the wall mass for Phase 2b, it was assumed there were no openings, so the weight
was distributed evenly throughout each wall type. Additional masses representing the parapet and dummy concrete blocks were then applied as a surface load in ETABS, equal to that in the testing (Peterman 2014). The seismic masses of the roof and floor in Phases 1 and 2b are given in Table 6-5.

Table 6-5: Seismic masses

<table>
<thead>
<tr>
<th></th>
<th>Phase 1 (kg)</th>
<th>Phase 2b (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>13,370</td>
<td>13,384</td>
</tr>
<tr>
<td>Floor</td>
<td>18,949</td>
<td>19,263</td>
</tr>
</tbody>
</table>

6.4.2.2 Verification Results

The first mode natural periods were determined for both the long (X) and the short (Y) directions. Figure 6-9 depicts the first mode shapes.

![Figure 6-9: First mode of vibration in the short and long directions](image)

The first mode natural periods found in the shake table tests and the analyses are shown in Tables 6-6 and 6-7 for Phases 1 and 2b, respectively. The percentages in brackets denote the analysis deviations from the test results.

Table 6-6: Phase 1 first mode natural periods:

<table>
<thead>
<tr>
<th>Direction</th>
<th>Test (Schafer et al. 2016) (s)</th>
<th>Present (s)</th>
<th>Leng (2015) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X - Long</td>
<td>0.32</td>
<td>0.32 (0%)</td>
<td>0.30 (-6%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>0.36</td>
<td>0.36 (0%)</td>
<td>0.32 (-11%)</td>
</tr>
</tbody>
</table>
### Table 6-7: Phase 2b first mode natural periods

<table>
<thead>
<tr>
<th>Direction</th>
<th>Test (Schafer et al. 2016) (s)</th>
<th>Present (s) (+%)</th>
<th>Leng (2015) (s) (+%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X - Long</td>
<td>0.20</td>
<td>0.21 (+5%)</td>
<td>0.22 (+10%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>0.30</td>
<td>0.32 (+7%)</td>
<td>0.28 (-7%)</td>
</tr>
</tbody>
</table>

It can be seen that the present analysis results are closer to the test results than Leng’s (2015) for both construction phases. Both models were able to simulate the significant decreases in the natural periods due to the increased building lateral stiffness from Phase 1 to Phase 2b, reflecting the effects of gravity walls and their elements.

Each peak storey drift was determined by averaging the corresponding displacements of the four corner nodes and dividing the result by the storey height. The X displacement was measured under seismic excitation in the long direction (CNP196), and the Y displacement was measured under excitation in the short direction (CNP106). Due to the small magnitude and susceptibility to random noise, the displacement in the perpendicular direction of each excitation is not considered. Figure 6-10 shows the present analysis results for the lower storey drift in Phase 1.

![Figure 6-10: Phase 1 lower storey drift in the long direction](image)

The peak storey drifts are shown in Tables 6-8 and 6-9 for Phases 1 and 2b, respectively. The percentages in brackets denote the analysis deviations from the test results. Drift time history results were not compared with the test data as the latter was not available.
Table 6-8: Phase 1 peak storey drifts

<table>
<thead>
<tr>
<th>Direction</th>
<th>Storey</th>
<th>Test (Schafer et al. 2016) (drift %)</th>
<th>Present (drift %)</th>
<th>Leng (2015) (drift %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X - Long</td>
<td>Lower</td>
<td>0.12</td>
<td>0.12 (0%)</td>
<td>0.10 (-17%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Lower</td>
<td>0.10</td>
<td>0.09 (-10%)</td>
<td>0.08 (-20%)</td>
</tr>
<tr>
<td>X - Long</td>
<td>Upper</td>
<td>0.11</td>
<td>0.09 (-18%)</td>
<td>0.03 (-73%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Upper</td>
<td>0.08</td>
<td>0.07 (-13%)</td>
<td>0.04 (-50%)</td>
</tr>
</tbody>
</table>

Table 6-9: Phase 2b peak storey drifts

<table>
<thead>
<tr>
<th>Direction</th>
<th>Storey</th>
<th>Test (Schafer et al. 2016) (drift %)</th>
<th>Present (drift %)</th>
<th>Leng (2015) (drift %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X - Long</td>
<td>Lower</td>
<td>0.04</td>
<td>0.04 (0%)</td>
<td>0.10 (+150%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Lower</td>
<td>-0.08</td>
<td>0.09 (+13%)</td>
<td>0.12 (+50%)</td>
</tr>
<tr>
<td>X - Long</td>
<td>Upper</td>
<td>-0.04</td>
<td>0.02 (-50%)</td>
<td>0.05 (+25%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Upper</td>
<td>-0.06</td>
<td>0.08 (+33%)</td>
<td>-0.05 (-17%)</td>
</tr>
</tbody>
</table>

Table 6-8 shows that the accuracy of the present model for the peak storey drifts in Phase 1 is less closely aligned than the natural periods in the same phase, but is more closely aligned than the beam-column model of Leng (2015).

In Phase 2b the present model’s results for the upper storey’s peak storey drifts deviated more from the shake table test results compared to the model of Leng (2015), as shown in Table 6-9. For the lower storey, the present model’s peak storey drifts remained reasonably accurate. It is uncertain why the present model’s deviation for the upper storey is as high as 50%, although the use of only one significant figure was likely a factor.

Each peak floor (or roof) acceleration was determined by averaging the accelerations of the four corner nodes in the relevant direction, expressed as a fraction of the gravity acceleration. The peak accelerations of the floor and roof are shown in Tables 6-10 and 6-11 for Phases 1 and 2b, respectively.
### Table 6-10: Phase 1 peak floor acceleration

<table>
<thead>
<tr>
<th>Direction</th>
<th>Diaphragm</th>
<th>Test (Schafer et al. 2016) (g)</th>
<th>Model (g)</th>
<th>Leng (2015) (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X - Long</td>
<td>Floor</td>
<td>0.142</td>
<td>0.100 (-30%)</td>
<td>0.170 (+20%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Floor</td>
<td>0.085</td>
<td>0.078 (-8%)</td>
<td>0.224 (+164%)</td>
</tr>
<tr>
<td>X - Long</td>
<td>Roof</td>
<td>0.177</td>
<td>0.170 (-4%)</td>
<td>0.179 (+1%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Roof</td>
<td>0.125</td>
<td>0.145 (+16%)</td>
<td>0.228 (+82%)</td>
</tr>
</tbody>
</table>

### Table 6-11: Phase 2b peak floor acceleration

<table>
<thead>
<tr>
<th>Direction</th>
<th>Diaphragm</th>
<th>Test (Schafer et al. 2016) (g)</th>
<th>Model (g)</th>
<th>Leng (2015) (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X - Long</td>
<td>Floor</td>
<td>0.106</td>
<td>0.092 (-13%)</td>
<td>0.431 (+307%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Floor</td>
<td>0.079</td>
<td>0.094 (+19%)</td>
<td>0.630 (+697%)</td>
</tr>
<tr>
<td>X - Long</td>
<td>Roof</td>
<td>0.133</td>
<td>0.156 (+17%)</td>
<td>0.398 (+199%)</td>
</tr>
<tr>
<td>Y - Short</td>
<td>Roof</td>
<td>0.119</td>
<td>0.187 (+57%)</td>
<td>0.884 (+643%)</td>
</tr>
</tbody>
</table>

The deviations from the test results of the present analysis results for the peak floor accelerations are relatively large compared to the results for the natural periods but are much smaller than those of Leng (2015). It appears that peak floor accelerations are more sensitive to modelling assumptions than natural periods and peak storey drifts.

The deviations in peak floor accelerations of the present analysis results could also be due to the seismic input that might not exactly represent the shake table tests. The seismic data used in the present analyses were obtained from the ground motion records shown in Figure 6-4 and Table 6-2 and were scaled to match the peak test input. During the shake table tests, there were likely to be deviations from the intended seismic input.

### 6.5 Application

One of the applications for the equivalent shear modulus method is to check the storey drift under serviceability lateral loads. CFS-NEES experimentation has demonstrated that a CFS intensive building is significantly stiffer and stronger than the engineering designs suggest because it acts as a system rather than a set of uncoupled shear walls (Schafer et al. 2016). Based on the findings, the drift of a multi-storey CFS intensive building should decrease when taking into account full-system behaviour. This section
demonstrates this behaviour by modelling a simplified version of an archetype mid-rise residential building and generating a storey response curve under serviceability wind loading along the short axis.

6.5.1 Method
An outline of the process used for the application is depicted in Figure 6-11. Design using flexible and rigid diaphragm assumptions is referred to as 'conventional-flexible' and 'conventional-rigid', respectively.

![Diagram of the process](image)

**Figure 6-11**: Comparison of storey drift for the equivalent shear modulus method against conventional methods

At the first step, the archetype floor plan was simplified for the comparison study. The shear walls were designed for maximum shear due to ultimate wind loading under both a flexible and rigid diaphragm
assumption. The storey drift under serviceability wind loading was calculated for the structure under both assumptions, using the designed shear walls. Next, the building was modelled in ETABS and the storey drift was determined with only shear walls in the model, followed by the addition of gravity components. The simplified archetype floor plan is depicted in Figure 6-12, with shear walls highlighted in red.

Figure 6-12: Simplified archetype building plan view

The floor plan was the same for all storeys with heights of 3.1 m. Shear walls were all 3.7 m in length. Windows and doors were included where appropriate, as shown in Figure 6-13, which also provides a reference number for each of the shear walls.
6.5.2 Design

Many components of the system have an impact on the lateral behaviour of the building and distribution of lateral forces, including the stiffness of the floors, walls and connections. As the behaviour of all these components is very complex, a conservative approach is to calculate the required strength for both a rigid diaphragm and flexible diaphragm then take the worst-case scenario between the two (AISI 2015). This allows the design to be encompassing, however, can lead to overdesign.

Under a flexible diaphragm assumption, the lateral load is attributed to shear walls based on tributary area and the lateral system is designed as a series of independent shear walls (Schafer et al. 2015). Under a rigid diaphragm, the floor system distributes the lateral force to walls at each storey level, based on the associated relative stiffness (Madsen et al. 2011). In this study, accidental torsion was neglected.

The design was carried out for a total wind loading of 1870 kN in the Y direction. Tables 6-12 and 6-13 depict the shear demand distribution for a flexible and rigid diaphragm assumption, respectively. Shear walls are numbered as labelled in Figure 6-13 above, only one half (and the centreline) of the building is designed as the floor plan is symmetrical about the centreline.
Table 6-12: Flexible diaphragm based shear distribution

<table>
<thead>
<tr>
<th>Shear wall</th>
<th>Tributary width (m)</th>
<th>Relative tributary width (%)</th>
<th>Shear (kN)</th>
<th>Shear demand per unit length (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.5</td>
<td>3.7</td>
<td>69.5</td>
<td>19.0</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>5.7</td>
<td>106.9</td>
<td>29.2</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>5.7</td>
<td>106.9</td>
<td>29.2</td>
</tr>
<tr>
<td>4</td>
<td>13</td>
<td>7.4</td>
<td>138.9</td>
<td>38.0</td>
</tr>
<tr>
<td>5</td>
<td>4.75</td>
<td>2.7</td>
<td>50.8</td>
<td>13.9</td>
</tr>
<tr>
<td>6</td>
<td>4.75</td>
<td>2.7</td>
<td>50.8</td>
<td>13.9</td>
</tr>
<tr>
<td>7</td>
<td>5.25</td>
<td>3.0</td>
<td>56.1</td>
<td>15.3</td>
</tr>
<tr>
<td>8</td>
<td>5.25</td>
<td>3.0</td>
<td>56.1</td>
<td>15.3</td>
</tr>
<tr>
<td>9</td>
<td>5.25</td>
<td>3.0</td>
<td>56.1</td>
<td>15.3</td>
</tr>
<tr>
<td>10</td>
<td>5.25</td>
<td>3.0</td>
<td>56.1</td>
<td>15.3</td>
</tr>
<tr>
<td>11</td>
<td>5.25</td>
<td>3.0</td>
<td>56.1</td>
<td>15.3</td>
</tr>
<tr>
<td>12</td>
<td>5.25</td>
<td>3.0</td>
<td>56.1</td>
<td>15.3</td>
</tr>
<tr>
<td>13</td>
<td>7</td>
<td>4.0</td>
<td>74.8</td>
<td>20.5</td>
</tr>
<tr>
<td>14</td>
<td>7</td>
<td>4.0</td>
<td>74.8</td>
<td>20.5</td>
</tr>
</tbody>
</table>

Table 6-13: Rigid diaphragm based shear distribution

<table>
<thead>
<tr>
<th>Shear wall</th>
<th>Wall length (m)</th>
<th>Relative stiffness (%)</th>
<th>Shear (kN)</th>
<th>Shear demand per unit length (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>2</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>3</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>4</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>5</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>6</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>7</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>8</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>9</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>10</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>11</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>12</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>13</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
<tr>
<td>14</td>
<td>3.7</td>
<td>4</td>
<td>71.9</td>
<td>19.7</td>
</tr>
</tbody>
</table>
Design wall capacity was determined based on recent strap-braced shear wall test results (Bhuiyan et al. 2019). A strap-braced shear wall is depicted in Figure 6-14, where $L$ is the strap length.

![Figure 6-14: Labelled strap-braced shear wall](image)

The testing was performed on a strap-braced shear panel which was 2.7 m in length 3.1 m in height. The straps used in the shear panel were 150 mm wide, 1.5 mm thick and made from G450 CFS. As all of the shear walls in the simplified archetype have the same length, the capacity of these walls was assumed to be directly proportional to the cross-sectional area of the bracing strap. Hence, the factored capacity of the tested shear wall was simply linearly interpolated for other strap sizes for the designs in this study. Three strap widths were designed, as shown in Table 6-14 with their factored capacity.
Table 6-14: Strap-braced shear wall capacities

<table>
<thead>
<tr>
<th>Strap width (mm)</th>
<th>Wall capacity (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>130</td>
<td>20.6</td>
</tr>
<tr>
<td>190</td>
<td>30.1</td>
</tr>
<tr>
<td>240</td>
<td>38.1</td>
</tr>
</tbody>
</table>

Shear walls were designed with a strap size such that the factored wall capacity exceeded the applied shear load calculated from the design envelope, as shown in Table 6-15.

Table 6-15: Strap-braced shear wall designs

<table>
<thead>
<tr>
<th>Shear wall</th>
<th>Governing design</th>
<th>Worst case shear (kN/m)</th>
<th>Required strap width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>2</td>
<td>Flexible</td>
<td>29.2</td>
<td>190</td>
</tr>
<tr>
<td>3</td>
<td>Flexible</td>
<td>29.2</td>
<td>190</td>
</tr>
<tr>
<td>4</td>
<td>Flexible</td>
<td>38.0</td>
<td>240</td>
</tr>
<tr>
<td>5</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>6</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>7</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>8</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>9</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>10</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>11</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>12</td>
<td>Rigid</td>
<td>19.7</td>
<td>130</td>
</tr>
<tr>
<td>13</td>
<td>Flexible</td>
<td>20.5</td>
<td>130</td>
</tr>
<tr>
<td>14</td>
<td>Flexible</td>
<td>20.5</td>
<td>130</td>
</tr>
</tbody>
</table>

The shear load decreases from the base to the top of the structure, however for simplicity, the walls designed for the highest base shear at the lowest storey were the same throughout the height of the building.

6.5.3 Storey drift

Storey drift was determined using the conventional flexible and rigid diaphragm assumptions as well as using the equivalent shear modulus method. The storey displacement is equal to the displacement at the base of the storey $u_b$ relative to the displacement at the top of the storey $u_t$. The storey drift is equal to the
storey displacement divided by the storey height \( h_i \) as shown in Equation 6-5.

\[
\text{Storey drift} = \frac{u_{t,i} - u_{b}}{h_i}
\]  

(6-5)

Wall displacement depends on the wall stiffness and load applied to each wall. The serviceability wind load was calculated to be 1,528 kN. Shear wall stiffness values were assumed based on the laboratory test (Bhuiyan et al. 2019). The stiffness for the 3.7 m long wall was interpolated using a linear relationship based on the length of the strap-bracing, \( L \) and the cross-sectional area of the strap. Based on the findings of Velchev (2008), it is assumed that the stiffness of a strap-braced shear panel primarily depends on the stiffness of the strap only. The stiffness of each wall is shown in Table 6-16, where \( k_w \) is the stiffness of the wall without hold-downs.

<table>
<thead>
<tr>
<th>( b ) (m)</th>
<th>( h ) (m)</th>
<th>( L ) (m)</th>
<th>Strap width (mm)</th>
<th>( k_w ) (kN/m)</th>
<th>( k_T ) (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.7</td>
<td>3.1</td>
<td>4.8</td>
<td>130</td>
<td>4,607</td>
<td>3,721</td>
</tr>
<tr>
<td>3.7</td>
<td>3.1</td>
<td>4.8</td>
<td>190</td>
<td>6,734</td>
<td>4,995</td>
</tr>
<tr>
<td>3.7</td>
<td>3.1</td>
<td>4.8</td>
<td>240</td>
<td>8,506</td>
<td>5,909</td>
</tr>
</tbody>
</table>

The stiffness of hold-downs was assumed to be 14,000 kN/m based on data from Simpson Strong-Tie (Simpson Strong-Tie Company Inc 2019). Equation 6-1 was used to calculate the total wall stiffness \( k_T \), as shown in Table 6-16 to account for deflection due to uplift at the hold-downs.

### 6.5.3.1 Flexible and Rigid

For the conventional flexible and rigid diaphragm assumptions, the storey drift was calculated using the average storey shear at each storey, due to surface wind loading. The displacement is based on the stiffness of the shear walls only and does not account for additional stiffness provided by non-structural wall panels or the rigidity of the wall to floor connections (Madsen, 2011). The wall deflections were determined by multiplying the total stiffness \( k_T \) by the applied load, as determined assuming either a rigid or flexible diaphragm. It was conservatively assumed that there was no downward force due to the permanent and imposed actions.
6.5.3.2 Model

To determine storey drift using the equivalent shear modulus method, the modified archetype building was modelled with shear walls, gravity walls and hold-downs in ETABS, as shown in Figure 6-15.

![Figure 6-15: Archetype building modelled with equivalent shear modulus method](image)

Equivalent shear modulus values for walls were calculated based on the wall stiffness, the stiffness $k_w$ was used as the hold-downs were modelled. The properties for the three designed shear walls are depicted in Table 6-17. All wall shell elements were assigned a thickness $t$ of 150 mm.

<table>
<thead>
<tr>
<th>Shear wall strap width (mm)</th>
<th>$k_w$ (kN/m)</th>
<th>$t$ (mm)</th>
<th>$G_{xz}$ (MPa)</th>
<th>$E_z$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>130</td>
<td>4,607</td>
<td>150</td>
<td>26.0</td>
<td>5</td>
</tr>
<tr>
<td>190</td>
<td>6,734</td>
<td>150</td>
<td>38.0</td>
<td>5</td>
</tr>
<tr>
<td>240</td>
<td>8,506</td>
<td>150</td>
<td>48.1</td>
<td>5</td>
</tr>
</tbody>
</table>
Gravity wall stiffness was based on laboratory test results of plasterboard sheathed walls in Australia (Shahi et al. 2014). The tested wall specimen had a height of 2.4 m and a width of 2.4 m. Walls were framed using web stiffened 350S141-30 C-shaped lipped studs at 600 mm spacing on-centre. Fibre cement boards with a thickness of 5 mm were used as sheathing. The net stiffness of the wall $k_w$ was reported to be 1,210 kN/m/m. This value is assumed for all gravity walls in the archetype structure and was also assumed to be linearly proportional to length; hence a constant equivalent shear modulus was used for all lengths. All doors were 25% sheathed, and windows were 55% sheathed, reducing the stiffness as shown in Table 6.18.

<table>
<thead>
<tr>
<th>$k_w$ (kN/m/m)</th>
<th>$G$ – Wall (MPa)</th>
<th>$G$ – Window (MPa)</th>
<th>$G$ – Door (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,210</td>
<td>7.6</td>
<td>5.7</td>
<td>3.8</td>
</tr>
<tr>
<td>1,210</td>
<td>7.6</td>
<td>5.7</td>
<td>3.8</td>
</tr>
</tbody>
</table>

Walls were unmeshed and edge constrained, allowing elements framing into the shell edge (rather than corner) to remain connected to the shell (Computers & Structures Inc 2017). Shear wall hold-downs were modelled with a stiffness of 14,000 kN/m using the spring and gap element, 10% of this stiffness was assigned to gravity wall hold-downs. Inter-storey straps were assumed to be pinned connections. The building elevation is depicted in Figure 6-16, highlighting the hold-downs.
Floor shells were modelled in-between each of the walls. A moment edge release was assigned to all edges of the floor shells such that no moment was carried between walls and floor, essentially making a pinned connection. As established in Section 6.4, a rigid diaphragm was assumed for the application. The total weight of the structure was approximated at 22,250 kN based on the archetype weight (Section 4.5.1). Self-weight was assigned to the walls at 1 kN/m$^3$ and floors at 3.9 kN/m$^3$, to obtain the total expected weight. Serviceability wind load was applied with a uniform surface load assigned to the Y-face of the structure. Nonlinear static analysis was used as it is required for the gap elements used to model the hold-downs to work.

Drift due to un-factored permanent and wind load was calculated for the model with shear walls only (Figure 6-17), as well as with shear and gravity walls as shown in Figure 6-18. The model consisting of only shear walls implemented a rigid diaphragm assigned to the nodes at each floor and the shear wall mass was increased to account for the floor weights.
Figure 6-17: Model of the archetype building with shear walls only

Figure 6-18: Model of the archetype building with shear walls and gravity walls
6.5.4 Storey drift results

The storey displacement at each floor is reported in Table 6-19, and the storey drift at each storey is compared in Figure 6-19. As the storey drift for the flexible diaphragm assumption varied depending on wall tributary area, the results for only the maximum deflection are reported. The allowable storey drift was assumed to be storey-height/360 under serviceability wind loading; hence, the allowable drift ratio was 0.278% as shown by the red line in Figure 6-19.

Table 6-19: Storey displacements for all calculation methods

<table>
<thead>
<tr>
<th>Storey</th>
<th>Rigid diaphragm</th>
<th>Flexible diaphragm</th>
<th>Model - Shear walls only</th>
<th>Model - All walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.00</td>
<td>1.37</td>
<td>2.55</td>
<td>0.32</td>
</tr>
<tr>
<td>6</td>
<td>3.10</td>
<td>4.10</td>
<td>4.06</td>
<td>1.10</td>
</tr>
<tr>
<td>5</td>
<td>5.10</td>
<td>6.84</td>
<td>5.63</td>
<td>1.91</td>
</tr>
<tr>
<td>4</td>
<td>7.20</td>
<td>9.58</td>
<td>7.11</td>
<td>2.72</td>
</tr>
<tr>
<td>3</td>
<td>9.20</td>
<td>12.31</td>
<td>8.49</td>
<td>3.53</td>
</tr>
<tr>
<td>2</td>
<td>11.30</td>
<td>15.05</td>
<td>9.71</td>
<td>4.30</td>
</tr>
<tr>
<td>1</td>
<td>13.30</td>
<td>17.78</td>
<td>10.33</td>
<td>5.56</td>
</tr>
<tr>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
It can be seen that the conventional flexible and rigid diaphragm methods produced the largest storey drifts, the rigid diaphragm assumption producing similar results to the shear walls only model. The model with both gravity and shear walls demonstrates a much lower storey drift at every storey. The storey drift at each storey is on average 70% less for the model with the contribution from all components as compared to the flexible diaphragm approach.

This demonstrates that the inclusion of shear stiffness of non-structural components can significantly impact the drift ratio. The allowable drift is exceeded at four of the storeys for the flexible diaphragm, three for the rigid diaphragm, two for the model with shear walls only and none for the model with all walls.

6.6 Discussion

The proposed equivalent shear modulus method has demonstrated to be a simple and efficient method to
accurately predict the global lateral behaviour of a CFS structure under serviceability loading. This method provides the basis for a tool that can facilitate the structural design process for mid-rise CFS. The verification results showed that, in comparison with the experimental testing, the model accurately predicted the natural period of both tested phases. The peak storey drift and peak storey acceleration were predicted with a lesser extent of accuracy in some cases, however, compared favourably with Leng’s (2015) high fidelity beam-column element model.

The application of the model for calculating the storey drift due to wind loading on the archetype building demonstrated a significant increase in stiffness when including the contribution of gravity walls. The results show that using conventional methods to calculate drift is conservative, as these methods do not take into account the full-system behaviour, potentially leading to overdesign. For this example, the failure of the lower three storeys in the serviceability check under a flexible diaphragm assumption would require changes to the structural design. However, the model demonstrated that with the stiffness of the full-system accounted for, these walls do not fail in serviceability. It should be noted that a number of simplifying assumptions were made in the archetype building model, such as the simplified floor plan and single-sheathed gravity wall stiffness, which would need to be refined for practical use. However, these were deemed suitable for the purposes of demonstrating how the proposed method could be implemented in industry for design checks.

The results in this chapter demonstrate that using a rigid diaphragm is sufficient to match the experimental data for the two-storey building. The use of a semi-rigid diaphragm was tested in the model by varying the floor stiffness properties as well as the floor-wall connection rotational stiffness. This provided similar results; however, the analysis took much longer than that for a rigid diaphragm, and it was necessary to introduce a number of assumptions. The use of a rigid diaphragm reduces the degrees of freedom as there is no in-plane membrane deformation, and hence, the model requires much less computing power for simulation. The assumption of a rigid diaphragm in the model can be justified both through the verification results as well as through the results of the experimental testing (Schafer et al. 2016), which found the diaphragm was relatively rigid based on the ratio of the maximum in-plane diaphragm deflection to the average drift of adjoining vertical elements, even though the structure was designed with the flexible diaphragm assumption (Peterman et al. 2016a). Modelling the stiffness of other components including the inter-storey chord straps for shear walls and the parapet stiffness, and refining the assumption for opening
stiffness could potentially improve the results. Confidence in the model could also be reinforced through further verification of test results for other construction phases and larger-scale structures.

The model is limited to serviceability loads in the elastic region; however, the inclusion of stiffness degradation to shell elements could be implemented in the future for use in capacity design. Other applications of the model are to check if plasterboard sheathing connected to the main structure is in danger of cracking under serviceability loads and analysis of hybrid systems such as CFS with the addition of RC walls and HRS.

A summary of the benefits of the equivalent shear modulus method are listed below:

- Full-system behaviour – Incorporates stiffness of shear walls, gravity walls and floor systems, allowing the designer to achieve a better understanding of the overall building lateral behaviour.
- Material data – Stiffness data from experimental tests as well as finely detailed FEA models can be used, and the equivalent shear modulus can be determined for both sheathed and strap-braced shear walls.
- Analysis time - Minimal degrees of freedom are required, so the computation time is very short. The analysis time does not increase with more components in the wall, e.g. for Leng’s (2015) beam-column element method, the simulation time increases for more layers of sheathing in each wall, as more elements are required.
- Modelling effort - The proposed method should reduce the time required to build a model, which is of great benefit to practicing engineers. The simplicity of the model also lends itself to the potential compatibility with BIM, through importing simple geometries from architectural models into ETABS or similar for serviceability checks and shear wall layout design.

6.7 Conclusions
This chapter has presented an efficient modelling method for practical 3D nonlinear elastic analysis of a multi-storey CFS building that incorporates the lateral stiffness contributions of non-structural shear components such as gravity walls and their elements. It represents each wall with an equivalent shear modulus four-node orthotropic shell element, drastically reducing the number of degrees of freedom and therefore the analysis time. The entailed modelling efforts are also significantly reduced compared to the
conventional frame modelling method.

Verification of the resulting building model against published shake table test results of a two-storey building prototype found that the proposed modelling method was able to account for the lateral stiffness of the full-system, and compared favourably against a published modelling method in terms of accuracy with respect to the natural periods, the peak storey drifts and the peak floor accelerations at two different construction phases. The application of the modelling method to the archetype mid-rise apartment structure demonstrated a significant reduction of storey drift when compared to calculations using conventional methods.

This study contributes to the development of loadbearing CFS in Australia by proposing a simple and practical modelling method with the potential for industry use in serviceability design. Incorporating the contribution of non-structural components to the lateral load resisting system allows the building to be designed as a complete system, receiving the benefits of the full-system behaviour, aligned with observed experimental behaviour.
Chapter 7 - Conclusions

7.1 Introduction

This thesis has presented three core studies relating to the implementation of loadbearing CFS in Australian mid-rise apartment structures. This chapter provides a summary of the key findings in reference to the research aims and objectives for each study. Recommendations are made for future research, followed by contributions and closing remarks.

7.2 Key findings

The initial exploratory study in Chapter 3 aimed to investigate the application of loadbearing CFS mid-rise construction in international markets with the objectives to:

- Explore the current state-of-the-art in mid-rise loadbearing CFS construction.
- Identify the advantages and disadvantages of loadbearing CFS.

The literature review identified the increase in construction of mid-rise apartment structures in Australia as well as the opportunity for an innovative construction method such as loadbearing CFS to better meet this demand than traditional construction methods. The desktop study and case study survey were valuable in capturing perspectives of construction professionals and identifying the state-of-the-art in international mid-rise loadbearing CFS construction. Findings from the chapter were triangulated to identify the advantages and disadvantages of the use of loadbearing CFS as well as best practices. The key findings from this initial study are summarised as follows:

- Multiple design solutions exist for mid-rise loadbearing CFS construction, as demonstrated by international case studies. Loadbearing CFS wall systems can be integrated with many flooring systems including CFS joist and truss floors topped with OSB or metal decking and hollow-core concrete. Lateral load resisting systems can be OSB, steel or gypsum-sheathed shear walls, strap-braced walls and CFS bolted moment frames. Chord studs or end posts can be designed with CFS studs or HRS sections for higher loads. RC or reinforced masonry cores can also be integrated with CFS construction to resist lateral loads.
- Construction speed and cost-effectiveness are of high import for mid-rise structures and were
determined to be the key advantages of CFS, based on the findings of the desktop study and case study surveys. CFS synergises with off-site manufacturing (OSM), which has been identified to be necessary for the future productivity of the construction sector (CRC Construction Innovation 2007). Repetitive pre-panelisation can lead to efficient and economical structures as well as fast on-site erection with reduced costs.

- Case studies highlighted a beneficial weight reduction of CFS in comparison to other traditional methods, for example, a 40% and 50% reduction for CFS framed six-storey apartments in the UK (Baker 2006b; Metek 2016a).

- CFS is best suited to floor plans with vertically aligning walls to ensure a direct load path; however, integration with HRS and RC can lead to efficient material use in more complicated designs. Increasing the number of trades required for construction has the potential to reduce efficiency, so coordination between CFS and HRS contractors is essential. With an exemplary design, residential loadbearing CFS structures of nine storeys have been constructed in the USA without the requirement of RC or HRS (Westerman 2016).

- A performance solution is required for mid-rise fire-resisting construction with CFS in Australia as there are no deemed-to-satisfy provisions for loadbearing CFS apartments with three or more storeys. Further development of standards relating to CFS are required to provide deemed-to-satisfy fire and acoustic solutions as proprietary systems can incur high testing costs.

Chapters 4 and 5 built upon the findings of Chapter 3 to further investigate the advantages of a lightweight CFS structure in an Australian context. The aim of the comparative archetypal case study in Chapter 4 and 5 was to investigate the structural feasibility of CFS in the Australian mid-rise residential market. The objectives were to:

- Compare the structural systems of RC and CFS for a typical Australian mid-rise apartment.
- Determine the effect of a CFS structure on foundation design.

A comparative case study method was implemented using a mid-rise residential archetype structure designed initially with a RC-framed structure and subsequently with a loadbearing CFS structure. The archetype building was designed to be typical of the sector, to obtain a degree of generalisability for the findings. The first objective is addressed in Chapter 4, where the preliminary structural design of the two
buildings is described, and design actions are compared. The findings from Chapter 4 were then implemented in Chapter 5 for the second objective, where the design of deep piled and shallow pad footing foundations for the CFS and RC structures were compared in two different locations (Sydney CBD and the Sydney suburb of Lindfield). The value in this comparison is the Australian context and the quantification of the benefits of a lightweight structure. The findings for the comparative archetypal case study are described below:

- The seven-storey archetype building was initially designed with a RC flat plate structure to be a typical mid-rise apartment building in Australia, and then subsequently designed with a loadbearing CFS structure. The CFS structure was designed with a ledger framing system. The gravity system consisted of loadbearing CFS stud walls of varying gauge, as well as HRS posts in the lobby and balcony areas. The flooring was designed with 203 mm deep CFS C-joists with a metal deck and gypsum topping. The lateral load resisting system comprised of predominantly steel-sheathed CFS shear walls as well as some HRS braced frames in the lower storeys.

- The archetype CFS structure was found to have a significantly lower weight than the RC structure with a reduction of 60%. The governing axial load case demonstrated a reduction of 53%.

- The lower self-weight of the CFS structure resulted in a lower seismic weight, with earthquake actions significantly reduced for the CFS structure. The governing lateral actions for base shear and overturning moment were shown to be reduced by up to 58% for the CFS structure as compared with the RC structure. The imposed and wind actions, however, were the same for both structures.

- The resulting design of foundations demonstrated a substantial saving in concrete and reinforcing steel volumes for the CFS structure over the RC structure. The concrete volume and steel reinforcing for pile foundations were reduced by 42% and 45%, respectively. The pad footings had a concrete volume and steel reinforcing reduction of 40% and 59%, respectively.

- The reduction in concrete and steel volumes for foundation design can mitigate costly rock drilling and potential dewatering and waste removal or remediation. Reducing the amount of time and cost in foundation construction can also lead to increased productivity, which the Australian construction sector is lacking.

- The structural design of CFS would be more accessible for structural practitioners with updated
structural ductility and structural performance factors for various CFS lateral load resisting systems in the Australian Standards (SA 2007). Current provisions result in conservative designs for mid-rise CFS structures without special testing.

- As the archetype building was designed to be suitable for a RC structure, the CFS design was not as optimal as the RC design. Hence, the comparison presented could be considered less favourable for the CFS building. However, this would be a likely scenario for implementation in the industry at present, as most architects in Australia design buildings for a RC structure. It is necessary for a CFS structure to be able to adapt to current mid-rise building designs, which was demonstrated to be possible in this study.

While Chapters 4 and 5 emphasised the beneficial implications of the use of loadbearing CFS in mid-rise apartment construction, Chapter 6 aimed to provide a practical contribution to facilitate the structural design process for mid-rise CFS in order to simplify the introduction of mid-rise CFS into the Australian market. The objectives were to:

- Develop an efficient method to simulate full-system serviceability behaviour of a CFS shear wall building.
- Demonstrate the application of the method to a mid-rise apartment structure.

Initially, recent developments in the understanding of full-system behaviour of CFS framed structures and previous methods for modelling this behaviour were reviewed. Due to the complex behaviour of the CFS system, it is imperative to improve the efficiency of computational tools for modelling CFS for practical use. The first objective was addressed with the development of a proposed modelling method for full-system behaviour of CFS structures. It uses shell elements with an equivalent shear modulus to check serviceability limit state designs. The FEA method was a beneficial tool for this study, allowing improved understanding and replication of CFS full-system behaviour and facilitating the design. FEA also allows further analyses to be undertaken without the requirement of expensive full-scale testing. This was used for the second objective by modelling the CFS archetype building and comparing the storey drift due to wind loading with conventional methods. The primary outcomes of this research are:

- The development of an efficient method for 3D structural analysis of CFS buildings that accounts for the lateral stiffness contributions of non-structural components, enabling the prediction of the
lateral elastic behaviour of the structure. Each shear or gravity wall is represented by an equivalent shear modulus four-node orthotropic shell element, determined from the experimental test of a representative wall panel, or from the analysis of a finely detailed finite element model of the panel.

- A two-storey CFS building was modelled and verified against full-scale shake table test results with respect to the natural period, the peak storey drift and the peak floor acceleration at two different construction phases. This demonstrates the ability of the modelling method to account for the lateral stiffness contributions of gravity walls and their elements.

- The application of the model for serviceability design checks on an archetype building indicates a significant decrease in the storey drift when accounting for the full-system behaviour of the structure.

- The equivalent shear modulus method forms the basis for a design tool that will allow engineers to predict the lateral behaviour of a CFS structure for serviceability loading, facilitating more efficient CFS building designs.

7.3 Recommendations for future research

The findings of this thesis have led to the identification of some areas of interest for future research, as outlined below:

- The development and testing of CFS axial and lateral loadbearing systems that make use of the current Australian CFS supply chain for low-rise structures would be beneficial for the industry. This could lead to the introduction of deemed-to-satisfy provisions in the Australian building regulations, making CFS more accessible for structural engineers.

- Structural solutions such as precast concrete lift cores and HRS moment frames can synergise well with CFS construction. Research into the integration with the current supply chain for HRS and precast concrete in Australia could lead to more efficient structures. Optimising structural solutions that make use of prefabricated modular bathroom or kitchen pods could also improve construction speed and minimise costs.

- The floor plan of the archetype structure in this thesis was not designed to be optimal for CFS. The development of architectural designs that exemplify the use of loadbearing CFS would be beneficial. Architectural design guides that identify best practice in designing floor plans suitable
for loadbearing CFS with consideration of constraints for prefabricated components could further support the uptake of loadbearing CFS in the mid-rise residential sector in Australia.

- An optimised design of the transfer slabs to better suit CFS would also be beneficial, creating a more efficient structure, further minimising pile and pad footing volumes for the CFS building.

- The equivalent shear modulus method proposed in this thesis could be further developed through addressing the assumptions made, such as the rigid diaphragm and equivalent axial rigidity of walls. Comparison with test data for more construction phases and structures, with different gravity and lateral load resisting systems, would improve confidence in the model. Future additions to the model could include nonlinearity in shell elements for predicting ultimate limit state behaviour and the introduction of RC and HRS to assist in hybrid construction.

7.4 Contributions and closing remarks

The overall aim of the thesis was to explore the feasibility of mid-rise loadbearing CFS construction in the Australian market from a structural perspective, with a view to facilitating its further adoption within the Australian construction industry. This aim has been achieved through the three core research studies resulting in a contribution to the body of knowledge for loadbearing CFS mid-rise construction. The advantages and disadvantages, as well as a number of best practices, have been identified to assist in the implementation of mid-rise loadbearing CFS by Australian practitioners.

The comparative case study has demonstrated the ability for a CFS system to adapt to a building designed for a traditional RC structure. The favourable comparison of the loadbearing CFS structure with the RC structure in terms of design actions and foundation design should encourage its adoption in the Australian mid-rise residential sector. A contribution has also been made to the literature in the area of efficiently modelling the full-system behaviour of CFS structures (Franklin et al. 2019). The proposed modelling method has also taken steps towards a contribution to industry by simplifying the structural design of the complex CFS system, imperative to facilitating the use of loadbearing CFS.

The mid-rise construction sector is in need of innovative solutions in order to increase productivity in-line with other sectors. It is believed that the uptake of loadbearing CFS in the mid-rise apartment sector would benefit the Australian construction industry and this thesis has made a valuable contribution to this aspiration.
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Appendices

Appendix A:

Journal Article

Appendix B:

Peer-reviewed conference papers


Appendix C:

Application for ethical approval

Participant Information Sheet
Appendix A

Journal Article

Efficient 3D lateral analysis of cold-formed steel buildings

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A R T I C L E   I N F O

Article history:
Received 9 April 2019
Received in revised form 23 May 2019
Accepted 23 May 2019
Available online 30 May 2019

Keywords:
3D building analysis
Cold-formed steel building
Equivalent shear modulus
Lateral behaviour
System behaviour
Shell element

A B S T R A C T

Non-structural components contribute significantly to the lateral stiffness of a cold-formed steel (CFS) building structure, but are cumbersome to model explicitly in the structural analysis. They are therefore commonly ignored in a 3D structural analysis, and their benefits are lost to the design. This paper proposes an efficient modelling method that enables practical and accurate 3D elastic analysis of a multi-storey CFS building structure to study its lateral behaviour within the serviceability limit state. Each shear or gravity wall is represented by an equivalent shear modulus four-node orthotropic shell element, which incorporates the lateral stiffness (or flexibility) contributions of all components including sheathing, braces and fasteners as present in the wall. The equivalent shear modulus is determined from the experimental test of a representative wall panel, or from the analysis of a finely detailed finite element model of the panel. The resulting two-storey building model, which has much fewer degrees of freedom compared to conventional models, is verified against full-scale shake table test results with respect to the natural period, the peak storey drift and the peak floor acceleration at two different construction phases. This paper demonstrates that the proposed modelling method not only saves analysis time considerably through the drastic reduction of degrees of freedom, but also compares favourably against a published modelling method in terms of accuracy and modelling efforts.

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1. Introduction

Cold-formed steel (CFS) members are increasingly used as primary structural components in building systems around the world. However, in Australia the use of CFS members for load bearing applications in mid-rise buildings has been extremely limited. Improvements to the analysis and design procedures in terms of efficiency and accuracy, and better understanding of the sway behaviour of CFS buildings could lead to greater uptake in the construction industry. The ability to incorporate the contributions of non-structural components to the lateral resistance of a CFS building will also be beneficial.

The lateral load resisting system of a CFS building generally comprises either strap-braced \cite{6} or sheathed shear walls \cite{11}. Currently, CFS lateral load resisting design codes require the shear walls to take the entire lateral load themselves. This requirement does not take into account the stiffening effect of non-structural wall sheathing and other components which are essential for fire and acoustic compliance.

Within the timber industry in the USA, full-scale building testing has been carried out \cite{15} resulting in improvements to design standards and modelling methods. Since then, full-scale tests of CFS building structures were undertaken in China \cite{14}, Italy \cite{8} and at the University at Buffalo, USA, as a part of the National Science Foundation funded Network for Earthquake Engineering Simulation (NEES) project: CFS-NEES \cite{21}. The NEES project involved full-scale two storey shake table tests of a CFS building at its different construction phases. The fully completed building’s response was up to 16 times stiffer than that of the earlier construction, which was without non-structural components \cite{12}. This result demonstrates the significant and beneficial effect of non-structural components, and highlights the importance of modelling non-structural components in the structural analysis.

A number of studies have developed various methods to model CFS shear wall behaviour, with a focus on seismic response. Shear walls in CFS building structures are commonly sheathed with Oriented Strand Board (OSB), which is rigid in plane while the wall panel deforms through the flexibility of the sheathing connections. This behaviour has been replicated by Buonopane et al. \cite{2} with the shear walls being modelled using a rigid diaphragm attached to nonlinear spring elements simulating the fasteners. Bian et al. \cite{1} extended this research, using the model to analyse shear and gravity wall systems. A number of other studies have used a similar method involving the modelling of shear wall connections \cite{12,5,7,19,23,25}. This fastener-based method can accurately simulate the behaviour of a shear wall but involves a large number of degrees of freedom which may be impractical for full-scale multi-storey building simulations, and may not be applicable to strap braced shear walls.

To simplify the modelling of shear wall behaviour, shear wall panels have been modelled using linear frame elements and...
nonlinear diagonal braces as an equivalent truss [7,9,10,13,22]. Fülöp and Dubina [9] developed a trilinear hysteretic model using frame elements to simulate single shear walls. Leng et al. [12] produced a number of models of varying fidelity using OpenSees for the CFS NEES building with the characterisation of nonlinear behaviour for shear walls, gravity walls, interior walls, and semi-rigid floor and roof diaphragms. Wall components were modelled taking into account the pinching effect in the hysteretic response on the basis of design equations and sub-system level tests. Leng et al. [12] extended the use of equivalent beam column elements by modelling sheathing subpanels. Li et al. [14] created a shear wall model based on the restoring force of wall fasteners, represented as an equivalent bracing. This method has reduced degrees of freedom compared to the fastener-based method, however many elements are still required for each wall in the building.

Further simplification of the shear wall representation has been developed by Martínez-Martínez and Xu [18] with the use of a 16-node equivalent modulus shell element model without hold-downs. The equivalent modulus is a representation of the sheathing and stud properties. A three-storey building was modelled using this method, and the results for lateral displacements compared favourably against a conventional FEA model. However, no comparisons were made against laboratory test results.

This paper proposes the use of an equivalent shear modulus four-node orthotropic shell element to model each wall for the purpose of simulating the elastic sway behaviour of a CFS multi-storey building, incorporating the lateral stiffness contributions of shear walls as well as non-structural components. The FE modelling herein will focus on accurately simulating the elastic stiffness of the system with a view to facilitating the serviceability limit state design of CFS mid-rise buildings under lateral loading such as wind. The advantages of the proposed modelling method are twofold. First, it can be used whether the sheathing is OSB, gypsum or steel sheeting, and it can even be used for strap-braced shear walls. Second, it is significantly more efficient than the existing modelling methods [1,2,7,9,10,12,14,19,22,23,25]. A bibliography of proposed numerical models can be found in a recent paper by Usefi et al. [24].

Due to the absence of quasi-static tests, the resulting building model will be verified against the results of shake table tests undertaken at low level seismic excitations [21], with respect to the natural period, peak storey drift and peak floor acceleration. Analyses will be taken at two different construction phases in order to demonstrate the lateral stiffness contributions of gravity walls and their elements.

2. Methodology

2.1. Shear and gravity walls

This paper uses an equivalent shear modulus four-node orthotropic shell element model each of the CFS shear and gravity walls. Fig. 1 depicts a four-node shell element of height $h$ and width $b$, modelling a wall of the same dimension. The thickness $t$ of the shell element is the same as that of the wall. The deflection of the shell element (i.e. wall) under a horizontal load $F$ is also depicted in the figure, where $\delta$ is the deflection due to shear deformation of the shell element and $\delta_{HD}$ is the upward deflection of the hold-down. A hold-down is an anchorage device used to hold down the bottom track at the chord stud’s location.

The total deflection $\delta_T$ of the shell element is given in Eq. (1) where $k_T$ is the total lateral stiffness

$$\delta_T = \delta_S + \frac{F}{k_T} \delta_{HD}$$

(1)

The hold-down deflection $\delta_{HD}$ is equal to

$$\delta_{HD} = \frac{b F}{k_{HD}}$$

(2)

in which the $k_{HD}$ is the stiffness of the hold-down.

The deflection $\delta_S$ due to shear deformation is given by Eq. (3) where $G_xz$ is the equivalent (in-plane) shear modulus of the shell element

$$\delta_S = \frac{G_xz t b}{k_T}$$

(3)

Eqs. (2) and (3) are substituted into Eq. (1) to give the equivalent shear modulus

$$G_x = \frac{h}{\left( \frac{b F}{k_T} \right) \frac{t b^2}{b k_{HD}}}$$

(4)

The total stiffness $k_T$ can be obtained from a laboratory test using Eq. (1), as illustrated in Section 3.1. Alternatively, it can also be obtained from the analysis of a finely detailed finite element model of the wall panel.

Fig. 1. Shell element geometry and deflections.
2.2. Hold-downs

Hold-downs are modelled using spring and gap link elements at the two base nodes of the shell element. Hold-downs have a very high stiffness in compression, which is simulated by the gap element. The gap element is massless and is assigned a high stiffness of $10^7$ kN/mm in the Z direction. In tension, the spring acts with a constant stiffness of 9.93 kN/mm for the shear wall analysed in the present work [13]. The hold-downs for gravity walls are modelled with one tenth of the stiffness of the shear wall hold-downs [1]. Shear anchors are not modelled as the hold-downs are restrained in the horizontal directions.

3. Verification

To verify the proposed methodology, a two-storey building [21] was modelled in ETABS at two construction phases, and the analysis results were compared with the experimental data and the beam-column analysis results of Leng [13] for the natural period, peak storey drift and peak floor acceleration (Phase 1: A1-3D-SDa, Phase 2b: A2b-3D-SDa). Shake table test results were used for verification due to the lack of quasi-static tests of full scale CFS buildings.

3.1. Tested wall stiffness values

The wall stiffness values were derived from the shear and unsheathed gravity wall tests undertaken as part of the CFS-NEES project [16]. The tested components had construction details consistent with those used in the full-scale shake table test [21]. The experimental shear wall specimen with OSB sheathing is shown in Fig. 2.

A number of CFS wall types were subjected to monotonic and cyclic loading under displacement control [16]. The specimens were 2.74 m high and 1.22 m or 2.44 m long, with either OSB sheathing, gypsum sheathing or no sheathing. The walls were framed with 600S162-54 studs and S/HDU6 hold-downs with a specified stiffness of 9.93 kN/mm. In the current study, the stiffness was estimated at a deflection equal to the highest expected deflection of the full-scale building when tested under low level seismic excitation [20]. The total stiffnesses $k_T$ of the 1.22 and 2.44 m long OSB sheathed shear walls with ledger were found to be 0.785 and 2.47 kN/mm, respectively [17].

3.2. CFS NEES shake table test

Shake table testing was undertaken for a full-scale two storey CFS framed building as part of the CFS-NEES project [21]. The building was designed using the allowable strength method for a location in Orange County, California. The specimen was 7 m by 15.2 m in plan and 5.8 m in height. There were several CFS shear walls and gravity walls with and without window openings. OSB sheathed CFS joist floor and roof panels were used in a ledger framing system. The shear walls utilised back-to-back chord studs, and were designed as laterally decoupled, requiring a hold-down at both ends of each wall segment. Simpson Strong-Tie S/HDU6 hold-downs were used at the base of the shear walls on the ground floor. Steel straps were used to transfer horizontal forces from the upper to the lower chords.

Testing was undertaken at a number of different construction phases aiming to quantify the effects of non-structural components on the building’s structural performance. In this paper, two of the phases are analysed, denoted as Phase 1 and Phase 2b.

Phase 1 consisted mainly of structural elements only: single-sided OSB sheathed shear walls, roof, floor and unsheathed gravity walls, as illustrated in Fig. 3. Phase 2b had the addition of exterior OSB sheathing on the gravity walls (i.e. sheathed on one side only). However, the two phases were designed to have similar total weights on the shake table through the use of supplemental concrete blocks and steel plates. The only (primary) variable affecting the frame’s responses under shaking was therefore the system stiffness.

Table 1 details the construction differences between the two phases. The framing labels can be explained through an example of the ground floor shear wall; 600S162-54. ‘600’ is the web depth in 100th inches, ‘S’ refers to stud or joist section, ‘162’ is the flange width expressed in 100th inches and ‘54’ is the minimum base metal thickness in 1000th inches. Field stud refers to any stud that is not a chord stud.

Two ground motion records of the 1994 Northridge earthquake were used in the tests [21] at a number of scaling levels, being from the Canoga Park and the Rinaldi receiving stations as provided by the Pacific Earthquake Engineering Research Center (PEER). The unscaled ground motions are shown in Fig. 4, while the actual peak accelerations analysed in the present work are given in Table 2. As indicated in the

![Fig. 2. Experimental specimen with OSB sheathing (Source: [16]).](image-url)
In the present work, the natural period was determined using free vibration analysis in ETABS [4]. To determine the peak storey drift and the peak floor acceleration, nonlinear time history analysis was used. Analyses were undertaken using factored seismic records to match the actual Phase 1 and Phase 2b Peak Ground Acceleration from the shake table tests [20].

### 3.2.1. Modelling walls, floor and roof

Fig. 5 shows the present building model in ETABS, with the blue colour denoting shear walls and the grey colour, gravity walls. In the tests, chord studs were connected between storeys with a steel strap, which was modelled in the present work by using a shared node between the upper and the lower shell elements.

The shear moduli for various lengths of shear walls in the building were determined through interpolation between the 1.22 and 2.44 m wide shear walls obtained from the laboratory tests [17]. For the sheathed gravity walls, due to the absence of laboratory tests, the elastic stiffness was assumed to be the same as the OSB sheathed shear walls.

Sheathed gravity walls with openings naturally have a reduced stiffness. As there is no test data available for such walls, the shear modulus of a wall with an opening is factored by the square root of the proportion of the wall area that is sheathed. For example, a wall with a 55% sheathed area has a shear modulus equal to 74% of that of a fully sheathed wall that was tested [17]. It will not be appropriate to linearly factor the shear modulus as an opening is typically located centrally within the wall, without reducing the wall width.

The vertical elastic modulus of the shell element $E_z$ was initially assumed to be 5 GPa as it was around the typical value for OSB panels [3]. However, the value can also be justified through the concept of effective shell areas in compression and tension due to overturning moment. An effective area is the area on either the windward or leeward side of the shell element that has an equivalent axial resistance to the

---

**Table 2**

Seismic record data.

<table>
<thead>
<tr>
<th>PEER record</th>
<th>CNP106</th>
<th>CNP196</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>Y</td>
<td>X</td>
</tr>
<tr>
<td>Axis</td>
<td>Short</td>
<td>Long</td>
</tr>
<tr>
<td>Actual phase 1 excitation PGA (g)</td>
<td>0.061</td>
<td>0.083</td>
</tr>
<tr>
<td>Actual phase 2b excitation PGA (g)</td>
<td>0.061</td>
<td>0.085</td>
</tr>
</tbody>
</table>

---

**Fig. 4.** Unscaled ground motions CNP106 and CNP196.

**Fig. 5.** Opposing views of 3D building model in ETABS.
Based on the information provided by Peterman [20]. In calculating the shell elements are determined by calculating the mass per volume. The elemental masses of the wall mass for Phase 2b, it was assumed there were no openings, the weight was distributed evenly throughout each wall type.

The masses of the walls, floor and roof were modelled in ETABS through the use of elemental mass and additional mass. Elemental mass is the self-weight of the components. The elemental masses of the shell elements are determined by calculating the mass per volume based on the information provided by Peterman [20]. In calculating the wall mass for Phase 2b, it was assumed there were no openings, so the weight was distributed evenly throughout each wall type.

<table>
<thead>
<tr>
<th>Component</th>
<th>$t$ (mm)</th>
<th>$G_{EY}$ (MPa)</th>
<th>$E_{Y}$ (GPa)</th>
<th>Mass (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear wall 1</td>
<td>164</td>
<td>17.9–24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Shear wall 2</td>
<td>164</td>
<td>17.9–24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Gravity wall 1</td>
<td>152</td>
<td>0.3</td>
<td>2.7</td>
<td>77</td>
</tr>
<tr>
<td>Gravity wall 2</td>
<td>152</td>
<td>0.3</td>
<td>2.7</td>
<td>34</td>
</tr>
<tr>
<td>Interior wall</td>
<td>92</td>
<td>0.5</td>
<td>2.7</td>
<td>65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>$t$ (mm)</th>
<th>$G_{EY}$ (MPa)</th>
<th>$E_{Y}$ (GPa)</th>
<th>Mass (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear wall 1</td>
<td>164</td>
<td>17.9–24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Shear wall 2</td>
<td>164</td>
<td>17.9–24.9</td>
<td>5</td>
<td>116</td>
</tr>
<tr>
<td>Gravity wall 1</td>
<td>152</td>
<td>19.3–28.1</td>
<td>2.7</td>
<td>103</td>
</tr>
<tr>
<td>Gravity wall 1 – opening</td>
<td>152</td>
<td>14.3–20.8</td>
<td>2.7</td>
<td>103</td>
</tr>
<tr>
<td>Gravity wall 2</td>
<td>152</td>
<td>14.4–21.1</td>
<td>2.7</td>
<td>56</td>
</tr>
<tr>
<td>Gravity wall 2 – opening</td>
<td>152</td>
<td>10.7–15.6</td>
<td>2.7</td>
<td>56</td>
</tr>
<tr>
<td>Interior wall</td>
<td>92</td>
<td>0.5</td>
<td>2.7</td>
<td>65</td>
</tr>
</tbody>
</table>

A damping ratio of 5% was assumed in all analyses.

Table 3
Phase 1 wall components.

Table 4
Phase 2b wall components.

Table 5
Seismic masses.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Roof (kg)</th>
<th>Floor (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1</td>
<td>13,370</td>
<td>18,949</td>
</tr>
<tr>
<td>Phase 2b</td>
<td>13,384</td>
<td>19,263</td>
</tr>
</tbody>
</table>

Additional masses representing the parapet and dummy concrete blocks were then applied as a surface load in ETABS, equal to that in the testing [20]. The seismic masses of the roof and floor in Phases 1 and 2b are given in Table 5.

### 3.2.2. Results

The first mode natural periods were determined for both the long (X) and the short (Y) directions. Fig. 6 depicts the first mode shapes. The first mode natural periods found in the shake table tests and the analyses are shown in Tables 6 and 7 for Phases 1 and 2b, respectively. The percentages in brackets denote the analysis deviations from the test results.

It can be seen that the present analysis results are closer to the test results than Leng’s [13] for both construction phases. Both models were able to simulate the significant decreases in the natural periods due to the increased building lateral stiffness from Phase 1 to Phase 2b, reflecting the effects of gravity walls and their elements.

Each peak storey drift was determined by averaging the corresponding displacements of the four corner nodes and dividing the result by the storey height. The X displacement was measured under seismic excitation in the long direction (CNP196), and the Y displacement was measured under excitation in the short direction (CNP106). Due to the small magnitude and susceptibility to random noise, the displacement in the perpendicular direction of each excitation is not considered.

Fig. 7 shows the present analysis results for the lower storey drift in Phase 1.

The peak storey drifts are shown in Tables 8 and 9 for Phases 1 and 2b, respectively. The percentages in brackets denote the analysis deviations from the test results. Drift time history results were not compared with the test data as the latter was not available.

Table 8 shows that the accuracy of the present model for the peak storey drifts in Phase 1 is not so good as for the natural periods in the same phase, but is better than the beam-column model of Leng [13]. In Phase 2b the present model’s results for the upper storey’s peak storey drifts deviated more from the shake table test results compared to the model of Leng [13], as shown in Table 9. For the lower storey, the present model’s peak storey drifts remained reasonably accurate.

Fig. 6. First mode of vibration in the short and long directions.
Table 8
Phase 1 peak storey drifts.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Storey</th>
<th>Test [21] (drift %)</th>
<th>Present (drift %)</th>
<th>Leng [13] (drift %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X – long</td>
<td>Lower</td>
<td>0.13</td>
<td>0.11 (0%)</td>
<td>0.09 (−17%)</td>
</tr>
<tr>
<td>Y – short</td>
<td>Lower</td>
<td>0.09 (−10%)</td>
<td>0.08 (−20%)</td>
<td>0.08 (−20%)</td>
</tr>
<tr>
<td>X – long</td>
<td>Upper</td>
<td>0.10</td>
<td>0.09 (−18%)</td>
<td>0.03 (−73%)</td>
</tr>
<tr>
<td>Y – short</td>
<td>Upper</td>
<td>0.08</td>
<td>0.07 (−13%)</td>
<td>0.04 (−50%)</td>
</tr>
</tbody>
</table>

Table 9
Phase 2b peak storey drifts.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Storey</th>
<th>Test [21] (drift %)</th>
<th>Present (drift %)</th>
<th>Leng [13] (drift %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X – long</td>
<td>Lower</td>
<td>0.04</td>
<td>0.04 (0%)</td>
<td>0.10 (150%)</td>
</tr>
<tr>
<td>Y – short</td>
<td>Lower</td>
<td>−0.08</td>
<td>0.09 (13%)</td>
<td>0.12 (50%)</td>
</tr>
<tr>
<td>X – long</td>
<td>Upper</td>
<td>−0.04</td>
<td>0.02 (−50%)</td>
<td>0.05 (25%)</td>
</tr>
<tr>
<td>Y – short</td>
<td>Upper</td>
<td>−0.06</td>
<td>0.08 (33%)</td>
<td>−0.05 (−17%)</td>
</tr>
</tbody>
</table>

Table 10
Phase 1 floor acceleration.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Diaphragm</th>
<th>Test [21] (g)</th>
<th>Model (g)</th>
<th>Leng [13] (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X – long</td>
<td>Floor</td>
<td>0.142</td>
<td>0.100 (−30%)</td>
<td>0.170 (120%)</td>
</tr>
<tr>
<td>Y – short</td>
<td>Floor</td>
<td>0.085</td>
<td>0.078 (−8%)</td>
<td>0.224 (104%)</td>
</tr>
<tr>
<td>X – long</td>
<td>Roof</td>
<td>0.177</td>
<td>0.170 (−4%)</td>
<td>0.179 (1%)</td>
</tr>
<tr>
<td>Y – short</td>
<td>Roof</td>
<td>0.125</td>
<td>0.145 (16%)</td>
<td>0.228 (82%)</td>
</tr>
</tbody>
</table>

It is uncertain why the present model’s deviation for the upper storey is as high as 50%, although the use of only 1 significant figure was likely a factor.

Each peak floor (or roof) acceleration was determined by averaging the accelerations of the four corner nodes in the relevant direction, expressed as a fraction of the gravity acceleration. The peak accelerations of the floor and roof are shown in Tables 10 and 11 for Phases 1 and 2b, respectively.

The deviations from the test results of the present analysis results for the peak floor accelerations are relatively large compared to the results for the natural periods, but are much smaller than those of Leng [13]. It appears that peak floor accelerations are more sensitive to modelling assumptions than natural periods and peak storey drifts.

The deviations in peak floor accelerations of the present analysis results could also be due to the seismic input that might not exactly represent the shake table tests. The seismic data used in the present analyses were obtained from the ground motion records shown in Fig. 4 and Table 2 and scaled to match the peak test input. On the other hand, during the shake table tests, there were likely to be deviations from the intended seismic input.

4. Conclusions

This paper has presented an efficient modelling method for practical 3D elastic analysis of a multi-storey cold-formed steel building that incorporates the lateral stiffness contributions of non-structural shear components such as gravity walls and their elements. It represents each wall with an equivalent shear modulus four-node orthotropic shell element, drastically reducing the number of degrees of freedom and therefore the analysis time. The entailed modelling efforts are also significantly less compared to the conventional frame modelling method.

Verification of the resulting building model against published shake table test results of a two-storey building prototype found that the proposed modelling method was able to account for the lateral stiffness contributions of gravity walls and their elements, and compared favourably against a published modelling method in terms of accuracy with respect to the natural periods, the peak storey drifts and the peak floor accelerations at two different construction phases.

Acknowledgements

The authors would like to thank the Australian Research Council for funding this research through the ARC Research Hub for Australian Steel Manufacturing under the Industrial Transformation Research Hubs scheme (Project ID: I11 30100017). The authors would also like to thank the Sustainable Building Research Centre at the Innovation Campus of the University of Wollongong for the use of its facilities.

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Appendix B

Peer-reviewed conference papers


COMPARISON OF DESIGN LOADS FOR COLD-FORMED STEEL AND REINFORCED CONCRETE MID-RISE STRUCTURES

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ABSTRACT
Cold-formed steel (CFS) has been demonstrated to be a practical loadbearing construction material for mid-rise apartment buildings internationally. However, the use of CFS for mid-rise residential buildings in Australia has been extremely limited. Reinforced concrete (RC) is conventionally used for mid-rise construction in Australia. Theoretically, CFS has a number of benefits over RC. However, within the literature, there is a lack of evidence of these advantages. One potential benefit is the higher strength-to-weight ratio of CFS. While this material property relates to weight reduction, other factors such as load factors and live loads also affect the design loads. This study aims to quantitatively demonstrate this advantage in application through design to the Australian standards. A typical RC structure mid-rise residential building was designed for this comparative study. The building is seven storeys above ground level with a single basement car park level, and is notionally located in Sydney. The building was designed for two structural systems (CFS and RC) to ensure structural stability and an efficient design. The design loads of the conventional RC system are compared with that of a CFS system with reference to the Australian standards. The CFS building demonstrates a reduction in weight of 60% when compared to the weight of the RC building of the same dimensions. The reduction in serviceability load is 54% and ultimate design load is 53%. The seismic loads for the CFS building are up to 58% less, whilst wind loads are the same for both structures. This study demonstrates the weight reduction and its implications resulting from changing a building design from RC to CFS. Future work should consider an analysis of the potential economies in foundation design through the use of a CFS superstructure. This research strengthens the case for implementation of CFS construction in the Australian mid-rise sector.

Keywords: Loadbearing cold-formed steel, mid-rise structure, structural design, comparative case study

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1. INTRODUCTION

Mid-rise apartment construction is rapidly increasing in Australia with a higher growth rate than all other dwelling types [1]. Mid-rise apartments are generally constructed with reinforced concrete (RC), requiring extensive time on site, formwork and expensive foundations due to the weight. RC is used due to availability in supply chain and current industry knowledge. Other lightweight solutions such as cold-formed steel (CFS) have been demonstrated to work internationally. CFS has a number of advantages noted in the literature including its high strength to weight ratio, ductility which prevents brittle failure, consistent material quality, non-combustibility, recyclability and high resistance to termites, mould and corrosion [2,3]. Many of the construction advantages for CFS framing are due to its compatibility with offsite manufacturing. The increase in construction speed is the most notable benefit of CFS, up to 50% faster than traditional methods [4].

The weight of a building is important for calculating the design actions which then leads to the design of its superstructure and substructure. A reduction in weight results in reduced substructure and foundation costs. Weight reduction can also be beneficial for difficult soil conditions such as collapsible soil that may have excessive settlement under additional load [5] meaning that a site that is unsuitable for a heavy RC building may be appropriate for a lighter CFS building. This paper focuses on the practical implications of lightweight construction on design loads for which there is a lack of quantification in current literature.

Many authors have used a case study methodology to compare differing facets of building structural systems. Paulson [6] compared wind and seismic base shears on a prototype building in a number of locations in the United States (US), finding that generally seismic base shear forces dominate over wind base shear forces for short and heavy structures. Lenon [7] designed a cross-laminated timber (CLT) building for wind and seismic actions and a comparison of wind loads for low, medium and high rise buildings has been carried out by Holmes [8]. Panchal and Marathe [9] compared design loads for a steel-concrete composite and RC high rise case study. Starossek [10] compared concrete and lightweight structures for bridges in a weight vs. cost context. Other studies have compared environmental effects of concrete and steel buildings e.g. [11,12]. The authors have been unable to find studies in the literature comparing the design loads for RC and CFS mid-rise apartment buildings. This study aims to compare the design actions of permanent, imposed, wind and seismic loads for RC and CFS structures with the intention of foundation cost analysis in future work.

2. ARCHETYPE CASE STUDY AND DESIGN

To assess the effect on design loads, a case study methodology has been employed. A typical RC structure mid-rise building was designed for a notional site in Sydney. The archetype was designed to comply with all necessary regulations [13,14,15]. A 3D architectural Revit model of the building and a floor plan are depicted in Figures 1 and 2 respectively.

![Figure 1: 3D Revit model of Archetype building](image-url)
The building is seven storeys above ground level with a single basement car park level. The building has a step-back for the upper two storeys as is common for a building of this size in this sector. The building is considered to be of importance level 2 with an ordinary consequence of failure [16]. Design working life is 50 years, giving an annual probability of exceedance of 1/500 was to be used for both wind and seismic design. The key building characteristics are presented in Table 1.

Table 1: Archetype building characteristics

<table>
<thead>
<tr>
<th></th>
<th>Value (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor to floor heights</td>
<td></td>
</tr>
<tr>
<td>B-G</td>
<td>2.7</td>
</tr>
<tr>
<td>G-1</td>
<td>4.0</td>
</tr>
<tr>
<td>1-2, 2-3, 3-4, 5-6</td>
<td>3.1</td>
</tr>
<tr>
<td>4-5</td>
<td>3.2</td>
</tr>
<tr>
<td>Total height</td>
<td>23.7</td>
</tr>
<tr>
<td>Spans</td>
<td>2.5 – 7.5</td>
</tr>
<tr>
<td>Length</td>
<td>54</td>
</tr>
<tr>
<td>Width</td>
<td>19</td>
</tr>
</tbody>
</table>

A preliminary structural design was undertaken with two structural systems (CFS and RC) to ensure structural stability and a relatively efficient design. Consideration is made for the superstructure only, as the substructure is beyond the scope of this paper. Figure 3 outlines the methodology for the study; establishment of archetype design parameters, determination of actions and comparison of governing load cases.

Design actions were calculated for both the CFS and RC structures, in a Sydney and Brisbane
location with two soil conditions, Class C and Class E, for a total of eight scenarios.

3. STRUCTURAL SYSTEM COMPARISON

The archetype building was designed for an RC structure initially, then for a CFS structure. Concrete members were designed using AS3600 [17] and CFS members were designed using ASD approach from the NA AISI standards as this method has been used in the US for multistorey CFS buildings and could be validated by a structural engineer. There are a number of differences in the loadbearing structure of the two systems as detailed in Table 2.

<table>
<thead>
<tr>
<th>System</th>
<th>Reinforced Concrete</th>
<th>Cold-Formed Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load bearing</td>
<td>RC unbraced blade columns continuous down the length of building. Dimensions of 1200 x 200 mm</td>
<td>152 mm deep CFS stud walls. Walls are continuous down the length of building. 600 mm on-centre spacing of studs, some double stud walls</td>
</tr>
<tr>
<td>Lateral load resisting</td>
<td>RC U-shaped limited ductile shear core. 200 mm thick</td>
<td>CFS shear walls. CFS stud walls are laterally braced by the wall sheathing.</td>
</tr>
<tr>
<td>Floor</td>
<td>Two-way PT flat plate. 200 mm deep</td>
<td>203 mm deep cold-formed steel joists at 600 mm on-centre spacing. 15 mm form deck with a 39.7 mm gypsum based floor topping. Joists are balloon frame</td>
</tr>
<tr>
<td>Roof</td>
<td>PT flat plate. 200 mm deep</td>
<td>CFS trusses at 900 mm on-centre. Trusses support a 40 mm top hat batten, fibreglass insulation and a 0.47 mm TCT metal roof deck</td>
</tr>
<tr>
<td>Façade</td>
<td>Combination of steel panels, brick and cement render</td>
<td>Combination of steel panels, brick and cement render</td>
</tr>
</tbody>
</table>

The CFS systems used are common for a US context and comply with Australian requirements. It should be noted that some hot rolled steel is required for the CFS building’s lower storeys to account for load transfer in the open area of the ground floor.

4. PERMANENT LOAD

Permanent loads consist of the self-weight of the load-bearing structural system and superimposed dead load. Superimposed dead load accounts for non-structural members including
the facade, floor cover (levelling grout, tiles, etc.), suspended ceiling and services. The self-weight of concrete is approximated at 25 kN/m³, a slab depth of 200 mm would provide a dead-load of 5 kPa. CFS load-bearing floor components (joist, metal deck, gypsum concrete overlay) are much lighter with a self-weight of 0.8 kPa. Superimposed on these loads is an allowance for partitions, floor cover, insulation, ceiling and MEP amounting to 0.9 kPa. The floor areas were taken from the Revit model and used for floor self-weight and superimposed dead load (ceiling, floor finish, partitions) of the area. A number of component quantities were taken from a Revit model of the building to get exact dimensions, including, loadbearing walls and façade. The computed loads are compared in Table 3.

Table 3: Permanent loads comparison

<table>
<thead>
<tr>
<th></th>
<th>CFS (kN)</th>
<th>RC (kN)</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>11537</td>
<td>40134</td>
<td>71.3%</td>
</tr>
<tr>
<td>Wall/column</td>
<td>6283</td>
<td>11327</td>
<td>44.5%</td>
</tr>
<tr>
<td>Façade</td>
<td>4288</td>
<td>4288</td>
<td>0.0%</td>
</tr>
<tr>
<td>Total</td>
<td>22108</td>
<td>55749</td>
<td>60.3%</td>
</tr>
</tbody>
</table>

The reduction represents the difference between the CFS and RC loads as a percentage of the RC load. The greatest savings come from the floor at 71% reduction, it should be noted that there is a partition allowance on both the floor loads. The wall/column comparison is the load bearing walls for CFS and the columns and shear walls for RC, the façade is the same for both buildings. CFS has a significant final weight reduction of 60%.

5. IMPOSED LOAD

The magnitude of the imposed load is dependent on the type of activity/occupancy and the location in the building [18]. Table 4 depicts the surface loads applied in each location of the building.

Table 4: Imposed surface loads

<table>
<thead>
<tr>
<th>Location</th>
<th>Load (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Residential</td>
<td>1.5</td>
</tr>
<tr>
<td>Corridor/stairs</td>
<td>4</td>
</tr>
<tr>
<td>Balconies</td>
<td>2.5</td>
</tr>
<tr>
<td>Roof</td>
<td>0.25</td>
</tr>
</tbody>
</table>

These uniformly distributed imposed actions were applied to the floor in SAFE 2014 as depicted in Figure 4. A reduction factor was applied in accordance with AS/NZS1170.1 [18] based on the area supported by an element (substructure). Live loads are the same for both RC and CFS buildings. For the purposes of determining overall structural actions, distributed surface loads were considered over point and line loads as they provide the greatest load.
The live loads for both buildings are the same as they have the same floor plan. The total reduced live load was calculated to be 6331 kN.

6. WIND ACTIONS

Design wind actions are calculated in accordance with AS/NZS 1170.2 [19] and are dependent on the location and importance of the building. Hence, wind actions are identical for the two building structures. For buildings less than 25 m in height, wind actions are approximated as an equivalent horizontal base shear and overturning moment. The natural frequency of building is greater than 1 Hz so the design is based on static wind loads and dynamic response is not considered. The surface of the balcony balustrades and external walls are considered as tributary area for wind pressure, with balconies taking up 25% of the building’s surface, the building’s windward face surface area is multiplied by 1.25. Frictional drag factor is not calculated for a building of these dimensions and torsional forces are only calculated for buildings over 70 m. The base shear is the addition of positive external windward force and negative external leeward force, overturning moment is the moment about the base of the structure due to the wind pressure. The actions shown in Table 5 are for the worst case scenario in each location.

<table>
<thead>
<tr>
<th>Location</th>
<th>Brisbane</th>
<th>Sydney</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Shear (kN)</td>
<td>2947</td>
<td>1837</td>
</tr>
<tr>
<td>Overturning Moment (kNm)</td>
<td>34921</td>
<td>21765</td>
</tr>
</tbody>
</table>

The wind loads for both types of structure in each location are the same as the dimensions do not change. Brisbane has a higher design wind speed than Sydney causing a larger base shear and overturning moment.

7. SEISMIC ACTIONS

The seismic actions are computed using AS1170.4 [20]. There are a number of variables including the building location, height and the stiffness and mass of the structural system. The structure has an earthquake design category of 2, meaning a static analysis is sufficient for this comparison. Horizontal equivalent static forces were computed and applied to the building distributed vertically. The design actions are dependent on structural ductility and structural performance factor; these are based on the materials and type of lateral load resisting system. The seismic-force-resisting system for RC is a limited-ductile shear wall core and for CFS, a sheathed shear wall system which is specified as ‘Other’ in the standard. The soil type plays a large role in seismic design loads, so two soil conditions were selected for each location. Soil Class C is
shallow soil and Class E is very soft soil. Table 6 depicts the base shear and overturning moments for each case.

Table 6: Seismic action comparison

<table>
<thead>
<tr>
<th>Location</th>
<th>Sydney</th>
<th>Brisbane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Class</td>
<td>C</td>
<td>E</td>
</tr>
<tr>
<td>Building Type</td>
<td>RC</td>
<td>CFS</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>3247</td>
<td>1372</td>
</tr>
<tr>
<td>Moment (kNm)</td>
<td>55505</td>
<td>22843</td>
</tr>
</tbody>
</table>

The base shear and moment are much higher for the RC structure than CFS for all cases due to the higher seismic weight which consists of the dead load and a factored live load for each floor. The seismic actions in Sydney are higher than Brisbane and actions are greater in very soft soil than shallow soil.

8. COMPARISON OF ACTIONS

Actions are put into limit state load combinations for either strength or serviceability design. Load combinations factor the loads, accounting for uncertainty and the probability of each combination. Some of the load combinations from AS1170.0 [16] are detailed in Table 7.

Table 7: Load Combinations

<table>
<thead>
<tr>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loads</td>
</tr>
<tr>
<td>G</td>
</tr>
<tr>
<td>Q</td>
</tr>
<tr>
<td>Wu</td>
</tr>
<tr>
<td>Eu</td>
</tr>
<tr>
<td>Combinations</td>
</tr>
<tr>
<td>G + Q</td>
</tr>
<tr>
<td>1.35G</td>
</tr>
<tr>
<td>1.2G + 1.5Q</td>
</tr>
<tr>
<td>1.2G, Wu, ΨQ</td>
</tr>
<tr>
<td>G, Eu, ΨQ</td>
</tr>
</tbody>
</table>

Note that in this study only loads in the same orientation are compared, hence the axial, lateral shear and moments are compared separately. Also for this reason, the axial loads in the load combinations containing lateral loads are not considered. The following sections compare the axial and lateral loads for each of the cases.

8.1 Axial Actions

The axial permanent and imposed load combination are depicted in Figure 5.
The governing load combination for both RC and CFS is $1.2G + 1.5Q$. Comparing this governing axial design load, CFS has a reduction of 53%. This load combination would be used for foundation strength design, a lower load means that a CFS structure could use lighter, less expensive foundations further reducing the load. The serviceability combination $(G+Q)$ reduction of 54% is significant for settlement of foundations. CFS would be beneficial especially for soil garnering large settlements which might be unsuitable for the construction of an RC structure.

### 8.2 Lateral Actions

Base shear and overturning moment are the two actions caused by the lateral loads, wind and earthquake. Figure 6 compares the un-factored loads in each location.

![Base Shear Comparison](image1)
![Overturning Moment Comparison](image2)

**Figure 5:** Axial load comparison

**Figure 6:** (a) Base shear comparison (b) Overturning moment comparison
It is clear that the CFS seismic loads are smaller than RC in all cases, but depending on the location, wind may govern over seismic loads. Table 8 and 9 show the governing load cases for each location and structure. In both tables, the comparison shows the CFS loads as a percentage reduction relative to the RC loads.

<table>
<thead>
<tr>
<th>Soil Class C</th>
<th>Sydney</th>
<th>Brisbane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure</td>
<td>RC</td>
<td>CFS</td>
</tr>
<tr>
<td>Governing Shear</td>
<td>Seismic</td>
<td>Wind</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>3247</td>
<td>1837</td>
</tr>
<tr>
<td>Governing Moment</td>
<td>Seismic</td>
<td>Seismic</td>
</tr>
<tr>
<td>Overturning Moment (kNm)</td>
<td>5505</td>
<td>22843</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil Class E</th>
<th>Sydney</th>
<th>Brisbane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure</td>
<td>RC</td>
<td>CFS</td>
</tr>
<tr>
<td>Governing Shear</td>
<td>Seismic</td>
<td>Seismic</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>6417</td>
<td>2712</td>
</tr>
<tr>
<td>Governing Moment</td>
<td>Seismic</td>
<td>Seismic</td>
</tr>
<tr>
<td>Overturning Moment (kNm)</td>
<td>109700</td>
<td>45148</td>
</tr>
</tbody>
</table>

In Sydney, with Soil C, seismic governs for RC, whereas wind governs for CFS. The overturning moment is slightly higher due to seismic load than under wind load. In Sydney with Soil E, seismic governs for both. In Brisbane with Soil C, wind governs both. In Brisbane with Soil E, seismic governs RC but wind governs CFS. Figure 7 depicts the governing loads for RC and CFS in each scenario.

Figure 7: (a) Governing base shear, (b) Governing overturning moments

All cases show that the design loads for RC are higher than those for CFS; except in Brisbane with soil class C where the design loads are the same. This is due to the low seismicity and high wind loads which are independent of the structure type. The much lower base shear applied to the
CFS building of up to 63% less than RC leads to a smaller foundation volume and less lateral displacement for foundations in soil. The lower overturning moment means less demand on the lateral load resisting system and uplift of foundations.

9. CONCLUSIONS
The aim of this study was to compare the design loads for an RC and CFS framed mid-rise apartment building. Code-based analysis was carried out on the case study building to find axial and lateral loadings. The study found that the weight of the archetype building is reduced by 60% with a CFS structure. The governing axial load case of permanent and imposed actions had a reduction of 53%. The governing lateral actions were also shown to be reduced for the CFS structure with the greatest reduction of 58% base shear and overturning moment occurring in Sydney with soil class E. These results can be explained by the light weight of the CFS structure compared to the RC structure. This study has demonstrated the effect of location and found it is more advantageous to use CFS in a location with higher seismicity and poor soil conditions. The actions calculated can be applied in future research to make a cost comparison in substructure design and earthworks, these items accounting for a significant percentage of overall construction costs. Subsequent research should consider other locations and soil types as well as differing building heights as cumulative load effects would be higher for high-rise structures. Foundation design comparison is the next step for this research.

10. ACKNOWLEDGEMENTS
Funding from the Australian Research Council Industrial Transformation Research Hubs Scheme (Project Number IH130100017) is gratefully acknowledged. The authors also acknowledge the project industry partners BlueScope, Stockland and Cox Architecture.

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THE CASE FOR COLD-FORMED STEEL CONSTRUCTION FOR THE MID-RISE RESIDENTIAL SECTOR IN AUSTRALIA: A SURVEY OF INTERNATIONAL CFS PROFESSIONALS

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*Corresponding author

Abstract. Australia has a growing population, especially in dense city regions; this has led to an increase in the number of mid-rise apartment buildings being constructed. More efficient methods of construction could enable the sector to meet this increased demand. Internationally, cold-formed steel (CFS) is used widely as the loadbearing structure within mid-rise apartment buildings. However, the use of CFS for the construction of mid-rise apartment buildings in Australia has been extremely limited. At the time of writing, Australian mid-rise apartment buildings are typically constructed using reinforced concrete, and the potential benefits of using alternative construction systems are not well-known. The Australian construction sector could benefit from an increased understanding of the advantages of CFS construction. Through the use of both a desktop study and a survey of international CFS industry experts, this study aims to explore the benefits and practices of CFS construction. The study identifies current CFS industry practices that have been used in completed projects in the UK, USA and Canada. A variety of loadbearing and lateral load resistance systems exist in practice, as well as fire and acoustic solutions. The prominent advantages of CFS are found to be speed of construction, cost effectiveness and non-combustibility. Potential improvements for design and construction are identified, including load-path alignment and solutions for long spans. Addressing these issues could lead to a more efficient solution for an Australian structural CFS system. The survey ascertains the positive benefits of CFS construction and common methods of use in practice. This addition to the body of knowledge helps to close the gap between research and practice and demonstrates that CFS offers potential benefits for the mid-rise apartment construction sector in Australia.

Keywords: Loadbearing cold-formed steel; mid-rise apartment; state-of-the-art; Australia; industry practice

1 INTRODUCTION

Due to Australia’s growing population in city centres, there is less land available for housing. This has brought about an increase in construction of higher density mid-rise and high-rise apartments. Reinforced concrete (RC) frame is the typical structural system used for this building type in Australia because of the supply chain and knowledge of the current construction industry. However, there are other lightweight structural solutions that can improve the construction process such as Cold-formed steel (CFS) and Cross-laminated timber (CLT). CFS is used as a load-bearing construction material extensively in regions such as the United States of America (USA) and United Kingdom (UK). A number of CFS benefits have been reported in literature such as high strength-to-weight ratio, lightweight, portability,
offsite manufacturability, recyclability, and termite and mould resistance (Yu, 2016), (Heywood, Lawson, & Way, 2012). However, there is a lack of research into the application of CFS in real world projects. To advance the use of CFS in Australia, more knowledge is required on the construction process. Construction is an applied field of research that can be viewed with interacting elements of both subjectivity and objectivity (Bourdieu, 1990). People play key roles in most aspects of construction, meaning that qualitative research methods are appropriate (Abowitz & Toole, 2010). It is important to learn from the construction industries in other countries implementing this type of construction method so as to avoid unnecessary problems. There are many differences between the Australian and international contexts in terms of standards and codes, but the learnings from international case studies of similar design are a good starting point for this research. This study aims to explore various solutions for mid-rise CFS residential and student accommodation construction and to understand the advantages and disadvantages of CFS use.

2 METHOD

The aim is achieved through obtaining quantitative and qualitative data on existing CFS construction and creating detailed case studies. Initially, an exploratory desktop study was undertaken; CFS case studies were found in the USA, UK and Canada. The research includes 22 buildings with information sourced from a number of publically available sources including steel institutes, associations, company websites and magazines. The exploratory case study research functions as a source of insight and ideas from construction industry professionals. A survey research method was employed to develop detailed case studies. An internet survey was selected to gather data due to its simplicity and accessibility for international participants (Robson, 2011). Construction companies and professionals with experience in CFS mid-rise construction were selected to participate in an online survey containing both open-ended and closed questions. Participants were asked to consider a particular CFS construction project they were involved with and answer questions regarding it. The survey questions prompted information such as key data on the building including time, cost and size, type of construction, reasoning for the design and other questions regarding building information modelling (BIM), compliance, sustainability and onsite construction logistics. Since no loadbearing CFS buildings over four storeys have been completed in Australia at the time of writing, the participants were international. The building projects were selected through meeting criteria to suit research aims including CFS structural system, residential, height of four or more levels and completed within the last 10 years. Non-probabilistic convenience sampling was used. A number of relevant companies were also emailed based on findings from the desktop study. There were seven respondents to the survey. The rationale for using companies as a source of data is that they can draw on industry experience to provide a deeper understanding of CFS construction. This research is limited by the low response rate, making comparative study difficult. Data were collected using an online survey tool (Survey Monkey) for the period of September 2016 to December 2016. Due to the small sample size the analysis was of a case study style and not statistical. Presented findings are summarized into general themes.

3 RESULTS

3.1 Desktop Study

A range of projects from USA, UK and Canada were identified. The earliest dated project was from 2004 to the most recent completed in 2016. The majority of the cases used CFS as
the main loadbearing structure, with some incorporating structural steel to enable long spans. Stick, panelised and modular construction types were implemented and a variety of roof, floor and wall systems were used. Table 1 summarises the 22 projects of which 15 are residential, six are student accommodation, two are hotels and one is a hospital.

Table 1 Desktop study buildings

<table>
<thead>
<tr>
<th>Location</th>
<th>No.</th>
<th>Year constructed</th>
<th>CFS Storeys</th>
</tr>
</thead>
<tbody>
<tr>
<td>USA</td>
<td>12</td>
<td>2004-2014</td>
<td>3-9</td>
</tr>
<tr>
<td>UK</td>
<td>8</td>
<td>Up to 2015</td>
<td>4-11</td>
</tr>
<tr>
<td>Canada</td>
<td>2</td>
<td>2011-2013</td>
<td>4-5</td>
</tr>
</tbody>
</table>

3.2 Surveys

Data were collected from seven survey respondents, with four of them providing more detailed information on the structural system of a specific building as summarised in Table 2.

Table 2 Survey structural systems overview

<table>
<thead>
<tr>
<th>Case</th>
<th>Location</th>
<th>Type</th>
<th>CFS Storeys</th>
<th>Loadbearing wall system</th>
<th>Roof system</th>
<th>Floor system</th>
<th>Bracing system</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>USA</td>
<td>Hotel</td>
<td>4</td>
<td>Depth: 92mm - 152mm</td>
<td>Thermoplastic polyolefin rigid insulation over 12mm metal deck &amp; CFS joist</td>
<td>40mm gypsum concrete over 12mm metal deck &amp; CFS joist</td>
<td>Sheet steel CFS shear walls, structural steel</td>
</tr>
<tr>
<td>B</td>
<td>Canada</td>
<td>Apartments</td>
<td>6</td>
<td>Depth: 162mm - 203mm</td>
<td>Flat roof - CFS trusses</td>
<td>203mm hollow-core concrete</td>
<td>CFS X-bracing and Masonry shaft</td>
</tr>
<tr>
<td>C</td>
<td>UK</td>
<td>Student</td>
<td>7</td>
<td>Depth: 92mm (Asymmetrical lipped C)</td>
<td>Flat roof - 254mm lipped C Cassette 1.5mm S390 steel</td>
<td>254mm lipped Cee Cassette</td>
<td>CFS X-bracing 80, 50 &amp; 40mm wide flat strap</td>
</tr>
<tr>
<td>D</td>
<td>UK</td>
<td>Student</td>
<td>7</td>
<td>Depth: 92mm Gauge: 1.5mm, 1.2mm and 1mm</td>
<td>Cassette - 254mm x 1.5/2.5mm joist, with 22mm OSB decking</td>
<td>Cassette - 254mm x 1.5/2.5mm joist, with 22mm OSB decking</td>
<td>CFS X-bracing</td>
</tr>
</tbody>
</table>

4. FINDINGS

As well as providing information on the systems used, survey respondents offered their views on the CFS construction process with some common themes arising. The three main advantages described by respondents were speed of construction, cost effectiveness and non-combustibility. A number of categories were determined based on the information provided by survey respondents and desktop studies: structure, time and cost, building information...
modelling (BIM), height, fire and acoustics, and sustainability. These categories are discussed in the proceeding section.

4.1 Structure

The loadbearing walls consist of studs that vary (as demonstrated in the surveys) in depth and gauge depending on the purpose and manufacturing capabilities. Yu (2016) states the typical range of section thicknesses is from 0.373mm to 6.35mm. From the survey responses, stud gauge was found to range from 1.2 - 2.4mm and stud depth was found to vary from 92 - 203mm. Connections between members were generally self-tapping fasteners. The steel strength is higher than that of hot-rolled steel with yield strengths of 380 - 390 MPa being reported, whilst in Australia high strength steel of 550 MPa is standard for CFS sections (Yu, 2016). There are different options for lateral load resisting system such as CFS shear walls, RC core, structural steel bracing or a combination. The Case A structure utilised CFS walls with a layer of thin sheet steel as well as some structural steel braced frames to resist shear. Figure 1 displays the third and fourth floor layout of Case A. The East-West direction had suites on the first floor which allowed the sheet steel shear walls to continue from the first floor to the roof. At the lower levels, the opposite side of the East-West corridor and the North-South wing was open area that was supported by structural steel. The upper storey shear walls on the North-South wing were supported by moment frames and braced frames. To simplify the foundations, the moment frames were aligned and spaced on every-other party wall.

![Shear wall layout Case A](image)

This structure was designed as a flexible diaphragm unlike a structure with a reinforced concrete core lateral resisting system. Another example of this style is the Pearl City Assisted Living Facility which used sheet steel and gypsum composite shear panels for lateral resistance and a tie-down anchor system to prevent shear wall overturning. Harbour Court Student Residence used CFS walls that were X-braced. On the lower ground a moment resisting frame with inclined struts was used for stability. Case B, C and D also use CFS X-bracing. Harvard Yards used welded structural steel braced frames for lateral load resistance. Convent Hill made use of cast in place reinforced concrete stairs and elevator cores, The 400, Washington State was provided with lateral resistance by reinforced concrete shear walls as
the building is in a high seismic zone. The loads were transferred through a rigid diaphragm concrete slab to the concrete shear cores. Many of the cases noted the reason for selecting CFS was its lightweight which can be up to 50% lighter than that of a concrete alternative (BuildSteel, 2017). This can be vital when constructing on sub-optimal soils, for example, St Mary’s Village Studios was built on a steep slope with ground anchor stabilised soil, a lightweight CFS construction method made this development possible. Respondents highlighted the importance of the load-bearing wall layout. Some cases had alignment issues which were solved through parametric design, architectural redesign and/or introduction of hot-rolled steel (HRS). Up to 100 tonnes of HRS was used in one case where there were long spans in the lobby area that CFS joists do not have the capacity to span. The respondent believed that coordination between CFS and HRS contractors is essential to reduce this type of over design. Alignment of the walls so that load paths are continuous down the height of the building makes CFS design simpler and works to its strengths.

4.2 Time and Cost

Time and cost have remained two of the key factors determining the success of a construction project throughout history (Chan & Chan, 2004). CFS construction can reduce the construction duration by up to 50% compared with traditional methods (Lawson, 2012). One survey respondent understood that the speed and cost benefits come in large part, from the ability to prefabricate large amounts of the building: “Panels were built one month in advance of need on site”. Pre-panelised construction allows fast installation of the building envelope and following trades early access (Lawson, 2012). Another respondent stated that their client finished the project two months ahead of schedule which is significant in an industry where budgets and schedules are more often underestimated (Aibinu & Pasco, 2008). Construction time can be further reduced because of the ability to concurrently phase CFS construction. Figure 2 depicts the construction schedule for Case B.

![Figure 2 Case B structure Gantt chart](image)

The building’s structure took 90 days to erect, roughly 10 days per floor. It is can be seen that as the floor was still in construction, the loadbearing walls and any HRS are erected.
There is also a slight overlap between the walls for one level and the flooring above it. Another example of phased construction is Elan Westside Apartments, Atlanta. To improve speed of construction, each floor was divided into three phases to be completed simultaneously in different areas of the building footprint. Some cases demonstrated the benefits of CFS use under different site constraints. Heathrow Hotel was built in a long narrow site with limited access. In this case CFS wall panels were selected over precast concrete due to the lightweight of the steel frames and the accuracy and speed of erection. The Sheridan College Student Residence in Canada opted to use CFS so that it was possible to complete the residence by the start of the university semester. Concrete did not need to be poured in the colder months when lengthy times are required to cure, greatly reducing construction time. Another example of time saving is Colborne Street, Brantford, Ontario. The developer of this four storey residential CFS building believed construction time was reduced by 5-8 weeks. The time saving was due to the fast installation of CFS framing at a rate of one floor per week. It was claimed that an extra $100,000 CAD in rent was earned because of the early occupancy. Many cases noted that there were savings in insurance premiums for construction with CFS. As steel is non-combustible there is reduced risk of fire as compared with timber and due to the short time on site the workers’ insurance premiums are reduced (BuildSteel, 2018). Respondents stated that savings also came from lower construction financing charges, pared site supervision needs, and less labour.

4.3 Building Information Modelling

BIM has been demonstrated to improve the process flow of construction projects in the past (Azhar, 2011). A McGraw Hill construction report stated that 45% of current BIM users will be using BIM on at least 60% of their projects (Construction, 2008), one survey respondent stated that BIM was used in 80% of company projects. Preferred programs were Revit and AutoCAD for architectural design, Excel, Tedds, Statica and Sketchup for structural analysis and in-house software for manufacturing. The developer of Elan Westside Apartments selected CFS because of the ability of the CFS engineer to create all of the structural framing drawings, layout drawings, shop drawings and material rolling information in a single model. For the Holiday Inn Express it was noted that BIM and the use of CFS were required for everything to fit together precisely. Plans showed where each wall panel was to be installed and in what order. Holes for services were already fitted into the panels during offsite fabrication. Another example of a project that used BIM for construction accuracy is the AQ Rittenhouse project. CFS was not used for any structural purposes; however it was used for external framing over 12 storeys. The framing was installed in very tight quarters and within a high-traffic area. The panelised external walls came with stud, track, connection framing, exterior sheathing and finishes applied from the factory. Infill walling was not used to prevent the slab edge from being exposed so hot-rolled steel double bent angles were installed at each slab, with the panels welded to these. BIM was used for accuracy in placement of slots in bent angles for studs and jambs.

4.4 Height

Building height is a limitation for CFS construction compared with concrete and structural steel construction. This is because the complexity of the construction system necessarily increases with building height, reducing feasibility. CFS studs often need to be doubled or tripled in the lower levels as the building height increases. At the time of writing, the highest CFS building in Australia was four storeys (Dynamic Steel, 2015). Australia currently has no “deemed to satisfy” provisions for apartments with three or more storeys using a CFS
structure (Australian Building Codes Board, 2016). However, international projects demonstrate that there is opportunity for much taller buildings to be constructed using loadbearing CFS. The iQ is an 11 storey student accommodation building in the UK. P.A.W. Structures installed the CFS structure in less than 17 weeks. An example from the USA is the Rodin Square Project, a nine storey residential building. The project uses ClarkDietrich HDS (Heavy-Duty Stud) structural studs with greater loadbearing capacity compared with standard stud shapes. Many of the first floor studs carry over 22,000 kilograms. The walls also provide lateral bracing for the building so no masonry core was required for the building. These cases demonstrate that with efficient design practices and potential improvements, it is possible to build CFS structures with more than three storeys.

4.5 Fire and Acoustics

Fire and acoustic requirements for CFS construction are provided through prescriptive design as there is a lack of understanding of the performance of CFS members, subsystems, and systems (Yu, 2016). Depending on the local standard used, there are different requirements for fire and acoustics. The USA, UK and Canada all have demonstrated solutions for their respective codes. A UK project makes use of mineral wool, sound resistant plasterboard and resilient bars to achieve an airborne sound reduction of over 63 dB. In 2014, Oval Quarter was the largest CFS project in the UK at the time of construction, valued at over £100 million. For the flooring, acoustic insulation was achieved with a 21mm deep screed board on top of CFS joists and two layers of 15mm fire resistant plasterboard with a service zone gap and a final layer of plasterboard ceiling below. The City Green project in the USA provided sound reduction through the combination of structural concrete floor sheathing panels topped with a poured-in-place underlayment. The underlayment had mats both above and below it achieving sound attenuation scores of STC (Sound Transmission Class) 55.5 to 59.77 and IIC (Impact Insulation Class) 58.5 to 62.4. Many of the case studies noted that another reason for CFS selection was its non-combustibility.

4.6 Sustainability

Some of the noted advantages with regard to sustainability include the amount of recycled content in the steel, reduction of construction waste and in terms of building energy sustainability, it provides additional room for insulation in the walls to reduce heating and cooling load on the building. One respondent stated that the CFS components achieve A+ and A ratings in the UK’s BRE Green Guide which ranks building materials and components from A+ to E, where A+ represents the best environmental performance (Building Research Establishment Ltd, 2018). CFS has a long lifespan as there are not the shrinkage and moisture problems that exist with timber construction. There are no requirements for termite proofing and CFS components can simply be recycled at the end of a building’s lifetime.

5 CONCLUSION

The paper has shown that CFS is a viable method for mid-rise residential construction and there are a number of benefits associated with its use. These findings demonstrate the range of potential system types possible for CFS structures. More effective ways are also being developed as use of CFS matures in these international markets. The advantages found consistently in the case studies, such as speed and cost effectiveness, are highly sought after in the construction industry. Some of the challenges with CFS construction were also mentioned, but these can be seen as opportunities for the development of a new system for Australia. This
research is limited in that the sources of information were public case studies produced, in most cases, by companies involved with the project. This means there is the potential for bias and for positive conclusions to be focussed on. There is also a lack of publically available information highlighting the need for academic research exploring the potential of CFS. Mid-rise CFS construction is still a relatively new method and can be improved through further investigation of current practice and development of new practices. The case studies reviewed reveal a number of different potential benefits in using CFS and how these are currently exploited. This present study can lead to more in-depth research providing greater insight into the use of CFS in mid-rise construction and how it could be implemented in Australia.

6 ACKNOWLEDGEMENTS

Funding from the Australian Research Council Industrial Transformation Research Hubs Scheme (Project Number IH130100017) is gratefully acknowledged.

REFERENCES


Appendix C

Application for ethical approval

Participant Information Sheet
A. CHECKLIST (for applicants)

Please check the Ethics web page for agenda deadlines [http://www.uow.edu.au/research/ethics/UOW009377.html](http://www.uow.edu.au/research/ethics/UOW009377.html) and ensure this checklist is completed before submission. Applications should be sent or delivered to:

Ethics Unit, Research Services Office
Level 1, Building 20 (North Western Entrance)
University of Wollongong NSW 2522

☐ Original Ethics Application plus appropriate number of copies.
  - Applications for the full Human Research Ethics Committee, Please note BOTH the Health & Medical HREC AND the Social Sciences HREC require 17 copies PLUS the original.
  - Applications to the Executive Committee of the HREC (expedited review) require only the original.
  - Double-sided printing is preferred.

☐ Participant Information Sheet/Package. *(Please include version number and date)*

☐ Consent Form/s. *(Please include version number and date)*

☐ Copies of questionnaire/s, survey/s or interview/focus group questions. *(Please include version number and date)*

☐ Copies of all material used to inform potential participants about the research, including advertisements and letters of invitation. *(Please include version number and date)*

☐ Evidence of permission to conduct research from site managers *(Not required for research sites within NSW Department of Health at this stage)*

☐ Evidence of approval/rejection by other HRECs, including comments and requested alterations to the protocol.

☐ Copies of Confidentiality Agreement templates for any third parties involved in the research.

☐ Copy of Research Contract for sponsored/contract research.

☐ Copy of Clinical Trial Insurance Requirements Form *(UOW researchers answering YES to Q.10 only)*

☐ Privacy Exemption Application *(Researchers answering NO to Q.38 only)*

For Clinical Trials also include:

☐ Protocol (17 copies)

☐ CTN or CTX Form (1 original copy)

☐ Summary Sheet (17 copies)

☐ Insurance Information (1 copy)

☐ Budget (17 copies)

☐ Clinical Trial Agreement (1 copy)

☐ Investigator’s Brochure (6 copies)
B. GENERAL INFORMATION

1. DESCRIPTIVE TITLE OF PROJECT:
A survey of cold formed steel (CFS) companies relating to specific mid-rise residential building projects.

2. 7 LINE SUMMARY OF PROJECT AIMS:
This research project is an exploratory study within the larger Steel research Hub project B2.1 based on the development of a prototype CFS building system for Australia. The study aims to explore various solutions for cold formed steel construction and compare international with Australian methods. It also aims to understand the advantages and disadvantages of cold formed steel use and increase the knowledge database of the Steel research Hub. This will be achieved through obtaining quantitative and qualitative data on existing cold formed steel construction and building detailed case studies for use in a state of the art paper.

3. PARTICIPATING RESEARCHERS:
Summarise the qualifications and experience of all personnel who will be participating in the project. **NB: For student research a Supervisor must be the Principal Investigator.**

<table>
<thead>
<tr>
<th>Principal Investigator/Supervisor</th>
<th>First Name</th>
<th>Family Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>Professor</td>
<td>McCarthy</td>
</tr>
<tr>
<td>Email: <a href="mailto:timmc@uow.edu.au">timmc@uow.edu.au</a></td>
<td>Phone No: +61 2 4221 4591</td>
<td></td>
</tr>
<tr>
<td>Qualifications</td>
<td>BE, MSc, PhD, MIEI Professor of Structural Engineering</td>
<td></td>
</tr>
<tr>
<td>Position</td>
<td>Professor of Structural Steel and Design, Director of Engineering and Mathematics Education Research Group</td>
<td></td>
</tr>
<tr>
<td>Role in Project, relevant research experience (if no experience describe how relevant experience will be obtained)</td>
<td>Supervisor. Experience in supervising industry linkage projects. Experience supervising MPhil and PhD students conducting surveys and interviews in the fields of sustainable construction and structural engineering.</td>
<td></td>
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<table>
<thead>
<tr>
<th>Second Investigator</th>
<th>First Name</th>
<th>Family Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>Doctor</td>
<td>Heffernan</td>
</tr>
<tr>
<td>Email: <a href="mailto:eheffern@uow.edu.au">eheffern@uow.edu.au</a></td>
<td>Phone No: +61 (0)2 4239 2143</td>
<td></td>
</tr>
<tr>
<td>Qualifications</td>
<td>BA(Hons), PG Dip Arch, MA, PhD</td>
<td></td>
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<tr>
<td>Position</td>
<td>Research Fellow SBRC</td>
<td></td>
</tr>
<tr>
<td>Role in Project, relevant research experience (if no experience describe how relevant experience will be obtained)</td>
<td>Supervisor. Experience in conducting online questionnaire surveys at both Masters and PhD level. Experience in conducting research within the field of sustainable construction including interviews and a range of qualitative and quantitative methods.</td>
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<table>
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<th>Co-Investigator/Student</th>
<th>First Name</th>
<th>Family Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>Mr</td>
<td>Franklin</td>
</tr>
<tr>
<td>Email: <a href="mailto:nf932@uowmail.edu.au">nf932@uowmail.edu.au</a></td>
<td>Phone No: 0450528007</td>
<td></td>
</tr>
<tr>
<td>Qualifications</td>
<td>BEng (Hons)</td>
<td></td>
</tr>
<tr>
<td>Position</td>
<td>PhD Student, Sustainable Buildings Research Centre, Faculty of Engineering and Information Sciences</td>
<td></td>
</tr>
<tr>
<td>Role in Project, relevant research experience (if no experience describe how relevant experience will be obtained)</td>
<td>Survey design and analysis of research data. Experience will be obtained through reviewing previous questionnaire surveys.</td>
<td></td>
</tr>
</tbody>
</table>

Please add or delete extra boxes for additional researchers

4. CONTACT DETAILS FOR CORRESPONDENCE: *(Please note that most correspondence is sent electronically so please ensure that email addresses are included)*

**Name:** Mr Nicholas Franklin

**Postal Address/School/Unit/Dept & Faculty:** Sustainable Buildings Research Centre, Faculty of Engineering, University of Wollongong, Wollongong NSW 2522

**Email:** nf932@uowmail.edu.au

**Phone:** Mobile:

If principal contact is not the Principal Investigator (PI) please provide the contact details for the PI:

**Name:** Prof Timothy McCarthy

**Postal Address/School/Unit/Dept & Faculty:** Sustainable Buildings Research Centre, Faculty of Engineering, University of Wollongong, Wollongong NSW 2522

**Email:** timmc@uow.edu.au

**Phone:** +61 2 4221 4591 Mobile:

5. EXPECTED DURATION OF RESEARCH: *(Please specify as near as possible start and finish dates for the conduct of research)*

**FROM:** September 2016 **TO:** December 2016
6. **Purpose of project:**
   Indicate whether the research is one or more of the following:

   - [x] Staff Research (University of Wollongong)
   - [ ] Staff Research (ISLHD)
   - [x] Student Research *(Please specify):*
   
   Course undertaken: **Doctor of Philosophy**
   
   Unit/Faculty/Department: **Sustainable Buildings Research Centre, Faculty of Engineering**
   
   Supervisor/s: ________________________________
   
   - [ ] Other *(Please specify):* ________________________________

7. **HAS THIS RESEARCH PROJECT BEEN REVIEWED BY ANY OTHER INSTITUTIONAL ETHICS COMMITTEE?**
   
   YES [ ] NO [x]

   If NO go to Section C. If YES:

   (a) What committee/s has the application been submitted to?

   (b) What is the current status of this/these applications? *Please include copies of all correspondence between the sponsor or researcher and the other ethics committee/s.*

C. **FINANCIAL SUPPORT FOR RESEARCH**

8. **WHAT IS THE SOURCE AND AMOUNT OF FUNDING FROM ALL SOURCES FOR THIS RESEARCH?**

<table>
<thead>
<tr>
<th>Source (Name of Organisation/Funding Scheme)</th>
<th>Amount</th>
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<tbody>
<tr>
<td>ARC Research Hub for Australian Steel Manufacturing PhD Scholarship</td>
<td>PhD Scholarship</td>
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</tbody>
</table>

   For sponsored research please include the budget for the trial including information about capitation fees, payments to researchers, institutions or organisations involved in the research, current and consequential costs and costs which may be incurred by participants. If the research is sponsored:

   (a) Is there any affiliation/association or financial interest between the researcher/s associated with this research and the sponsor/funding body/supplier of a drug, surgical device or other therapeutic device to be used in the study?

   YES [ ] NO [x]
If YES please detail.

(b) Are there any conditions placed on this research by the funding body?

YES ☐ NO ☒

If YES, please provide details and provide a copy of the contract/letter of agreement with the funding organisation detailing the terms on which the research is being supported.

(c) Is a copy of the HREC approval to be forwarded to the granting body?

YES ☐ NO ☒

If YES, please advise of any deadlines.

D. RESEARCH METHODS

9. RESEARCH CATEGORIES:

Please mark the research categories relevant to this research proposal. At least one category should be marked for each grouping. You should mark as many categories as are relevant to the proposed research. For OTHER please specify.

(a) RESEARCH PROCEDURES USED:

☐ Anonymous questionnaires/surveys
☐ Coded (potentially identifiable) questionnaires/surveys
☒ Identifiable questionnaires/surveys
☐ Examination of student work, journals etc
☐ Examination of medical, educational, personnel or other confidential records
☐ Observation (overt)
☐ Observation (covert)
☐ Interviews (structured or unstructured)
☐ Telephone interviews
☐ Procedures involving physical experiments (eg exercise, reacting to computer images)
☐ Procedures involving administration of substances (eg drugs, alcohol, food)
☐ Physical examination of participants (eg blood glucose, blood pressure and temperature monitoring)
☐ Collection of body tissues or fluid samples
☐ Surgical procedures
☐ Other ________________________________

(b) RESEARCH AREAS:
☒ Qualitative research
☐ Social science research
☐ Humanities research
☐ Educational research
☐ Health research
☐ Psychological research
☐ Comparison or evaluation of drugs, surgical or other therapeutic devices
☐ Comparison or evaluation of clinical procedures
☐ Comparison or evaluation of counselling or training methods
☐ Investigation of the effects of an agent (drug or other substance)
☐ Investigation of biomechanical processes
☐ Biomedical research
☐ Epidemiology
☐ Genetic research
☐ Other ________________________________

10. DOES THE PROJECT INVOLVE THE USE OF A THERAPEUTIC INTERVENTION, DRUGS, A SURGICAL DEVICE, OR A PHYSIOLOGICAL TRIAL?

YES ☐ NO ☒

If NO to Q.11.

If YES:
(a) Please give details of the type of intervention and provide evidence that appropriate indemnity and compensation arrangements are in place to ensure adequate compensation to participants for any injury suffered as a result of participation in the trial (indemnification forms). If the research is being undertaken in a private practice please provide evidence of adequate and appropriate insurance coverage.

(b) Is the research registered:
☐ As a CTN Trial with the TGA
☐ As a CTX Trial with the TGA
☐ On any national or international clinical trial registers
☐ Other (please detail) ________________________________

11. RESEARCH DESIGN AND JUSTIFICATION:
Describe what you want participants to do and justify the design with reference to relevant literature. Please provide an explanation in terms that can be understood by a non-expert reader. A flow chart or other diagram illustrating the sequence of research activities should be included if possible. For research involving a treatment or physical intervention (eg clinical studies, physiological trials, mental health interventions) a protocol should be provided.

Participants (overseas and Australian experts) will be asked to complete a survey containing both open-ended and closed questions. The survey will be sent via email. Participants will be asked to consider a particular project they were involved with and answer questions regarding it.

Survey questions layout:
1. Key data on the building including time, cost and size
2. Type of construction for roof, walls, floor
3. Reasoning for the design, what went well and what didn’t’ throughout the construction process
4. Other questions regarding building information modelling, compliance, sustainability and onsite construction logistics.

Numbers 3 and 4 in the layout are required for the study because they give the participant the opportunity to provide an opinion and potentially useful information that would not be gathered with closed questions.

They will be asked to provide, if possible:
1. Plan view of a floor - To get apartment ratios and areas
2. Elevation view - To get floor heights
3. Construction details - For thermal and acoustic details
4. Publications - For any further information for case study

As the research will be part of an exploratory study, it will also be used to define the future focus of the student’s PhD. The survey was selected as the information collected can be used to create detailed case studies which can be put into a state of the art research paper. This research will also aid the objectives of the Steel Research Hub in creating a prototype cold formed steel building.

Sequence of research activities:
Desktop study > Survey > Data Collection > Analysis > Dissemination

12. STATISTICAL DESIGN:

Any research project that involves the collection of data should be designed so that it is capable of providing information that can be analysed to achieve the aims of the project. Usually, although not always, this will involve various important statistical issues and so it is important that the design and analysis be properly planned in the early stages of the project. You should seek statistical advice. The University of Wollongong has a Statistical Consulting Service that provides such advice to research students and staff undertaking research. Are statistical issues relevant to this project?

YES ☐ NO ☒

If NO to Q.13.

If YES:
(a) Have you discussed this project with the Statistical Consulting Service or any other statistical advisor?

YES ☐ NO ☐
If NO, please explain why not.

(b) Provide the calculations used to determine the appropriate sample size. If no power calculations have been done please explain the reason for choosing the sample size.

E. ETHICAL CONSIDERATIONS

13. What are the ethical considerations relevant to the proposed research, specifically in relation to the participants’ welfare, rights, beliefs, perceptions, customs and cultural heritage? How has the research design addressed these considerations? Consideration should be at both individual and collective levels.

Burden of time – The survey method was selected because it can be done on the participant’s time and they will have a reasonable deadline to reply by. The research questions are specific to what the study requires.

Sensitivity of company information – The participants will be made aware that the information will be shared with the Steel Research Hub

Choice of anonymity – The participant can chose to remain anonymous in reporting of research but their identity will be known to the researchers

Withdrawal – Participants will be advised and assured that they may choose to withdraw participation at any point before or during the completion of the survey or within a month; the participant may withdraw any data they have provided to that point.

F. RISKS AND BENEFITS

14. Does the project involve the risk of emotional distress or physical harm, or the use of invasive procedures (e.g., blood sampling)?

YES ☐ NO ☒

If YES:
(a) What are the risks?

(b) Explain how the risks of harm or distress will be minimised. In the case of risks of emotional distress, what provisions have been made for an exit interview or the necessity of counselling?

15. Is information about criminal activity likely to be revealed during the study?

YES ☐ NO ☒

If YES, have you included a caution regarding any relevant mandatory reporting requirements in the Participant Information Sheet?
16. Detail the expected benefits of the study to the participants and/or the wider community.

Participants – Connection to Australia for overseas participants through cold formed steel research. There is also potential for overseas companies to expand into the Australian market.

Wider community – Research for improvement in the use of cold formed steel in Australia may lead to more sustainable medium density housing for the community. This research could further expand the cold formed market and lead to job creation in the local community.

G. PARTICIPANTS

17. MARK THE CATEGORIES RELEVANT TO THIS PROPOSAL:

☐ Healthy members of the community
☐ University students
☒ Employees of a specific company/organisation
☐ Members of a specific community group, club or association
☐ Clients of a service provider
☐ Health Service clients (eg users/clients of a Health Service)
☐ School children
☐ Hospital in-patients
☐ Clinical clients (eg patients)
☐ Aboriginal/Torres Strait Islander people
☐ Members of socially disadvantaged groups
☐ Cadavers/cadaveric organs
☐ Other (please specify) ____________________________

18. EXPECTED AGE/S OF PARTICIPANTS - PLEASE MARK ONE OR MORE

☐ Children (under 14 years)
☐ Young people (14-18 years)
☒ Adults (> 18 years)

19. What is the rationale for selecting participants from this/these group/s?

The participants invited to participate in this research are employees of an overseas or Australian company with experience in a particular building project. The building projects were selected through meeting a criteria suiting the research aims. The people will initially be identified either through contacts at Bluescope Steel, SBRC and within the University of Wollongong or via publicly available information on the internet.
H. RECRUITMENT

20. How will potential participants be approached initially and informed about the project? For example, direct approach to people on the street, mail-out to potential participants through an organisation, posters or newspaper advertisements etc. Please explain in detail and include copies of any letters, advertisements or other recruitment information.

initial approach will be via email to their professional email account

21. Where will potential participants be approached by the researchers to seek their participation in the research, and where will research activities involving participants be conducted?

potential participants will be approached via email to their professional email account to seek participation in research. activities will be conducted on the internet through email.

22. How many participants in total do you anticipate will be involved in the project? If the research has several stages and/or groups of participants, please provide the total number of participants expected as well as the number and participant group involved in each stage.

5-10 depending on participation

I. CONSENT PROCESS

Generally the consent of participants must be obtained prior to conducting research. If you do not intend to seek people’s permission to use information about them which may be identifying, you may need an exemption from State and Federal Privacy requirements. This is addressed in Section J.

Attach copies of any letters of invitation, information packages, consent forms, proxy/substitute consent forms, debriefing information, identification cards, contact details cards, etc and ensure they include a version number and date.

23. Will consent for participation be obtained from participants or their legal guardians?

YES ☒  NO ☐

If NO, go to Q.31.
24. **How will consent for participation be obtained?**

- ☐ In writing
- ☐ Verbally
- ☒ Tacit (For example, indicated by completion and return of survey)
- ☐ Other (please specify) _____________________________
- ☐ Consent not being sought

*Please explain why the method chosen is the most appropriate and ethical.*

A tacit method for consent was chosen because the participants are professional people and the subject of the survey relates only to their role within a project they have worked on professionally and is limited to non-sensitive information. It is therefore seen to be unnecessary to obtain signed consent forms, and it is also easier for the participant and reduces the need for scanning and returning signed documents.

25. **Is it anticipated that all participants will have the capacity to consent to their participation in the research?**

- YES ☒
- NO ☐

If NO, please explain why not (eg children, incompetent participants etc) and explain how proxy or substitute consent will be obtained from the person with legal authority to consent on behalf of the participant.

26. **For participants who have the capacity to consent, how does the process ensure that informed consent is freely obtained from the participant?**

Potential participants are free to choose whether they consent and reply to the initial contact or not. Participants will be advised and assured that they may choose to withdraw participation at any point before or during the completion of the survey or within a month; the participant may withdraw any data they have provided to that point.

27. **Are any participants in a dependant relationship with the researcher, the institution or the funding body (eg, the researcher’s clinical clients or students; employees of the institution; recipients of services provided by the funding body)? If so, what steps will be taken to ensure that participants are free to participate or refuse to participate in the research without prejudice or disadvantage?**

- No
28. How does the project address the participants’ freedom to discontinue participation? Will there be any adverse effects on participants if they withdraw their consent and will they be able to withdraw data concerning themselves if they withdraw their consent?

There will be no adverse effects on participants if they withdraw their consent. If participants withdraw during the project within a month, they will be able to withdraw any data they have provided.

29. Does the project involve withholding relevant information from participants or deceiving them about some aspect of the research?

YES ☐ NO ☒

If YES, what is the justification for this withholding or deception and what steps will be taken to protect the participants’ interest in having full information about their participation?

30. Will participants be paid or offered any form of reward or benefit (monetary or otherwise) for participation in the research? If so, please detail and provide a justification for the payment, reward or benefit.

No

J. CONFIDENTIALITY AND PRIVACY

31. How will the privacy of individual subjects be protected when recording and analysing the data?

Participants will be asked to give consent for use of their names and company names in the survey. They will also be given the option to remain anonymous.

32. Will information collected from data or interview be published or reported?

YES ☒ NO ☐

If YES, what form will this take? All uses of data must be explicitly consented to.

The information will potentially be used within the student’s PhD thesis, academic publications, and steel research hub reports.

33. Will any part of the research activities be placed on a visual or audio recording (eg digital audio/visual recordings or photographs)?

YES ☐ NO ☒

If YES:

(a) What will the recording be used for?
(b) Who will see/hear the recording?

34. Data (including questionnaires, surveys, computer data, audio/visual digital recordings, transcripts and specimens) must be securely stored at all times. Where will the data be held and who will have access to it? (Please include building and room numbers if relevant)
   (a) During the project?
   Soft copy data will be securely stored on the SBRC computer system. Any hard copy data will be stored in a locked SBRC cabinet
   Members of Steel research hub will have access to the data
   (b) On completion of the project?
   Data will be stored securely and archived according to UOW archiving policies

35. Data should be held securely for a minimum of 5 years (15 years for clinical research) after completion of the research. How long will the data be stored for? If it is not being stored, please provide an ethical justification for this.
   Data will be stored securely and archived according to UOW archiving policies

36. Does this project involve obtaining identifiable information (eg, data) from a third party without prior consent from the participant or their legal guardian?
   YES ☐ NO ☒
   If NO, you have completed the questionnaire. Please ensure that the form has all the appropriate signatures and attachments and complete checklist before submission.
   If YES, go to Question 37.

37. Who will be providing the information? Please specify whether the organisation will be Commonwealth, State or private, and include copies of any correspondence regarding permission to access this information from a responsible officer of the agency.
38. Will the information be de-identified during collection, use or disclosure?

YES ☐  NO ☐

If NO, you must apply for an exemption to the State and Federal Privacy Acts. Please complete the Privacy Exemption Application Form available from the Forms section of the Ethics web page.

IF YES:
(a) Who will be de-identifying the information? Is this a person who would normally have access to the information?

(b) How and when will the data be de-identified?
K. DECLARATION BY INVESTIGATORS

Principal Investigator:

- I certify that I am the Principal Investigator named on the front page of this application form.
- I undertake to conduct this project in accordance with all the applicable legal requirements and ethical responsibilities associated with its carrying out. I also undertake to take all reasonable steps to ensure that all persons under my supervision involved in this project will also conduct the research in accordance with all such applicable legal requirements and ethical responsibilities.
- I certify that adequate indemnity insurance has been obtained to cover the personnel working on this project.
- I have read the NHMRC’s National Statement on Ethical Conduct in Human Research and the Australian Code for the Responsible Conduct of Research. I declare that I and all researchers participating in this project will abide by the terms of these documents.
- I make this application on the basis that it and the information it contains, are confidential and that the Human Research Ethics Committee of the University of Wollongong/Illawarra Shoalhaven Local Health District will keep all information concerning this application and the matters it deals with in strict confidence.

Professor Tim McCarthy 23/8/15

Name (please print) Signature Date

Signature/s of other researcher/s: The first named researcher will assume responsibility for the project in the absence of the Chief Investigator. All investigators must sign the application.

Doctor Emma Heffernan 25/8/15

Name (please print) Signature Date

Nicholas Franklin 25/8/15

Name (please print) Signature Date

Include additional lines if necessary
L. **APPROVAL BY HEAD OF UNIT**

This person must not be a member of the research team.

I am aware of the content of this application and I am satisfied that:

- All appropriate safety measures have been taken;
- The research is in accordance with UOW/ISLHD Policy;

and approve the conduct of the project within this unit.

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**NOTE: RESEARCH MUST NOT COMMENCE UNTIL THE APPLICATION HAS BEEN APPROVED BY THE HREC**
PARTICIPANT INFORMATION SHEET FOR EXPERT SURVEYS

PROJECT TITLE: A survey of cold formed steel companies relating to specific mid-rise residential building projects

BACKGROUND & PURPOSE OF THE RESEARCH
This is an invitation to participate in a study conducted by researchers at the Sustainable Buildings Research Centre (SBRC) at the University of Wollongong, Australia. This research aims to gain an understanding of the state of the art in cold formed steel construction for mid-rise residential buildings. This will be achieved through the analysis of case study buildings using a questionnaire survey to obtain quantitative and qualitative data on existing cold formed steel construction. This research forms part of a PhD funded by the Australian Research Council (ARC) through the Research Hub for Australian Steel Manufacturing.

INVESTIGATORS
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METHOD AND DEMANDS ON PARTICIPANTS
If you choose to participate, you will be asked to complete an online survey with an expected duration of 30 minutes. You will be asked to consider a particular construction project your company was involved with and answer questions regarding it. The survey will be laid out with the following section structure:

1. Key data on the building including time, cost and size
2. Type of construction for roof, walls, floor
3. Reasoning for design decisions, what went well and what didn’t throughout the construction process
4. Other questions regarding building information modelling, compliance, sustainability and onsite construction logistics.

The research data will potentially be used within my PhD thesis, academic publications, and steel research hub reports.

POSSIBLE RISKS, INCONVENIENCES AND DISCOMFORTS
The main issue is the time required to complete the survey. Your involvement in the study is voluntary and you may choose to withdraw your participation at any point before or during the completion of the survey or within a month; sensitive information may be redacted or de-identified. Soft copy data will be securely stored on the SBRC computer system. Any hard copy data will be stored in a locked SBRC cabinet. Refusal to participate in the study will not affect your relationship with the University of Wollongong or the Steel Research Hub.

ETHICS REVIEW AND COMPLAINTS
This study has been reviewed by the Social Sciences Human Research Ethics Committee of the University of Wollongong. If you have any concerns or complaints regarding the way this research has been conducted you can contact the UOW Research Services Office on +61 2 4221 3386 or email rso-ethics@uow.edu.au

Thank you for your interest in this study.